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Survey and dynamic behaviour of the Our Lady of Conception Church, Portugal

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ABSTRACT

The Our Lady of Conception church (Fig. 1) is located in village of Monforte (Portugal) and is not in use nowadays. The church presents structural damage and, consequently, a study was carried out. The study involved the survey of the damage, dynamic identification tests under ambient vibration and the numerical analysis.

The church is constituted by the central nave, the chancel, the sacristy and the corridor to access the pulpit. The masonry walls present different thickness, namely 0.65 m in the chancel, 0.70 m in the sacristy, 0.92 in the central nave and 0.65 m in the corridor. The masonry walls present 8 buttresses with different dimensions. The total longitudinal and transversal dimensions of the church are equal to 21.10 m and 14.26 m, respectively.

The survey of the damage showed that, in general, the masonry walls are in good conditions, with exception of the transversal walls of the nave, which present severe cracks. The arches of the vault presents also severe cracks along the central nave. As consequence, the infiltrations have increased the degradation of the vault and paintings. Furthermore, the foundations present settlements in the Southwest direction.

The dynamic identification test were carried out under the action of ambient excitation of the wind and using 12 piezoelectric accelerometers of high sensitivity. The dynamic identification tests allowed to estimate the dynamic properties of the church, namely frequencies, mode shapes and damping ratios.

A FEM numerical model was prepared and calibrated, based on the first four experimental modes estimated in the dynamic identification tests. The average error between the experimental and numerical frequencies of the first four modes is equal to 5%.

After calibration of the numerical model, pushover analyses with a load pattern proportional to the mass, in the transversal and longitudinal direction of the church, were performed. The results of the analysis numerical allow to conclude that the most vulnerable direction of the church is in the transversal one and the maximum load factor is equal to 0.35.

Fig. 1. Our Lady of Conception church

KEYWORDS: Masonry, church, survey, dynamic tests, numerical analysis.

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1 INTRODUCTION

The Our Lady of Conception church (Fig. 1) is located in the village of Monforte (Portalegre, Portugal) at 600 m from the downtown and outside the ancient medieval walls of the village, which previously protected the village of Monforte. This church is part of the complex of three churchs (church of *Calvário* and the church of *S. João Baptista*), which corresponds to the magical-religious triangle of the place of village of Monforte. The churches were metrically positioned in correspondence with the vertices of a triangle. The triangle is a geometrical shape used in the mystical and religious traditions, through which the central mystery of Christian is represented – The Trinity. Furthermore, the triangle together with the number three is symbol of the highest wisdom and of the perfect harmony, and represents the spiritual perfection and the Supreme Being.

Although currently it is not known the exact construction date of the churches, taking into account the documents present in the Municipal Archive of the Council of Elvas the Our Lady of Conception church was the first one to be built in the 17th century, followed by the church of *S. João Baptista* (18th century) and finally the church of *Calvário* (end of 18th century or beginning of 19th century). However, this chronology is not correct when compared with the documents of 18th century, namely the National Inquiry carried out after the Lisbon earthquake of 1755, in which it is verified that the church of *Calvário* already existed with the current configuration [Silva 2000]. In the absence of historical evidence that allow to accurately date the construction of the churches, the following historical context can be assumed: (a) Our Lady of Conception church: 17th century; (b) church of *Calvário*: 18th century; (c) church of *S. João Baptista*: end of 18th century.

The Our Lady of Conception church presents features that allow to identify it as a church of Manuelino-Mudejar style and is composed by the central nave, the chancel, the sacristy and the corridor to access the pulpit. The masonry walls present different thickness, namely 0.65 m in the chancel, 0.70 m in the sacristy, 0.92 in the central nave and 0.65 m in the corridor. The masonry walls present 8 buttresses with different dimensions. The total longitudinal and transversal dimensions of the church are equal to 21.10 m and 14.26 m, respectively. The central nave has 14.26 m and 6.33 m in the longitudinal and transversal direction, respectively. The maximum external height of the church is equal to 10.3 m. The Our Lady of Conception church is classified as a Public Interest Building and presents several pathologies in the structural and decorative elements, which require a multidisciplinary teamwork to evaluate the conservation status of the church, involving civil engineers, architects, restoration technicians, geologists and archaeologists.

