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STRUCTURAL REHABILITATION OF OLD MASONRY CHURCHES. A COST-ORIENTED CASE STUDY

BY

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Abstract. The rehabilitation stage of old masonry orthodox church buildings represents an important challenge from both architectural point of view, by conserving, repairing or replacing the interior and exterior ecclesiastical elements, and from structural point of view by designing an appropriate strengthening solution aiming to minimize the impact upon the building. The most common strengthening solutions, triggering certain seismic protection levels, imply high costs. Taking into account the large number of churches for which rehabilitation is necessary and the relatively low financial availability, the cost optimization of the rehabilitation process is being emphasized. This paper presents a case study consisting of a decayed old church built in the XIXth century, for which, different levels of structural rehabilitation are analysed, leading to a gradual increase of the seismic protection level. Also, the execution costs are appraised for each seismic protection level. Based on the provisions of the active norms, the analysis and the evaluation of the strengthening solutions are carried out analytically, with a a software that uses the finite element method (FEM). The results of the study are outlined and a cost sensitivity analysis of the different rehabilitation levels is presented.

Key words: rehabilitation of old masonry churches; FEM analysis, cost optimization.

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1. Introduction

Churches have always represented a distinct part of the built heritage due to their social and cultural importance. When compared to nowadays regular constructions, for which the expected life is fifty or one hundred years, old churches must be carefully and more wisely treated as their life expectance is much higher that the latter. The damage and decay extent of old churches is, in most cases, proportional to their age, as deterioration is an inherent process of service-life or, even more pronounced, of the lack of use. The structural damages are direct consequences of the effects of aggressive environmental actions (earthquakes, soil settlements, variations of the underground water level, climate) (Lourcenco, 2013), of the miss-conception and execution errors (empirical approaches) and due to the lack of maintenance during their in-service life.

The rehabilitation of damaged old orthodox churches gathers complex processes, from both architectural and structural point of view. The interventions must be conceived in such way that the subject of the church is not altered while the structural safety level is increased up to a well-established point. ICOMOS Organization (ICOMOS, 2003) have established a set of recommendations for the analysis, conservation and structural restoration of architectural heritage which state that, in general, the process of determining the rehabilitation procedures should include the following preceding steps: (a) acquisition of data: information and investigation, (b) historical, structural and architectural investigation, (c) survey of the structure, (d) field research and laboratory testing and (e) monitoring. Next, the structural behavior is analyzed, aiming to identify the loads acting upon the building, the characteristics of the materials and the static scheme. The third step consists in diagnosis and safety evaluation. Diagnosis is based on qualitative approaches (historical and archaeological research combined with direct observations of the damage extent) and quantitative approaches (direct identification of the material properties, structural analysis and structural health monitoring). The safety evaluation is the point in which the decision for intervention is determined assuming a certain safety level (appropriately justified), which may be lower than the one applied for the design of new buildings. The last step is the one in which the strengthening solutions and technologies are selected, the cost of the interventions are evaluated and the execution documents are drafted. The whole process is graphically presented in Fig. 1.

There is a strong correlation between the applied safety level and the financial impact (costs) of the interventions. Thus, the safety level must be carefully assessed, trying to obtain a balance point between structural safety and feasibility of the project.

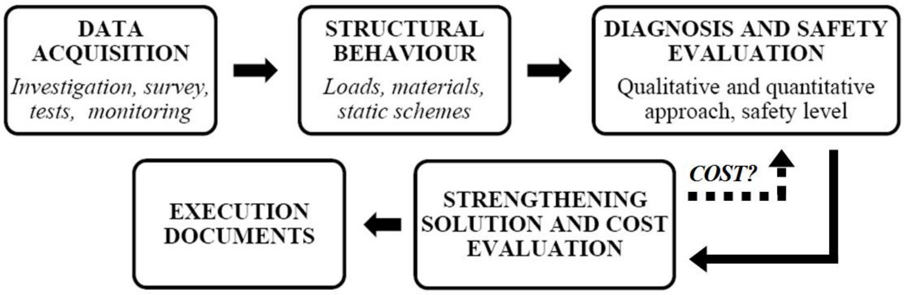


Fig. 1 – Rehabilitation methodology (adapted from Lourcenco, 2013; ICOMOS, 2003).

