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SEISMIC RESPONSE OF HISTORICAL MASONRY CHURCHES: THE CASE OF THE BANAT ROMANIAN REGION

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Abstract

The seismic events of the recent decades allowed us to comprehend the vulnerabilities of the historical built heritage and the fundamental role of prevention and safeguard strategies. This led us to study the seismic performances of historical constructions in Europe. This on-going research focuses on the built heritage in Romania, with particular attention to six typical masonry churches located in the Banat region, whose structural configuration is characterized by a long vaulted central nave and a bell tower included into the main façade. We firstly compare the seismic vulnerability of the six churches and estimate an intervention priorities scale through simplified risk assessment methods. Then, we present a more detailed vulnerability investigation focused on one of the churches, by means of global finite element modal linear analyses and assessment of local failures through adaptive limit analysis.

Keywords: historical masonry churches, seismic assessment, territorial scale assessment, local failures, modal linear analyses, vulnerability investigations.

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1 INTRODUCTION

In the recent years the safeguard of our cultural heritage is becoming an increasingly important theme, especially referring to the protection against environmental threats such as earthquakes. Due to the cultural heritage consistence and the very often limited economic resources, authorities need to proceed incrementally through multilevel methodologies for the definition of a priority analysis and intervention scale. More in-depth analyses, included the experimentation by means of local and global models, are then implemented. This task seems to be more challenging if the reference sample is represented by a large number of existing masonry buildings. The most difficult task is to preserve the functional and structural aspect of these constructions, also keeping the historical-monumental value intact. Furthermore, each of these buildings is made with different construction techniques and materials, generating increasingly different three-dimensional configurations.

The work presented in this paper is part of an ongoing research focused on existing masonry buildings located in the Romanian country [1, 2]. In particular, the study intends to analyze the seismic response of some Orthodox churches belonging to the Banat area. In this paper's first section different vulnerability assessment methodologies are presented and compared on a sample of six similar churches. Additionally, more detailed analyses on one church assumed as case study are carried out to simulate the seismic response of these particular buildings. The aim of the paper is to experiment both Italian and Romanian approaches for existing masonry buildings applied on a Romanian churches sample.

One may note that the analyzed churches are all characterized by a rectangular plane configuration ending with a circular or polygonal apse on one side and incorporating one bell tower on the other side, centrally in the main façade. Internally, all the representative Orthodox churches elements are in this order located: the narthex (pronaos), the central nave (naos), the transept, the iconostasis (non-structural painted wall between naos and transept) and the sanctuary (altar area). Figure 1 synthetizes churches features previously mentioned.

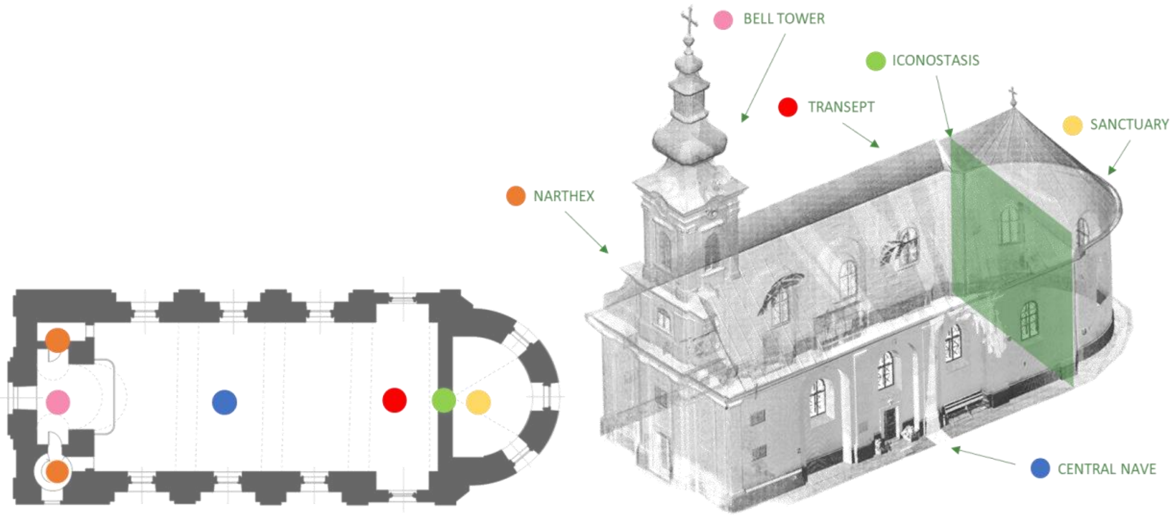
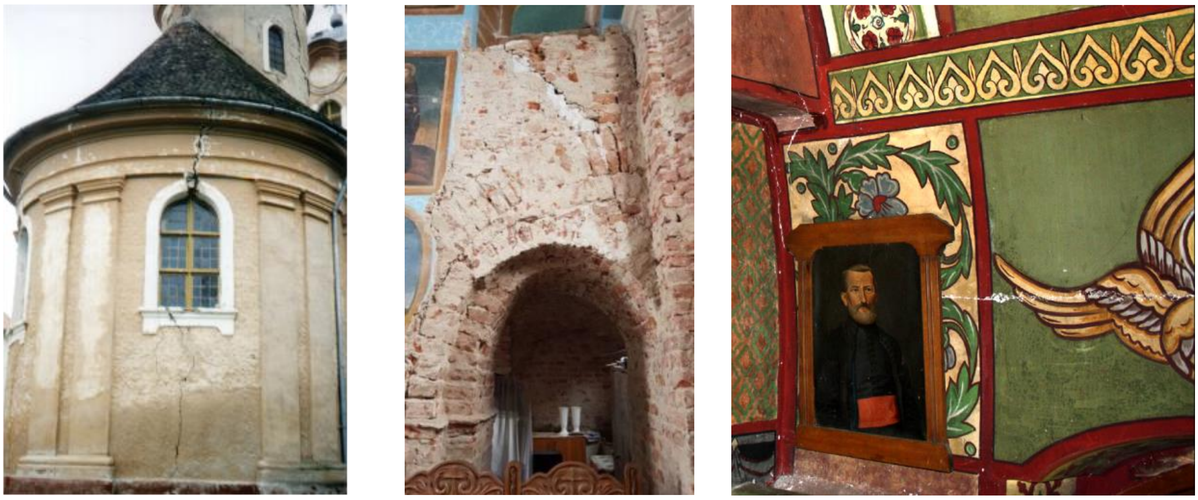


Figure 1: Main architectural elements of an Orthodox church (the picture shows the Învierea Domnului Church in Belinț) [2].

As regarding materials, the six churches are made of brick or mixed stone-brick masonry bearing walls, a system of brick masonry vaults (in some cases wooden or plasterboard-wooden plank vaults occur), a brick or mixed stone-brick masonry bell tower with a wooden spire at the top, a wooden mezzanine located in the narthex and a pitched wooden roof.

2 BACKGROUND

Previous research on a Banat church in Birda [3], investigated after the occurrence of 1991 earthquakes in Banloc, has suggested the division of the church into five rigid blocks (one on the apse area, three blocks on the nave and one between the bell tower and the bearing walls) respecting the cracks detected on walls [1]. Furthermore, it has been pointed out the importance of the vertical seismic components, which are not negligible compared to the horizontal ones. The most damage affected area was the apse one, interested by two vertical deep cracks on the walls, as illustrated in Figure 2a. Later research conducted on other two churches located in Beregsău Mare and Chizătău cities has demonstrated that only churches at a maximum distance of 50 km from the epicenter (the Birda one was 15 km far) showed clear cracks caused by the earthquake (vertical cracks on the apse area and diagonal crack on the bell tower), while the other ones recorded damage triggered by the differentiated settlement of the foundations, lack of interventions and deficient conformation, as shown in Figure 2b and 2c [1]. Briefly, even if the expected PGA was respectively 0.15 g and 0.20 g for Chizătău and Beregsău Mare (61 and 42 km from the epicenter), no earthquake-caused-damage occurred in these two churches.

