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Seismic Assessment of Typical Historical Masonry Churches in Banat region, Romania - Part I

Anna Lo Monaco[a], Nicola Grillanda[b], Iasmina Onescu[c], Mihai Fofiu[c], Francesco Clementi[d], Michele D'Amato[a], Antonio Formisano[e], Gabriele Milani[b], Marius Mosoarca[c*]

[a]*Department of European and Mediterranean Cultures, Architecture, Environment and Cultural Heritage (DiCEM), University of Basilicata, Via Lanera 20, Matera 75100, Italy*

[b]*Department of Architecture, Built Environment and Construction Engineering (A., B., C.), Politecnico di Milano, Piazza Leonardo da Vinci 32, Milan 20133, Italy*

[c]*Faculty of Architecture and Urban Planning, Politehnica University Timisoara, Traian Lalescu 2/A, Timişoara 300223, Romania*

[d]*Department of Civil and Building Engineering, and Architecture (DICEA), Polytechnic University of Marche, Via Brecce Bianche 12, Ancona 60131, Italy*

[e]*Department of Structures for Engineering and Architecture, School of Polytechnic and Basic Sciences, University of Naples "Federico II", Piazzale Tecchio 80, Naples 80125, Italy*

Abstract

The Banat seismic area in Romania is characterized by shallow earthquakes that generate in historic buildings different failure mechanisms. Religious buildings represent one of the most important historical buildings in Romania, with interesting structures and beautiful architectural details, but many of them in a poor conservation state.

The failure mechanisms of the historical Romanian Baroque churches recorded after the earthquakes of 1990 and 1991 are due to the high value of the vertical components of the seismic action, the reduced number of cycles.

The article presents the values of the seismic risk factor determined for three churches: Chizatau, Beregsau Mare and Birda churches with the ETABS software according to the Romanian design code P100-3 / 2019 in which the effects of the vertical seismic components are not taken into analysis.

Keywords: Historical churches; Seismic assessment; Local failures; Non-linear analyses; shallow earthquakes

* Corresponding author. Tel.: +40740612757
E-mail address: marius.mosoarca@upt.ro

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1. Introduction

Romania is a European country, located in the Eastern part of Europe, with moderate seismicity in Banat seismic area, located in the western part of the country and interesting architecture. One of the most representative architectural edifices are the religious ones, as Romania is an Orthodox country with many historical churches, that are well preserved.

Churches represented in the past and still represent in the present a very important social and cultural asset of each local community. Most of the historical churches still exist in good shape after hundreds of years, even though they are made either in wood, or masonry. The masonry ones present usually stone or masonry foundations, masonry massive walls and masonry vaults with/or timber framework.

Many of the Romanian churches were built before the existence of any design code, so their structural system is a complex one, which needs a very close investigation to be understood and preserved in harmony with the authenticity of the building (Lo Monaco A., 2022).

The seismic assessment of such complex historic structures represents a difficult task, due to the consideration of both architectural and structural variables, but also due to the community implications of such an intervention. Several studies were previous made to investigate the typical failure mechanisms for historic religious buildings located in seismic area, but most of them are based on the failure rigid blocks method for Roman Catholic churches type. For the Orthodox churches type, which are different in terms of shape in plan, elevation and building system, a study was made in 2013, based on a comparison between numerical analysis results and real damages observed in Banat region, after the earthquakes of 1991 (Mosoarca M., 2013), highlighting the failure mechanism observed after past earthquakes, which are in harmony with the expected failure mechanism from the numerical analysis.

2. Case study buildings

2.1. Sf. Mare Mucenic Gheorghe church in Beregsau Mare

Beregsau Mare is a village from 1335, located in Timis county at 18 km away from Timisoara city, with almost 1800 inhabitants. The Orthodox church „Sf. Mare Mucenic Gheorghe" was first mentioned in 1793, as a consignment of its building phase that took place between 1809 and 1812.

