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Structural Assessment and Strengthening of a Historic Masonry Orthodox Church

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Abstract: This study provides insight into the structural assessment, diagnosis, and strengthening of the medieval church of Tazlău Monastery in Piatra Neamț, Romania. The first part of the paper briefly presents the wider context of strengthening and preserving heritage churches and monastic buildings and describes the architectural setting and the structural features of the traditional Romanian Orthodox churches. The second part of the paper is a case study related to the rehabilitation of a medieval heritage church, which is the paramount building of a larger monastic complex. Erected in 1496, the church of the Nativity of the Blessed Virgin Mary closely follows the medieval traditional Orthodox patterns from both architectural and structural points of view. Structural assessment and diagnosis revealed that degradations were induced and developed throughout the life of the structure due to approximately 24 earthquakes (estimated at over 6.0 magnitude) having endangered the structural safety of the building and the mural iconography. After the structural diagnosis, a combined and complex method of strengthening consisting of both grouting and introducing steel rods in vertically drilled galleries along the entire height of the walls was selected. The main advantage of applying this combined strengthening strategy was a remarkable enhancement of the structural seismic performance of the church building.

Keywords: heritage; strengthening; structural assessment; masonry; church

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1. Introduction

Over the course of history, various edifices have been built as a result of specific religious, political, social, or economic requirements, edifices that are now included in either the national or the world heritage. The value that classifies these buildings as heritage is given by a certain combination of their history, greatness, architectural and structural features, and religious, cultural, and traditional importance [1].

A large share of the historical monuments are buildings with structures made of brick and/or stone masonry walls and vaults. The passage of time has left its mark on these structures; a number of factors, such as earthquakes, change of use, moisture, lack of maintenance, and other natural or physical factors, have contributed to the continuous degradation of the built cultural heritage. The need to strengthen and rehabilitate historical monuments is due to the fundamental requirement of any modern society to preserve, conserve, and enhance its heritage so that it remains a center of cultural wealth and also to provide people a sense of place and identity through a stable connection to the past [2–4].

In Romania, The Ministry of Culture and National Identity [5] has identified and listed all the heritage buildings in the National Heritage Buildings Catalogue—Year 172 (XVI)—No. 646 bis., volumes I, II, and III, respectively [6]. According to this catalog, a large share of these heritage buildings are churches and monastic assemblies with exceptional religious, cultural, and social value and are still in service as active places of worship, education, and community gathering. Within this frame, prioritizing the structural assessment, diagnosis, and strengthening intervention across the church portfolio is a necessity to prevent losses in terms of cultural heritage, maintenance costs, and, in severe cases, human lives [6,7].

The seismic vulnerability of old masonry churches has been extensively studied and described in various works [8–12]. Their unsatisfactory structural behavior is mainly related to the typical architectural patterns and geometrical features of the church, the low tensile strength of the masonry, which makes them sensitive to out-of-plane loads, and the lack of sufficient stiff elements in horizontal directions, which may favor the so-called "box behavior" of the structure [13,14].

Up to this point, numerous heritage churches and monastery buildings have undergone various repairing and strengthening procedures with the objective of improving their structural capacity. The most commonly utilized strengthening methods in Romania include the following.

- In 1990, Professor A. Cișmigiu proposed the "framing system" method as a solution for strengthening masonry structures. This system involves enclosing the structural masonry components in a new reinforced concrete framework. Through mechanical means and bonded connections between the reinforced concrete system and masonry elements, the resultant framing system functions as a unified system when subjected to seismic actions [15,16].
- Strengthening using metallic elements can involve various methods, such as repairing specific degraded members, enhancing the overall horizontal load-carrying capacity by encasing the masonry in a spatial steel framework, adding steel plates on contour frames, or restraining the displacements by using horizontal tie-rods. Some of these methods were later reconfigured using fiber-reinforced polymer composite materials instead of steel elements [17].
- Mortar jacketing consists of the application of a self-supporting reinforced cement mortar matrix. This technique is suitable for both stone and brick masonry elements, but since the meshes are usually applied on both faces of the walls, its practical use is severely limited. In fact, when referring to the heritage masonry buildings in Romania, this kind of intervention works on both the inside and outside faces of the walls in only a small fraction.
- Structural repointing of the masonry elements consists of the embedment of steel bars (or composite bars/strips) within the horizontal mortar joints [18]. It usually contributes to the fast increase of the tensile strength of the masonry members, yet it is difficult to apply to stone masonry members due to the geometrical irregularities. It should also be noted that even in the case of brick masonry elements, the effectiveness of this method is particularly concentrated on the outer surfaces of the walls.
- The base isolation method consists of decoupling the masonry superstructure from the foundation and installing an isolating system that possesses high horizontal flexibility and sufficient vertical rigidity [19]. Although this method proved to be effective in protecting masonry structures from high seismic activity, there are some important drawbacks, including the risk of masonry alteration during the decoupling procedure, the need for regular maintenance of the isolation system, and the prohibitive costs.
- The installation of vertical steel rods (post-tensioned in some cases) in drilled galleries along the entire height of the masonry walls has the benefit of preventing local/general collapses of the masonry elements by enhancing the overall ductility of the structural system [20]. Additionally, it improves the in-plane cross-sectional strengths and post-elastic strain capacity. These features are advantageous in preventing the failure mechanisms induced by either the axial forces or the combined effects of bending moments and axial loads.

The selection of a strengthening method from the ones listed above involves numerous considerations, such as the age and the condition of the structure, the materials used in the construction, the desired outcome, and the available budget. In addition, it is essential to notice that nowadays, for a new building, the difference between the architecture and structure is well defined; when referring to historical monuments, especially old masonry churches, the connection between the structure and the shape of a building appears in reciprocal conditioning [21]. This is why, in most cases, the massive load-bearing structures of the historical monuments provided the stability of these constructions over time. Therefore, the diagnosis and rehabilitation of historical monuments should be based on complex assessments performed at the interface between history, architecture, art, and structural engineering. In this frame, it is impossible to select a general valid strengthening solution for old masonry structures. In most cases, a customized approach is essential to achieve successful results.

The goal of the research reported herein is to provide insight into the structural assessment and rehabilitation solutions that were applied to a medieval heritage church located in Piatra Neamț, Romania. The church is the centerpiece of a large monastic complex, and its construction spans several centuries, with additions and modifications made by different princes, rulers, and benefactors. Throughout its history, this heritage building has faced many challenges, including fires, earthquakes, and invasions, but it has managed to survive and continues to play an important role in the community, still serving as a center of spirituality, education, and culture.