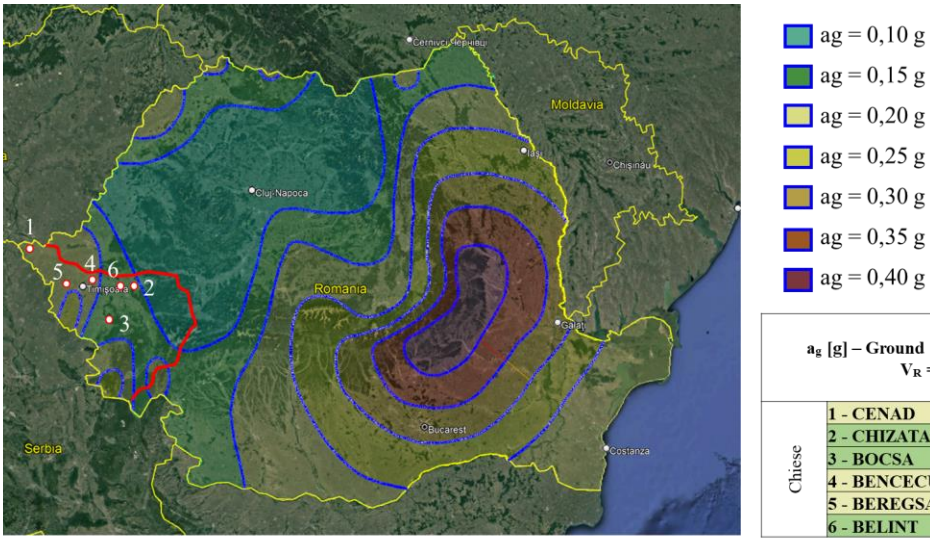


a b c

Figure 2: Cracks recorded: a) on the apse area of Birda church [3], b) between narthex arches and bell tower walls in Beregsău Mare church, and c) on the walls, near the bell tower arches of the Chizătău church.

2.1 Romanian seismicity

Romanian country is located in the Eastern part of Europe and is one of the European highest seismicity countries. The European seismic hazard map [4] shows the Peak Ground Acceleration (*PGA*) expected to be reached or exceeded with a 10% probability (P_{VR}) in 50 years, whose values repeat themselves on average every 475 years. Romanian seismic hazard is estimated to be moderate. Romanian regulations [5] recommends a more detailed hazard map in which the PGA fluctuated between the values of 0.10 g and 0.40 g, assuming P_{VR} = 20% in 50 years and a return period *IMR*= 225 years (T_R for Italian regulations), as shown in Figure 3. In the future the Romanian hazard map will be update following European standards, so an *IMR* = 475 years and a P_{VR} = 10% in 50 years should be used. Figure 3 also shows the expected *PGA* in the six different locations, being the six churches analyzed located in these areas.



a_g [g] – Ground acceleration at LSLS V_R = 50 anni		
Chiese	1 - CENAD	0.20
	2 - CHIZATAU	0.15
	3 - BOCSA	0.15
	4 - BENCECU DE JOS	0.20
	5 - BEREGSAU MARE	0.20
	6 - BELINT	0.15

Figure 3: Romanian seismic hazard map (T_R = 225 years, P_{VR} = 20% in 50 years) [5] and expected PGA in the six different locations.

2.2 Case studies: six churches in the Banat Romanian area

Figure 4 represents the localization of the Banat area and of the six churches listed below. They are located all in the Timiş district, excepted the Bocsa one, located in the Caraş-Severin district.

1. *Pogorârea Sfântului Duh* (Holy Spirit Descent) in **Cenad**
2. *Nasterea Maicii Domnului* (God's Mother Nativity) in **Chizatau**
3. *Sfint Nicolae* (Saint Nicholas) in **Bocsa**
4. *Sfantul Nicolae* (Saint Nicholas) in **Bencecu de Jos**
5. Sfântul Gheorghe (Saint George) in **Beregsau Mare**
6. Învierea Domnului (Jesus Resurrection) in **Belint**

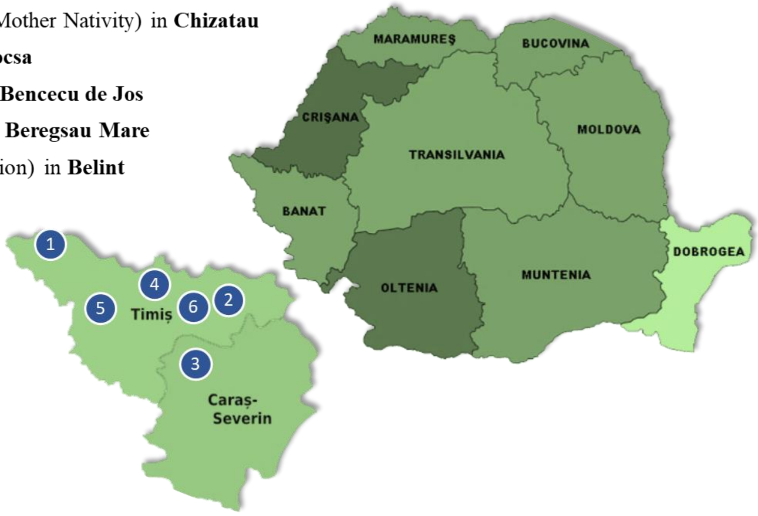


Figure 4: Localization of the Banat region and of the six churches investigated [2].

Table 1 synthetises the main information for each studied church. Two pictures and the plan configuration are also linked to each one. Churches are all made of brick clay masonry bearing walls, excepted Bocsa church (made of mixed stone-brick masonry), and continuous foundations. The Belinț church, in particular, presents mixed stone-brick masonry only from the foundation level up to the ground level. All cracks recorded on these churches are due to different foundation settlements. Only the churches of Bocsa and Beregsău Mare are near the epicentre of the 1991 earthquake, 45 and 42 km far from Banloc respectively.

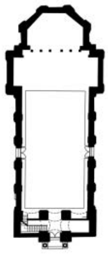
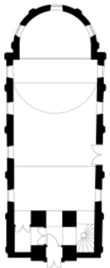
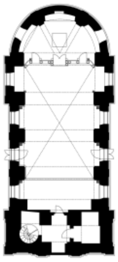
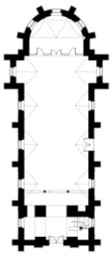
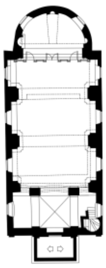
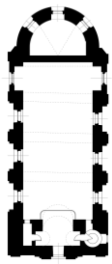
N.	External picture	Interior picture	Plan	Information
1				Construction period and location: 1888, flat rural area of Cenad. Walls material and thickness: brick masonry, 70-75 cm. Vaults: wooden barrel vault with lunettes. Bell tower height: 26.15 m. Damages recorded: cracks on walls, damaged plaster and paintings. Previous interventions: metal ties.
2				Construction period and location: 1827, flat rural area of Chizătău. Walls material and thickness: brick masonry, 57-97 cm. Vaults: wooden boards barrel vaults and arches. Bell tower height: 23.21 m. Damages recorded: cracks on walls, damaged plaster and paintings, cracks between tower walls and longitudinal walls. Previous interventions: -
3				Construction period and location: 1795-1911, city of Bocsa. Walls material and thickness: stone-brick masonry, 100-160 cm. Vaults: brick masonry barrel vaults and arches. Bell tower height: 35.33 m. Damages recorded: - Previous interventions: restoration in 2016 involving foundations, floors, roofs and the use of metal ties.
4				Construction period and location: 1899, hilly area of Bencecu de Jos. Walls material and thickness: brick masonry, 55-75 cm. Vaults: plasterboard and wooden plank barrel vault with lunettes. Bell tower height: 23.27 m. Damages recorded: cracks on walls, damaged plaster and paintings. Previous interventions: 2015, repair of the façade and the roof.
5				Construction period and location: 1793-1810, flat rural area of Beregsău Mare. Walls material and thickness: brick masonry, 35-75 cm. Vaults: brick masonry barrel vaults and arches. Bell tower height: 27.96 m. Damages recorded: cracks on walls, damaged plaster and paintings. Previous interventions: metal ties.
6				Construction period and location: 1797, flat rural area of Belinț. Walls material and thickness: brick masonry, 70-170 cm. Vaults: brick masonry barrel vaults and arches Bell tower height: 25.80 m. Damages recorded before: vertical cracks in the apse area. Previous interventions: complete restoration in 2014-2020, involving foundations, bell tower, walls, roofs, openings and finishes.

