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Analysis and Design of a Base Isolation System for an Old Church with Masonry Structure

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Summary

The studies that consider the evaluation and the rehabilitation processes of historical masonry structures consist of structural analysis, aiming to better understand the seismic behaviour of these buildings, and also, may refer to the assessment of the safety level with respect to different rehabilitation strategies.

This paper present the case of an old historical masonry church for which a base isolation system has been developed. Aspects regarding the modelling process, finite element method analysis, modal and static linear analysis are presented. The efficiency of the base isolation system that has been designed for this specific structure is also verified using a time history analysis.

KEYWORDS: FEM, base isolation, masonry church, time history analysis.

1. INTRODUCTION

Historic buildings provide the most tangible legacy of our past civilization and in some cases they speak clearer than any remaining manuscripts. Historic masonry structures have low ductility, and, due to their stiff and brittle structural components, are usually severely damaged during strong earthquakes.

The main reason for damage is a lack of ductility that prevents a structure from being able to sustain the displacements and distortions caused by severe earthquakes. Damage caused by earthquakes to historic buildings is irreversible and these lost documents cannot be retrieved. The goal should then be to strengthen these structures in a manner that requires the least intervention and the greatest care to preserve authenticity. This goal, reflected in such conventions as the Venice Charter, poses real challenges with traditional masonry structures subjected to earthquakes [1].

Seismic isolation systems work by decoupling the building or structure from the horizontal components of earthquake ground motion by interposing a layer of low horizontal stiffness between the structure and its foundation. This layer prevents the transmission of accelerations from the ground to the building. This decoupling action is affected by mounting the building on a system of bearings that are stiff in

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the vertical direction (in order to support the building weight) but soft in the horizontal direction [2].

With base isolation, the earthquake energy that would have been transferred to the structure gets absorbed at the base level. In addition, the period of the isolated structure is increased which typically results in a reduction in seismic demands.

In these ways, ductility demand to the structure is greatly reduced. Displacement across the isolation system can be somewhat controlled by the addition of damping.

Seismic base isolation has so far been applied mainly in newly designed structures but has shown great promise in the improvement of existing structures with stiff and brittle structural systems. The advantage of this method is in the minimal intervention on the existing structure and the protection of architectural integrity. A disadvantage is the loss of the cultural layer and any archaeological remains as part of the base isolation installation. In the case of compact and exceptionally valuable historic structures of smaller proportions, base isolation becomes an acceptable, although expensive solution [1].

2. STUDY CASE

The studied church is dedicated to St. Nicholas and was built in Iasi, in 1592 on the site of an older church, erected by Alexandru Lăpuşneanu. Over time, the church was damaged and repaired several times.

The design objective for seismic strengthening of the church was to provide global and local performances that exceeded the requirements of P100-3/2008. The enhanced global performance targets at design earthquake (DE) and maximum considered earthquake (MCE) were:

- DE (475 year): Performance of between immediate occupancy (IO) and life safety (LS)
- MCE (2475 year): Performance of between LS and collapse prevention (CP)

These performance targets exceed the current common seismic retrofit requirements of LS and CP at DE and MCE, respectively P100-3/2008. Accelerations and drift ratios were reduced to level below the limiting values that initiate either in-plane or out-of-plane failure of vulnerable URSM walls.

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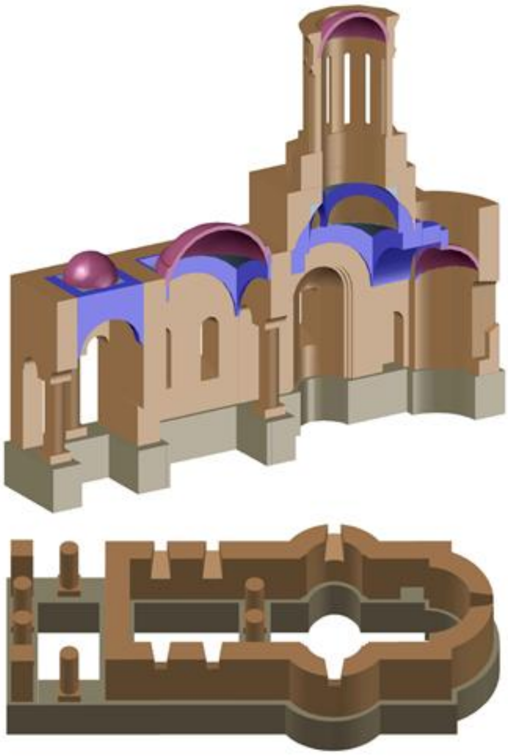

Figure 1. Isometric view of Aroneanu Church

Due to economic advantages (reduced prices) and the specific behaviour of friction pendulum isolators (the mass centre coincides with the centre of stiffness), they were chosen as the optimal solution.