2. The Church of the Nativity of the Blessed Virgin Mary of Tazlău Monastery

An Orthodox Church building consists of three interconnected spaces: the narthex (and porch in some cases), nave, and Sanctuary, which, as a whole, represent the architectural setting for the Liturgy (Figure 1) [22,23]. These spaces are arranged along an axis that is aligned from west (the narthex and porch) to east (the Sanctuary). The narthex and the porch are usually separated from the nave through an internal wall. The nave is separated from the altar by a screen called iconostasis. Thus, from an architectural point of view, an Orthodox Church building is centered around the Sanctuary and, in particular, the altar table. The latter, from the religious point of view, symbolizes the mystical and eternal presence of the heavenly throne of God [24]. In Romania, the traditional monastery churches follow a unique pattern of being long and narrow, with only one apse that spans the entire width of the church and one roof characterized by a large overhang (Figure 1).

Figure 1. Tazlău monastery: (**a**) The church of the Nativity of the Blessed Virgin Mary of Tazlău monastery, (**b**) Sanctuary and altar table, (**c**) 3D model—isometric view, (**d**) Longitudinal section and arrangement of spaces.

The architecture of the church of the Nativity of the Blessed Virgin Mary is a testament to the skill and creativity of the builders and architects who constructed it. It is characterized by a combination of Byzantine, Gothic, and Moldavian influences, creating a unique and harmonious blend of styles. The Tazlău Monastery is not only an architectural masterpiece but also a repository of artistic treasures. The interior of the church is adorned with frescoes and murals that depict scenes from the Bible and the lives of the saints. The frescoes were painted by some of the most talented artists of their time, and their style and technique are representative of the Moldavian school of painting. The monastery also houses a rich collection of religious objects, including icons, liturgical objects, and manuscripts.

The structural system of the church of the Nativity of the Blessed Virgin Mary, erected between 1496 and 1497, consists of single-leaf, river stone masonry walls varying in thickness between 1.65 and 1.05 m [25]. The vaults are made of brick masonry in the form of a dome, which rests on stone pediments and arches. The church tower, which starts at the level of the nave dome, is made of brick masonry. This configuration proves that medieval craftsmen were aware of the seismic activity of the site, and they empirically designed a gradual transition between the rigid elements (stone walls) and the more flexible elements (brick masonry tower and vaults). The same gradual transition approach was adopted for the tower and the walls of the nave. Two consecutive horizontal bases were constructed, the first in a square-shaped pattern and the second in a star-shaped pattern (Figure 2).

Figure 2. The interior of the tower, perspective at the square-shaped and star-shaped bases.

The mass use of bricks in Romania dates back to the 14th century, during the reign of the Wallachian prince Vladislav I (1364–1377) [26–28]. The bricks used in the construction of the church of the Nativity of the Blessed Virgin Mary were handmade from local clay mixed with water and molded into shape before being dried in the sun, according to the common medieval manufacturing methods [29–31]. They were then burned in kilns at high temperatures to make them hard and durable. These bricks have certain distinct characteristics. They are (most of them) large in size, rectangular in shape, and have a reddish-brown color.

In 1858, a closed porch was built on the west façade. The west wall of the porch was reconstructed in 1985, and the vault corresponding to the porch was reconstructed using reinforced concrete. The foundations of the church were built according to an empirical-intuitive concept and consist of raw river coarse stone blocks bonded with lime mortar, with no protection against underground agents. The foundation depth varies between 1.30 and 1.90 m.

According to the geotechnical study of the foundation soil, the land on which the church is located has global and local stability ensured. The monastery site is located in the Sub-Carpathian depression Tazlău-Casin, near the bank of the Pestioșu River and the southwest extremity of the Russian–Moldavian platform. The groundwater table at the site was measured in two test holes to be 2.5 m and 1.60 m, respectively, below the ground surface. The description of the foundation soil based on the boring information from the two 6 m depth boreholes drilled at the investigated site is given in Tables 1 and 2.

Table 1. Lithologic description for the investigated site based on the first borehole.

Borehole No. 1
0–0.25 m Vegetable soil with stone fragments
0.25–2.1 m Brownish-red silty clay with fine sand and small gravel fragments
2.1–2.8 m Sand and gravel in a clay matrix
2.8–4.7 m Grey clay with brownish regions
4.7–6 m Clayey sand with gravel fragments

Table 2. Lithologic description for the investigated site based on the second borehole.

Borehole No. 2
0–0.5 m Vegetable soil with brick fragments
0.5–1.6 m Brownish-red silty clay with fine sand and small gravel fragments
1.6–2.7 m Clayey sand with small gravel fragments
2.7–4.5 m Brownish-red clay with sandy regions
4.5–6 m Clayey sand with gravel fragments and brownish-red fine sand

3. Structural Assessment and Diagnosis

3.1. Visual Investigation

During the on-site visual investigations, it was observed that cracks (developed as a result of previous earthquakes) had occurred in the most vulnerable elements. In the Sanctuary, continuous cracks were identified between the window and the vault keystone (Figure 3a). In the nave, cracks were identified both in the apse and in the transition arch at the entrance of the narthex (Figure 3b). Severe degradations were also found at the level of the church tower that starts at the level of the nave dome. These degradations are located both in the masonry and in the nearby roof timber framing system (Figure 3c). In the narthex, various cracks were identified above the windows and the door.

As can be noticed in Figure 4, the cracks occurred in the most vulnerable parts of the structure [32,33] in both horizontal and vertical members. Moreover, the crack pattern corresponds to the typical mechanisms of damage/failure of the Orthodox churches. The losses in terms of structural redundancy were confirmed by a continuous longitudinal fracture, starting from the porch and continuing up to the Sanctuary, dividing the nave into two parts [34,35]. Similarly, the multiple transversal fractures that developed in the upper areas (the less rigid ones) of the Sanctuary, nave, narthex, and porch confirmed that the structure was additionally subdivided into multiple parts. Moreover, a separation tendency has been observed between the porch and the narthex, indicated by the continuous development of cracks in the longitudinal walls [36].