Table 1: The six Banat churches analysed.

3 TERRITORIAL SCALE VULNERABILITY INVESTIGATIONS

The first level of assessment, which typically involves comparative assessments at a territorial level, plays such an important role in screening and in-depth analysis planning. In this section are presented three different vulnerability assessment methods. The first one reports vulnerability curves obtained by considering structural, architectural, artistic, urbanistic and social-economic parameters. The second one puts into account environmental threats actions, differentiated in sporadic event and continuous processes, and parameters related to the investigate construction aspects. The last one, provided by Italian regulations, requires on-site surveys for the building's knowledge and allows to define the structure seismic capacity expressed in terms of peak ground acceleration (*PGA*).

3.1 Empirical assessment

This methodology comes from Benedetti and Petrini European method [6], afterwards adapted on Banat Romanian region by Onescu and Mosoarca [7] and finally implemented by Onescu [8]. The analysis provides a vulnerability form to complete jointly to visual inspections. 37 parameters are divided into structural and cultural sections. The final vulnerability index is obtained separately for the two sections ($I_{V\,STRUCT}$ and $I_{V\,CULT}$) by summing individual parameter contributions, as explained in Equation 1 and Equation 2. Each parameter contribution is instead calculated by multiplying the assessed vulnerability class to the associated weight. Equation 3 expresses the calculation of the expected damage state for each church, related to the expected macroseismic intensity.

$$I_{V\,STRUCT} = \sum_{i=1}^{10} s_i \times w_i \tag{1}$$

$$I_{V\,CULT} = 0.70 \times \sum_{i=1}^{10} s_i \times w_i + 0.15 \times \sum_{i=11}^{28} s_i \times w_i + 0.10 \times \sum_{i=29}^{33} s_i \times w_i + 0.05 \times \sum_{i=34}^{37} s_i \times w_i \tag{2}$$

$$\mu_D = 2.5\left[1 + tanh\left(\frac{I + 12.50 \times V_{CULT} - 13.1}{\Phi}\right)\right] \tag{3}$$

The procedure is firstly applied following Benedetti and Petrini method [6], in which only structural parameters are considered. Secondly, cultural value is included in the evaluation. In Figure 5 and Figure 6 are presented the two different approaches by means of individual vulnerability curves (on the left) and main vulnerability curves for the six churches investigated (on the right). The six churches result to be in a medium seismic vulnerability, included in a damage state range of D2-D3 referred to the expected local macroseismic intensity. It is pointed out that, once the cultural value is considered, the medium seismic vulnerability of each church decreases by 5%. This phenomenon is justified by the fact that the six churches analyzed are different from a structural point of view, but similar from an architectural, artistic, urbanistic and social-economic one.

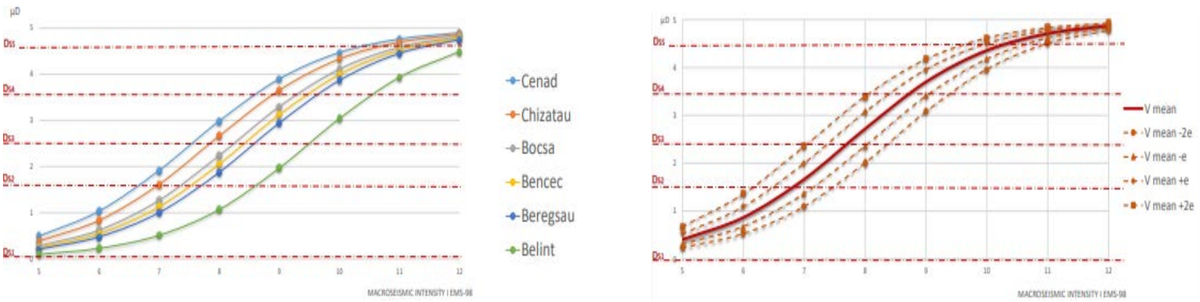


Figure 5: Empirical assessment with only structural parameters. On the left individual curves, on the right main vulnerability curve for the ivestigated churches.

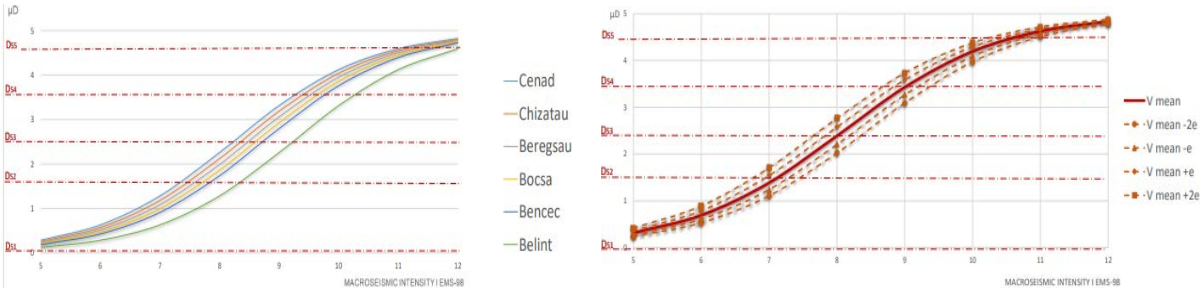


Figure 6: Empirical assessment with cultural parameters. On the left individual curves, on the right main vulnerability cruve for the ivestigated churches.

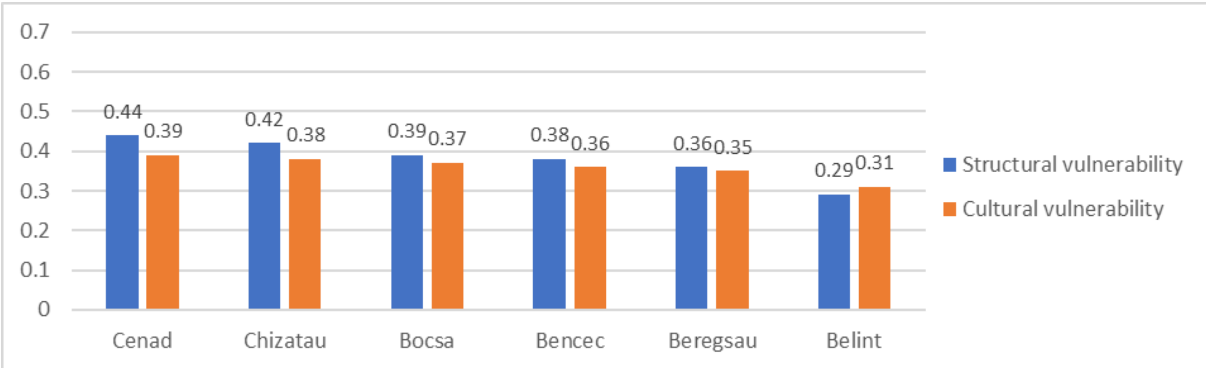


Figure 7: Vulnerability indexes with structural and cultural parameters.