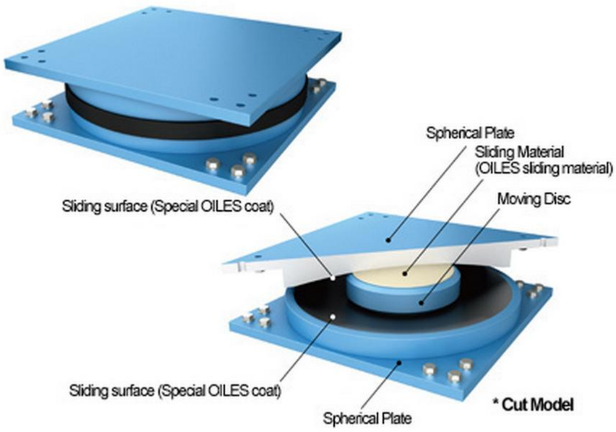

Figure 2. Spherical seismic isolators

The sliding isolation devices presented in figure 3 are consisting of three main steel parts with inner sliding surfaces. The shape of the internal part is always spherical and allowing rotations and horizontal sliding displacements as well. The device is transmitting the vertical loads and is providing free horizontal flexibility, while dissipating energy.

The proposal of the seismic isolators in plan layout, in accordance with geometrical considerations, stability and vertical loads balancing, is presented in figure 3.

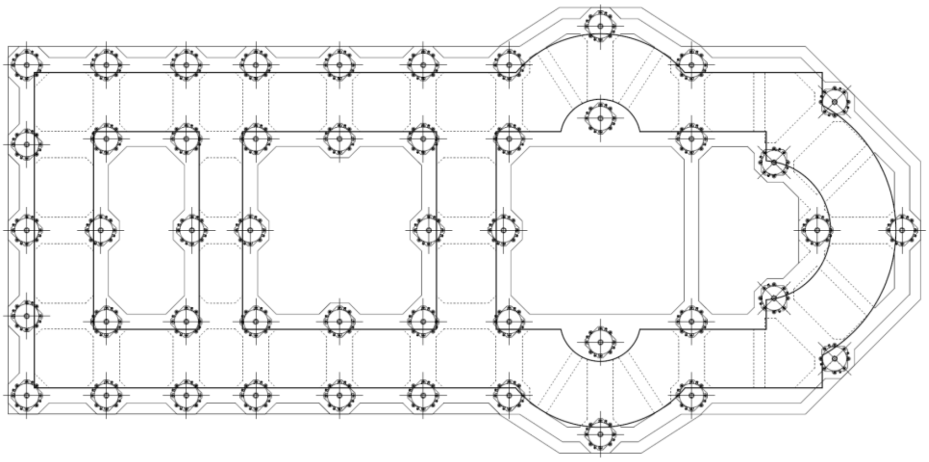

Figure 3. In plan layout of the seismic isolators

The seismic isolators were chosen (radius and friction coefficient) by iterative computation proposing a target period of isolated system and a maximum displacement, based on the following relationships [3]:

$$R = g * \sqrt{T/2\pi} \tag{1}$$

where:

- R is the curvature radius of the seismic isolators;

- T the period of the isolated system;

$$\xi = \frac{2}{\pi} * \left(\frac{\mu}{\mu + \frac{D}{R}}\right) \tag{2}$$

where:

- ξ is the equivalent damping of the isolated system;

- μ isolator friction coefficient;

- D maximum displacement of the isolators;

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$$k_{eff} = V * \left(\frac{1}{R} + \frac{\mu}{D}\right) \quad (3)$$

where:

- k_{eff} effective stiffness of the isolators;

- V is the total weight of the isolated structure;

$$T_{eff} = 2\pi * \sqrt{\frac{V}{k_{eff} * g}} \quad (4)$$

where:

- T_{eff} is the effective period of the isolated system;

3. MODAL AND LINEAR STATIC ANALYSIS

The analysis models were developed by using Etabs V9.7.2. software (fig. 4). The model contains 1500 shell elements both for walls and vaults, with a meshing step of 0.4m. The difficulties in the modelling process consisted of simulating the geometry of the church, with curved walls and vaults at different levels.

