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Installation of a base isolation system made of friction pendulum sliding isolators in a historic masonry orthodox church

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ABSTRACT

The first part of the paper focuses on briefly presenting the background referring to the necessity of rehabilitating the heritage church buildings and to the general principles and strategies that dominate the rehabilitation processes and techniques. Also, some of the most important buildings that underwent through a base isolation process are exemplified, from ancient to present times, both world wide and in Romania. The second part of the paper presents a case study referring to the first historic heritage church building that was base isolated in Romania, St. Nicolae Aroneanu Orthodox Church, located in Iasi County. Being erected in 1594, the church follows the traditional orthodox architectural patterns of those times, from both esthetical and structural points of view. The rehabilitation necessity was imperative, since the church was severely damaged by numerous strong seismic actions during the last 400 years. After extensive and wide-ranging investigations and evaluations, the base isolation method was selected as the appropriate rehabilitation solution. The system consists in installing 48 friction pendulum sliding (FPS) isolators, between two horizontal reinforced concrete carrying elements that were casted at the infrastructure level, followed by decoupling the superstructure of the church from the existing foundation system and transferring it to the seismic isolators. Since it has never been previously applied to Romanian heritage church buildings, the execution process was divided into several technological stages, each of them being extensively discussed, by highlighting the advantages and drawbacks that arose. The main advantages that derived from applying the base isolation rehabilitation strategy refer to a considerably improved response of the structure to the high intensity seismic actions that are specific to the north-eastern part of Romania and to a significant reduction of the drift displacements and of the shear forces in the structural masonry walls. Nevertheless, by far the most important advantage consists in the fact that all necessary rehabilitation works are performed at the infrastructure level, without generating risks of damaging the heritage architectural and artistic components of the church.

1. Introduction

Romania has a large number of heritage buildings, which are widely distributed in both the Bucharest area and also in the other territories of the country. The Ministry of Culture and National Identity has summarized the entire stock of heritage buildings and listed them in the National Heritage Buildings Catalogue – Year 172 (XVI) – No. 646 bis., published on July 16th, 2004, volumes I, II, and III respectively [1]. The large number of heritage buildings, many of them having exceptional cultural and social value, triggers extended and important attention with respect to the responsibilities and actions that have to be applied in order to preserve these buildings. According to the national catalogue, the heritage buildings are classified in two general levels, with respect to their importance (class A and B) and with respect to their type (I, II, III and IV). Table 1 presents the total number of heritage buildings located in Romania, with respect to these two levels of classification.

Worldwide, approximately 70% of the total number of buildings are composed of structural load-bearing masonry systems. Therefore, one of the most consistent research areas refers to the specific methodologies of evaluating and strengthening/rehabilitation these types of structural systems. In Romania, among the constructions that have structural load-bearing masonry systems, a large number is being represented by the religious buildings, most of them being in service for very long periods of time (usually more than 100 years). Therefore, in many cases, these buildings are severely damaged due to the action of different factors (environmental conditions, natural decay of construction materials, lack of maintenance, recurrent seismic actions, etc.)

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Table 1
Heritage buildings and sites distribution in Bucharest and Romania [1].

	No. of heritage buildings	I Archaeology	II Architecture	III For public	IV Memorials	Class A monuments (highest class)
Bucharest	2627	190	2089	112	236	247
Romania	29,425	9585	17,708	678	1464	6640

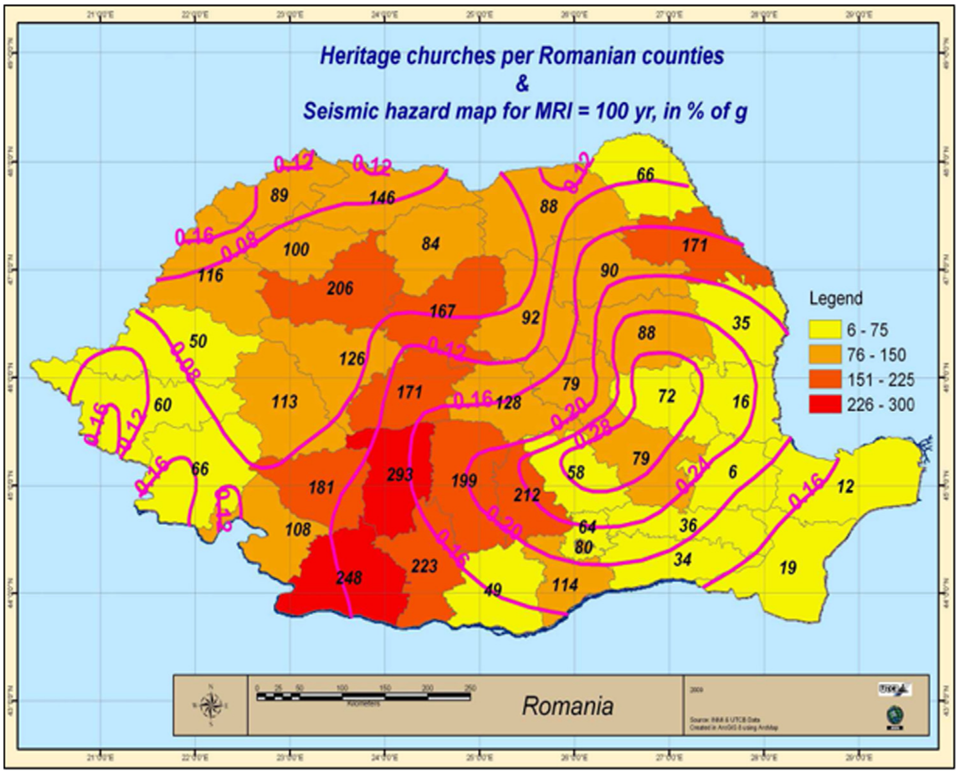

Fig. 1. The distribution of heritage church buildings in Romania correlated with the map of the peak ground accelerations for seismic actions having a return period (MRI) of 100 years [2].

being prone to imminent collapse. Fig. 1 presents the total numbers of heritage church buildings in Romania, with respect to the corresponding level of seismic hazard [2].

2. Background of base isolation concept for historic heritage buildings

2.1. Rehabilitation strategies and methods for historic heritage buildings. Advantages of the base isolation concept

According to the Romanian Norm M.P. 025-04-“Methodologies of risk evaluation and strengthening solutions for heritage buildings” [3] which follows the general principles presented in the “Venice Charter for the Conservation and Restoration of Monuments and Sites” [4] and in Eurocode 8 [5], the restoration processes of the heritage buildings must comply with the following set of requirements:

- efficiency: the intervention works must be efficient and their efficiency must be demonstrated through qualitative and quantitative analysis;
- compatibility: the intervention works must be compatible with the original structural system and building materials/elements in terms of chemical, mechanical and architectural particularities;
- durability: the intervention works must be carried out by using materials and processes that have certified durability, being similar to that of the initial materials/elements of the building;
- reversibility: the intervention works should be reversible in order to allow for future intervention processes to be performed.

Until present times, many heritage church buildings have been subjected to various strengthening processes, aiming to increase their structural performance. Some of the most frequent strengthening solutions, also referred to as traditional systems, that were successfully applied in Romania consist of:

- the “framing system” is one of the first solutions of strengthening, being proposed by prof. A. Cișmigiu in 1990. This system consists of encasing the structural masonry elements in a new reinforced concrete system. By connecting the r.c. system with the masonry elements (through mechanical means and physical adhesion), the hybrid components which are obtained respond as a unitary system when subjected to seismic actions [6,7,8];
- strengthening with metallic elements may refer to: particular strengthening of damaged elements, improving the general horizontal load bearing capacity by in-plan or spatial framed structures, strengthening the masonry walls with contour frames made of steel plates, limiting the displacements of the masonry elements by applying horizontal tie-rods;
- combined and complex methods of introducing post-tensioned vertical steel rods in drilled galleries along the entire height of the masonry elements. This strengthening method has a major effect in preventing local or general collapses of masonry elements, by improving the general ductility of the structural system. Also, it increases the in-plane cross-sectional strengths and post-elastic strain

capacity for the failure mechanisms induced by axial loads or combined effects of bending moments and axial forces. The strengthening technique consists of increasing the axial loads in the masonry walls by post-tensioning the high-strength steel rods. The additional axial forces generate a more uniform distribution of the compressive stresses that compensates the relatively low tensile strength of the masonry [9].