2. Description of "St. Dimitrie" Church

2.1. Architectural and Structural Description

"St. Dimitrie" Church is located in Vicoleni village, Botoşani county, and it was erected in 1942. The church (Fig. 2) has dimensions about 18 × 8 m^2 in plan, with a maximum height of the tower about 21 m. The nave has a simple, rectangular shape, with buttresses at the corners of the west façade and

Fig. 2 – "St. Dimitrie" Church – Photo and CAD Model.

at the mid-span of the northern and southern façades, under the square base of the tower. The walls of the church, 60 cm thick, are made of burned clay bricks with limestone and sand mortar. The foundations, made of unreinforced concrete, are continuous under the structural walls. The interior of the church follows the traditional layout, being composed of porch, narthex, nave and altar (Fig. 3). The flat cap over the narthex has a wood structures and bears on the longitudinal and transversal arches. The tower has an octagonal outer layout and a circular one at the interior. The square base of the tower bears on two pairs of arches. The apse of the altar has a polygonal layout at the exterior and a semi-circular one at the interior. Similar to the cap above the narthex, the semi-calotte over the altar has also a wooden structure.

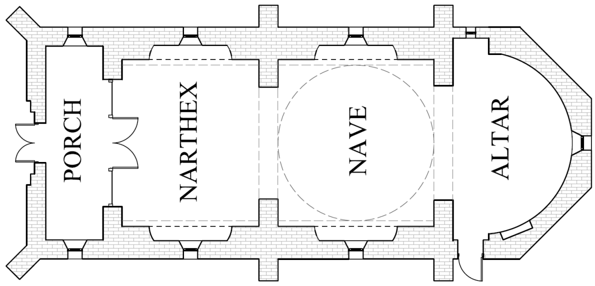


Fig. 3 – Plan of the church.

2.2. Qualitative Diagnosis

The first phase of the qualitative diagnosis of the church structure consisted in direct observation of the damage extent by investigations carried out on site. After a careful evaluation of the cracks pattern and location it has been conclude that, due to consecutive earthquake actions and because of the soil settlements, the structure of the church is divided in distinct quasi-rigid blocks, confirming the damage mechanism proposed by Prof. Al. Cişmigiu. The mechanism, having a three-dimensional distribution is characterized by two main processes (Cişmigiu A., 1996):

a) a longitudinal fracture that, in almost all cases extends from the porch to the altar, dividing the structure in two, relatively symmetrical, blocks;

b) multiple transversal fractures in the vulnerable areas of the porch, nave and narthex (wall sections weakened by embrasures, load concentrations and irregular load distributions, local soil settlements).

The most important damages that were identified during the site investigations were the two pairs of cracks (opened up to 15 mm) in the northern and southern walls, starting at the square base of the tower and, we assume, ending at the foot of the foundations (Fig. 4). The cracks have a vertical direction, on the exterior sides of the buttresses, and were caused due to great variation of the loading state (high concentrations of the vertical loads under the tower) combined with the low tensile strength of the masonry and low bearing capacity of the foundation soil. The cracks are also visible from the inside, their path following the exterior boundary of the transversal arches that carry the tower. Also, other vertical cracks and fissures were identified above and under the embrasures of the windows or doors and in the key stones of the interior arches. After evaluating the damage mechanism, a total of 10 distinct quasi-rigid blocks have resulted (Fig. 4). The number of the separation blocks illustrates the vulnerability of the church structure.

Fig. 4 – Cracks / fissures path and location
(Soveja & Budescu, 2014).

3. Quantitative Diagnosis

The quantitative diagnosis consisted of structural analysis aiming to identify the safety level. Based on the mechanical properties of the constituent materials and following the provisions of the active Romanian norms, the elastic and strength properties of the masonry were calculated, being presented in Table 1.

