Seismic vulnerability of historical orthodox churches in Romania: numerical modelling and retrofit solutions

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Abstract. Historical orthodox churches in Romania are generally masonry structures, built without a clear understanding of earthquake demands. Past earthquakes have shown specific damage patterns for these buildings: damages in domes and bell towers, damages of the church nave (particularly a general longitudinal dislocation), damages of the infrastructure. As some of the existing buildings have high architectural or historical value, their structural strengthening is important. Typical failure mechanisms for churches under earthquake loads are identified. Based on these mechanisms, the methods for assessing the seismic vulnerability of existing churches are discussed. A historical listed church is considered as case study. The considered numerical model is presented. Qualitative and quantitative results obtained from the numerical model for the un-retrofitted building are analysed. A retrofit solution is proposed. Comparisons are made between the expected structural behaviour of the church in its present state and after structural strengthening. Conclusions are drawn regarding the seismic vulnerability of this type of buildings.

1. Earthquake damage patterns for orthodox churches
An important part of the cultural heritage buildings in Romania is represented by masonry churches. The preservation of these monumental buildings with cultural and artistic value is a stringent issue, considering their earthquake vulnerability. This vulnerability is both related to the particularity of the construction materials as to the specific structural and architectonic configuration. Masonry typically displays limited compression and low tensile and shear strength. For the old buildings, materials often deteriorated in time, when lacking proper maintenance and repairing works, due to mechanical stresses (in particular during earthquake events) and environmental actions, which lead to a further decrease of their initial strength.

In terms of geometry, orthodox churches have few transverse walls to connect the longitudinal facades. The walls are high and slender. The roofs are vaulted and most of them have the specific shapes of Byzantine vaults and domes with indirect support by the pillars through the pendentives. The bell towers are often slender. The combination of slender walls, flexible floors, inadequate connection between the different parts and eccentric position of bell towers make the churches particularly vulnerable to earthquakes.

Due to their specific characteristics, churches exhibit typical earthquake damage, as identified in [1] and [2]. The general deterioration pattern consists in three types of damages: damages of bell towers, damages of the church nave, damages of the foundations. The bell towers represent some of the most vulnerable elements, in many cases partial or even full collapse being observed [1]. As the towers are slender and can suffer a high dynamic amplification during earthquakes, important tensile and shear stresses can occur. Depending on their geometry and the shape of the window openings, failure areas are typically observed at the lower level of the windows or at the base of the tower (figure 1).
Damage of the church nave consists in longitudinal and transverse dislocations (figure 1). The longitudinal dislocation almost always extends from the altar to the porch, dividing the church in two halves. It is due to the lack of transverse walls, the lack of a rigid connection at the roof level and the out of plane forces induced on the façade walls by the vaulted roofs. The transversal fractures generally appear in the axes of the naos, narthex and porch [1] and are systematically observed in the weaker parts of the nave walls, more precisely near the narrow window openings [1], [2].

Figure 1. Typical earthquake damage patterns for churches and identification of some of the blocks in which the church is separated

The combined longitudinal and transversal dislocations lead to the fragmentation of the church into multiple independent blocks and failure mostly occurs out of the plane of the facades. Examples of such blocks are highlighted in figure 1 (right). The probable block geometry for different geometrical typical configurations of orthodox churches is studied in [3]. Sometimes the longitudinal and transversal fractures of the nave also propagate to the foundations. This is most often the case for the longitudinal crack at the base of the window in the axis of the altar [1], but it is also observed for the foundations of the longitudinal façade walls [1], [2].

2. Numerical modelling for the assessment of earthquake vulnerability of churches

The numerical modelling of historical masonry structures is an intricate issue. This is mainly due to the degrading nonlinear anisotropic cyclic behaviour of masonry and to the complex interaction between in plane and out of plane failure patterns. With the increase in computational power in recent years, it is possible to create refined finite element models that can reproduce this behaviour. As shown in [4], accurate results can be obtained by using numerical models where the non-linear behaviour of masonry is described by a continuum plasticity-based damage model. In these models, different strength, stiffness degradation and recovery effects are defined for tension and compression and the main failure mechanisms are tensile cracking and compressive crushing.