However, the application of these strengthening systems to the structures of the heritage church buildings generated extensive debates and uncertainties related to the reversibility and compatibility degrees of these approaches. Thus, by taking these drawbacks into account, the base isolation method is emphasized since it is considered to be more appropriate for these types of heritage buildings due to the limited interventions that are made to the structural system [10].

The base isolation process consists of decoupling the superstructure from the horizontal displacements of the ground and creating additional damping effects into the system, through an isolating system which possesses sufficient vertical rigidity and high horizontal flexibility [11,12]. In this way, during the seismic action, the displacements will occur at the isolation system level, leading to a significant decrease of the relative displacements and loads induced to the superstructure. Among the strengthening solutions that can be applied to the masonry heritage buildings, the base isolation method is probably the most suitable one with respect to the previously mentioned set of requirements that are given in the international conventions and which must be considered when deciding on the opportunity of intervention. The most important advantage consists in the fact that the interventions are made only at the infrastructure level. Therefore, eventual damages that might be produced to the architectural or artistic components (interior paintings, frescoes, exterior paraments and decorations, etc.) can be successfully avoided [13].

2.2. Base isolated heritage buildings in the world and in Romania

The base isolation principle, even if it nowadays consists of modern and complex materials or techniques, has been used from ancient times [14,15]. As an example, the foundations of the Temple of Artemis in Greece, which is one of the largest temples and is included in the list of the seven wonders of the ancient world, were made of large stone blocks separated by a thin layer of clay, mixed with ashes and charcoal [16]. This structural innovation probably represents the first step in applying the base isolation system. Based on similar approaches, other types of constructions were base isolated in ancient times (Chinese bridges, temples and monasteries, Inca walls and several Roman temples) [14,15].

Up to the present times, there is only a very limited number of heritage church buildings that have been base isolated, most of them being located in Italy, while in Romania there are none, so far. One of the first modern base isolations has been applied to the San Giovanni Church, located in the province of Perugia, Italy, in 2004. The stone masonry building was severely damaged during an earthquake in 1997. The adopted base isolation system consisted of 8 High Damping Rubber Bearings - HDRBs and 6 Sliding Devices (SDs). The isolation devices were positioned between the carrier frame, made of reinforced concrete, and the initial foundations, which were continuous under the stone walls. The procedures of installing the isolators were done by using pairs of hydraulic jacks, positioned on each side of the bearing device [17].

Mele et al. [18] presented a case study focusing on specific particularities of base isolation system for heritage buildings. Four masonry church buildings were selected, (San Giovanni a Mare – SGMR, San Ippolisto – SI, San Giovanni Maggiore – SGM and San Paolo Maggiore – SPM) and certain design approaches of the base isolation systems were analysed and discussed. Two of the churches were relatively small and were considered as light-weight structures, thus the base isolation system consisted of a limited number of isolation devices. Thus, for SGMR church, having maximum in-plan dimensions of 19.5 × 38.2 m, 28 High Damping Rubber Bearings (HDRBs) isolators, with different diameters (Ø550 – 20 pcs., Ø650 – 4 pcs. and Ø750 – 4 pcs.), and 10 SDs isolators have been selected. For SI church, having maximum in-plan dimensions of 22.9 × 46.2 m, the base isolation system was composed of 28 HDRBs isolators (Ø600 – 16 pcs., Ø700 – 2 pcs. and Ø800 – 10 pcs.) and 5 SDs. For the other two church buildings, having similar shapes but larger dimensions, of approximately 37.5 × 66.5 m, the base isolation system resulted in 57 HDRBs (Ø600 – 20 pcs., Ø700 – 14 pcs., Ø800 – 7 pcs., Ø900 – 8 pcs., Ø1000 – 4 pcs. and Ø1250 – 4 pcs.) for SGM church, and 74 HDRBs (Ø600 – 20 pcs., Ø700 – 26 pcs., Ø900 – 16 pcs. and Ø1100 – 6 pcs.) for SPM church. The fundamental periods of vibration for the non-isolated structures were in the range of 0.45–0.70 s on transverse direction, and 0.39–0.55 s on longitudinal direction. After the base isolation systems were applied, the dynamic properties have changed considerably. The calculated fundamental periods of vibration and the participating mass factors had values of 2.95 s/100% for SGMR, 3.10 s/97% for SI, 3.10 s/99% for SGM and 3.30 s/99% for SPM.

Other examples of base isolating the heritage buildings are presented in [15]. The paper introduces some basic concepts regarding the application of the base isolation system to both new and existing buildings. The authors emphasize the case of heritage buildings in Italy that are severely damaged due to the large number of earthquakes. Moreover, they highlight the fact that the base isolation method, framed as “the real challenge of seismic isolation”, can be successfully applied to this type of buildings, despite the increased level of difficulty in both design and site execution stages. Some of the heritage buildings with masonry structures presented in the paper have been severely damaged after the L’Aquila earthquake in 2009, such as: the historical Palace Ciuffini-Cricchi-Volpi was isolated with 28 HDRBs isolators having diameters of 550 mm and 25 SDs isolators, both being installed between the new levels of sub-foundations made of reinforced concrete, yielding the isolated fundamental period to 2.02 s and the maximum displacement to 146 mm; villa La Silvestrella was base-isolated by using 25 HDRBs isolators with diameters of 450 mm and 23 SDs, allowing a maximum displacement of 300 mm and yielding the fundamental period of vibration to 2.35 s; Emiciclo palace was also base-isolated by using 61 HDRBs and 47 SDs, allowing a maximum lateral displacement of 300 mm.

A new seismic isolation system, called “Seismic Isolation Structure for Existing Buildings” (SISEB), was presented in [19]. The system consists in an isolated platform, made of two reinforced concrete

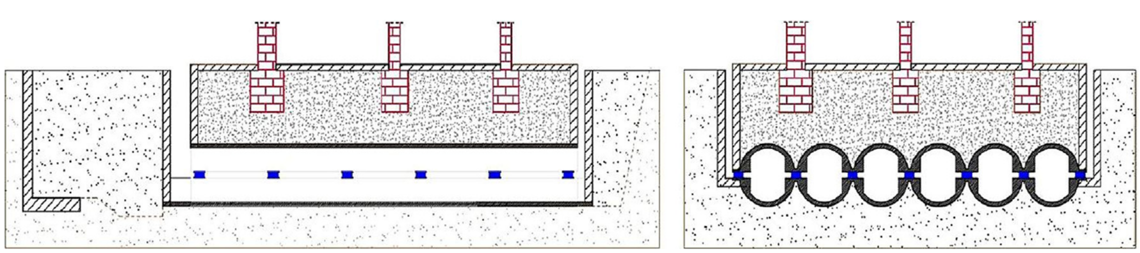

Fig. 2. Longitudinal and transverse cross-sections of the base-isolation system for existing buildings [19].

cylindrical elements/sectors, inserted under the existing foundation of the building, with no direct contact between the latter (Fig. 2). The construction stages of the system consist of:

– excavating a trench on one side of the building and installing the 2 m diameter pipes (in order to allow further or later maintenance works);
– removing the connection elements from the corresponding position of the isolation devices, joining the adjacent reinforced concrete pipes and installing the isolation devices;
– removing the remaining connection elements, thus separating the upper and the lower cylindrical sectors;
– the final stage consists of executing the boundary vertical walls and a rigid connection between the building and the isolation devices. Also, some technological openings need to be created, in order to execute future maintenance works or to replace the seismic isolators.