Table 1
Elastic and Strength Properties of the Materials

Characteristic	Value
Characteristic compressive strength of the bricks, f_b, [N/mm^2]	5
Mean compressive strength of the mortar, f_m , [N/mm^2]	0.4
Safety coefficient, *CF*	1.35 (according to P100-3/2008 §4.1)
Partial safety factor, γ_M	2.75 (according to P100-3/2008, Annex D §3.4.1.3.1.2)
Design compressive strength of the masonry, f_d , [N/mm^2]	1.52 (according to P100-3/2008, Annex D §3.4.1.3.1.1)
Design tensile strength of the masonry, f_{td} , [N/mm^2]	0.016 (according to P100-3/2008, Annex D §3.4.1.3.1.1)
Longitudinal elastic modulus of the masonry, E_z , [N/mm^2]	1,172 according to CR6-2012)

The next step consisted in evaluating the loads acting on the structure. The dead loads and the live loads were calculated according to the specific technical norms, being briefly summarized in Table 2. The seismic load was evaluated according to P100-1/2006 and P100-3/2008 norms, with the following particularities: the design safety factor for the structure, γ_I, was taken 1 as the building is of regular importance, the design ground acceleration, a_g, is 0.12 g,

Table 2
Dead Loads and Live Loads

Load	Value
Dead Loads (according to SR EN 1991-1-1-2004)	
Roof cover, [kN/m^2]	0.05
Roof structure, [kN/m^2]	0.75
Masonry elements (specific weight) , [kN/m^3] *the exact weight of the masonry elements is automatically calculated by the FEM software, depending on their geometry	18.00
Concrete elements (specific weigh) , [kN/m^3] *the exact weight of the concrete elements is automatically calculated by the FEM software, depending on their geometry	25.00
Live load (according to SR EN 1991-1-1-2004)	
Inner spaces, [kN/m^2]	3.00
Outer spaces (on the roof) , [kN/m^2]	0.75
Environmental loads (according to CR 1-1-3/2012)	
Snow load, [kN/m^2]	2.20

the corner period, T_c , is 0.7 s and the behavior factor, *q*, was taken 1.5. The seismic action has been simulated in the FEM software by a response spectrum analysis using the design accelerations spectra (Fig. 5) provided by the national code P100-1/2006. The loads combinations were defined according to CR-0/2012 norm in fundamental and special loading cases for both ultimate limit state (ULS) and serviceability limit state (SLS).

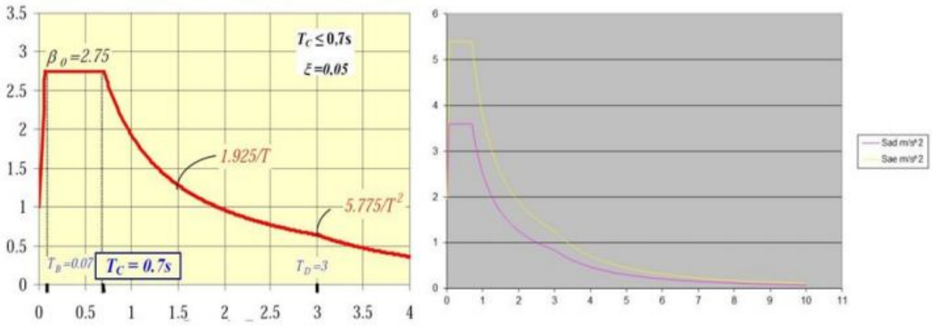


Fig. 5 – Elastic and design accelerations spectra.

The structure of the church has been modeled in Etabs V9.7.4. FEM software by applying a 3-D macro modeling strategy where the masonry panels are taken as homogeneous anisotropic continua elements (Lourenco, 1996). This strategy is a simplifying one, by neglecting the distinctive characteristics of the constituent materials (bricks, mortar and interfaces between them). The dynamic behavior of the structure and the interaction between elements have