The church was built in masonry clay brick with lime, with masonry foundations, masonry 65-92 cm perimetral walls, 36 cm thickness masonry vaults and wooden framework above the vaults. The building has an orthogonal plan, with 24.15 meters x 9.80 meters maximum plan dimensions (Fig. 1a), a maximum height of around 12 meters for the central part and 23.65 meters for the tower (Fig. 1b) and very interesting interior paintings (Fig. 1c), as well as wooden structure for the tower (Asociatia Restauro, 2019). The survey of the buildings was done using 3D scanners.

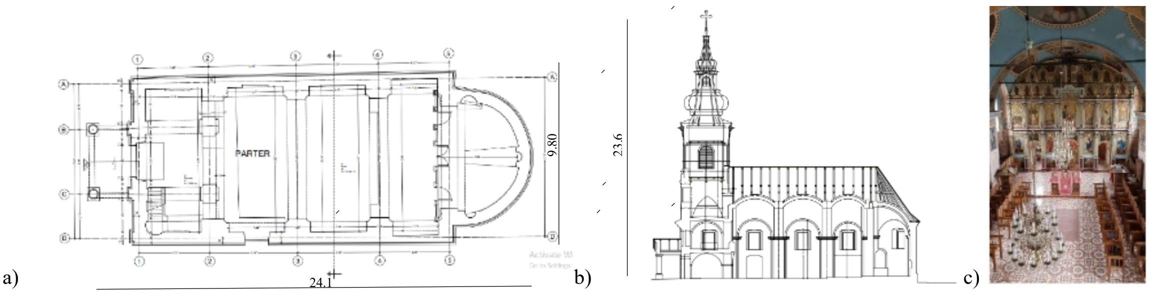


Fig. 1. Plan, section and interior paintings of the „Sf. Mare Mucenic Gheorghe" church in Beregsau Mare village, Romania

2.2. Nasterea Maicii Domnului church in Chizatau

Chizatau is a small village located in Timis county, with less than 1000 inhabitants. The villages dates from year 1359 and is famous because of the religious chorus, a very respected artistic formation. The church of Chizatau village is named „ Nasterea Maicii Domnului" is located in Chizatau village at no. 273 and was built in 1827.

The church was built in masonry clay brick with lime, with stone foundations, masonry walls of 57-97 cm thickness, masonry vaults and wooden framework above the vaults. The building has an orthogonal plan, with 25.33 x 9.27 meters maximum plan dimensions (Fig. 2a), a maximum height of 10.26 meters for the central part and 23.21 meters for the tower (Fig. 2b) and very interesting interior paintings (Fig. 2c), as well as wooden framework (Tonca D., 2021).

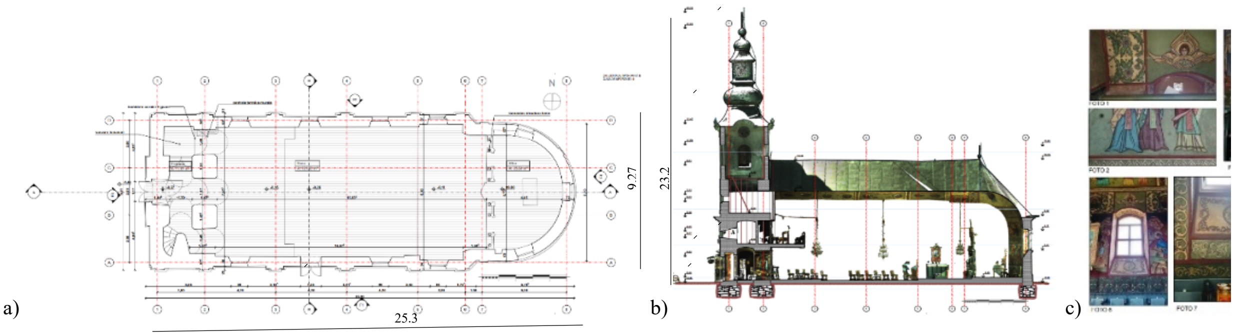


Fig. 2. Plan, section and interior paintings of the „Nasterea Maicii Domnului" church in Chizatau village, Romania