3. ST. Nicolae Aroneanu church – Short presentation

From functional and architectural points of view, the Orthodox Church buildings are characterized by certain particularities, as presented in Fig. 3, with respect to their shape, structure, types of elements etc. The vertical structural system of St. Nicolae Aroneanu church, erected in 1594, consists of brick masonry walls, being 1.20 m thick. At the ground level, the walls are resting on continuous footings made of cut stones, bound with lime mortar, having thicknesses similar to those of the above brick masonry walls. The foundations of the church are made of raw coarse stone blocks (limestone and sandstone), bound with lime mortar, having a depth of approximately 1.60 m, being 1.9–2.15 m below the present-day ground level. All horizontal structural elements of the church, consisting of domes, arches and pendants, are made of bricks with nominal dimensions of (13.5–14) × 28 × 5 cm. For the domes, the bricks were arranged askew with respect to the radius, resulting in a thickness of almost 20 cm. The dome over the narthex has a variable thickness, decreasing from the base to its central top part. The tower of the church starts from the level of the nave dome and it is also made of brick masonry [9].

The structural system is characterized by a set of particularities demonstrating that the initial craftsmen who built the church were aware of the important seismicity of the site and tried to empirically design the building in such a way that it could withstand the seismic actions. The in-plan layout of the church is symmetrical with respect to the longitudinal axis and has a rectangular shape, with lateral apses extended with less than ¼ of the nave span. The church is seismically adapted on vertical direction also because the transition between the walls of the nave and the tower is done in a progressive manner, through two consecutive horizontal bases, the first one being square-shaped and the second one being star-shaped.

This configuration provides a gradual transition from the rigid level of the nave walls to the slender (more flexible) level of the tower. Moreover, the filler which covers the exterior surfaces of the arches and domes, even if it represents an extra load that leads to higher seismic forces, has a well-defined function of increasing the axial compressive forces in these elements, thus improving their stability and decreasing the lateral displacements. Furthermore, this filling material is partly responsible for obtaining a superior horizontal rigidity at the roof level, hence improving the general seismic response of the building. Another specific element consists of wood tie-rods that were introduced into the massive masonry walls for improving their ductility [9].

During the site investigations that were made in order to establish the damage extent of the church building as a result of the previous

Fig. 3. St. Nicolae Aroneanu church: northern façade, the interior tower, 3D model – isometric view, plan view of walls and vaults, longitudinal section.

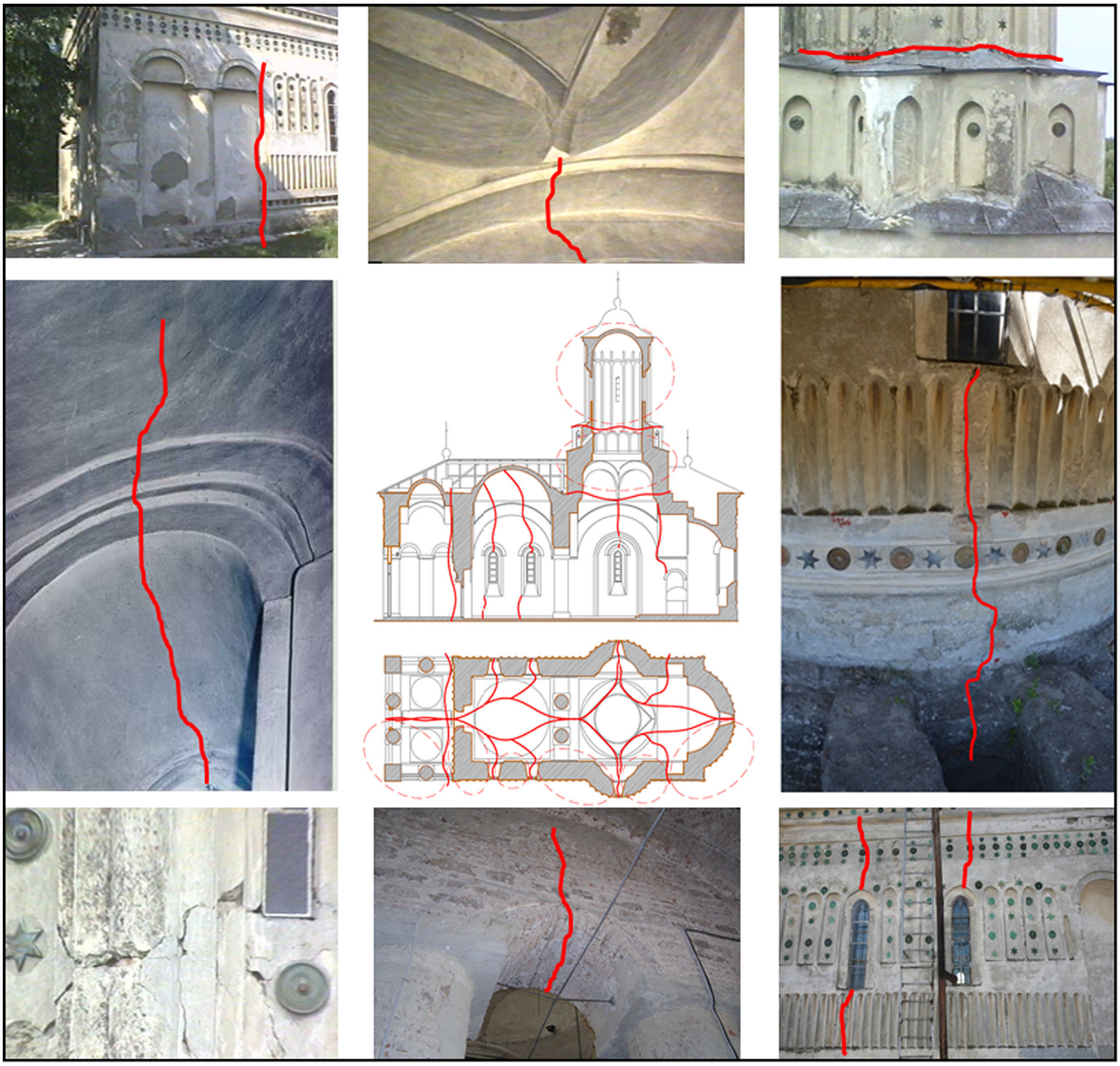

Fig. 4. Failure mechanisms for the St. Nicolae Aroneanu Church [9]. General longitudinal fracture of the church and multiple transverse fractures of the porch, narthex, nave and sanctuary.

seismic events, it has been observed that the cracks and fissures occurred in the most vulnerable elements [20,21], both vertical and horizontal structural components, and correspond to the typical damage mechanisms of the orthodox churches (Fig. 4), demonstrating important losses in terms of structural redundancy [22,23,24]. Thus, the general longitudinal fracture, extending from the porch up to the sanctuary, proved that the nave was divided in two quasi-symmetrical parts, while the consecutive transversal fractures that occurred in the less rigid areas of the porch, narthex, nave and sanctuary confirmed, as expected, that the structural system was additionally subdivided into multiple parts. Moreover, it has been observed that the porch tended to separate from the narthex, since the cracks in the longitudinal walls were continuously developing [25].

4. Structural rehabilitation solution

By evaluating the architectural and the artistic components of the church and by taking into account the damage extent of the structural system, it has been decided that the traditional strengthening procedures could not be applied for St. Nicolae Aroneanu church. Thus, the suitable approach consisted of decoupling the structure from the ground by installing a base isolation system. The design stage of the system is only briefly summarized in this paper due to content limitations. The complete design process will be presented in a future paper.