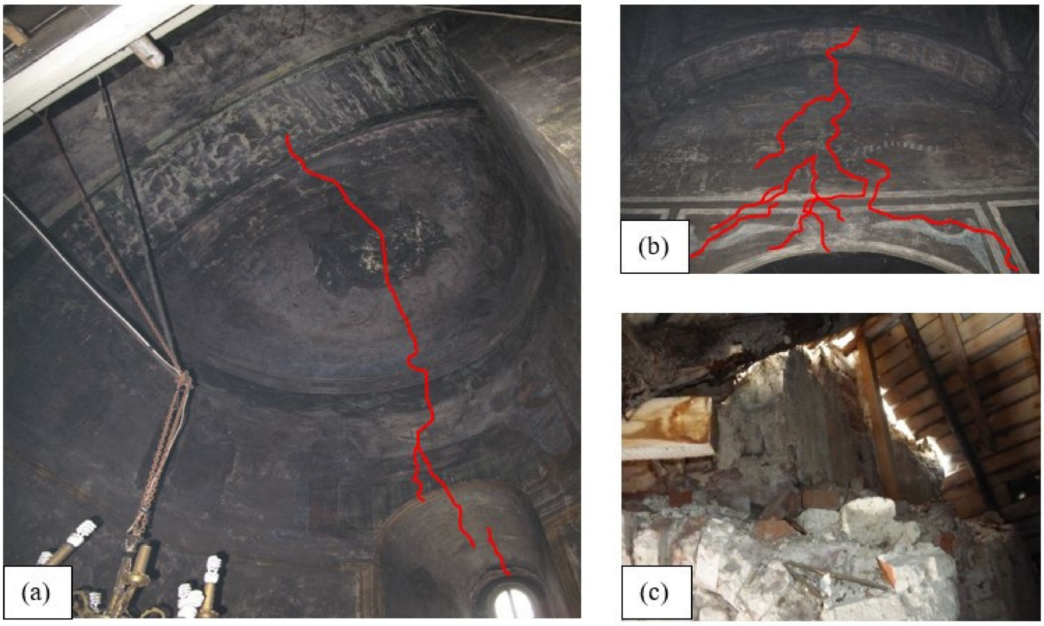

Figure 3. Visual investigations: (**a**) Continuous cracks between the window and the vault keystone—Sanctuary, (**b**) Cracks in the transition arch from the entrance in the narthex, (**c**) Degradations located both in the masonry and the nearby roof timber framing system.

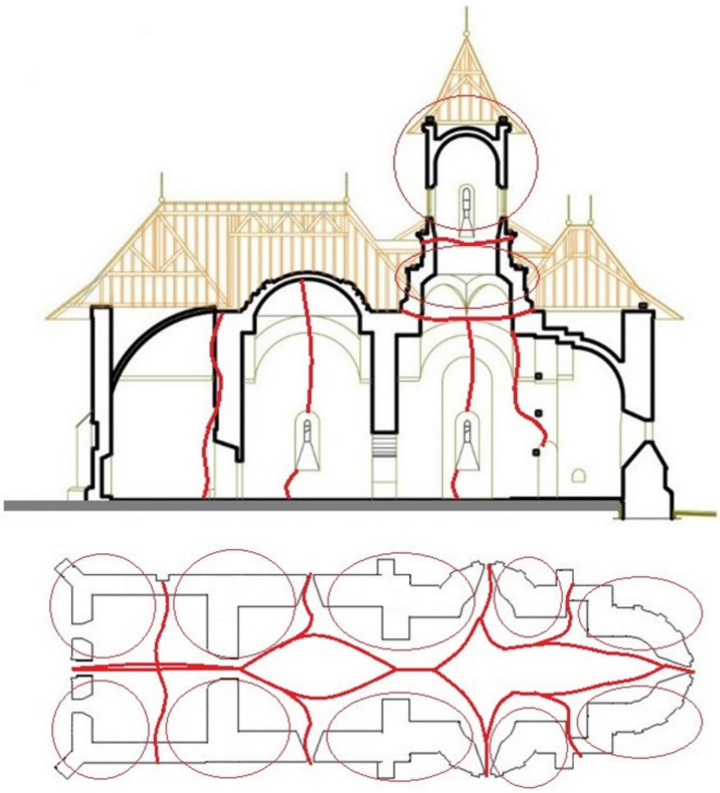

Figure 4. Crack pattern. Multiple transverse fractures (porch, narthex, nave, and Sanctuary) and longitudinal fractures of the church.

As a general rule, the uncontrolled presence of water is considered one of the most important factors that threaten heritage assets throughout the entire history of architecture [37,38]. In the case of the church of the Nativity of the Blessed Virgin Mary, it was observed that water had been absorbed from the ground through the phenomenon of capillarity due to the specific porous nature of stones and mortar (Figure 5) [39]. It is well known that this process (referred to as rising damp) is harmful to all porous materials as it may favor the occurrence of frost-induced damage, making them vulnerable to additional degradation mechanisms, such as biological growth or wind erosion [40]. By visual observation, the church is deeply affected by rising damp, and at least two common degradation patterns occurred on the external surface of the walls, color change (darkening) (Figure 5a) and efflorescence (Figure 5b).

Figure 5. Examples of degradation patterns in the church of the Nativity of the Blessed Virgin Mary: (**a**) darkening due to rising damp in the façade, (**b**) salt efflorescence.

3.2. Structural Analysis

In order to structurally assess and diagnose the damage described in the previous section, the church was modeled and analyzed (non-linear static) in Extended Three-Dimensional Analysis of Building Systems (ETABS) software using shell elements (Figure 6). One of the key features of ETABS software is its ability to model various complex structures in three dimensions. In this case, it enables designers to accurately model the behavior of heritage masonry structures under different loading conditions and assess their safety and performance.

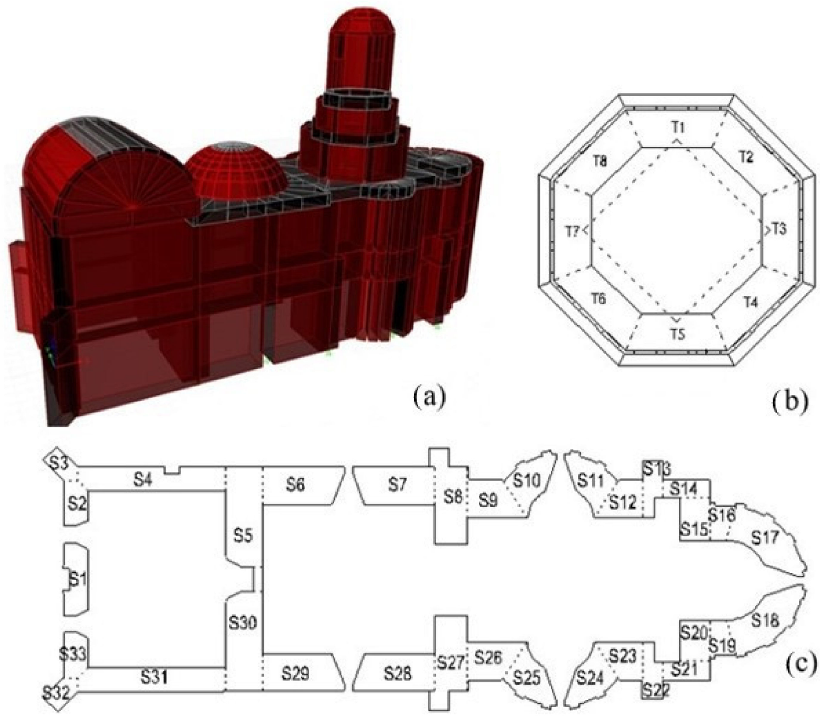

Figure 6. Numerical model: (**a**) Isometric view, (**b**) Structural elements distribution—tower level, (**c**) Structural elements distribution—church level.