The difficulties related to this approach, besides the significant computational efforts, are related to correctly define the input data. In fact, for old complex monumental buildings, there are numerous uncertainties regarding both the material properties and the geometrical or stiffness characteristics. Regarding the materials, old buildings generally suffered multiple changes through time (local degradations, reparations or modifications using different materials, etc.) so that the material characteristics are not uniform as would be for new constructions. Even more, there is a difficulty related to measuring the material characteristics, as only limited destructive tests are possible. In terms of global parameters, the presence of structural elements with curved geometry (arches, vaults,
domes), the stiffness of the floors and the connection between orthogonal walls or between walls and floors highly influence the dynamic behaviour of the structure. Therefore, it is difficult to calibrate the FEM model as to correctly represent the actual building behaviour and there are uncertainties related to the obtained results.

For this reason, in many cases a simplified approach is adopted. As shown in [5], churches can be subdivided into macro-elements - architectonic elements characterised by a proper seismic behaviour, almost independently from the rest of the structure (facade, dome, bell tower etc.). For each macro-element, the damage modes and the possible kinematic mechanism of collapse can be identified. Complete numerical models can be obtained by defining the laws governing the connections between the various macro-elements. The studies described in [6], [7] and [8] conclude that macro-modelling are adequate solutions for the seismic assessment of unreinforced masonry buildings, as they require lower computational resources, allow easier interpretation of results and provide satisfactory accuracy when compared to the more complex FEM approach.

Still, for practicing engineers even more simplified methods might be necessary. Many commercially available structural modelling software, to which they are accustomed, often only allow considering the in-plane behaviour of masonry walls. Also, for the current structural design, linear models are often used. In the case of churches, where the seismic behaviour is dominated by the nonlinear out-of-plane response, it is obvious that such numerical models are not accurate for the evaluation of the structural safety of the existing buildings. Yet, a combined method can be easily used in practice. For the assessment of the out-of-plane failure modes for the initial building, the maximum forces for each macro-element can be computed as described in [9], without making use of complex nonlinear numerical models. The strengthening solution generally consists in introducing elements that stiffen the roof and tie together the walls such that out-of-plane failure is avoided, combined with new elements that strengthen the walls in their plane. Having this in mind, a numerical model that only accounts for in-plane failure can estimate with a good degree of accuracy the necessary strength and stiffness of the newly introduced structural elements. Such an approach is further described.

3. Case study: historical listed church in Buzău county

3.1. The existing building: Gârlași Church (St. Constantine and Helen)

The construction of the analysed church (as written in the carved stone above the door of the porch) began in 1780. During its existence, it suffered numerous damages due to earthquakes and fires. Following the earthquake of 1802, the vaults of the church broke down. In 1806 the church burned and was later repaired. In 1835, a new tower of the church was built, but during the 1838 earthquake several new cracks appeared in the walls. In 1932, extensive restoration works began, which lasted until 1935. During the 1977 earthquake, the church suffered further damage, mainly cracks in the two pillars inside the nave, cracking of interior and outer walls, serious damage at the ceiling of the porch and at the two vaults of the nave and the altar. To prevent further damage, the circular columns were clad in reinforced concrete and horizontal steel ties were installed at the base of the vaulted roofs in 1980.

3.2. Strengthening solution

In order to enhance the seismic strength of the existing building, new reinforced concrete elements connected to the existing masonry are proposed. Their position and dimensions consider the initial geometry, such that they do not affect the volume and the interior decorations of the listed building. A general connection between the walls at the roof level is ensured by creating a continuous tie-beam at the base of the vaults on the perimeter of the building connected by low-height transversal beams cast on top of the existing vaults (figure 2 - left and figure 3).

Vertical tie-columns are embedded in the existing masonry walls and clad on the exterior with brick masonry so that the exterior aspect of the building is unchanged. These tie columns (figure 2) are
continuous from the foundation to the top of the walls where they are connected by a tie-beam (figure 3).

For the bell tower, 8 vertical tie-columns on the interior of the existing walls are connected by horizontal tie-beams at the top of the wall, above the windows and at the base of the tower. These tie-columns rest at their base on transverse beams that transmit the charges to the exterior walls (figure 3). The existing foundations are strengthened by a new RC foundation beam placed on their outer side (figure 2 - right).

3.3. Numerical modelling
The numerical model was implemented using the Etabs software [10]. The masonry walls as well as the foundations were defined using shell elements. The vaults were defined using linear elements in order to account for the lack of bending connection at their base to the walls. The geometry of the wooden roof elements was not included in the model, an equivalent weight was defined on the vaults and walls. In figure 4 the geometry of the existing building (left) and the strengthening RC elements, highlighted in purple (right) are shown, as they were defined in the model.