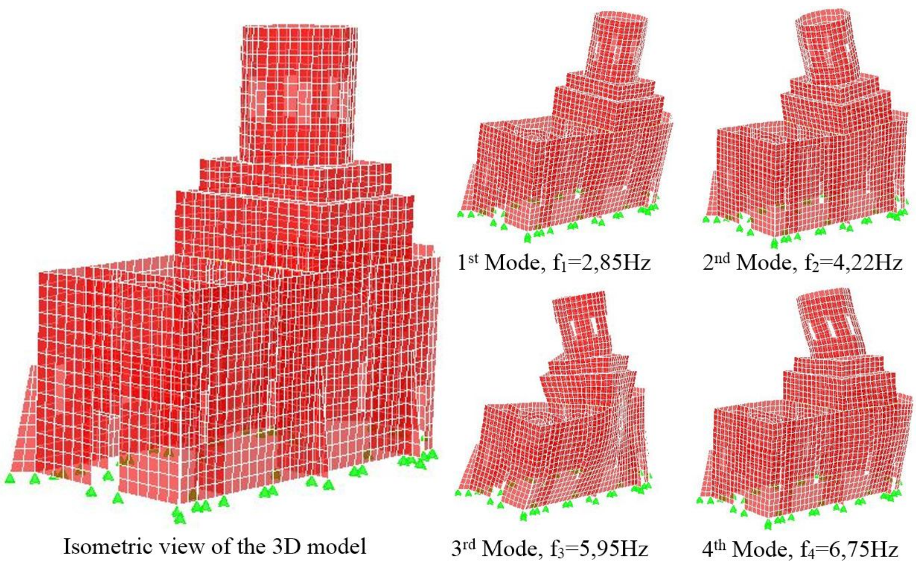
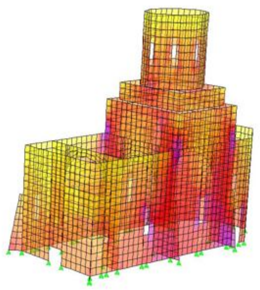
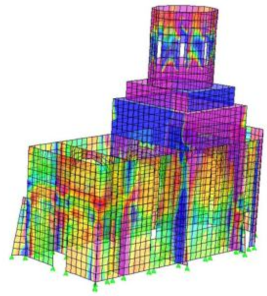


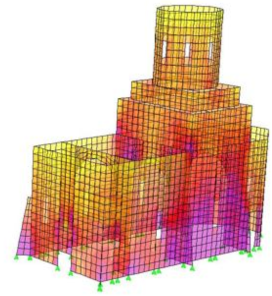
Fig. 6 – Isometric view of the 3d model and the first four modes of vibration.

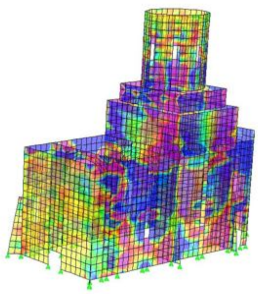
been obtained by running linear elastic and dynamic analysis of the 3-D model in the specific loads combinations mentioned before. The modal analysis comprised twenty modes of vibration such that the mass participation factor was greater than 90%. Fig. 6 presents the 3-D model of the church and the first four modes of vibration with their corresponding frequencies. It can be seen that the natural frequency of the structure is 2.89 Hz.

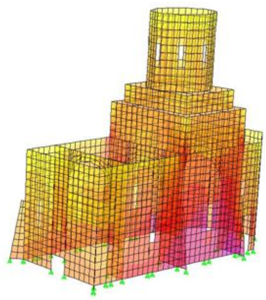
The structural walls have been modeled as shell elements and the loads in each of the latter are calculated by integrating the stresses obtained by means of linear elastic analysis. Fig. 7 presents the stress maps of the masonry walls under different loads combinations.


σ_z stresses in earthquake combination – X direction


τ_{xy} stresses in earthquake combination – X direction


σ_z stresses in earthquake combination – Y direction


τ_{xy} stresses in earthquake combination – Y direction


σ_z stresses in fundamental combination

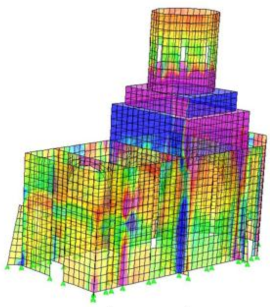

τ_{xy} stresses in fundamental combination

Fig. 7 – Stress maps of the walls under different load combinations.

The load bearing capacities of the walls have been calculated using a simplified approach, provided by the Romanian norm, P100-3/2008. This method enables the evaluation of the capacities under three distinct failure mechanism, namely: rocking (flexural failure), sliding shear and diagonal shear cracking. The analytical equations are presented in Table 3.