By definition, shell elements are thin, two-dimensional elements that can be used to model the surface of a structure (Figure 7). The main advantage of using shell elements in modeling heritage churches is the ability to accurately capture the shape and curvature of walls, arches, and domes [41]. Another advantage is that the shell elements can accurately capture the behavior of the masonry structures under different loading conditions. Masonry structures are typically subjected to complex loading conditions, including vertical and lateral loads and earthquake forces. Shell elements are capable of capturing these complex loading conditions and accurately simulating the behavior of the structure under different loading scenarios [42]. In addition, shell elements are computationally efficient, making them a preferred choice for large-scale models of heritage masonry structures. Solid elements allow for accurate control of the stress distribution within the structural elements, but they are computationally expensive and can quickly become unsuitable for large-scale models. Using shell elements to model the exterior of the structure reduces the computational load, making it easier to analyze and modify the model.

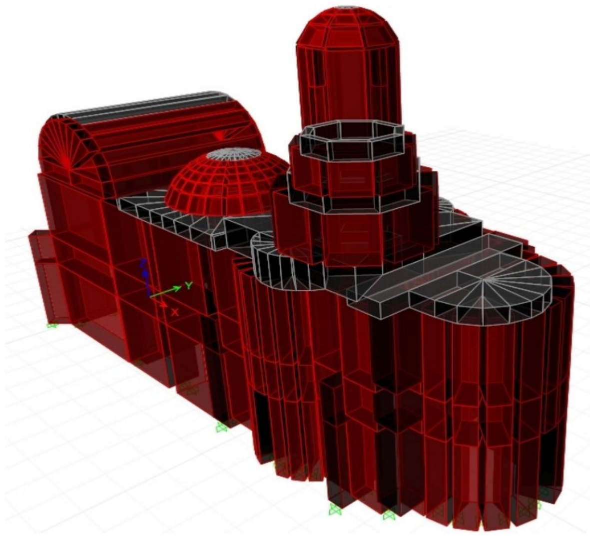

Figure 7. Perspective view—3D model. Shell elements.

In the first stage, specimens were extracted from the church walls and the mechanical characteristics were determined. This approach provides truthful values for the numerical modeling–material property assessment. Therefore, compressive tests according to the norms EN 1926:2006 [43], ASTM C67/C67M-21 [44], and SR EN 1015-11/2002 [45] were performed on bricks, stones, and mortars (5 specimens for each material). After applying the safety factors, the compressive strengths of all the masonry materials were determined (Table 3).

Table 3. Compressive strengths of the masonry materials.

Brick Compressive Strength (Mean Value) [N/mm^2]	Stone Compressive Strength (Mean Value) [N/mm^2]	Mortar Compressive Strength (Mean Value) [N/mm^2]
5.770	70.740	1.098

The compressive strength of the stone was determined by testing 5 cylindrical specimens with diameters ranging from 72 mm to 68 mm and heights of 54–55 mm. The specimens were carefully extracted from the topmost part of the stone walls, and the geometrical configurations specified in [43] were obtained using water jet cutting. The load was applied on smooth and flat surfaces that had been previously prepared to eliminate any abrupt irregularities. No capping with mortar was needed since the tolerances were below the limits indicated in the standard [43].

The compressive strength of the bricks was determined by testing 5 specimens extracted from the topmost part of the church tower. The procedures described in [44] were followed. However, it was not possible to extract whole brick specimens; therefore, the compressive strength was computed in the report to the area of the partial brick specimens.

The compressive strength of the mortar was determined by testing 5 specimens (mortar joints) extracted from the stone walls, according to the provisions given in [45]. The thicknesses of the extracted joints varied between 35 and 42 mm. The densities of the specimens varied between 1583 and 1610 kg/m^3.

The dead loads were generated automatically by the ETABS software based on the dimensions and the weights of materials and elements (Table 4). Snow loading (2.0 kN/m^2) was computed according to norm CR 1–1–3/2012 [46], and the live load was set as a surface load of 0.75 kN/m^2 on the attic [47].

Table 4. Dead loads and live loads.

Dead Loads				Live Load [kN/m^2]
Roof System [kN/m^2]	**Stone Masonry Elements [kN/m^3]**	**Plaster [kN/m^3]**	**Reinforced Concrete Elements [kN/m^3]**	
1.5	19.5	20.0	25.0	0.75

Seismic action was computed according to the Romanian norm P100-3/2019 [48], taking into consideration an average reference return period of 100 years. The design peak ground acceleration (PGA) was considered as 0.24 g and the corner period, Tc, as 0.7 s. The behavior factor, q, was considered 1.5 (appropriate value for heritage buildings made of unreinforced masonry), while the importance factor, γ_I, was taken as 1.2. The normalized elastic response spectrum, β, was 2.75, and the damping, ξ, was considered to be 8%.

Based on the modal analysis, the dominant modes of vibration were identified. The first eigenmode consists of combined translation and rotation along the transversal direction, while the second is characterized by translation along the longitudinal direction. The fundamental period of vibration was determined as 0.197 s, and the mass participation factors for the first five modes of vibrations were 74% in the transverse direction and 70% in the longitudinal direction.

According to the Romanian norm P100/3-2019 [48], the seismic classes of risk in which an existing building can be included are established by evaluating the index R_3 (the corresponding structural seismic safety level). For masonry structures, R_3 is defined as the ratio between the sum of the shear capacities of the structural walls and the sum of the effective shear loads produced by the seismic action (Equation (1)) [48].

$$R_{3i} = \frac{S_{cap,i}}{F_{b,i}} \tag{1}$$

where

- $S_{cap,i}$ is the shear capacity of the structural wall "*i*", determined based on the specific failure mechanism;
- $S_{cap,i} = V_{f1}\ or\ V_{f2};$
- $V_{f1} = \frac{N_d}{c_p \lambda_p} \nu_d (1 - 1.15\nu_d)$—the shear force associated with the eccentric compression failure of an unreinforced masonry wall subjected to the design axial force;
- $\lambda_p = \frac{H_p}{l_w}$—the shape factor;
- H_p—the height of the wall;
- l_w—the length of the wall;

- c_p—the coefficient that accounts for the bearing conditions (2 for cantilever wall, 1 for clamped wall)
- $\sigma_0 = \frac{N_d}{tl_w}$—the compressive stress corresponding to the design axial force;
- t—the thickness of the wall;
- $\nu_d = \frac{\sigma_0}{f_d}$;
- f_d—the design compressive strength;
- $V_{f2} = \min\left(V_{f21},\ V_{f22}\right)$—the shear capacity of the unreinforced masonry wall;
- $V_{f21} = f_{vd}D't$—the design shear force corresponding to the failure by sliding in mortar joints;
- D′—the length of the compressed region of the wall;
- f_{vd}—the shear strength associated with failure by sliding in mortar joints;
- $V_{f22} = \frac{tl_w f_{td}}{b}\sqrt{1 + \frac{\sigma_0}{f_{td}}}$—the design shear force corresponding to failure by diagonal fracture;
- $1.00 \leq b = \lambda_p \ \leq 1.5$;
- f_{td}—the design tensile strength
- $F_{b,i}$ is the shear load produced by the seismic action to the structural wall "*i*";
- $F_{b,i} = \frac{G_i}{\sum G_i}F_b$;
- G_i is the mass of the structural wall "*i*";
- ΣG_i is the mass of the whole building.