In the strengthened configuration model, several construction stages were defined in order to account for the stress levels that exist in the masonry elements during the consolidation works. The analysis assumes elastic behaviour (considering the ductility factor). Nonlinear behaviour was only defined at the level of the foundations, as to account for compression only in the soil. The dynamic characteristics and the distribution of lateral forces were based on modal analysis. The load cases were defined using equivalent lateral forces along the height of the building and considering different predominant direction for the earthquake.
4. Results and discussion
Comparisons were done between the initial and strengthened situation in terms of stresses and displacements. Regarding the initial configuration, a qualitative analysis was done as to identify the most vulnerable elements. As shown in [11], the identification of the most vulnerable elements can be done, with a good degree of accuracy, based on the normalised virtual work. In figure 5 the potential failure areas for the walls and vaults, as determined based on this method, are highlighted.

Regarding the analysis in terms of stresses, both principal compression and tensile stresses were computed, for each load case, and the zones with high stresses were identified and analysed. The computed absolute maximum values of stresses are diminished by only a few percent after consolidation, but the extents of areas with high stresses is reduced.
In figure 6, the principal compression stresses diagrams are traced. The highlighted (purple) areas correspond to values higher than 2 MPa. The principal tensile stresses diagrams are traced in figure 7, where the highlighted (blue) areas correspond to values higher than 0.6 MPa. It can be observed that the elements where an exceedance of these chosen values can occur are substantially reduced after consolidation and therefore the risk of damage occurring in the existing walls is diminished.
Figure 6. Principal compression stresses in the masonry walls for the initial (left) and the strengthened (right) configuration.

Figure 7. Principal tensile stresses in the masonry walls for the initial (left) and the strengthened (right) configuration.

The axial and flexion forces in the masonry vaults were also analysed. In terms of pure axial forces (figure 8), the maximum efforts are practically equal before and after consolidation (the maximum computed compression stress value is equal to 0.65 MPa). In figure 8, the compression forces are shown, not the stresses.

Figure 8. Compression in the masonry vaults under earthquake loads for the initial (left) and the strengthened (right) configuration.
In terms of bending forces (figure 9), the strengthening solution leads to a diminishment of almost 50% of the maximum computed values in the initial situation. It is obvious that the risk of cracking or breaking of the vaults is greatly reduced after the intervention. In figure 9, the bending moments on the elements are shown, not the stresses. The maximum computed stress is 1.44 MPa.

![Figure 9](image)

**Figure 9.** Flexion in the masonry vaults under earthquake loads for the initial (left) and the strengthened (right) configuration

For the check of the foundations, a Winkler model (linear force-deflection relationship) was considered. No tensile forces in the soil are allowed in the numerical model. The strengthening solution for the foundations was chosen such that the maximum pressure on soil during earthquake is similar to the maximum pressure under vertical loads for the existing building. As previously stated, the numerical model took into account the stresses in the existing elements during the construction works. In figure 10, left, the computed soil pressure after completion of the intervention works is traced. It can be noted that a significant pressure exists under the old foundations, while the new foundations are less charged, due to limited additional deflection. A similar situation appears under earthquake loads (figure 10, right), where the maximum soil pressure values correspond to areas under the initial foundation beams.

![Figure 10](image)

**Figure 10.** Reaction on foundations after completion of strengthening works (left) and for one of the analysed earthquake load scenarios (right)
5. Conclusions
The historic masonry churches exhibit specific damage patterns under earthquake loads. Their seismic behaviour is very complex, being governed by nonlinear out-of-plane response. As shown in the literature, the vulnerability of this type of buildings can be precisely established using refined FEM models or more simplified macro-models. There are two significant problems related to implementing these methods in practical cases. Firstly, these methods are very sensitive to input data (geometric, stiffness or material properties) that is difficult to assess for historical buildings. Secondly, practicing engineers are more accustomed to using commercial software that does not always allow for defining complex material behaviour.

A simplified analysis method is conducted, that is based on the fact that strengthening solutions generally consists in introducing elements that stiffen the roof and tie together the walls such that out-of-plane failure is avoided, combined with new elements that strengthen the walls in their plane. Analysis is made on linear numerical models, considering code-based ductility factor and material properties. Construction stages are defined in order to account to stress levels in the existing elements. Compression-only behaviour in the foundation soil is considered. This solution can be used for assessing the strengthened situation, while for the initial evaluation it must be combined with a separate analysis of the out-of-plane strength for each macro-element. Results show that the strengthening solution is appropriate for reducing stress levels for vaults, masonry walls and foundations alike. The numerical models allow to conduct code-base deflection and strength checks for the masonry and RC elements. As this method is easy to implement it can be used in current practice for the design of rehabilitation works.

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