3.2 LV0 and LV1 analyses

Referring to the historic-monumental heritage, Italian regulation [9] provides three Level of Valuation (LV1, LV2 and LV3), denoted by an increasing level of knowledge and analyses complexity. LV1, based on simple on-site surveys, is adopted for territorial scale assessments. For churches and other building with big central naves without intermediate floors, the LV1 simplified model enables the calculation of the vulnerability index (i_v), the peak ground acceleration ($a_{LSLS}\,S$) and the acceleration factor (f_a), by means of Equation 4, 5 and 6. Life-Safety Limit State (*LSLS*) is applied.

$$i_v = \frac{1}{6} \cdot \frac{\sum_{k=1}^{28} \rho_k \cdot \left(v_{k_i} - v_{kp}\right)}{\sum_{k=1}^{28} \rho_k} + \frac{1}{2} \tag{4}$$

$$a_{LSLS} S = 0.025 \cdot 1.8^{5.1-3.44 i_v} \quad [g] \tag{5}$$

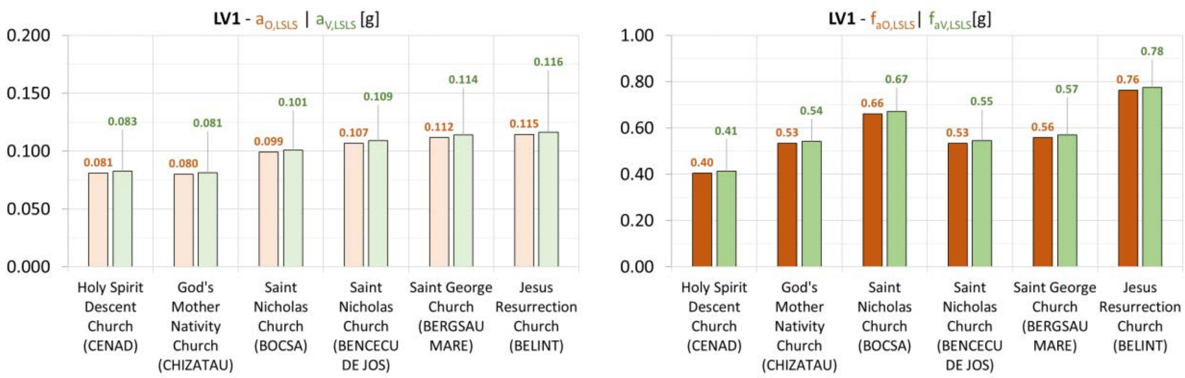
$$f_{a,LSLS} = \frac{a_{LSLS}}{a_{g,LSLS}} \tag{6}$$

In Equation 4 twenty-eight collapse mechanisms are considered: we assign a wight for the k-th mechanism (ρ_k), a score for the associated vulnerability (v_{ki}) and a score for the associated seismic-resistant advice (v_{kp}). ρ_k is equal to 0 if the mechanism considered is absent, while if it is present it ranges between 0,5 and 1. Scores for v_{ki} and v_{kp} may be 0, 1, 2 or 3. Once obtained i_v, the *PGA* at *LSLS* is calculated by means of Equation 5. Finally, once calculated a_{LSLS} by dividing *PGA* for *S* (*S* equal to S_S*S_T, depending on subsoil and topographic categories), the acceleration factor $f_{a,LSLS}$ could be obtained in Equation 6, being $a_{g,LSLS}$ is the expected ground acceleration for *LSLS* in that location. Since Romanian spectrum vertical components are not indifferent, we calculate both horizontal and vertical contributes for $a_{g,LSLS}$ and $f_{a,LSLS}$, by hypothesizing the C subsoil category and assuming the T1 topographic category (churches are located in flat areas). A Confidence Factor FC equal to 1.35 is considered. Table 2 synthetize all LV1 results:

Church	i_v	$a_{g,LSLS}$ [g]	$a_{O,LSLS}/FC$ [g]	$f_{aO,LSLS}$ [g]	$a_{V,LSLS}/FC$ [g]	$f_{aV,LSLS}$ [g]
1	0.587	0.2	0.081	0.405	0.083	0.414
2	0.566	0.15	0.080	0.534	0.081	0.542
3	0.460	0.15	0.099	0.661	0.101	0.672
4	0.450	0.2	0.107	0.534	0.109	0.545
5	0.428	0.2	0.112	0.558	0.114	0.570
6	0.389	0.15	0.115	0.763	0.116	0.775

Table 2: LV1 analysis results.

Figure 8a shows the comparison between horizontal and vertical ground accelerations calculated at *LSLS*, while Figure 8b the horizontal and vertical acceleration factors one. Considering an $a_{g,LSLS}$ of 0.15 g or 0.20 g depending on the case, it can be pointed out that no church results to be verified. If a FC inferior to 1.35 is assumed, only the Belint church could reach a capacity close to the expected acceleration on the site.



a b

Figure 8: (a) Horizontal and vertical ground accelerations and (b) acceleration factors on the six churches.

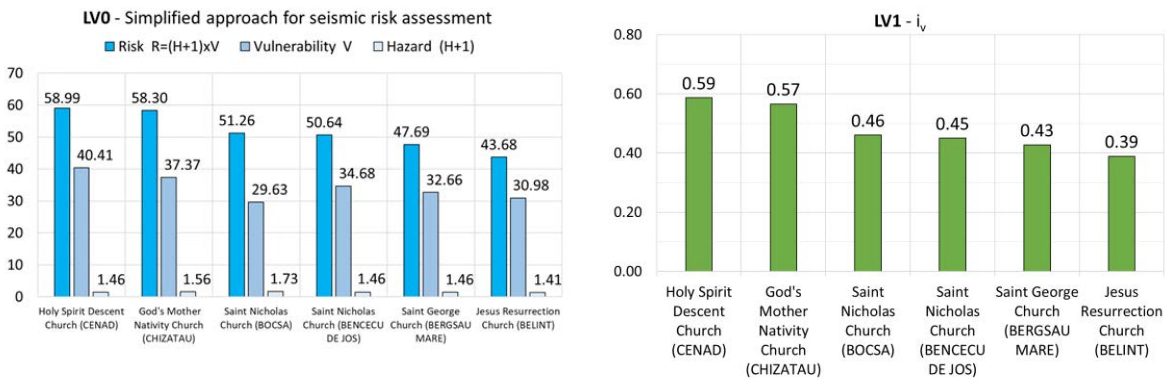
Recently, in continuity with the Italian Directive multilevel approach, in D'Amato M. et al. [10] a LV0 level (prior to LV1) of risk assessment is proposed, which is based on the application of three tools providing a score to hazard (H), exposure (E) and vulnerability (V). A simple version of LV0 method provides only H and V scores to calculate the risk R by means of the following Equations 7, 8 and 9:

$$R = [H+1] \times V \tag{7}$$

$$H = \sum_{k=1}^{7} h_{k,i} \tag{8}$$

$$V = \sum_{k=1}^{13} \rho_{k,i}\, v_{k,i} \tag{9}$$

The hazard H put into account seven environmental sporadic events and continuous processes, while the vulnerability V considers thirteen scored and weighted structural parameters. Figure 9 shows LV0 and LV1 results by comparing the scores obtained for the risk R and the vulnerability index i_v.



a b

Figure 9: (a) LV0 and (b) LV1 results.

3.3 Comparison of the three analysis methods

Figure 10 compares all the vulnerability assessment methods used before, leading to the conclusion that they totally agree on the results. The most vulnerable church is the Cenad one, while the least vulnerable is the Belinț one. Chizătău, Bocsa, Bencecu de Jos and Beregsău Mare churches follow the Cenad one, from the most to the least vulnerable.