From a geometrical point of view, the church is characterized by maximum in-plan dimensions of 22.70 × 9.90 m, generating a built-up area of 176.60 m^2. The maximum heights of the nave are 7.80 m at the eave level and 9.90 m at the ridge level while the maximum heights of the tower are 17.60 m and 20.00 m, respectively. The total mass of the church is close to 15,300 kN. As it was previously mentioned, the structural masonry walls of the church are made up of bricks that are bound with lime mortar. Based on the investigations that were performed in the laboratory and on-site, consisting of both destructive and non-destructive test methods, the physical and mechanical properties of the masonry elements were obtained, and the corresponding design values were calculated (Table 2).

The seismic action was taken according to the Romanian norm P100-3/2008 [26], having an average reference return period of 100 years. The design peak ground acceleration (PGA) was considered as 0.2 g (196.2 cm/s^2) and the corner period, T_c, as 0.7 s. The behaviour factor, q, was equal to 1, appropriate for unreinforced masonry structures, while the importance factor, γ_I, was taken as 1.2. For the non-

Table 2
Physical and mechanical properties of the masonry elements.

Element	Property	Value
Bricks	Density	1646 kg/m^3
	Characteristic compressive strength	5.00 MPa
Lime mortar	Density	1643 kg/m^3
	Characteristic Compressive strength	2.50 MPa
Masonry walls as structural elements	Design compressive strength	0.51 MPa
	Design tensile strength	0.05 MPa
	Longitudinal modulus of elasticity	1400 MPa
	Transverse modulus of elasticity	560 MPa

isolated model, the normalized elastic response spectrum, β, had values of 2.75 and the damping, ξ, was considered to be 5%.

The non-isolated structure was first modelled in ETABS software, using shell elements. By running the preliminary linear elastic analysis, based on the response spectrum method, the maximum relative displacements were determined, being 10.2 mm on longitudinal direction and 34.1 mm on transverse direction. From the modal analysis, the dominant modes of vibration were identified. The first one consisted of translation and rotation along the transverse direction, while the second one of translation along the longitudinal direction. The fundamental period of vibration was 0.23 s and the mass participation factors for the first four modes of vibrations were 51% on longitudinal direction and 70% on transverse direction.

The iterative pre-design process of the FPS isolators and the selection of the corresponding appropriate values for radius and dynamic friction coefficient have been performed by setting target values for the fundamental period of vibration and for the maximum displacement at the ground level. The target period was selected in order to obtain a lower value of the normalized elastic response spectrum, β, resulting in smaller shear forces induced in the masonry walls, compatible with the total shear capacity of the structure. This process follows the criteria and the provisions given by the Romanian seismic design norm P100/1-2006 [27], that imposes a fully elastic behaviour of the base-isolated structure.

Thus, for FPS isolators with radius, R, of 4 m, friction coefficient, μ, of 3% and maximum displacement, D, of 250 mm, the effective stiffness of the base-isolation system and the effective vibration period of the isolated structure can be calculated with Eqs. (1) and (2), respectively.

$$k_{eff} = V\left(\frac{1}{R} + \frac{\mu}{D}\right) \tag{1}$$

where

V – total mass of the isolated structure

$$T_{eff} = 2\pi\sqrt{\frac{V}{k_{eff}\,g}} \tag{2}$$

where

g – gravitational acceleration

The effective damping of the isolated structure, ξ_{eff}, calculated with Eq. (3), was 20.6%. Hence, the correction factor of the damping, $\eta = \sqrt{\frac{10}{5+\xi_{eff}}}$, was 0.624.

$$\xi_{eff} = \frac{2}{\pi}\left(\frac{\mu}{\mu + \frac{D}{R}}\right) \tag{3}$$

Based on the displacement spectrum and by correlating the values of the damping and of the effective vibration period of the isolated system, the effective maximum displacement, d_{cd}, in each FPS isolator has been calculated, with Eq. (4). Therefore, the process has been reiterated until the conditions imposed by the target values for fundamental period and displacement have been fulfilled.

$$d_{cd} = S_d(T_{eff}, \xi_{eff}) \tag{4}$$

where

d_{cd} – design maximum displacement of the FPS isolator
S_d – displacement spectrum

For the isolated model, the preliminary analysis approaches were similar to those presented for the non-isolated case. However, since the effective vibration period increased to T_{eff} = 2.80 s, the new value of the normalized elastic response spectrum, was calculated (β_{Ti} = 0.69). Under the effect of the design seismic action, the effective relative displacements, at the FPS isolator level, were 193.5 mm on longitudinal direction and 197.6 mm on transverse direction. The relative displacements of the church (base level vs. top of the tower) were close to zero, hence insignificant. The modal analysis showed that the displacement response of the church has changed into an evident translation along the two principal directions and the the mass participation factors, for the first four modes of vibrations, were greater than 99%. However, additional linear dynamic time-history analyses have been performed, by using two artificial accelerograms that were scaled to fit the requirements of the site, in order to check if the effective relative displacements do not exceed the allowable values, imposed by the FPS isolators. The results proved to be in good agreement with the initial ones.

Thus, the constructive system that was applied consists of: a horizontal r.c. carrying frame being 30 cm below the level of the perimetral footway; a r.c. raft foundation under the existing foundations; the base isolation system being composed of 48 friction pendulum sliding (FPS) isolators installed between the two r.c. elements; ventilation and maintenance exterior culvert and a r.c. slab over the carrying frame (Fig. 5). The positions of the FPS isolators, with respect to the ground floor plan of the church, is presented in Fig. 6 while their general layout and technical characteristics are presented in Fig. 7 and Table 3, respectively.

5. The execution technology for the base-Isolation system

Since this project was the first seismic base isolation process of a historic heritage church building in Romania, the execution technology, consisting of both general approaches and in-detail procedures, have been carefully analysed, discussed and decided between the specialists that were involved in the project (design team, contractors, site supervisors, equipment manufactures, etc.). Therefore, the execution stages are presented below.

Stage	Description of the works
1	Execution of the top r.c. carrying frame, consisting in a network of beams at both interior and exterior faces of the existing foundations. In the position where the FPS isolators were installed, the interior and exterior beams have been connected by transverse concrete straps (beams) that cross the existing foundations.
2	Execution of the transverse r.c. straps at the level of the raft foundation, similar to those done in stage 1, used as temporary supports for the hydraulic jacks installed to sustain the top r.c. carrying frame.
3	Completing the r.c. raft foundation and the perimetral foundation beams.
4	Installing the FPS isolators (without loading), the lifting devices and the displacement monitoring systems.
5	Decoupling the superstructure from the existing foundations by horizontally chain-cutting the existing foundations and transferring the loads to the hydraulic jacks and to the raft foundation.
6	Uplifting the structure.