This paper presents the work carried out on the structural behaviour of the Our Lady of Conception church, namely the survey of the structural pathologies, the dynamic identification tests and the numerical analysis.

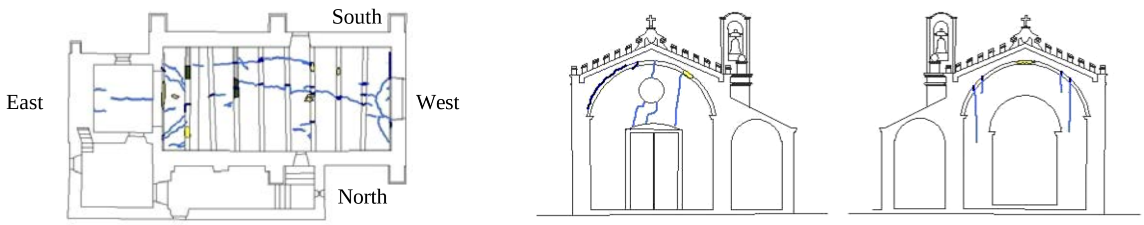
Figure 1. Our Lady of Conception church.

2 RESTORARION AND CONSERVATION

Several interventions of restoration and conservation on the Our Lady of Conception church were carried out. The interventions can be divided into two groups chronologically spaced in about 190 years: (a) interventions carried out between 1775 and 1840; (b) interventions carried out in 1973. In the first period, and due to the infiltrations of rainwater, minor repairs and replacements of tiles on the roof were carried out. These works were performed using traditional materials at that time. Furthermore, some paintings were painted again and some carpentry works were also carried out. For more details about these interventions see [Monforte Municipal Council 1774-1853]. In the second group of interventions (1973), the rebuilt of the roofing of the church was carried out by Directorate General for National Buildings and Monuments (DGEMN, Portugal). According to DGMEN [1973] the following works were delineated: (a) removal of roof in ruin and its structure; (b) demolition of some masonry elements of the roof; (c) construction of reinforced concrete ring beams; (d) construction of the roof slab (prestressed concrete rib and block slab), including the insulation with an asphaltic product; (e) reconstruction and repair of battlements and pinnacles; (f) repair of the gargoyles; and (g) whitewash painting. The works in the roof were carried out using modern materials and no compatible to the original materials, leading to the conclusion that this intervention was harmful for the conservation of the paintings. Besides these two main interventions, small repairing works with mortar were carried out in the past, mainly at the barrel vault and roof of the corridor.