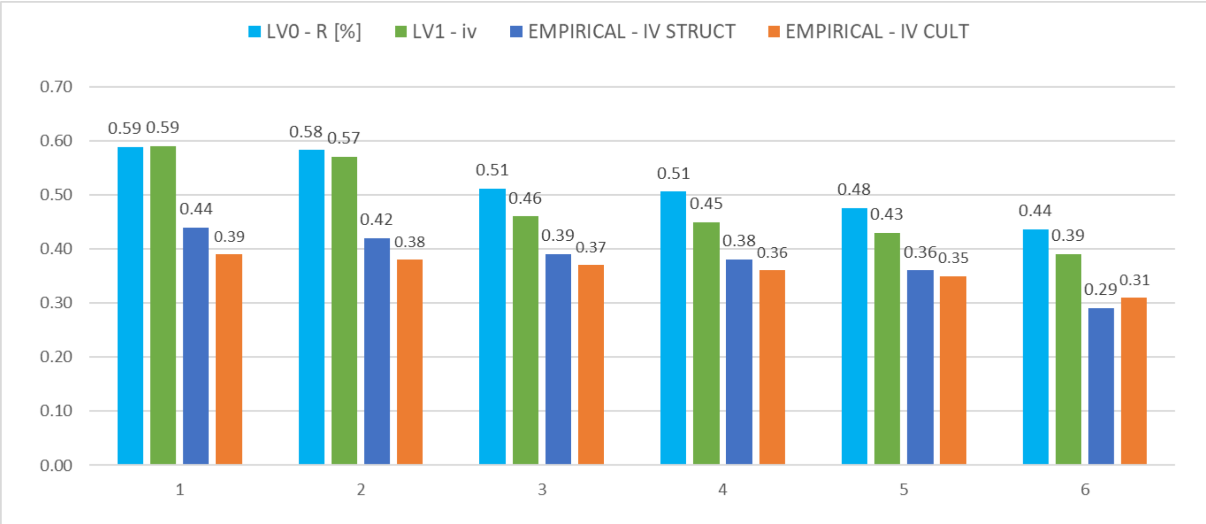


Figure 10: LV0 – *R [%]*, LV1 - i_v, EMPIRICAL $I_{V\ STRUCT}$ and EMPIRICAL $I_{V\ CULT}$ results.

4 SEISMIC ASSESSMENT OF THE BELINT CHURCH

The following paragraphs are focused on the church of Belinț, assumed as case study because of its advantageous position in previous assessments. Its modelling does not take into account actual restoration interventions, rather we want to investigate previous damage recorded on the church. In particular, we focus on vertical cracks located along the nave in correspondence of windows, especially on the transept ones (Figure 11). As authors argue in [3], churches far more than 50 km from Banloc (Belinț is 65 km far), don't report seismic-caused cracks, but only vertical cracks due to differential settlements. This may be confirmed by the foundation consolidation intervention in Belinț church, as shown in Figure 11. Firstly, local failure analyses are carried out, then global behavior is put into account. Vertical seismic components are in any case considered.

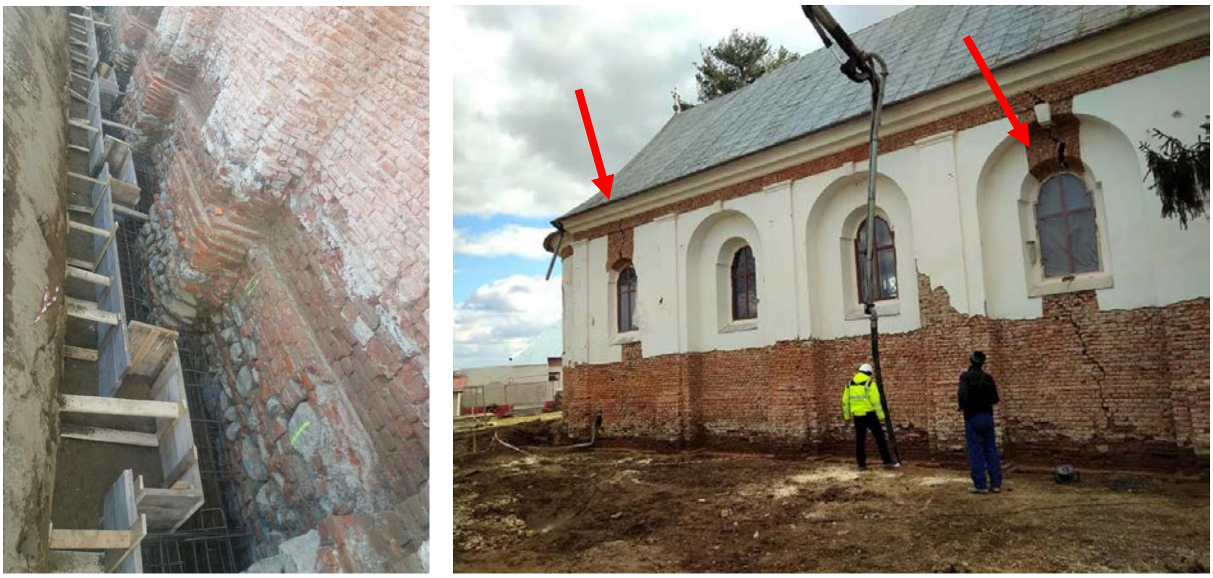


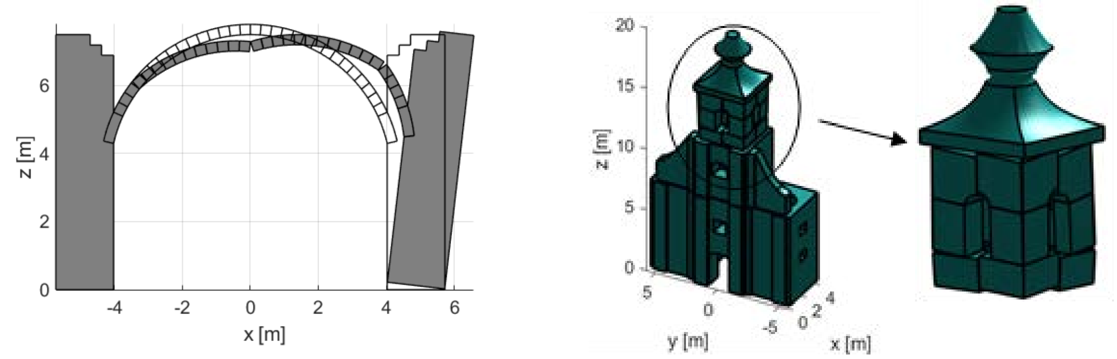
Figure 11: Foundation consolidation and previous cracks.

4.1 Assessment of local failures through kinematic limit analysis

The analyses on the Belinț church have been developed in two steps. In the first one, some local mechanism under horizontal loads have been computed. Secondly, the effects of a vertical seismic component have been investigated by applying some displacement configurations at the bottom. Different macro-blocks have been analyzed in this context, taking into account the damage observed on the church.