2.3. Sf. Gheorghe church in Birda

Birda has been documented with this name since 1690, and in 1761, on the official map of the Banat administration, the locality with the name of Pirda appears. The Serbian monastic settlement, "Sfântul Gheorghe", from the village of Mănăstire, is located near the river Bârzava and the forest. The architectural style of the monastery is Byzantine, and the painting on the vault is in fresco. The fame of the monastery is related to the name of Saint Ghorghe. After 1944, the monastery became for a time a barracks, then an office building for the Agricultural Production Cooperative in the village.

The church was built in masonry clay brick with lime, with stone foundations, masonry thick perimetral walls, masonry vaults and wooden framework above the vaults. The building has an orthogonal plan, with 20.95 meters x 9.40 meters maximum plan dimensions (Fig. 3), and very interesting interior paintings (Fig. 3), (Mosoarca M, 2013).

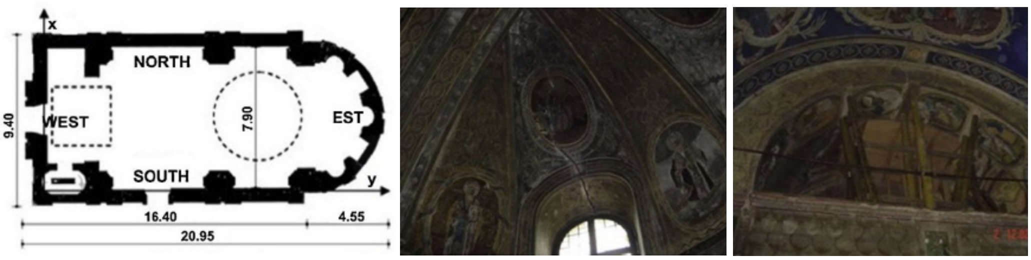


Fig. 3. Plan of the „Sf. Gheorghe" church in Birda village, Romania and interior paintings

3. Current state of the structures

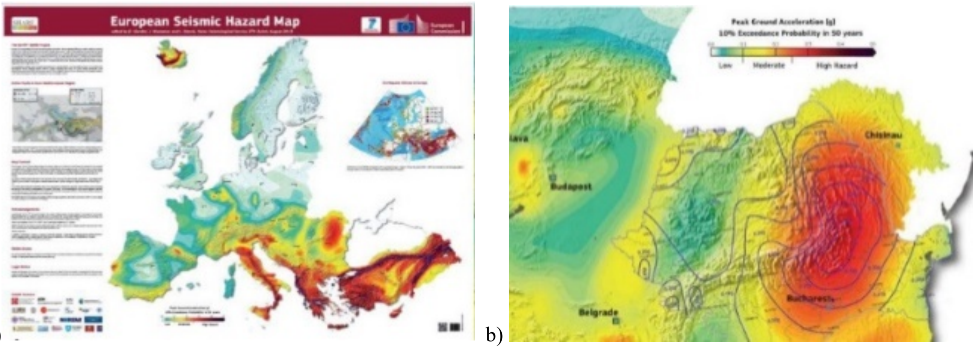
3.1. Seismic area description

By looking at the SHARE earthquake hazard map of Europe (Fig. 4a) (D. Giardini, 2016), Europe has zones with high seismic hazard, the map was evaluated with a time-independent, probabilistic approach. Although the southern part of Europe has the largest risk in terms of seismic actions with peak ground accelerations exceeding 0.5g, the central-eastern part is dominated by a confined region high ground acceleration, the region in question is the Vrancea region, the most dangerous seismic zone in Romania, followed by Banat seismic area. By overlaying the Romania norm peak ground accelerations map (P100-1, 2013) over the SHARE earthquake hazard map (Fig 4b), it can be observed that Banat region has its own epicenters for earthquakes. In Table 1 the distance from the epicenter of the earthquake in Banloc from 1991 for the churches is presented and it can be observed that the church from Birda is 15 km from the Banloc epicenter, so it fits the category of "epicentral earthquake" while the other two churches are at 42 and 62 km from the epicenter.