Table 3
Load Bearing Capacities of a Masonry Wall

Failure mechanism	Equation
Rocking	$V_{f1} = \frac{N_d}{c_p \lambda_p} v_d \left(1\text{-}1.15 v_d\right)$
Sliding shear	$V_{f21} = f_{vd} D' t$
Diagonal shear cracking	$V_{f22} = \frac{t l_w f_{td}}{b} \sqrt{1 + \frac{\sigma_0}{f_{td}}}$

Where: N_d – axial force of the wall, λ_p – shape coefficient of the masonry wall, c_p – boundary coefficient of the masonry wall, v_d – normalized axial force, f_{vd} – design bed-joint sliding strength, D – length of the compressed area of the wall (imposing linear distribution of the compressive stresses), t – the thickness of the wall, l_w – length of the wall, f_{td} – design tensile strength of the masonry, b – shape coefficient with values between 1.00 and 1.50, σ_0 – mean compressive stress corresponding to the design axial force (N_d).

For each wall of the structure, on both longitudinal and transversal directions, all three load bearing capacities are evaluated and the one with the minimum value is divided to the effective shear force, which has been obtained from the linear static analysis. This ratio determines the safety factor of the wall. The seismic safety factor, denoted R_3, is obtained by applying eqs. (1),…,(3), on each of the two principal directions(Table 4). The global seismic factor of the structure is obtained by averaging the two values obtained on longitudinal and transversal directions. The value of the global seismic factor determines the risk class of the structure, which, according to Table 5 is the 1st one.

$$R_3 = \frac{\sum_{jd} V_{fd} + \sum_{kf} V_{ff}}{F_b}, \qquad (1)$$

where: $\sum_{jd} V_{fd}$ is the sum of the bearing capacities of the walls with ductile fracture; $\sum_{kf} V_{ff}$ is the sum of the bearing capacities of the walls with fragile fracture; F_b is the base shear force of the structure.

The bearing capacities of the walls are taken with the following values:

$$V_{fd,i}(V_{ff,i}) = 0 \text{ for } R_{3,i} < 0.5, \tag{2}$$

$$V_{fd,i}(V_{ff}) < 1.5F_{b,i}. \tag{3}$$

Table 4
Seismic Safety Factors (R_3)

Direction	Value	Global
Longitudinal	0.246	**0.183**
Transversal	0.121	

Table 5
Seismic Risk Classes Associated to R_3 (according to P100-3/2008)

Seismic risk class			
I	II	III	IV
R_3, [%]			
<35	35…65	66..95	96…100

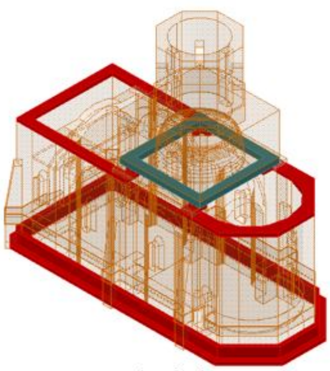
4. Structural Rehabilitation Solutions

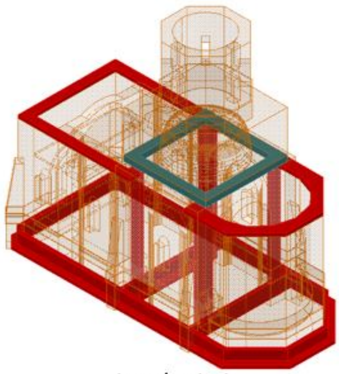
Taking into account that the seismic risk class of the church is the 1st one, for which structural interventions are mandatory, four rehabilitation solution are proposed. The impact and the complexity of the rehabilitation solutions are increased consecutively, aiming to obtain higher values of the seismic safety factors from one solution to another. The first solution consists in strengthening the existing foundations by an adjacent continuous R.C. beam and of a R.C. girdle executed on the top side of the masonry walls. The second and the third solutions supplements the previous one with vertical R.C. elements executed at the inner side of the masonry walls, that connect the foundation beam to the girdle executed on top of the walls. Also, the high load concentrations in the masonry walls near the transversal arches that carry the tower are eliminated by executing a R.C. frame. The fourth solution introduces R.C. strengthening elements to the tower, consisting in two girdles, at the bottom and at the top side of the masonry, connected with vertical elements. The works implied by all solutions are briefly presented in Table 6 and Fig. 8.