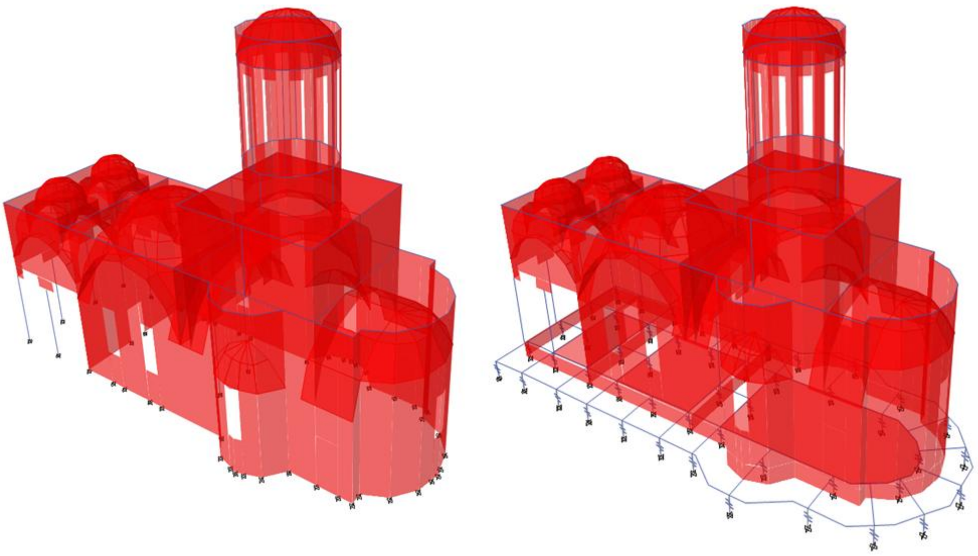

Figure 4. Structural analysis of Aroneanu Church

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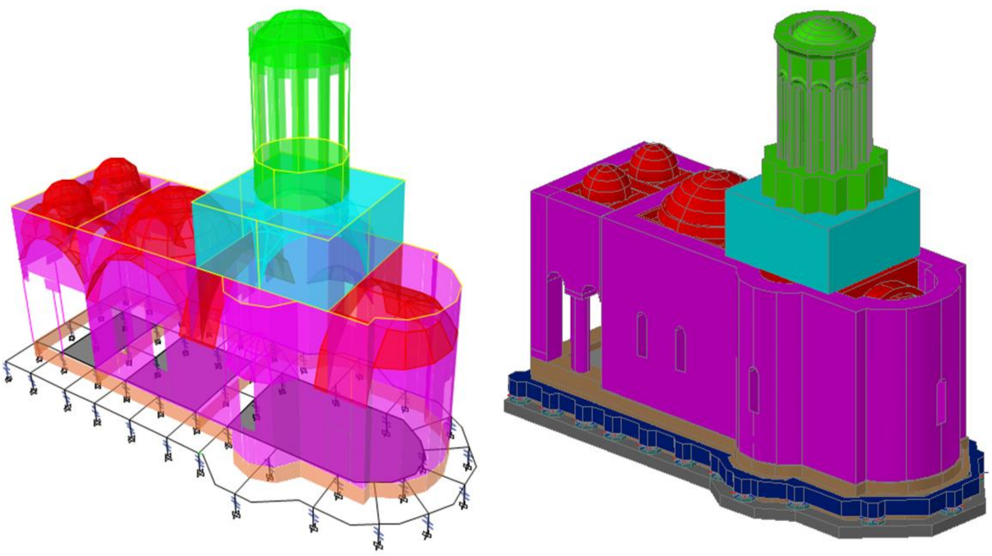

Figure 5. Model calibration

The calibration of the structural model was done by gradual correlation of the own weight loads with the three-dimensional survey of the structure (fig. 5).