Table 1. Distance from Banloc epicenter

Distance from Banloc epicenter	
Birda	15 km
Beregsau Mare	42 km
Chizatau	61 km

Although the churches are not far from each other in terms of distance, due to the specific seismic map of Romania they are in seismically active areas having different peak ground acceleration of 0,20g for Beregsau Mare and Birda and 0,15g for Chizatau, as presented in Fig. 4b.



a) b)

Fig. 4. a) SHARE European Seismic Hazard Map 2013; b) SHARE map and P100-1 2013, peak ground accelerations for Romania overlay

3.2. Current technical condition of structural elements

As for the current state of the churches the description of each structural part will be treated separately. For the foundations in all cases, they are continuous under the masonry walls, one made from stone masonry and the other of brick masonry, with mortar. All buildings present vertical cracks near the windows on the outside of the façade as presented in Fig. 5a for Beregsau Mare, Fig. 5b for Chizatau church and Fig. 5c for Birda church.

For Beregsau Mare church, the vault over the narthex, nave and altar is made of brick masonry. The floors of the tower are on wooden beams with plank floors. Following the on-site inspection, cracks in the masonry of the vaults and arches were found in the areas where the plaster was laid. The frame is made of wood, made of rafters with a section of 12x15 cm., braces with a section of 13x16 cm., and collar ties with a section of 13x16 cm. The first farm near the tower is broken.

As for the Chizatau church the vault over the nave and altar is made of softwood boards. Following the on-site inspection, cracks in the plaster were found. Behind the iconostasis the longitudinal walls are connected with a metal tie. The frame is made of purlins with a hand-processed wooden section with a size of approx. 12x15, wooden props 2x12x12, handmade rafters 11x11 and handcrafted wooden tie beams 14x16.

Inside the Beregsau Mare church, cracks were identified between the longitudinal walls and the altar wall, at the arches of the narthex balcony and cracks in the narthex vault have been identified (Fig. 6a). For Chizatau church cracks were identified between the tower walls and the longitudinal walls as presented in Fig 6b.

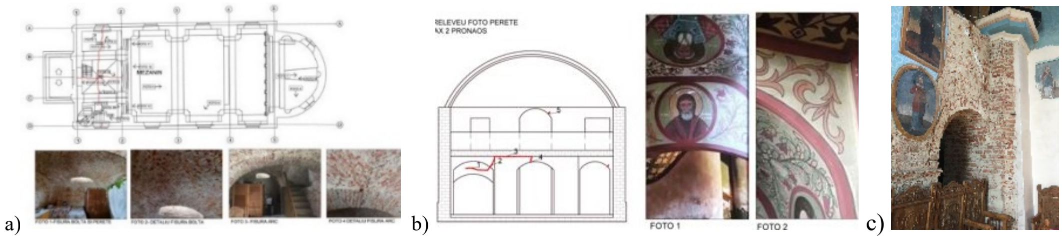


Fig. 5. Structural damages identified on interior walls for: a) Beregsau Mare church; b) Chizatau church; c) Birda church

For Birda church, which was close to the epicenter, by examining the positions of the cracks after the earthquake in 1991, the followings damages were found: the East tower has the upper closure in a vault made of bricks and has no significant cracks, the West and East towers present diagonal cracks characteristic to seismic actions and horizontal cracks due to torsion at the level of the windows (Fig. 6c). The largest cracks are developed in the eastern wall next to the altar because it is situated furthest away from the center of rigidity. (Mosoarca M., 2013)