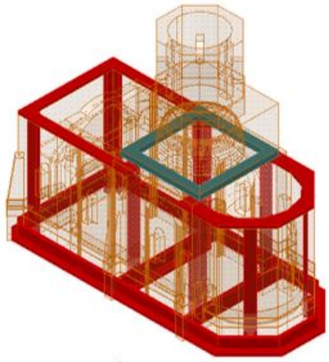
For each of the four rehabilitation systems, the seismic safety factor has been calculated according to the provisions of P100-3/2008 norm and those of other active norms (P100-1/2006 and SR EN 1992-1-1-2004). The safety factors and the corresponding seismic risk classes are presented in Table 7.

Table 6
Description of The Rehabilitation Solutions

Rehabilitation process / Solution	1st	2nd	3rd	4th
R.C. continuous foundation beam, adjacent to the existing foundations	•	•	•	•
R.C. girdle at the top side of the exterior masonry walls	•	•	•	•
R.C. transversal beams under the transversal arches that carry the tower		•	•	•
R.C. vertical elements (lamellar cross-section) at the inner corners of the transversal arches under the tower		•	•	•
Vertical R.C. elements at the inner side connecting the exterior foundation beam with the top girdle, in the porch and altar			•	•
Vertical R.C. elements that strengthen the square base of the tower				•
Top and bottom R.C. girdles at the octagonal masonry tower				•
Vertical R.C. elements connecting the top and bottom girdles of the tower				•
Grout injecting the cracks and the volume of the masonry (with limestone and cement based mortar)	•	•	•	•


a. 1st solution


b. 2nd solution


c. 3rd solution

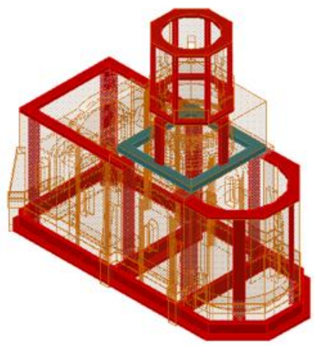

d. 4th solution

Fig. 8 – Isometric views of the structural rehabilitation solutions.

Table 7
Seismic Safety Factors (R_3) and Corresponding Seismic Risk Class

Rehabilitation solution	Transversal	Longitudinal	Global	Associated risk class
Unstrengthen	0.121	0.246	**0.183**	**Ist**
1st solution	0.187	0.319	**0.253**	**Ist**
2nd solution	0.577	0.743	**0.660**	**IIIrd**
3rd solution	0.895	0.969	**0.932**	**IIIrd**
4th solution	1.068	1.155	**1.111**	**IVth**

4. Structural rehabilitation solutions

One of the aims of this paper is to evaluate the economic impact of the rehabilitation process, focusing on the execution phase. Thus, for each of the four strengthening solutions that were presented in the previous chapter, the associated execution costs are evaluated. The analysis only refers to those construction works which are specific to the structural rehabilitation process, such as: earth works (excavating, filling and compaction), dismantle works and slits in the masonry walls, formworks, reinforcements, concrete works, scaffolding and timbering, grout injecting the cracks and the volume of the masonry walls.

For each item of work, the resource, labor and equipment usage rates have been taken according the Romanian rehabilitation collection of work items (RpC) (Lupăşteanu, 1997). The associated costs data for all resources are according to those used in present times, in Romania. The evaluation refers only to direct cost, without including the contract margin, as they may vary from one contractor to another.

Fig. 9 presents the variation of costs for each of the seven work items that are analyzed in this study. It can be seen that the costs of all work items increase with the extent of the rehabilitation processes (from the 1st to the 4th solution), the only exception being the earth works, for which, there are no differences between the 3rd and the 4th solution. The highest growth rate can be observed for the dismantle and slits work, for which the costs of the 4th solution are almost 6 times higher than those of the first 1st one. The lowest growth rate is for the earth works, for which the costs of the 4th solution is only 1.5 times higher than those of the 1st one. Another important parameter is the impact of each work item with respect to the global cost of the corresponding solution.

Fig. 10 presents for each of the four rehabilitation cases the divided cost and the associated percentages for all seven work items that are analyzed. As expected, the reinforcement works imply the highest costs in all four cases, followed by the concrete works and by the scaffoldings and timberings which come second only in the 4th solution where the tower is strengthened.