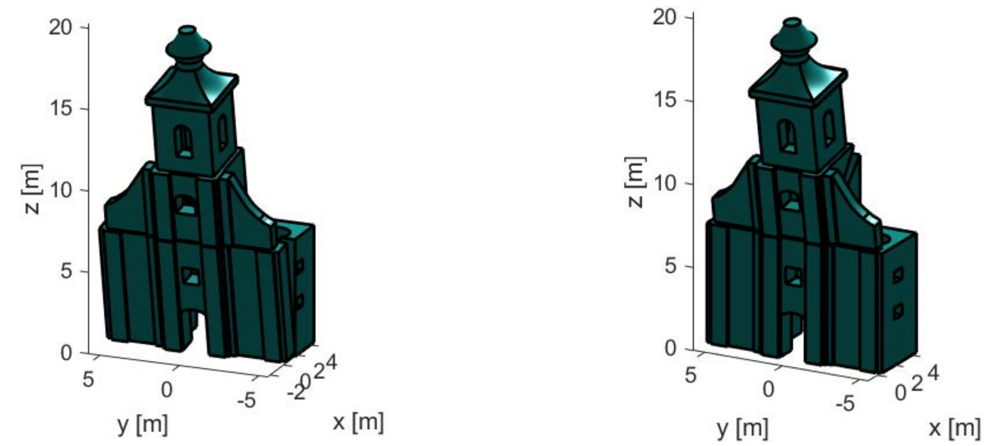
Possible seismic-induced local failures are here analyzed by following a computational upper bound limit analysis method. The attention is here focused on pre-defined macro-blocks representing structural partitions (façade and tower, arches, nave walls and apse) rather than a model of the whole church, as recommended in [3] for limit state analyses in typical Romanian churches. Each macro-block can be furtherly discretized into smaller elements, each one assumed infinitely rigid and resistant. An approach based on NURBS solids objects were recently proposed by some of the authors specifically for 3D limit analysis problems [11]. The interaction between adjacent rigid elements is modeled at the interface-level. Infinite compression and shear strength and null tensile strength can be hypothesized in agreement with the classical no-tension material proposed by Heyman [12], thus allowing to write the kinematic problem in terms of unilateral contact problem [13]. Alternatively, a Mohr-Coulomb frictional behavior can be used to include both crushing and shear failures. The choice depends also on the modeling strategy followed, as it will be seen later. The mechanism is described by a set of kinematic variables, each one constituting one of the 6 degrees of freedom of each element's centroid. A live load, i.e. a specific load configuration that depends on a load multiplier, is defined: typically, a distribution of horizontal load proportional to masses is used for local failures induced by horizontal seismic action. Otherwise, when the aim is to investigate the effect given by a vertical seismic action, a displacement configuration can be applied to the external boundary (typically at the bottom) to simulate the differential soil movements along the vertical direction. The solution, in terms of load multiplier and associated mechanisms, can be obtained by writing a simple linear programming problem. The objective function is given as the difference between the internal dissipation power and the external power associated to permanent loads. When the internal dissipation power is null, as in the no-tension material case, the minimization problem is equivalent to the maximization of the permanent loads-power [13]). For a detailed mathematical dissertation on the method, we refer to [11, 13, 14].

The results associated to the analyses under horizontal load have been reported in Figure 12. In particular, the load bearing capacity of the arches within the nave has been studied. The arch has been considered supported by two abutments whose thickness coincide with the thickness of the longitudinal nave walls, whereas the presence of the barrel vault has been included through a distributed mass at the extrados of the arch. Some local mechanisms have been then investigated for the tower, such as the simple overturning (global and partial) and the collapse of the belfry. A modeling strategy via NURBS solids [11] has been adopted for these mechanisms, avoiding a computationally more demanding representation for such a complex geometry. Given the massive thickness of walls constituting the façade-tower system, the associated safety indexes resulted higher than 1. However, more complex local mechanisms will be investigated in the future, considering for instance the other typical failures for towers [15].



a b

$\alpha_0 = 0.04$; $a_0^* = 0.33$ m/s^2; $a_0^*/a_{0\,\min}^* = 0.45$ $\alpha_0 = 0.44$; $a_0^* = 3.95$ m/s^2; $a_0^*/a_{0\,\min}^* = 3.74$



c d

$\alpha_0 = 0.19$; $a_0^* = 1.38$ m/s^2; $a_0^*/a_{0\,\min}^* = 1.87$ $\alpha_0 = 0.32$; $a_0^* = 2.33$ m/s^2; $a_0^*/a_{0\,\min}^* = 3.16$

Figure 12: Local mechanisms investigated for the Belint church: (a) collapse of the arch in the central nave, (b) belfry failure, (c) global and (d) partial overturning of the tower-façade system.

Figure 13 and Figure 14 report the first results obtained with the analyses under vertical settlements. The aim of these analyses is to derive a first approximated cracks pattern associated to a vertical excitation, with particular reference to the surface waves due to near-field earthquakes [16]. A proper numerical investigation should take into account the dynamic effects of the seismic action, and thus non-linear dynamic analysis approaches should be followed. If the attention remains focused on the limit behavior of the structure, i.e. within the context of limit analysis, the vertical seismic excitation can be represented by a differential vertical displacement and devoted numerical tools can be followed (see again [13]). In particular, the result will be commented in terms of deformed structure and related crack pattern. The arch subjected to vertical settlement (Figure 13) shows three flexural hinges as expected [13]. The nave wall has been studied under two settlement configurations: a bilinear negative displacement with minimum values on the fourth column (Figure 14a), and a piecewise linear displacement (Figure 14b). These two particular configurations are aimed to represent surface waves characterized by different wavelength. This case deserves several and more detailed investigations, thus the result here presented can be considered a starting point for more extended research. A discretization into few quasi-regular rigid blocks has been adopted to maintain low the computational effort. A Mohr-Coulomb frictional behavior has been assigned to each interface (null tensile stress, 4.14 MPa compressive strength, 0.05 MPa in cohesion and 0.4 as tangent of the friction angle). However, to overcome the simplified discretization, the internal dissipation along vertical interfaces has been computed according the homogenization theory presented by de Buhan & de Felice [17] (whereas for the inclined interfaces on arches the initial resistance parameters has been maintained). Finally, a non-associative behavior has been assigned in shear to exclude dilatancy effects [14]. The results show the presence of vertical cracks, with some of them due to the shear effect. It is interesting to mention that some vertical cracks have been observed on the nave walls close to the transept. Further investigation on these aspects will be presented in upcoming works.

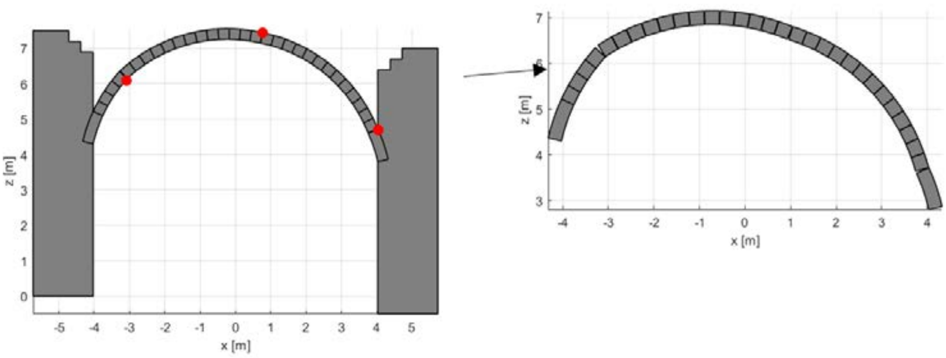
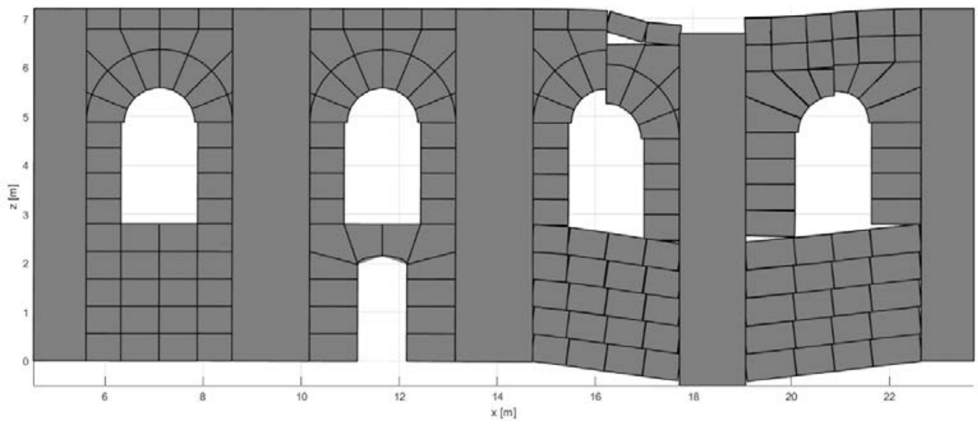
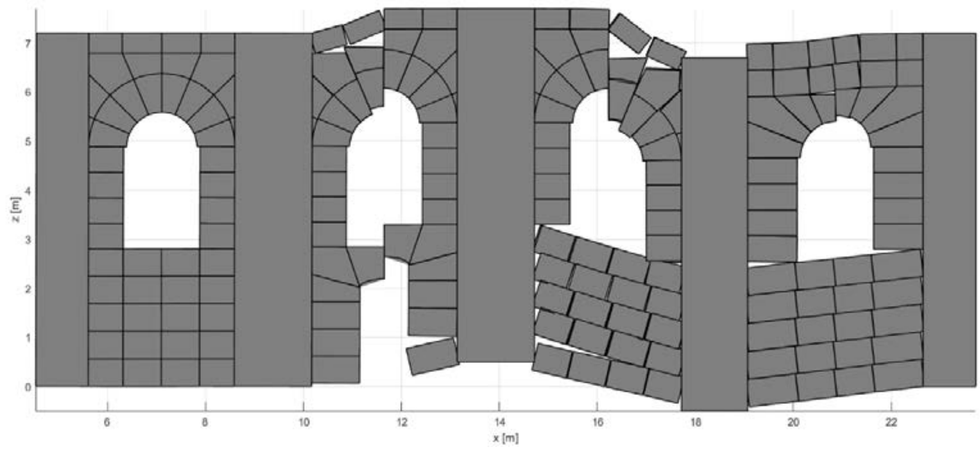