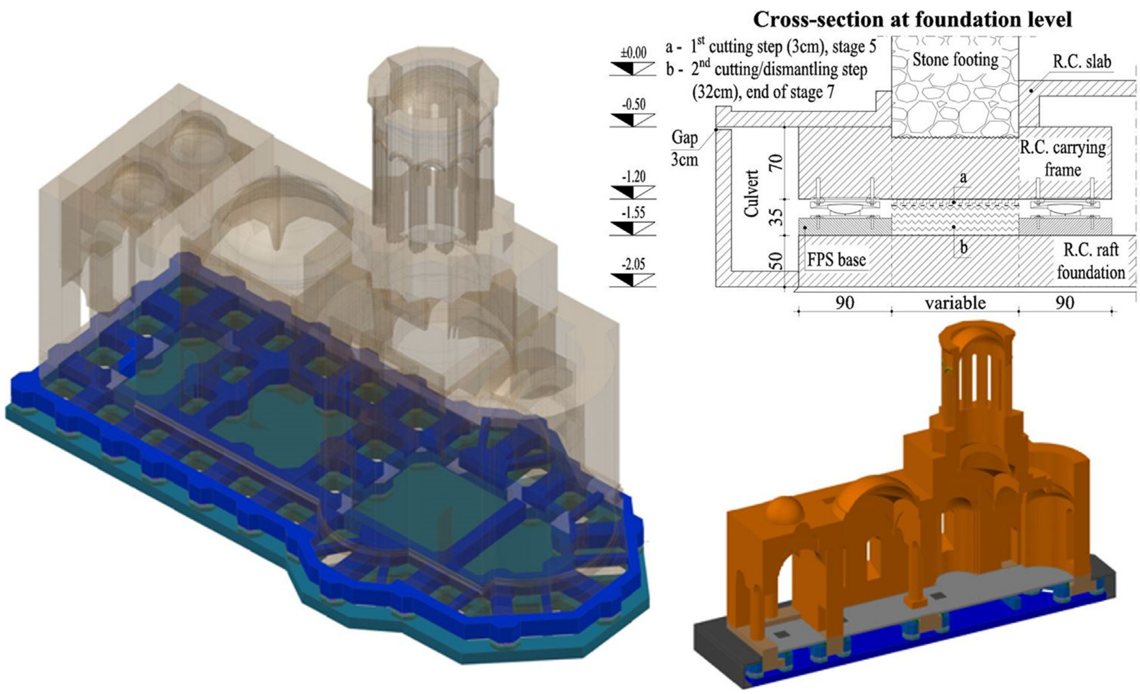

Fig. 5. Reinforced concrete carrying frame and raft foundation – isometric views and cross-sectional details.

7 Transferring the loads from the hydraulic jacks to the FPS isolators.
8 Execution of the interior r.c. slab, at the top part of the carrying frame.
9 Execution of the exterior culvert, used for ventilation and maintenance works.
10 Installing the structural monitoring system, that will be used during the in-service life of the building.

Stage 1. The top r.c. carrying frame was executed 30 cm below the sidewalk level, being composed of a network of beams at both the interior and exterior faces of the existing foundations. Additionally, the interior and exterior carrying frames have been connected by transverse r.c. straps that cross the existing foundations. In order to avoid the initiation of additional damages to the existing structure, the execution of the top r.c. carrying frame system has been divided into four main stages, with respect to the main functional compartments of the church (porch, narthex, nave, sanctuary). Moreover, each of the four main stages has been carried out into three sub-stages (Fig. 8). Thus, for each main stage, the first step consisted in cutting the channels in the existing foundation with a diamond-chain cutting machine followed by successive reinforcing and concrete casting the transverse straps. Next, the interior and exterior frames have been reinforced and casted. In order to avoid the technical limitations with respect to the positions of the concrete casting joints, the longitudinal reinforcements of consecutive segments of the carrying frame have been connected by parallel threaded couplers, instead of applying the traditional method based on overlapping. At the bottom faces of the interior and exterior beams, on the future position of the FPS isolators, steel plates have been embedded in concrete. Their position has been carefully disposed by topographic methods in order to ensure both longitudinal and transverse collinearity of the isolators.

Stage 2. After the top r.c. carrying frame was finished, local excavation were made in order to facilitate the execution of the bottom r.c. transverse straps, at the level of the raft foundations. The technology of execution was similar to that of the top straps. The bottom straps have been used as temporary supports for the hydraulic jacks that were installed to sustain the top frame while the general excavation was

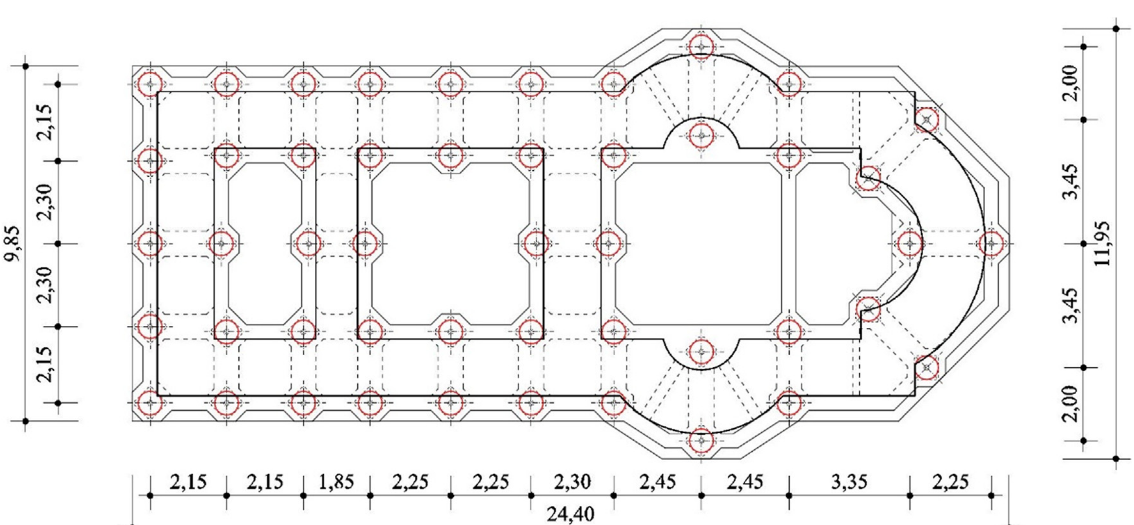

Fig. 6. Horizontal section – position of the FPS isolators.

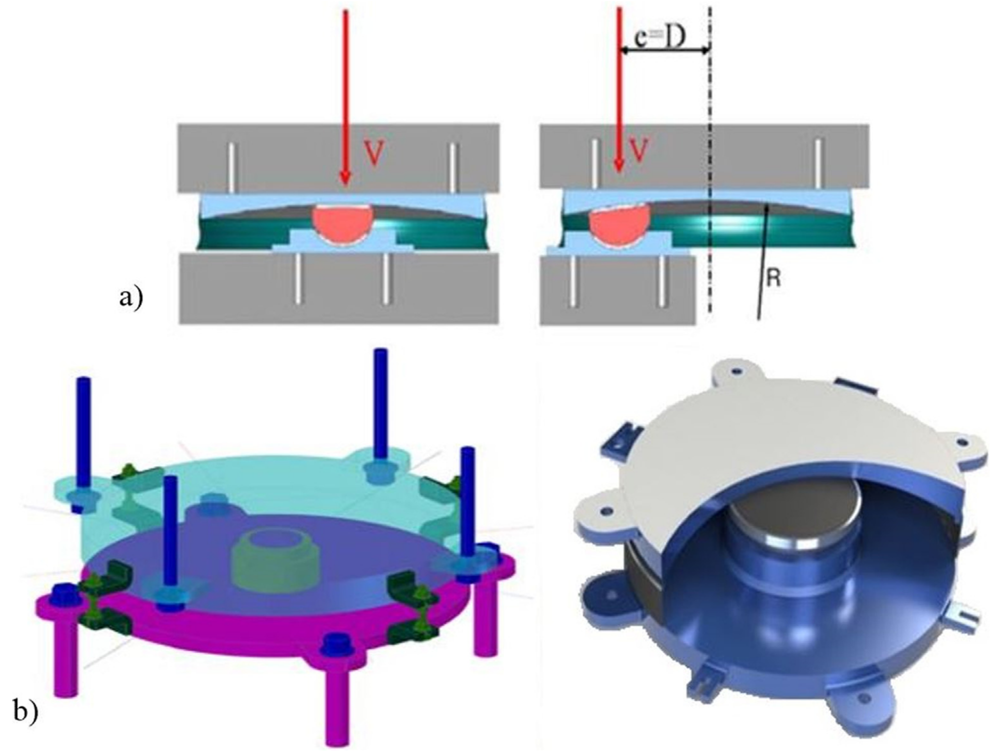

Fig. 7. FPS isolators – (a) sliding range (b) isometric view.

Table 3
Technical characteristics of the FPS isolators.