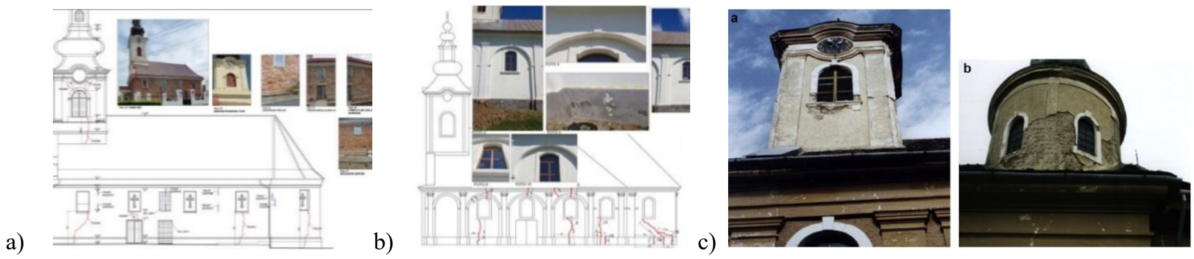


Fig. 6. Structural decay: a) in Beregsau Mare; b) in Chizatau; c) diagonal cracks and horizontal cracks in Birda (Mosoarca M., 2013)

4. Linear analysis

For the spatial analysis ETABS software was used, a behavior factor q=1.5 was considered for the structure. The principal vibration modes, maximum displacement levels and relative level displacements (drift ratio), vertical displacements (settlements) at the foundation level and the shear force in the walls were computed.

The degree of seismic assurance R3 according to the Romanian code P100-3 highlights the strength and ductility of the structure, in relation to seismic requirements. R3 for the structure is determined at the level above the theoretical embedding level and, where appropriate, at the other levels if they are deficient in rigidity or strength compared to the base level.

The seismic risk classes are as follows: Seismic risk class 1 for buildings with susceptibility to collapse; Seismic risk class 2 which includes buildings susceptible to major damage due to the design earthquake; Seismic risk class 3 which includes buildings susceptible to moderate damage due to the design earthquake; Seismic risk class 4 which includes buildings to which the expected seismic response under the effect of the design earthquake, is similar to that expected for constructions designed on the basis of normative design codes.

It is computed by dividing the design value of the shear force associated with the strength of the vertical element at the level considered and the design value of the shear force in element i, resulting from structural calculation in the relevant seismic load combination, according to Equation 1 (P100-3, 2018).

$$R_3 = \frac{\sum V_{Rdi}}{\sum V_{Edi}} \tag{1}$$

R3 is determined for both OX and OY directions with the relevant arrangement of the shear walls for both directions, for the existing structure a trust factor CF=1.35 was considered corresponding to a knowledge level KL1. The load bearing capacity of structural walls subjected to in plane forces is based on the type of failure expected. The shear force associated with the eccentric compression failure of an unreinforced masonry wall subjected to the design axial force Nd is calculated with the relation presented in Equation 2 (P100-3, 2018):

$$V_{f1} = \frac{N_d}{(c_p \cdot \lambda_p)} \cdot (1 - 1.15 \cdot v_d) \quad (2)$$

Whereas the shear strength of the unreinforced masonry wall is given by the relationship presented in Equation 3 (P100-3, 2018):

$$V_{f2} = min(V_{f21}, V_{f22}) \quad (3)$$

Where the design value of the shear breaking force in the horizontal joint V_{f21} is calculated by the formula presented in Equation 4 (P100-3, 2018):

$$V_{f21} = \frac{1.33}{CF \cdot \gamma_M} \cdot \left(f_{vk0} \cdot \frac{l_{ad}}{l_c} + 0.4 \cdot \sigma_d\right) \cdot t \cdot l_c \quad (4)$$