Figure 13: Analysis of the arch subjected to differential vertical settlements: deformed structure scaled by 50 (vertical displacement applied equal to 1 cm) and detail of the masonry arch scaled by 150.



a



b

Figure 14: Analysis of the nave wall under vertical settlements, deformed structures scaled by 50 (maximum displacement applied equal to 1 cm): (a) bi-linear and (b) piecewise linear settlement configurations.

4.2 Global seismic behavior assessment through linear analysis

A global finite element model (FEM) is now considered. Mechanic parameters are calculated following Italian regulation [18] and referring to Romanian literature values. Table 3 reports the design parameters calculated on a clay brick masonry.

f_c (N/mm^2)	compressive strength	1.83
t_0 (N/mm^2)	shear strength in the absence of normal stress (irregular texture)	0.05
f_{v0} (N/mm^2)	shear strength in the absence of normal stress (regular texture)	0.07
f_t (N/mm^2)	tensile strength (as 10% f_c)	0.18
E (N/mm^2)	modulus of normal elasticity	953
G (N/mm^2)	tangential modulus of elasticity	318
W (kN/m^3)	specific weight	16

Table 3: Design mechanic parameter for a clay brick masonry.

Elastic and design spectra for both horizontal and vertical components are also defined by following Romanian regulation [5]. The maximum expected acceleration for vertical components is calculated by multiplying the a_g = 0.15 g for 0.7. Figure 15 shows response spectra obtained:

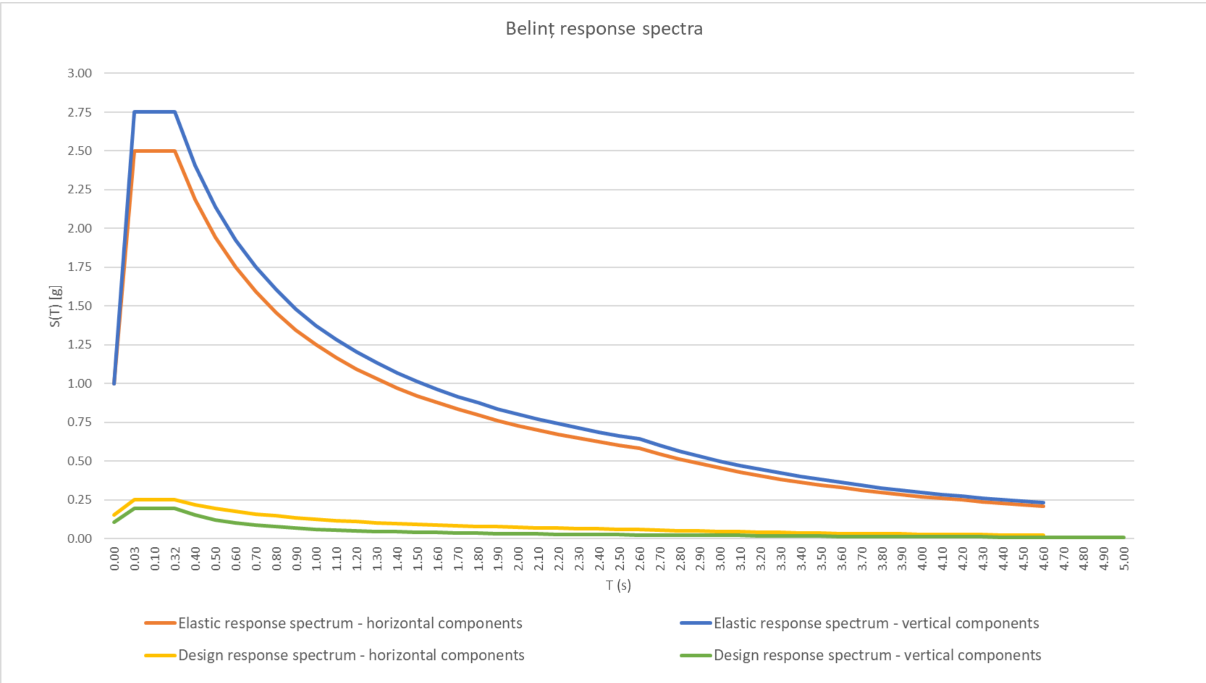
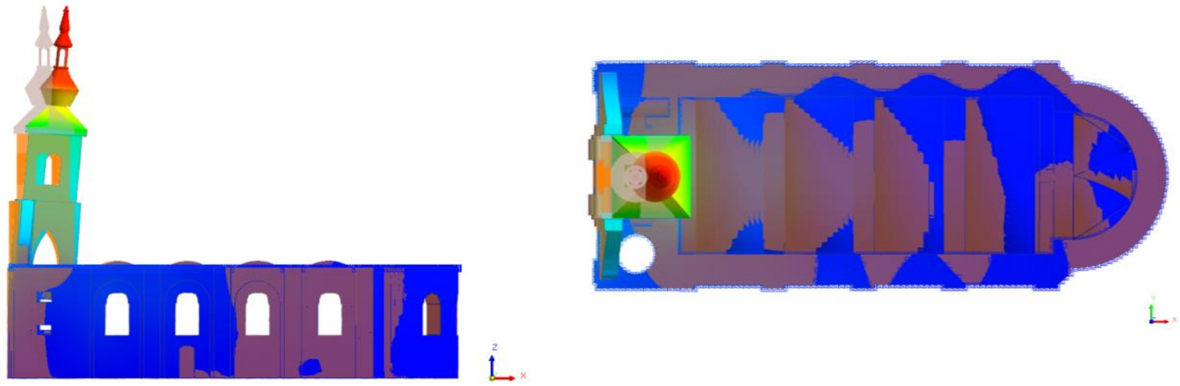


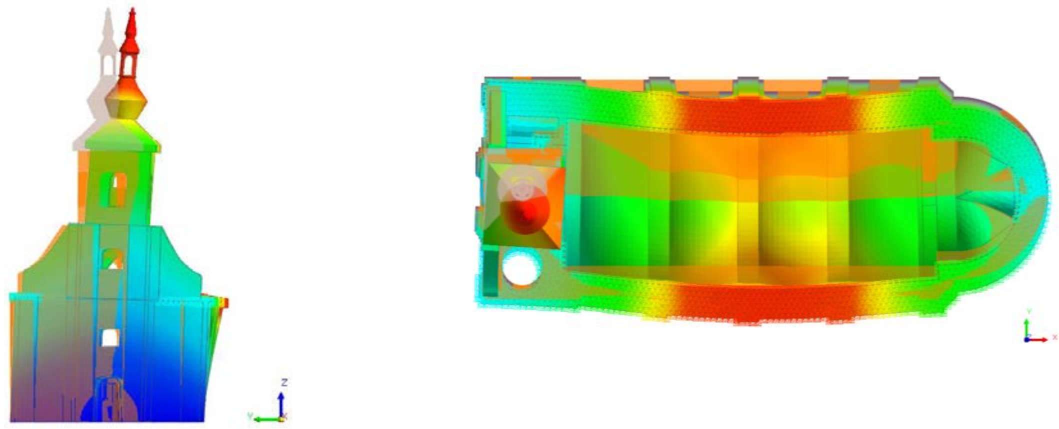
Figure 15: Belinț response spectra.