Characteristic	Notation	Value
Maxium axial forces	max N_{cap}	960 kN
N_{cap} – maximum axial force of the isolator	max N_{eff}	700 kN
N_{eff} – maximum axial force from the structure		
Maximum horizontal displacement	D_{long}	± 250 mm
	D_{trans}	± 250 mm
Rotation	max Φ	± 0.003 rad
Radius	R	4000 mm
Dynamic friction coefficient	μ	0.03

made, necessary for the execution of the raft foundation (Fig. 9).

Stage 3. The foundation system is composed of an interior continuous r.c. raft having a thickness of 50 cm, exterior perimetric continuous r.c. beams having cross-sectional dimensions of 70x50 cm and the transverse straps executed in stage 2 (Fig. 10). The main requirements of the foundation system consist of ensuring a uniform distribution of the loads to the soil level and, also, to possess sufficient stiffness in order to allow the FPS isolators to function properly. Thus, the maximum relative displacements of the foundation system were limited to 5 mm. Similar to the top r.c. carrying frame, the R.C. raft foundation system was also executed in 4 successive stages, similar to those presented in Fig. 8.

Stage 4. The FPS isolators were installed between the top carrying frame and the raft foundation, on the position of the steel plates that were embedded in the top beams. At the bottom part, the isolators were not in contact with the raft foundation since their base was executed in

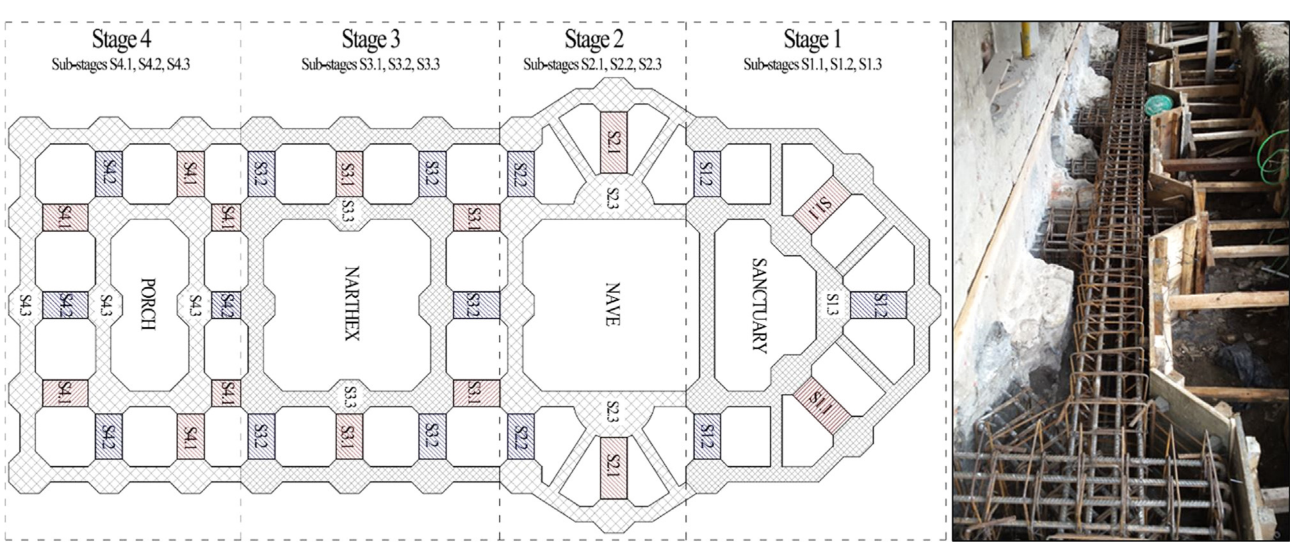

Fig. 8. Execution of the top r.c. carrying frame and transverse r.c. straps.

Fig. 9. Position of the hydraulic jacks and LVDTs.

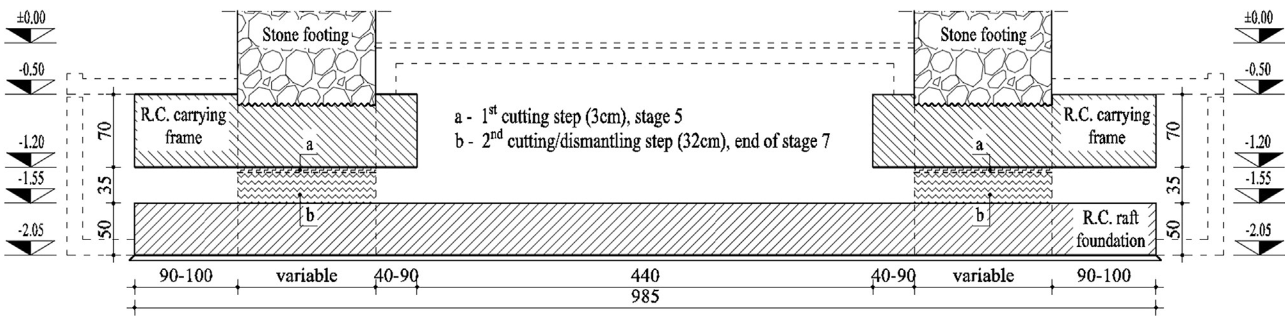

Fig. 10. Transverse cross-section at the foundation level.

a future stage. After the FPS isolators were installed, their position was precisely set with respect to the horizontal direction, by using a high-precision automatic digital level, allowing a maximum inclination of 0.3%. Also, in this stage, the lifting system was installed, being composed of 48 hydraulic jacks (Fig. 11). Each of the latter had a maximum lifting force equivalent to 700 kN. An acquisition system and an operating software have been specially designed for this project. Thus, the hydraulic jacks could be lifted or lowered individually, in groups or entirely, under force/pressure and displacement control.

All hydraulic jacks were instrumented with linear variable differential transformer (LVDT), having a maximum recording error of 0.01 mm, being connected to the same acquisition system, in order to constantly record the displacements between the superstructure and the infrastructure along the entire process of base isolation (Fig. 12). Besides that, an auxiliary displacement and settlement monitoring method has been implemented, being composed of 10 steel reference marks (RM1-RM10), socketed in the top and bottom r.c. elements, and 4 steady reference points (RS1-RS4), installed in positions that are not affected by the excavating works or by other technological processes. For the steady reference points, 4 r.c. piles were drilled and casted, having diameters of 300 mm and depths of 7 m. The steel reference points were embedded in the top part of the piles. The auxiliary monitoring method consisted in cyclic optical surveys of high precision, aiming to identify potential local or general settlements of the infrastructure elements or changes in the horizontal position of the building, especially during the load transfer stage (Fig. 13). Before chain-cutting the existing foundations, hence decoupling the infrastructure from the superstructure, the position of the socketed steel marks and the value of the LVDTs were recorded, being considered as equilibrium reference values during the lifting and lowering stages, since these values corresponded to the original position of the structure.

Stage 5. In this stage the infrastructure was decoupled from the superstructure by chain-cutting the existing foundations. The decoupling process was done progressively, by following a successive cutting order of isolated foundation plots, as presented in Fig. 14, in order to achieve a relatively uniform transfer of the loads to the hydraulic jacks. The existing stone foundation was cut with a diamond chain cutting machine, specially customized for this project. The dimension of the diamond chainsaw and the cutting speed were carefully selected in order to obtain a gap with a thickness of 3 cm (Figs. 5 and 10) and to minimize the vibrations that could damage the interior frescoes. After the decoupling process was finished, the superstructure was entirely supported by the hydraulic jacks.

Stage 6. In order to provide sufficient operating space between the top carrying frame and the raft foundation, necessary for completing the installation of the FPS supports, the superstructure was lifted. The operation has been done under displacement control, by imposing successive increments of 1 mm, up to a maximum value of 25 mm. Also, during the lifting process, the structure was brought to the initial horizontal equilibrium position, because minor local settlements were recorded by the LVDTs during the foundation cutting process, due to the load transfer from the existing footing to the temporary hydraulic jack supports (Fig. 15). Along the entire lifting process, the hydraulic jacks have been equipped with both mechanical and hydraulic blocking systems in order to avoid potential damages due to local pressure losses or even failures.