For the unstrengthened model, the structural seismic safety level was 42% for the nave and 20% for the tower in the transverse direction and 63% for the nave and 20% for the tower in the longitudinal direction, thus, ranking the church in the first seismic risk class. The latter is considered to be the most concerning condition since it corresponds to a high risk of imminent collapse.

The iterative pre-design process of the strengthening system was performed by imposing target values for the strengthened church numerical model for the fundamental period of vibration. The latter aims at obtaining a lower value of the normalized elastic response spectrum, β, thus, diminishing the shear forces induced in the masonry walls. This procedure follows the provisions given by the Romanian seismic design norm P100/1-2013 [49].

4. Strengthening System

The strengthening system was designed to prevent the failure of the church and to avoid any significant losses in terms of religious heritage. The methodology and the technology were carefully discussed, analyzed, and decided between the stakeholders involved in the rehabilitation process (contractors, designers, site supervisors, etc.). The constructive system that was selected and applied consisted of ternary hydraulic lime-based fluid grouting into the masonry walls, grouting to stop the capillary water transport, reinforced concrete pads below the level of the existing perimetral foundation, vertical steel rods inserted in drilled galleries along the entire height of the walls, reinforced concrete girders at the attic level, reinforced concrete girders at the lower and upper level of the tower, and substitution of the degraded roof timber framing system.

Ternary natural hydraulic lime-based (NHL 5 mortar) grouting was applied to the masonry walls and foundation to restore the continuity, uniformity of strength, compactness, and cohesion without modifying the morphology features of these structural elements. According to previous studies, grouting using an injectable mix is considered to be one of the most suitable techniques for strengthening stone masonry heritage buildings [50]. In fact, it was experimentally proven that an adequately designed mixture could fill even the smallest cracks (≈0.20–0.30 mm) and voids. The first applications of grout consisted of pure cement-based mixtures. Later, it was proven that the injectability of cement-based grouting is inadequate when small voids and cracks need to be filled (due to clogging) [51,52]. On the other hand, the use of pure lime-based grouts may not be sufficient for developing satisfactory mechanical strengths. Thus, lime-based grouts with the addition of cement (also referred to as binary or ternary grouts) were developed. These types of grouts were proven to be efficient by various research teams [53,54].

The initial mixture (pure lime-based) design for this study did not provide satisfactory mechanical strengths ($\leq$5 N/mm^2). Thus, the solution adopted includes adding 10% cement to the existing mix. The recipe and the experimentally determined physical and mechanical strength of the utilized grout are given in Table 5.

Table 5. Grout mixture. Physical and mechanical properties.

Natural Hydraulic Lime [kg/m^3]	**Cement I 52.5 R [kg/m^3]**	**Water [L/m^3]**
650	130	715
	Specimen dimensions	
L [mm]	**L [mm]**	**h [mm]**
150	150	150
	Specimen masses	
S_1 [kg]	**S_2 [kg]**	**S_3 [kg]**
7.070	7.110	7.220
	Specimen apparent densities	
ρ_1 [kg/m^3]	**ρ_2 [kg/m^3]**	**ρ_3 [kg/m^3]**
2095	2107	2139
	Compression strengths	
R_1 [N/mm^2]	**R_2 [N/mm^2]**	**R_3 [N/mm^2]**
5.46	5.99	5.36

Holes were drilled on both faces of the walls in an orthogonal network (maximum 5–6 holes per sqm). PVC (polyvinyl chloride) nozzles, 13 mm in diameter, were inserted into the holes (Figure 8a). Based on the previous experience of the site supervisors and experts, the grout was introduced through the PVC nozzles at a quasi-constant pressure of 3 bar. Grouting started from the bottom of the stone walls to their top. As the ternary lime-based grout was injected into the stone masonry through the PVC nozzles, the aesthetical outlook of the church remained unchanged, and the architectural authenticity was, thus, preserved.

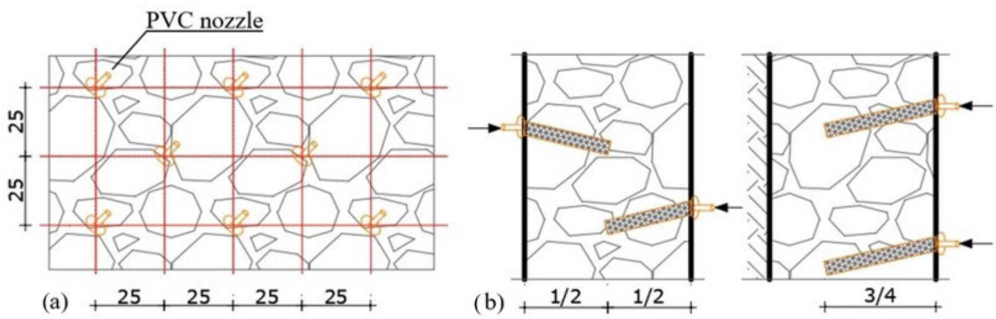

Figure 8. Natural hydraulic lime-based (NHL 5 mortar) grouting: (**a**) Insertion of the PVC nozzles (dimensions in cm), (**b**) Grouting depth compared to the width of the walls.

During the visual investigations, it was observed that in addition to the cracks, the lower parts of the walls were highly exposed to varying moisture stresses due to ground water, seepage water, and soil moisture. Moreover, the plinth of the floors was exposed to stresses caused by ice, splashing water, and de-icing salt. In order to stop the capillary water transport, grouting with a hydrophobic mixture was carried out at the foundations and above all the masonry plinths from both sides of the structure (Figure 9). The hydrophobic material that was used is composed of the ternary lime-based grout previously described, mixed with 10% waterproofing mixture, according to the specifications given by the manufacturer in the technical data sheet [55]. Based on the previous experience of the site supervisors and experts, the hydrophobic grout was introduced through the PVC nozzles at a quasi-constant pressure of 3 bar. In this manner, a moisture equilibrium was established in the stone masonry above the sealing level.