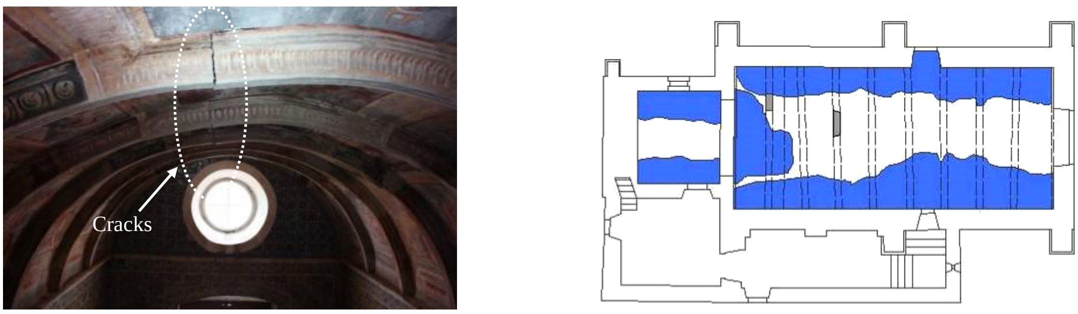
3 SURVEY OF CONSERVATION STATUS

The church presents a crack pattern with significant damage (Fig. 2), namely in the intrados of the barrel vault and in the transversal masonry walls of the central nave (façade and chancel arch). The arches of the vault presents severe cracks along the central nave, mainly near to the central alignment and South masonry wall (Fig. 3a). Furthermore, the vault presents many damp patches that were caused by infiltrations of rainwater from the roof (Fig. 3b). Over time, the infiltrations have increased the degradation of the masonry vault and paintings inside.



(a) (b) (c)

Figure 2. Crack pattern: (a) ceiling of the chancel and barrel vault; (b) façade; (c) chancel arch.



(a) (b)

Figure 3. Damage: (a) cracks at the arches of the vault; (b) damp patches.

In general, the masonry walls are in good conditions. However, the masonry walls present detachment of the plaster, mainly due to capillary rising damp (Figs 4a and 4b). The foundation presents settlements in the Southwest direction (Fig. 4c), which can be associated to the damage in the vault. The roof presents vegetation, mainly in the North side, and broken tiles. Furthermore, minor damages were also observed in the non-structural elements.

(a) (b) (c)

Figure 4. Details of the damage in the masonry walls and foundation: (a) plaster detachment; (b) capillary rising damp; (c) settlement of the foundation.

4 DYNAMIC IDENTIFICATION TESTS

Dynamic identifications tests were carried out, aiming at tuning a numerical model of the church. Ambient vibrations due to the movements of the building in the absence of induced forces were recorded to estimate natural frequencies, mode shapes and damping coefficients.

4.1 Configurations of the tests

The instrumentation required to conduct this work consists of 12 accelerometers (10 V/g, frequency range from 0.15 to 1000 Hz, dynamic range ± 0.5g), coaxial cables and one 24 bits data acquisition system with software developed by University of Minho, Figs 5a and 5b. In order to improve the quality of signals, sensors were installed on wooden bases bonded to the measurement points, see Figs 5c and 5d. The structure was studied under the action of ambient excitations; i.e. natural vibrations to which the structure is normally subjected (wind or traffic).

The accelerometers were placed at the points of the structure characterized by higher modal displacements, which have been previously identified through a preliminary FEM model of the church. Based on observations obtained with this model, the dynamic test was prepared according to three different configuration setups, where only horizontal accelerations were record. For the first setup seven accelerometers were used, while in the second and third twelve accelerometers were used. In each setup a different sensor arrangement was done, except for the first two sensors, which were positioned as reference points. In this way, all the signals record in different points at different times can be correlated, making possible to estimate the modal displacements in a higher number of points compared to the number of available sensors. The accelerometers were placed all over the structure, and in particular: (a) in the nave at a 5 m height from the base of the wall; (b) in the sacristy at about 3 m; (c) in the chancel at 3.8 m; and (d) in the corridor at about 2 m. The reference accelerometers were positioned at the centreline of the main façade in the longitudinal direction and in the lateral longitudinal wall in the transversal direction. On each setup 10 min of ambient vibration were record at 200 Hz sampling rate, in which a slight wind was exciting the structure.

(a) (b) (c) (d)

Figure 5. Dynamic identification tests: (a) and (b) details of the data acquisition system; (c) positioning of the wooden base of one of the points of measurement; (d) of the particular arrangement of the accelerometers inside the church.

4.2 Results

To process the acquired data the Stochastic Subspace Identification Principal Component (SSI-PC) method [Peeters & De Roeck 1999] was chosen. This method deals directly with time series (SSI-DATA, driven stochastic subspace identification), and it was used to estimate the modal parameters with high resolution. Table 1 presents the results in terms of natural frequencies and damping ratios. Fig. 6 presents the first four mode shapes.