Stage 7. Once the structure has been lifted, the position of the FPS isolators was rechecked and, after minor calibrations, their base was executed. For the latter, a fluid micro concrete (class C30/37) has been used, having graded aggregates with maximum size of 8 mm and very low contraction and expansion coefficients (Fig. 16). After the concrete cured, the lowering process started. During this stage, the pressure in the hydraulic jacks was simultaneously decreased, being correlated with the displacements given by the LVDTs in order to ensure a constant and balanced descending process. After the load was entirely transmitted to the FPS isolators, the horizontal position of the superstructure was verified (based on LVDTs recordings and on topografic methods), aiming to determine if it corresponds to the one which was recorded in stage 4, considered as equilibrium reference value. The maximum relative deviations ranged between −0.25 and +0.30 mm.

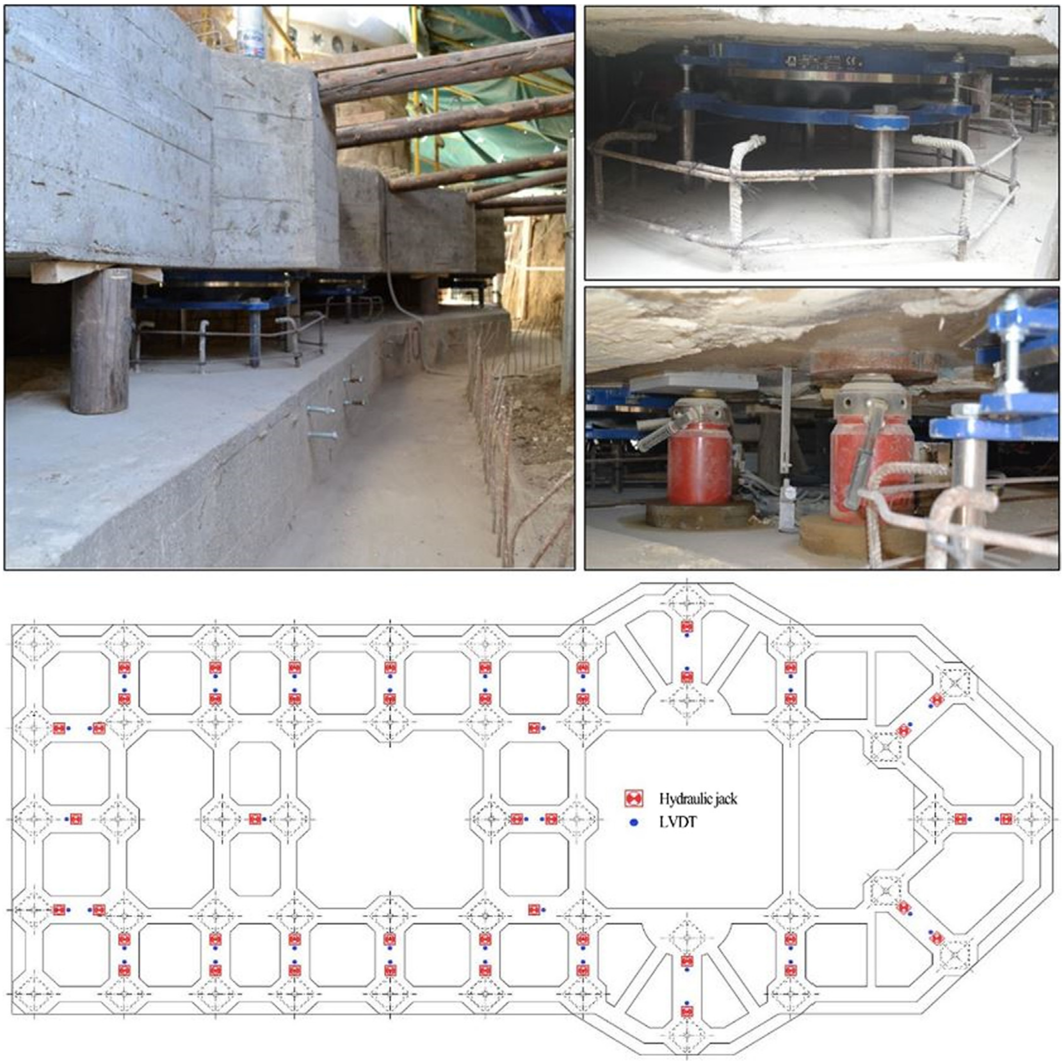

Fig. 11. Setting the hydraulic jacks and the LVDTs.

The last step of this stage consisted in dismantling the remaining part of the stone footing, between the top carrying frame and the raft foundation, until a clear seismic gap of 35 cm has been obtained, under the entire church (Figs. 5 and 10).

Stages 8 and **9** consisted in executing the interior r.c. slab, over the top carrying frame, and the exterior culvert, used for ventilating the infrastructure and for operating inspections and maintenance works at the FPS isolators. After the base isolation process has ended, the hydraulic jacks and the LVDTs have been removed and the church was instrumented with an additional monitoring system (**Stage 10**), that will be used during its in-service life, aiming to determine its real behaviour and response, under both gravitational and seismic actions. The system is composed of 8 3-D accelerometers, among which one is installed near the site of the church, inside a concrete pit for recording the local ground motions, and seven are installed on the superstructure of the church (five on the top surface of the r.c. carrying frame, one on the top side of the structural masonry walls of the nave and one on the top side of the structural masonry walls of the tower). The devices are connected to an acquisition system with a continuous program of recording.

6. Discussions and conclusions

The base isolation system that was applied to St. Nicolae Aroneanu church represents a major step forward for the Romanian civil engineering industry, since it is the first historic heritage church building that goes through such a process. Nevertheless, especially during the execution phase of the project, multiple and complex technological challenges were entailed. The most important challenge that had to be overcome during the installation process consists in ensuring the local and general equilibrium conditions of the church. The procedure of dividing the intervention works in consecutive sequences, carried out in delimited plots, proved to be a successful approach, since no damages were generated in the structure. Moreover, especially during the lifting and lowering stages, the integrated acquisition and control system had a critical function, because it enabled the possibility of adjusting the pressure in the hydraulic jacks based on the effective displacements recorded by the LVDTs. Thus, after the church was transferred to the FPS isolators, the initial equilibrium conditions were not altered, since its relative vertical position was identical to the original one.

Another essential feature of the rehabilitation process refers to the execution of an exterior culvert, at the infrastructure level, that improves the natural and mechanical ventilation, in order to avoid damp penetration and condensation effects which can seriously damage the interior and exterior frescoes [28].