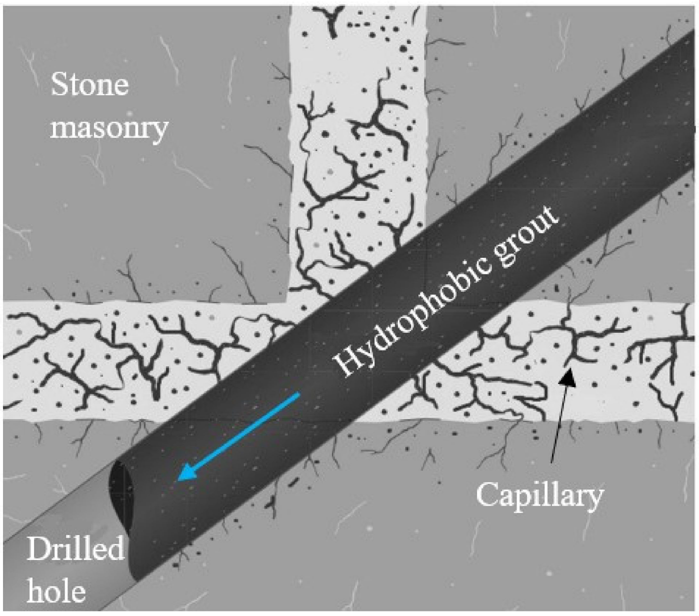

Figure 9. Grouting with hydrophobic mixture.

The solution adopted for the waterproofing of the floor was the extraction of the original stone tiles and excavation of 80 cm of soil, followed by the installation of 50 cm of highly compacted clay soil, a 10 cm gravel layer, and a 0.2 mm PVC membrane (Figure 10). Above these layers, a floor heating and cooling system was installed. It should also be noted that the flooring was constructed using almost all of the original stone tiles.

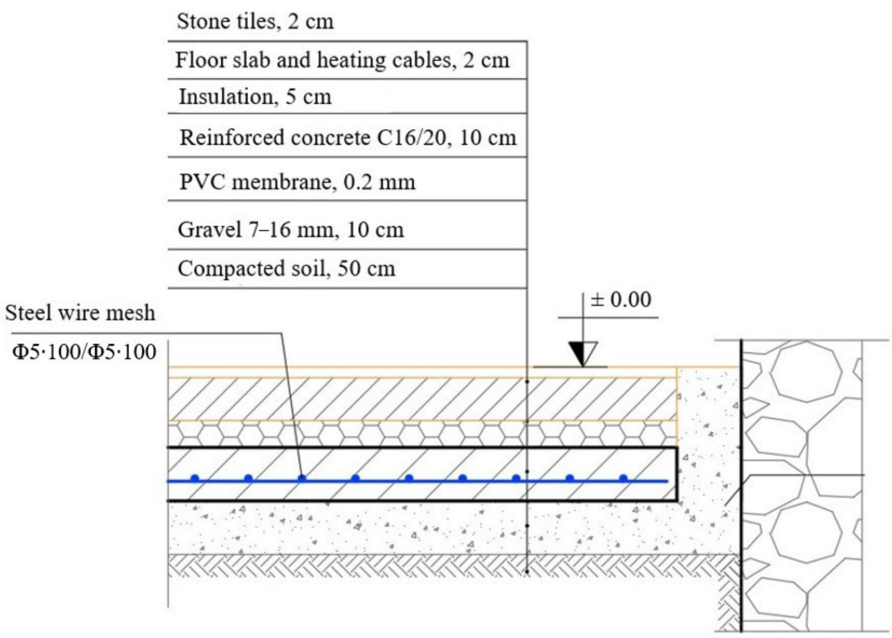

Figure 10. Floor detail.

Reinforced concrete pads were installed below the existing perimetral foundation (Figure 11). Reinforced concrete girders were constructed at the attic level and at the lower and upper levels of the tower (Figure 12a). After that, galleries were drilled along the entire height of the walls, and steel rods were introduced without post-tensioning (Figure 12b). The latter were anchored at the lower part to the reinforcements of the pads, and, at the upper part, a mechanical connection composed of a nut and steel bearing plate was used.

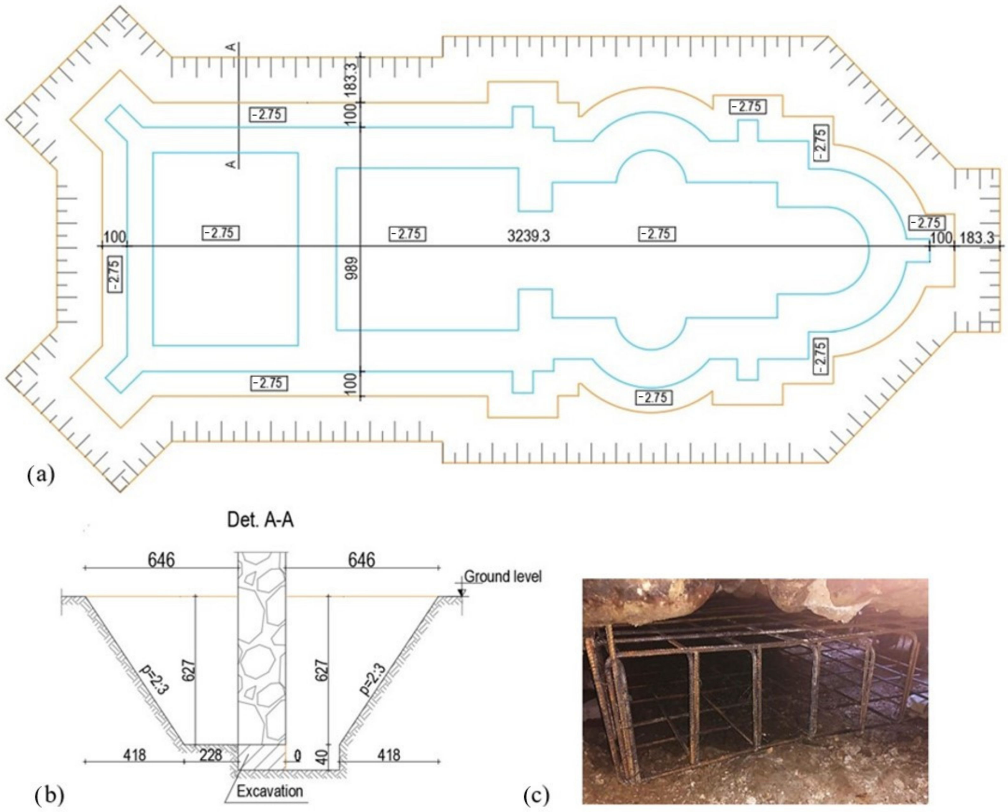

Figure 11. Reinforced concrete pads (**a**) Excavation plan, (**b**) Detail A-A, (**c**) Reinforcement of the pads (dimensions in cm).