As can be observed in Table 1, eight natural frequencies were estimated, ranging from 6.23 Hz up to 14.00 Hz. The Coefficient of Variations (CoV) for the three measuring setups is lower than 2.29%, indicating a good quality of the frequency estimates. In terms of damping ratios, an average value equal to 2.45% was obtained but with and average CoV equal to 25.76%. The higher range for CoVs indicates that damping was estimated with more difficulty and therefore its values has to be carefully take into account for further analysis. Concerning the mode shapes (Fig. 6), the first mode is the first out-of-plane bending mode for the longitudinal walls in the x direction at 6.23 Hz. The second mode mainly excites again the masses of the longitudinal walls in the out-of-plane direction at 8.17 Hz but now in opposite phases. The third mode at 10.09 Hz is the second bending out-of-plane mode for the longitudinal walls. The fourth mode activates the main façade in the longitudinal direction. As expected, the first modes mobilizes the heavy masonry walls on their lower stiffness (out-of-plane), especially on the transversal direction due to the geometry of the structural elements. The buttresses and the relative reduced height of the structure certainly contributed to the high value for the first natural frequency.

Table 1. Results of the modal identification tests by the SSI-PC method.

Mode Shape	f [Hz]	σ_f [Hz]	*CoV* [%]	ξ [%]	σ_ξ [%]	*CoV* [%]
Mode 1	6.23	0.04	0.71	1.85	0.36	19.46
Mode 2	8.17	0.03	0.42	1.72	0.20	11.63
Mode 3	10.09	0.10	0.96	2.21	0.77	34.84
Mode 4	10.46	0.24	2.29	2.52	1.44	57.14
Mode 5	11.62	0.08	0.68	2.73	0.43	15.75
Mode 6	12.84	0.10	0.78	1.64	0.48	29.27
Mode 7	13.69	0.18	1.31	3.91	0.11	2.81
Mode 8	14.00	0.12	0.86	2.90	1.02	35.17

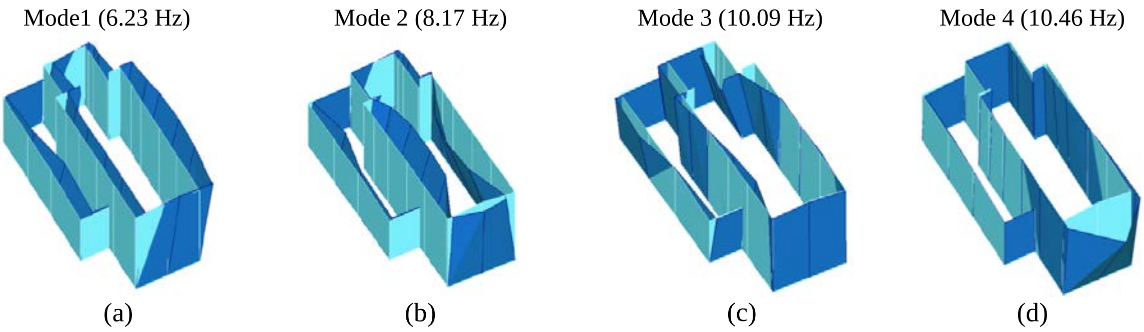


Figure 6. Experimental mode shapes for the first four modes.

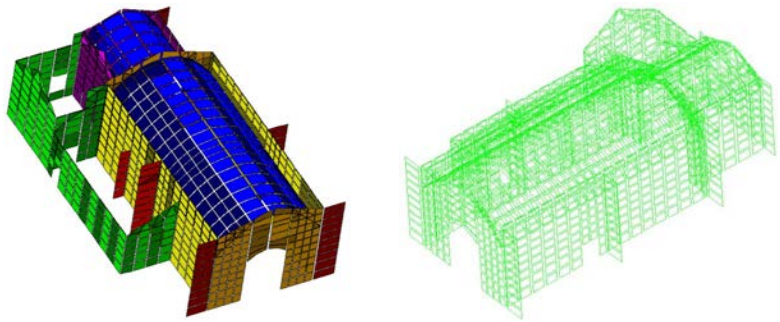
5 NUMERICAL ANALYSIS

To better analyse the structural safety under the seismic action, a numerical FEM model was prepared. First, the model was calibrated using the results of the dynamic identification tests, and later the safety was evaluated through the pushover analysis. The following sections present the preparation of the numerical model, the model updating analysis and the main results for the safety analysis.