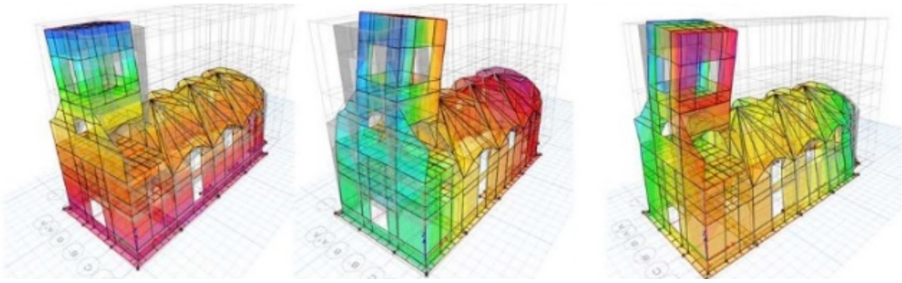
and the The design value of the breaking force by diagonal cracking V_{f22} is calculated by the formula presented in Equation 5 (P100-3, 2018)

$$V_{f22} = \frac{t \cdot l_w \cdot f_{td}}{b} \cdot \sqrt{1 + \frac{\sigma_0}{f_{td}}} \quad (5)$$

In order to determine the pressure and settlement at the base of the foundations, in the calculation program spring-type elastic supports with a rigidity of 15000 kN/m/sqm were defined. These supports were applied to some surface elements arranged at the base of foundation with dimensions similar to the size of the existing foundations.

4.1. Sf. Mare Mucenic Gheorghe church in Beregsau Mare

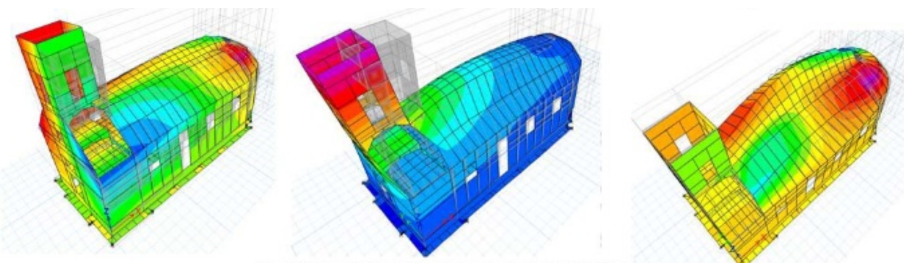
The main vibration periods are presented in Fig. 7 for translation in transversal and longitudinal direction and for torsion. The effective pressure on the ground, at the base of the foundations in the tower area, exceeds the corrected conventional pressure of the foundation ground. Therefore, it was proposed to consolidate the foundations by widening the foundations by 40 cm and making a perimeter beam of reinforced concrete. For the degree of seismic assurance R3 table 3 presents the ration for both directions.



Mode 1: Oy transversal translation T=0.246s Mode 2: Ox longitudinal translation T=0.175s Mode 1: Torsion T=0.128s

Fig. 7. Main vibration periods.

4.2. Nasterea Maicii Domnului church in Chizatau



Mode 1: Oy transversal translation T=0.44s Mode 2: Ox longitudinal translation T=0.33s Mode 1: Torsion T=0.23s

Fig. 8. Main vibration periods

The main vibration periods are presented in Fig. 8 for translation in transversal and longitudinal direction and for torsion. For the degree of seismic assurance R3 table 3 presents the ration for both directions for the strengthen and unstrengthen building.

4.3. Sf. Gheorghe church in Birda

The main vibration periods are presented in Fig. 9a for translation in transversal and longitudinal direction and for torsion. In the analyses of seismic behavior of Church of St. Gheorghe Birda, Timis County, Romania, based on the theory of transfer mechanisms, the church was divided into five rigid blocks according to the observations of cracks recorded after 1991 earthquake in the Banat area. Transfer blocks are shown in Fig. 9b.

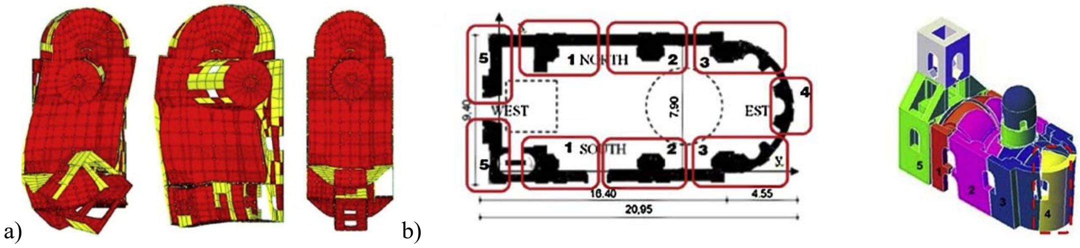


Fig. 9. a) Main vibration periods; b) Rigid blocks division for analysis.