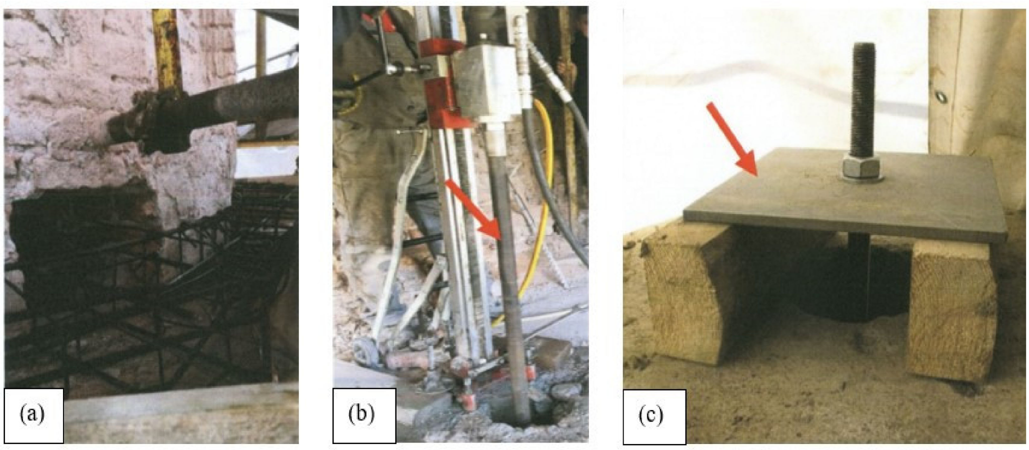

Figure 12. Vertical steel rods. Stages of installation: (**a**) Reinforced concrete girders, (**b**) Drilling of galleries, (**c**) Insertion and fixing of the steel road.

The installation stages of the vertical steel rods are illustrated in Figure 12. The positions of the reinforced concrete girders are highlighted in red in Figure 13. Figure 14 illustrates the configurations of the reinforced concrete girders and the connections between them and the vertical steel rods are depicted in Figure 15.

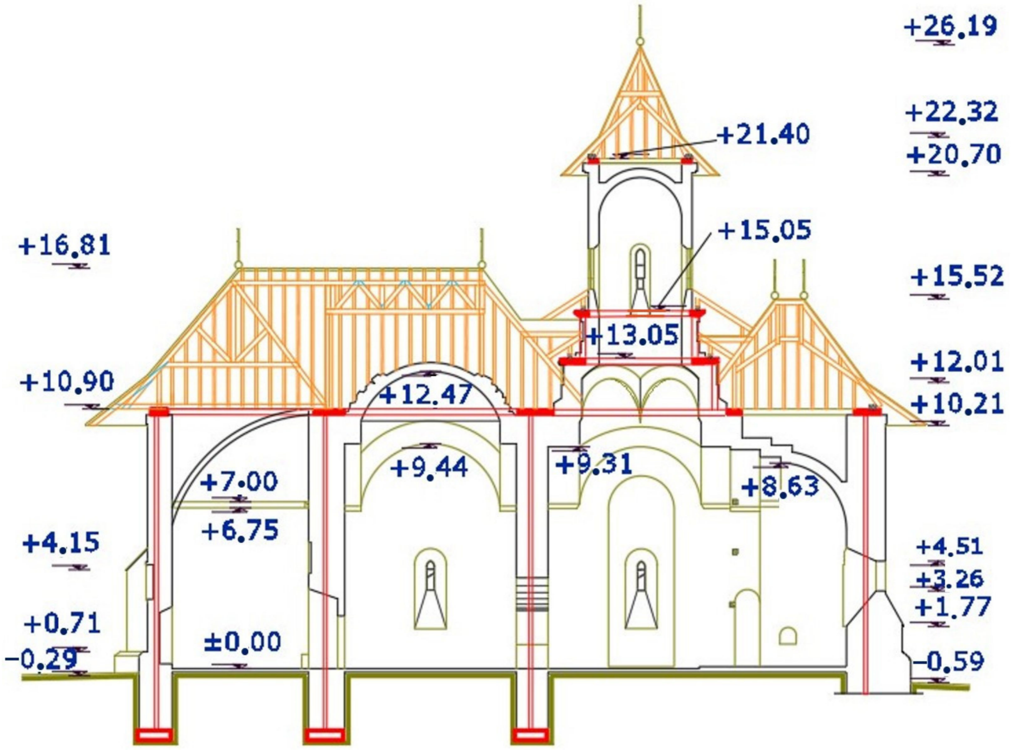

Figure 13. Position of the reinforced concrete girders (dimensions in m).

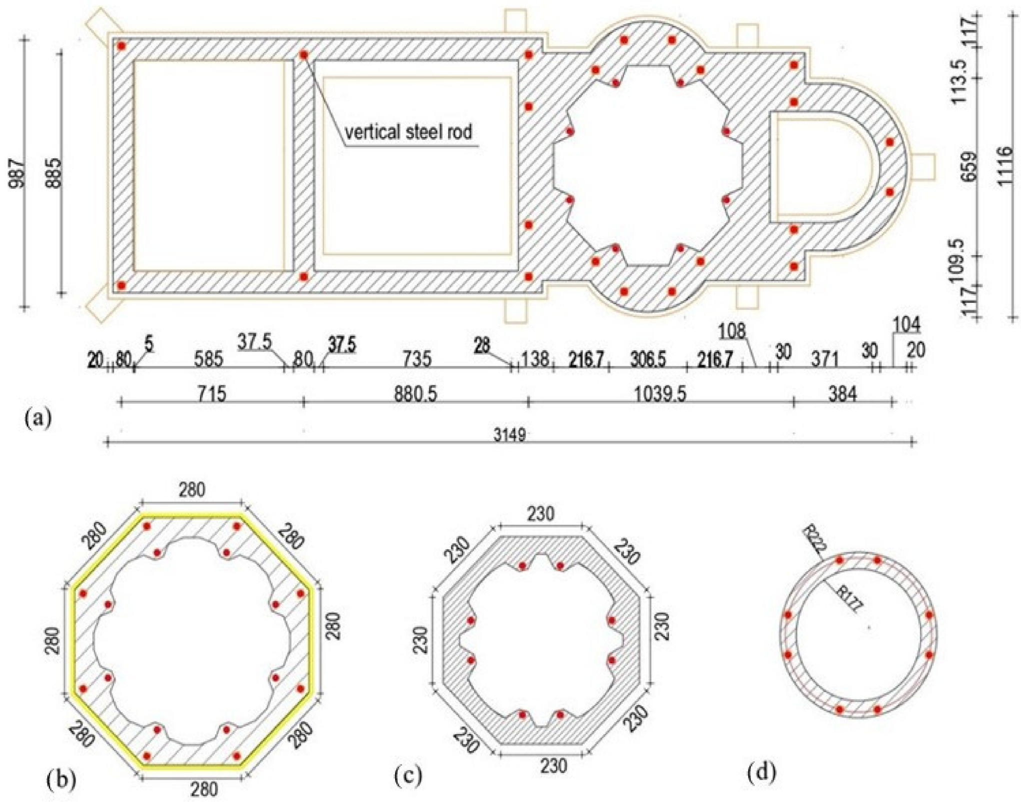

Figure 14. Reinforced concrete girders (dimensions in cm): (**a**) Attic level; (**b**,**c**) Lower levels of the tower, (**d**) Upper level of the tower (dimensions in cm).