The church is modelled by using three dimensional meshes for masonry panels and wooden mezzanine and pinnacle, while the wooden roof is put into account as simple loads combined in the seismic combination and distributed on the upper surface of the perimeter wall panels. Then modal linear analyses are carried out on the Belinț church model. As reported in Figure 16a, the first structure vibrational mode interests only the bell tower translating along X axis (T=0.25 s, Mass ratio=14.14%). The second one also involves the nave and interests the translation along Y axis (T=0.20 s, Mass ratio=56.46% - Figure 16b). Finally, the third vibrational mode involves the structure torsion around Z axis (T=0.16 s, Mass ratio=26.43% - Figure 16c).

a – MODE 1 (T=0.25 s, Mass ratio=14.14%)



b – MODE 2 (T=0.20 s, Mass ratio=56.46%)



c – MODE 3 (T=0.16 s, Mass ratio=26.43%)

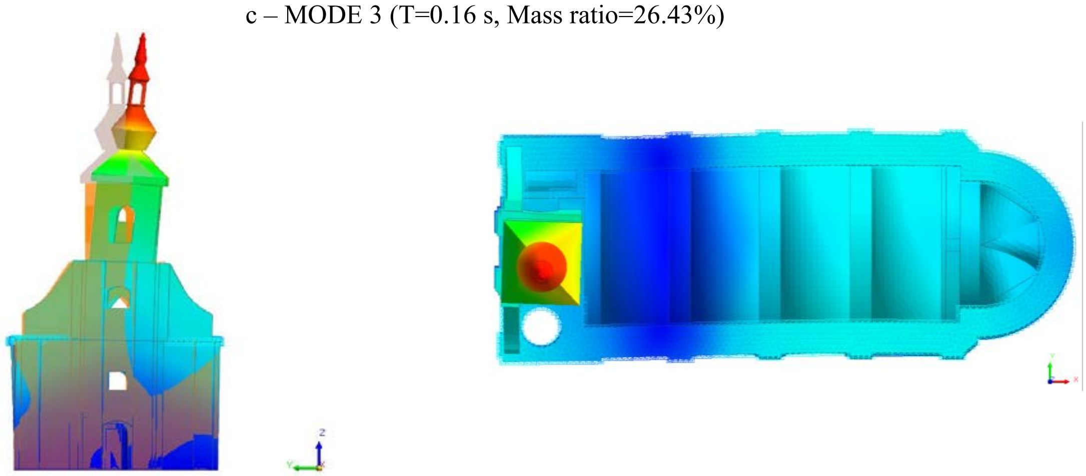
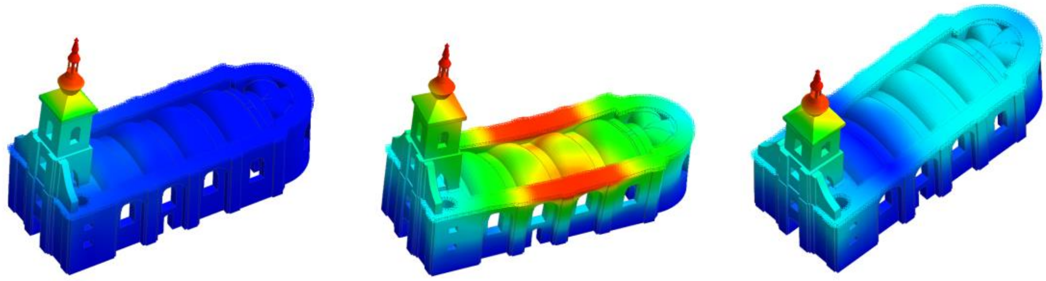


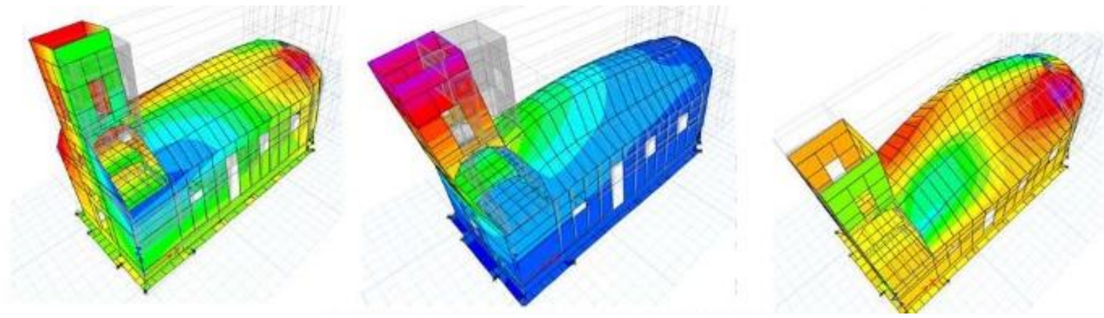
Figure 16 : (a) MODE 1, (b) MODE 2, (c) MODE 3. – Results obtained from MIDAS FEA NX [21]

4.3 Comparison between Banat church models

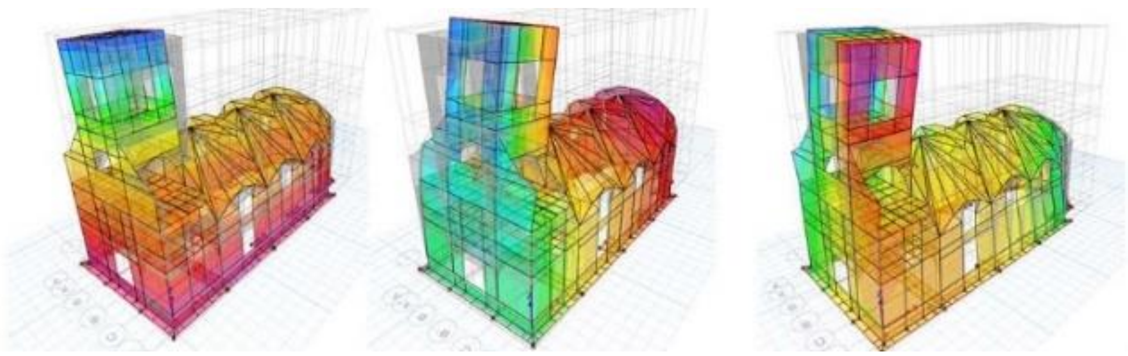
Figure 17 shows the comparison among Belinț church and the churches of Chizătău and Beregsău Mare [1] modal analyses. Due to differences in the plan configuration and vaults materials (see Table 1), Mode 1 and Mode 2 invert their translational direction: the two churches analyzed in [1] translate along Y in the first mode and along X in the second one, while Belinț presents an opposite situation. Mode 3 is a torsional vibration mode for all churches.



(a) Mode 1: Ox transversal translation T=0.254s Mode 2: Oy longitudinal translation T=0.20s Mode 1: Torsion T=0.16s



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



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

Figure 17: Principal vibrational modes for the churches of (a) Belinț, (b) Chizătău and (c) Beregsău Mare.

5 CONCLUSIONS AND FUTURE GOALS

In this paper six churches of the Banat Romanian region are analyzed. In particular, three territorial scale assessments methods are applied and compared, showing surprisingly agreement among them. Then more detailed preliminary investigations are carried out for the Belinț church, regarding the implementation of local failure assessments through kinematic limit analysis and of global modal linear analyses. Regarding modal linear analyses, one may conclude that there is a correlation between Romanian and Italian approach. In the future the study will continue in implementing non-linear analyses on the churches investigated to predict a possible damage status.

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