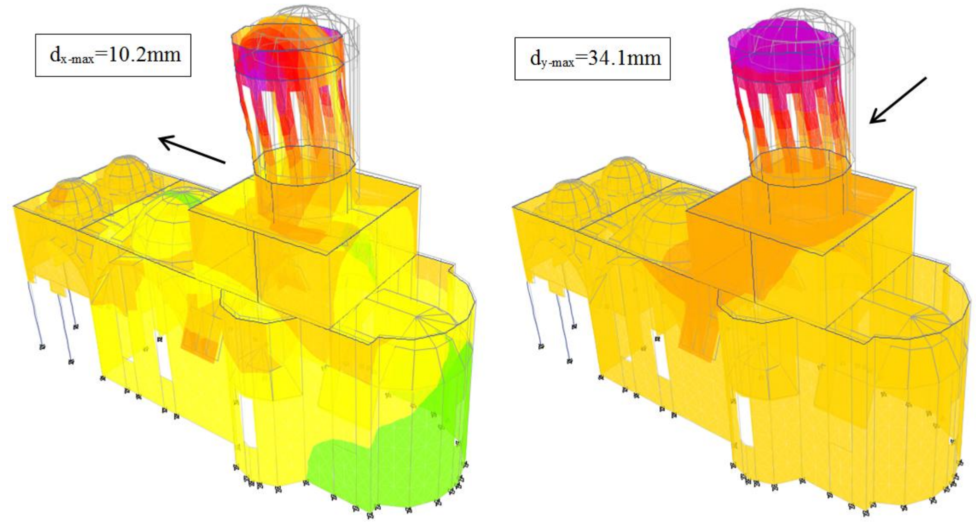

Figure 6. Maximum displacements from linear static analysis of Aroneanu Church with fixed base

The maximum displacement of the structure in the fixed base analysis have resulted at the upper level of the tower, with values of d_{x-max} = 10.2mm in the longitudinal direction and d_{y-max} = 34.1mm in the transversal direction (fig. 6).

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In the case of the isolated base analysis, the maximum displacements have been recorded at the base level, with values of $d_{x\text{-}max}$ = 193.5mm in the longitudinal direction and $d_{y\text{-}max}$ = 197.6mm in the transversal direction, lower than the allowable maximum displacement of the isolators (d_{max}=250mm) (fig. 7).

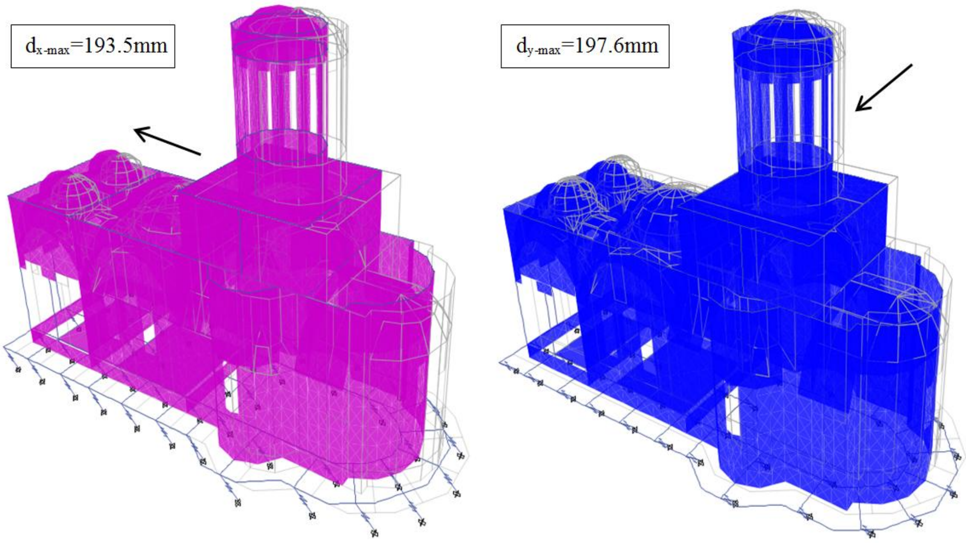

Figure 7. Maximum displacements from linear static analysis of Aroneanu Church with isolated base

2. CONCLUSIONS

Analysing the results from both fixed and isolated base analysis has been observed that modal participation factors for first 2 vibration modes in the case of the isolated base structure is 99% on both directions, whilst in the fixed base case, for the first 4 vibration modes, the sum of the modal participation factors is 51% on longitudinal direction and 70% on transversal direction.

The base shear force is reduced by 45% in the case of the base isolated structure.

The maximum displacements of the structure due to seismic action, modifies from 10.2mm and 34.1mm at upper level of the tower (fixed base) to 193mm and 197mm at the base level (isolated base).

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Table 1. Result from both fixed and isolated base structure analysis

Comparison criteria	Direction	Structure: Fixed base structure	Structure: Isolated base structure
Vibration mode / Period/ modal participation factors	x	2 / 0.22s / 7% 4 / 0.10s / 41%	2 / 2.78s / 99%
	y	1 / 0.26s / 14% 3 / 0.16s / 52%	1 / 2.82s / 99%
Sum of the modal participation factors (first 4 vibration modes)	x	51%	99%
	y	70%	99%
Base shear force	x	2221.64 kN	1165.39 kN
	y	2133.12 kN	1158.10 kN
Relative displacements (base-tower)	x	10.2 mm	0 mm
	y	34.1 mm	0 mm
Base displacements	x	0 mm	193.5 mm
	y	0 mm	197.6 mm

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