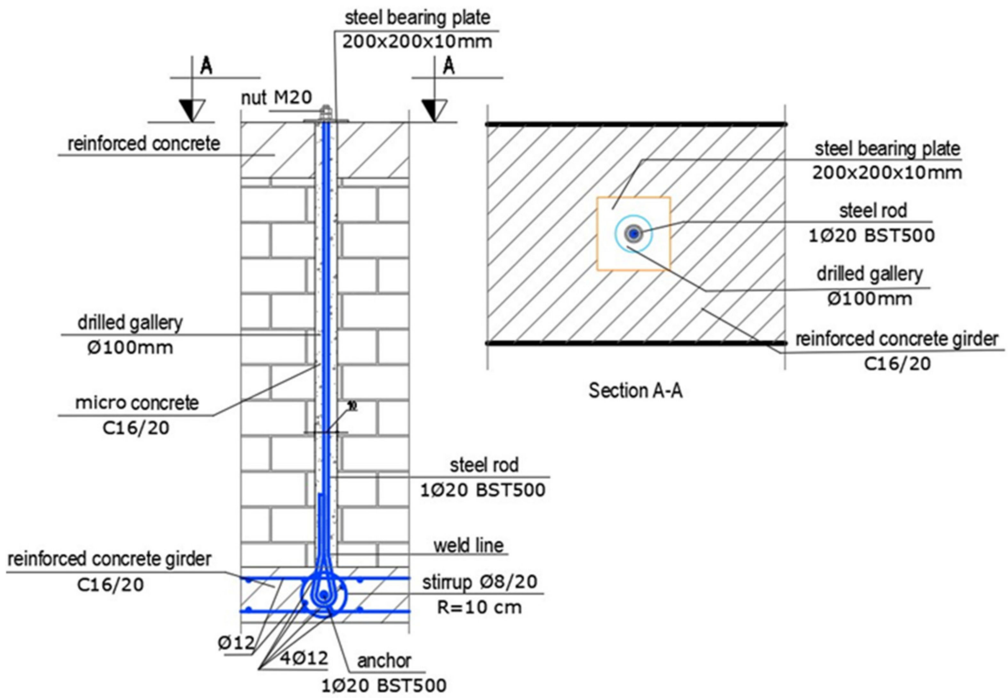

Figure 15. Drilled galleries and steel road connections.

5. Structural Analysis of the Strengthened Model

The strengthened structure was modeled in the ETABS software using shell elements. The RC elements were added to the unstrengthened model described in Section 3, and the loads were redefined. In order to obtain a complete and precise numerical model, the ultimate stresses corresponding to the vertical steel rods were determined on the base of the ultimate horizontal displacement of the pier. In fact, it was previously numerically proven by various research teams that if one considers a reinforced pier, the resultant force–axial displacement law is the same as for an unreinforced pier, as long as the axial force component that is supplied by the tie rods is added to the external vertical load. More details about this modeling strategy and the corresponding correction coefficients can be found in [56].

Regarding the definition of the material property, based on additional testing, it was found that the compressive strength of the masonry had been raised by 30% due to the natural hydraulic lime-based fluid grout injections. At the same time, the behavior factor, q, was considered 2 (appropriate value for strengthened masonry buildings) [48].

The fundamental period of vibration of the strengthened model was determined as 0.164 s, being 16.75% lower than the fundamental period of vibration of the initial unstrengthened model. Moreover, the mass participation factors increased to 81% in the transversal direction and 79% in the longitudinal direction.

For the strengthened model, the structural seismic safety level is 67% for the nave and 67% for the tower in the transverse direction and 96% for the nave and 67% for the tower in the longitudinal direction, thus, ranking the church in the third seismic risk class. The latter is characterized by relatively minor and insignificant damage induced to the masonry structure in the case of an earthquake occurrence.

6. Discussions and Conclusions

One of the essential features of this rehabilitation process refers to grouting with a hydrophobic mixture. The main benefit of this intervention is blocking the water penetration and the capillary water transport, which can cause damage beyond repair to the interior frescoes.

It is generally accepted that churches made of masonry structures were not conceived to withstand horizontal loads. In the majority of cases, even at low levels of ground acceleration, partial collapses of macro-blocks (e.g., elements of façades, apses, tympana, porches) are activated. The strengthening system described in this work consists of natural hydraulic lime-based grouting, reinforced concrete pads below the level of the existing perimetral foundation, vertical steel rods in drilled galleries along the entire height of the walls, reinforced concrete girders at the attic level, reinforced concrete girders at the lower and upper level of the tower, and substitution of the degraded roof timber framing system, which highly improved the structural behavior of the church. Firstly, the continuity, uniformity of strength, compactness, and cohesion of the walls were restored due to the ternary lime-based grouting. Then, the "framing system", conceived by encasing the stone and brick masonry elements in a new reinforced concrete system, led to a significantly improved seismic response of the structure. In this respect, under the effect of the designed seismic actions, the fundamental period of vibration diminished, the mass participation factors increased, and the R3 indexes (the corresponding structural seismic safety level) increased, thus, ranking the church in a low-risk seismic class.

Comparing the financial impact of the selected strengthening system as part of a wider rehabilitation solution with that of more advanced strengthening systems (based on base isolation or use of fiber-reinforced polymers materials) [22,57–59], there is no doubt that the costs of the first are considerably lower, the total price of this project is approximately 1500 EUR/m^2 (value expressed in terms of built-up area).

As a strengthening solution, the addition of reinforced concrete elements may have both positive and negative impacts on the architectural and historic value of an old masonry structure. On the one hand, adding reinforced concrete elements can provide structural stability, allowing the building to withstand seismic actions and ensuring its longevity. On the other hand, the addition of reinforced concrete elements can negatively impact the architectural features of heritage buildings. However, in the case described in this paper, the architectural value was not altered. The intricate details and the unique design elements, which are testimonies of the skills, creativity, and craftsmanship of the original builders and architects, were preserved.

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