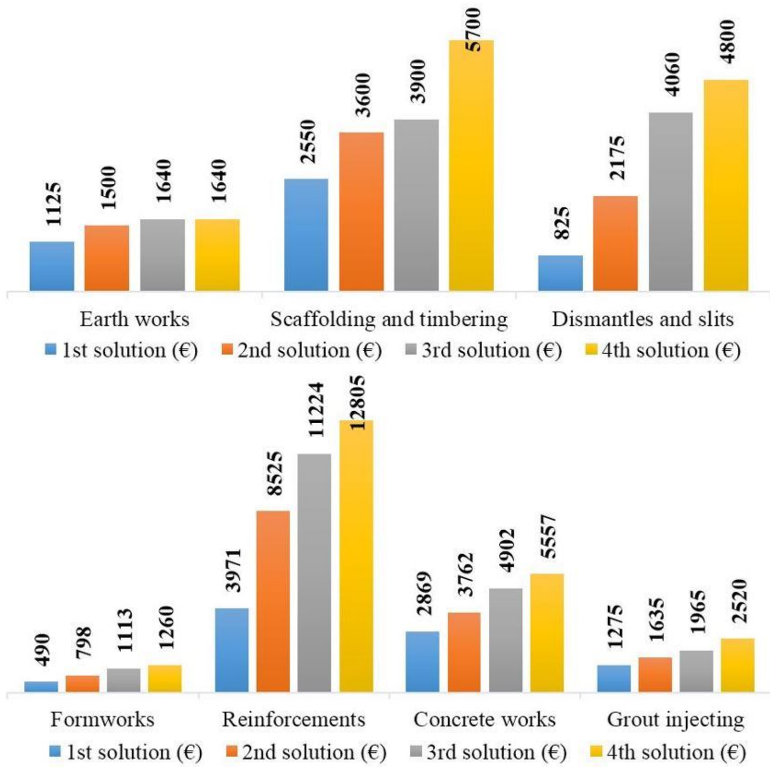


Fig. 9 – Variation of costs for each work item.

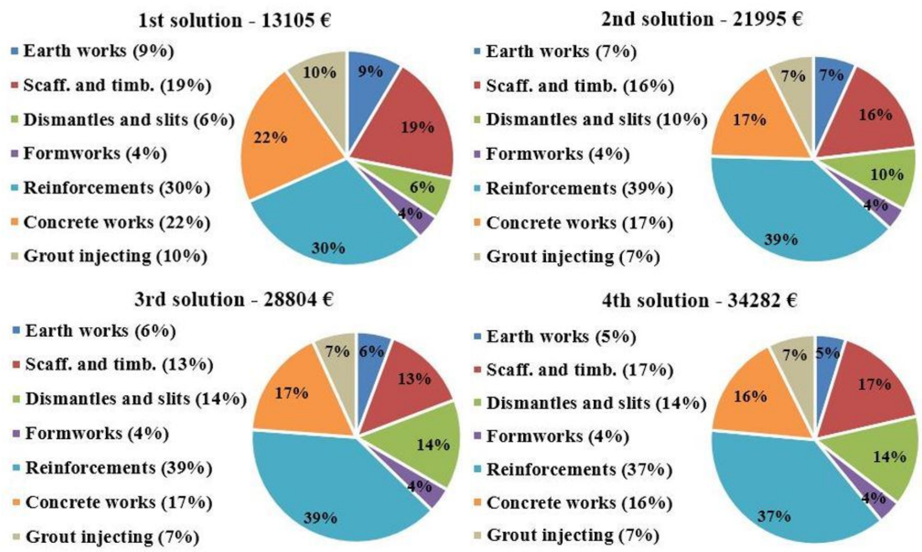


Fig. 10 – Divided costs and their corresponding percentages.

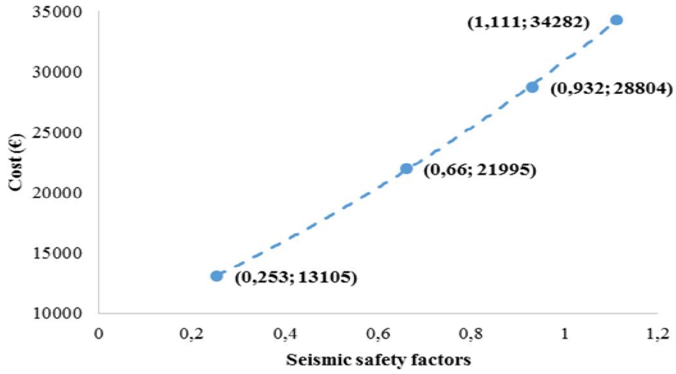


Fig. 11 – Seismic safety factors *vs*. costs.