Because the greatest damages were recorded in the nave and apse altar, a special attention was given to these areas. The analysis predicts that the ultimate limit state will be achieved by the collapse of block 4, which will collapse before blocks 2 and 3. This damage is confirmed by the biggest masonry rupture of the apse area after the 1991 earthquake. The torsion appears due to the large distance between the mass center (MC) and the rigidity center (RC) and the big difference between the rigidity in the nave and narthex. The main vibration periods are presented in Table 2. for translation in transversal and longitudinal direction and for torsion for all three churches.

Table 2. The main vibration periods.

	Transversal (seconds)	Longitudinal (seconds)	Torsional (seconds)
Beregsau Mare	0.128	0.175	0.246
Chizatau	0.230	0.330	0.440
Birda	0.210	0.340	0.480

4.4. Strengthening solutions

In accordance with the provisions of the technical expertise, the load bearing of structure generally shows damages generated by the differentiated settlement of the foundations, the deficient conformation and the lack of maintenance in time similar to other studies (Mosoarca M., 2012, Gioncu V., 2009). The proposed consolidation for the load-bearing structure is based on the damages identified following the in-situ investigations and the recommendations of the technical expertise. The interventions on the structure aim at preserving the cultural, artistic and historical value of the building. The proposed measures were based on the principle of reversibility of interventions, with interventions that are minimally invasive and are based on ensuring compatibility between materials.

- The foundations will be consolidated by making concrete substructures in sections of 60cm-100cm with a height of 40 cm.
- For the existing masonry walls the load bearing capacity was not necessary to be increased so the retrofitting procedure targets the existing cracks, so several different FRP materials can be used.

Restoring the continuity of the cracked masonry will be done by inserting stainless steel bars on the outside in the horizontal joints every 3 seats over which will be mounted fiberglass mesh with metal insert Geosteel Grid from Kerakoll.

Table 3. The degree of seismic assurance R3

Unconsolidated existing structure q = 1.5			
Earthquake direction SX + right	Earthquake direction SX- left	Earthquake steering SY + right	Earthquake direction SY- left
R3 = 0.91	R3 = 0.77	R3 = 0.87	R3 = 0.79
Seismic risk class IV	Seismic risk class III	Seismic risk class III	Seismic risk class III
R3 insurance degree for seismic actions in the plan of the walls Chizatau			
Unconsolidated existing structure q = 1.5			
R3 = 0.93	R3 = 1.22	R3 = 0.62	R3 = 0.51
Seismic risk class IV	Seismic risk class IV	Seismic risk class II	Seismic risk class II
Consolidated structure q = 1.5			
R3 = 0.93	R3 = 1.22	R3 = 0.82	R3 = 0.69
Seismic risk class IV	Seismic risk class IV	Seismic risk class III	Seismic risk class III

5. Conclusions

Following the analysis of the seismic response of the church through computer simulations with the ETABS program and in-situ investigations, the following conclusions resulted:

1 Although the churches were analyzed for a PGA value of 0.20g and 0.15g and the simulations indicate a high level of seismic damage, in reality these results were not confirmed by the pattern of cracks and the level of damage.

2. In situ, the seismic failure blocks for PGA 0.20 and 0.15g cannot be identified, because there are no cracks to delimit them.

3. Churches located more than 50 km from the source record vertical cracks due to settlements in the type in which the churches in the epicenter develop vertical cracks in the apse of the altar and diagonal and horizontal cracks in the bell tower.

4. Although the churches are similar to St. George of Bidra located in the epicenter of earthquakes in 1990 and 1991, there are different failure mechanisms between them, located about 50 km from the source and the epicenter due to the presence of high values of vertical seismic components.

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