4.1 Preparation of the numerical model

In case of historical masonry constructions, the difficulty of modelling the real mechanical behaviour of masonry (composite material with units and joints) is one of the main reasons to assume a (simplified) homogeneous material. Another difficulty is to simulate accurately the complex geometry, which leads to complex models with a high number of elements and long time consumed for running the non-linear analyses. Thus, simplified elements are commonly adopted for simulating the reality (e.g. flat shell elements to simulate masonry walls). Although these simplifications reduce the computation efforts, they increase the uncertainties in the calibration of numerical models, namely in the geometry and material properties, and connection between different elements.

The numerical modes was prepared using curved shell and curved beams elements, with quadratic interpolation, based on the the theory of Mindlin–Reissner, which includes the shear deformation. The curved shell is a more economical choice in comparison with the solid elements. The curved shell were adopted for the modelling of masonry walls, for the buttresses and for the roofs; while the curved beam elements were used for the modelling of reinforced concrete beams. Fig. 7 presents the numerical model. Fig. 7a presents the different materials considered for the later updating analysis, Fig. 7b the size of the mesh elements with the indication of the controlled points for the pushover analysis, and Fig. 7c a detail of the vault of the nave simulated with curved shell elements.



(a) (b)

Figure 7. Numerical model: (a) distribution of different materials (the roof is not represented and has different material properties); (b) FE mesh of elements.

4.2 Calibration of the numerical model

Based on the results obtained from the modal identification, an updating analysis to tune the numerical modal was carried out. In general, the model updating can be done by affecting various parameters: geometrical and/or mechanical. The goal of the update is to minimize the differences between the estimated and calculated structural response. During this process, a certain number of updating variables has to be selected to minimize the differences (e.g. the Young's modulus of masonry walls).

The model was calibrated according to the procedure described by Douglas and Reid [1982], which uses an optimization process to minimize the residuals between the experimental and the numerical response. Since in this case only dynamic response of the structure was known, frequencies were chosen to compute the residuals in the optimization process. It should be stressed that dynamic behaviour of a structure is strongly affected by changes in geometry, the variations of the boundary conditions, the mass distribution and the degradation of the mechanical properties of materials. Therefore, these aspects were taken into account in the selection of the updating variables.
Seven updating variables were considered, namely Young´s modulus of the transversal walls of the nave, Young´s modulus of the longitudinal walls of the nave, Young´s modulus of the vault, Young´s modulus of the walls of the chancel, Young´s modulus of the roof, Young´s modulus of the walls of

the corridor and sacristy and Young´s modulus of the buttresses (Fig. 7a). It is noted that the best results are achieved not necessarily through an increase in the number of updating variables to be updated. Furthermore, the values of each variable, as well as the respective lower and upper limits of their possible range, should be based on experimental data so that consistent results can be obtained.

The model updating was carried out taking into account the first four experimental modes. The results show that the calibrated model approaches to the dynamic behaviour of the structure, since the average absolute error between the experimental and numerical frequencies is equal to 5% and the experimental and numerical mode shapes are similar (Figs 6 and 8). As a result of the model updating the updated Young's modulus were obtained [Besca 2012].