The relation between the seismic safety factors and the costs of the corresponding rehabilitation solutions is presented in Fig. 11. The first solution triggers the smallest seismic safety factor (R_3^{I} = 0.253), being below the minimum value, which, according to P100-3/2008 norm is 0.65. Still, the cost evaluation for this solution is of great importance, particularly when compared to the cost of the second one, for which the safety factor is greater than the minimum one (R_3^{II} = 0.66). When comparing the latter, the costs of the second one are 68% higher than those of the first one while the seismic safety factor is 2.60 times higher, respectively. For the third solution, R_3^{III} is 0.932 and when compared to the previous seismic safety factor it shows an increase of 41%. The costs of the third solution are 31% higher than those of the second one. The fourth solution is the only one which has a seismic safety factor greater than one (R_3^{IV} = 1.111) being 1.19 times higher than the third one and having the same rate of growth when costs are compared.

5. Conclusions

When comparing the ratios of consecutive seismic safety factors, it can be seen that all of them are greater than one but the rate of growth decreases from one rehabilitation solution to another (2.6 for 1$^{\mathrm{st}}$ to 2$^{\mathrm{nd}}$, 1.41 for 2$^{\mathrm{nd}}$ to 3$^{\mathrm{rd}}$ and 1.19 for 3$^{\mathrm{rd}}$ to 4$^{\mathrm{th}}$). On the other hand, in terms of costs, the most advantageous solution is the second one as it produces the highest increase of safety factor (2.6 times) with a much smaller increase of costs (only 68%). When comparing the 2$^{\mathrm{nd}}$ solution to the 4$^{\mathrm{th}}$ one, the safety factors is 1.68 times higher while the costs are increased with 56 %. It is clear that as the extent of works increases, the rate of growth in terms of costs is higher than that of the seismic safety factor, changing the upword sloping yield as seen in Fig. 11.

The main assertion which is outlined by this paper is that the costs of the rehabilitation process of old church buildings are strongly connected to the designed seismic safety level of the structure. The extend of the strengthening works should be first correlated to the provision of the active norms, in terms of achieving the minimum imposed safety level, and, beyond this limit, the largness of the rehabilitation solution should be associated with the available financial resources, by analysing the feasibility of the project, and aiming to obtain the optimum relation between costs and benefits.

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REABILITAREA STRUCTURALĂ A BISERICILOR VECHI CU STRUCTURĂ DIN ZIDĂRIE. STUDIU DE CAZ PRIVIND ANALIZA COSTURILOR

(Rezumat)

Reabilitarea bisericilor vechi, de rit ortodox, cu structură din cărămidă reprezintă o provocare reală, atât din punct de vedere architectural, prin conservarea, refacerea sau înlocuirea elementelor de parament sau interioare, cât şi din punct de vedere structural, prin elaborarea unui sistem optim de consolidare, care să minimalizeze impactul asupra clădirii. Soluțiile de reabilitare structurală, transpuse şi în diferite niveluri de siguranță seismică, implică, de cele mai multe ori, costuri ridicate. În contextul fondului construit numeros şi a disponibilității financiare reduse, se reliefează necesitatea optimizării costurilor aferente intervențiilor structurale. În acest sens, lucrarea de față prezintă un studiu de caz, în cadrul căruia, pentru clădirea degradată a unei biserici construite în secolul al XIX-lea se propun soluții de consolidare de amploare diferită care să conducă la îmbunătățirea graduală a nivelului de asigurare la seism. De asemenea, pentru fiecare nivel de asigurare la seism, se apreciază costurile aferente lucrărilor de reabilitare. Evaluarea şi stabilirea fiecărei soluții de consolidare se realizează analitic, în concordanță cu reglementările tehnice aflate în vigoare, cu ajutorul unui program de calcul ce foloseşte metoda analizei cu element finit. Rezultatele obşinute sunt sintetizate, iar pe baza acestora se comentează unele aspecte legate de optimizarea nivelului de intervenții structurale prin prisma costurilor aferente.