Due to the isolation of the base, the structure has significantly improved its seismic response. The modal analysis of the isolated model demonstrates that the displacements of the church are similar to those of rigid bodies, consisting in translations along the two principal

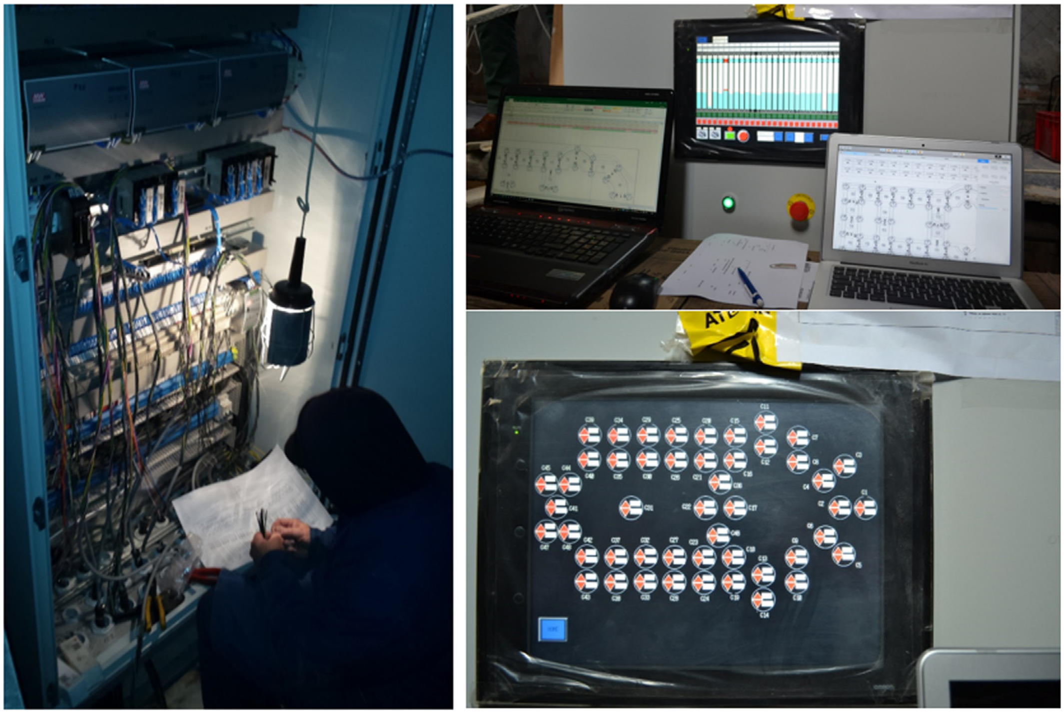

Fig. 12. Data acquisition system.

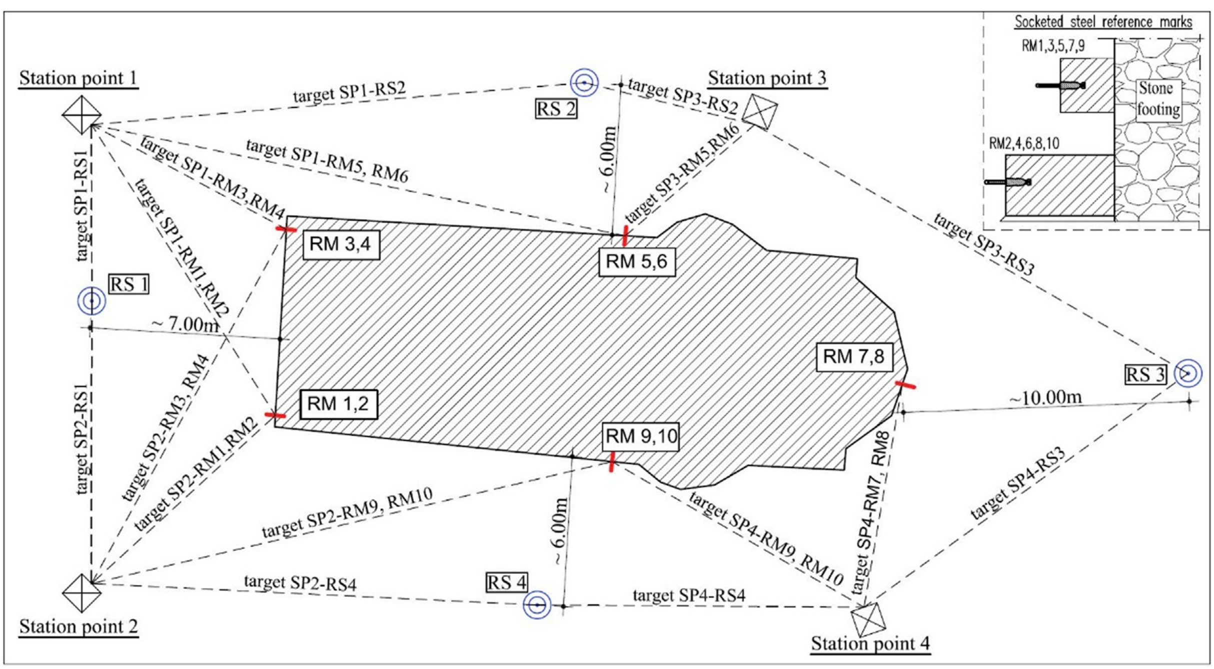

Fig. 13. Displacement and settlement measurements based on survey methods.

directions, and the mass participation factors have increased to 99%. Under the effect of the design seismic actions, the maximum displacements at the level of the isolators are 193.5 mm on longitudinal direction and 197.6 mm on transverse direction, both being smaller than the allowable ones, while the drift of the superstructure is insignificant.

Moreover, another important effect consists in reducing the effective shear forces at the base level of the building, due to the longer period of vibration ($T_{eff} = 2.80$ s) and to the enhanced damping (ξ_{eff}, = 20.6%). According to the Romanian norm P100/3-2008 [26], the seismic risk classes of existing buildings are established by evaluating their corresponding structural seismic safety level (denoted R_3). For unreinforced masonry structures, R_3 is strongly influenced by the ratio between the shear capacity of the structural walls at the base level and the effective shear loads produced by the seismic action. For the non-isolated case, the structural seismic safety level was 0.39 on longitudinal direction and 0.27 on transverse direction, ranking the church in the first seismic risk class, considered as the most dangerous one because it corresponds to imminent risk of collapse. After the base isolation system was applied, R_3 increased to 0.96 and 0.88, respectively, ranking the church in the third seismic risk class, characterized by minor damages induced to the structure in case of design earthquake occurrence.

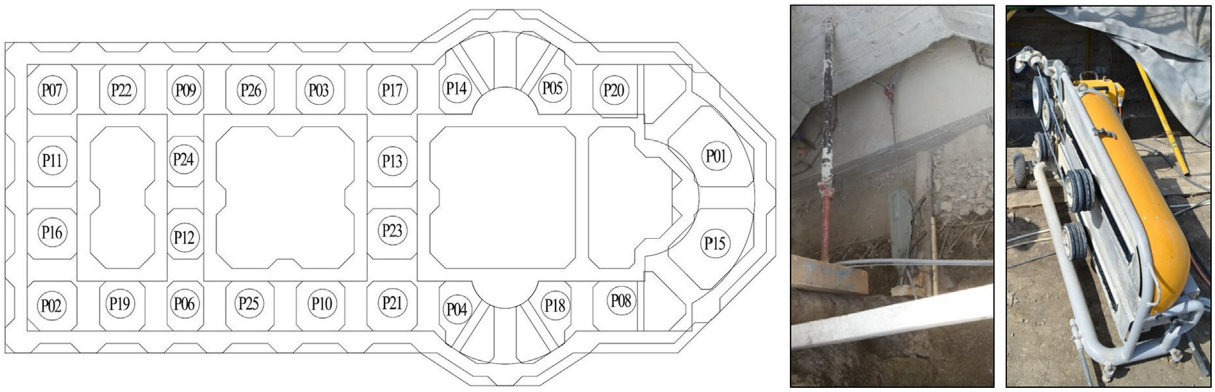

Fig. 14. Cutting sequence of the existing foundations and diamond chain cutting machine.

Fig. 15. Lifting process and the digital software for controlling the pressure in the hydraulic jacks.

Fig. 16. Casting the base of the FPS isolators.

Comparing the financial impact of the base isolation process, as a seismic protection solution, with that of the traditional strengthening techniques, there is no doubt that the costs of the first are considerably higher, since the total price of the project was close to 2200 EUR/m^2 (expressed in terms of built-up area). However, taking into account that St. Nicolae Aroneanu church comprises important architectural and artistic components (interior paintings, frescoes, exterior paraments) that could have been irreversibly damaged by applying traditional strengthening techniques, it becomes even more undeniable that the base isolation process was the appropriate procedure to reduce its seismic vulnerability.

Conflict of interest

None.

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