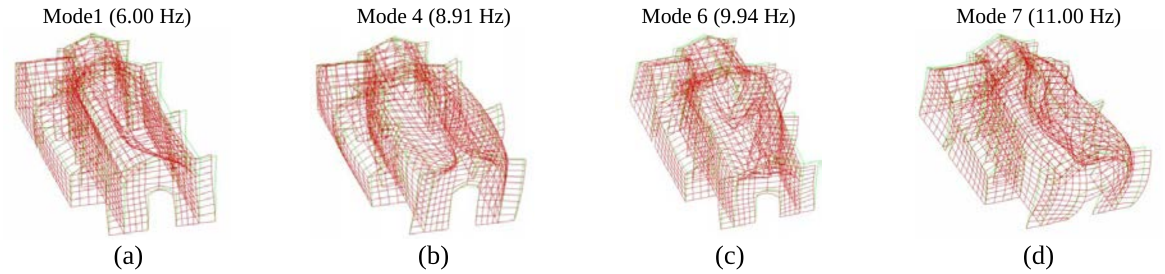


Figure 8. Numerical mode shapes.

4.3 Pushover analysis

Pushover analyses with a horizontal load pattern proportional to mass were carried, aiming at evaluating the dynamic behaviour of the church. For the material constitutive model the Total Strain Fixed Crack Model, which it is a model of distributed and fixed cracks based on the total strains, was adopted. For more details on the material properties see [Besca 2012]. Several pushover analyses in the positive and negative longitudinal (Y) and transversal (X) direction were carried out. Furthermore, the response of the structure was evaluated in several points of control, which presented similar behaviour. Fig. 9 presents the capacity curves of the pushover analyses in the longitudinal direction. The maximum load factor is equal to 0.60, which corresponds to a high value, and the displacement at the top of the façade is small (about 2 mm). These aspects are related to the high stiffness in the longitudinal direction (buttresses). On the other hand, the maximum load factor in the transversal direction is about 0.35 and presents higher deformation (Fig. 10). Thus, the results of the pushover analysis show that the transversal direction is the most vulnerable direction of the church and the maximum load factor is equal to 0.35.

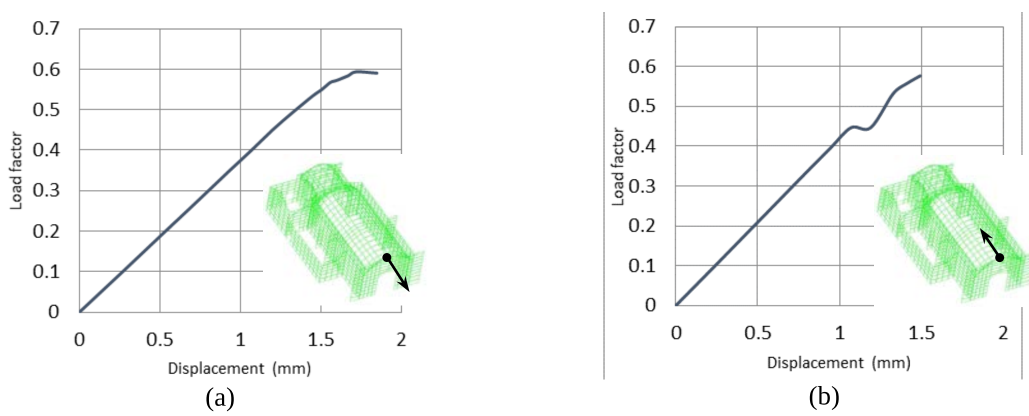
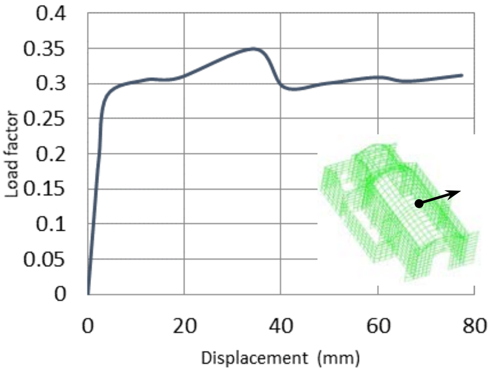
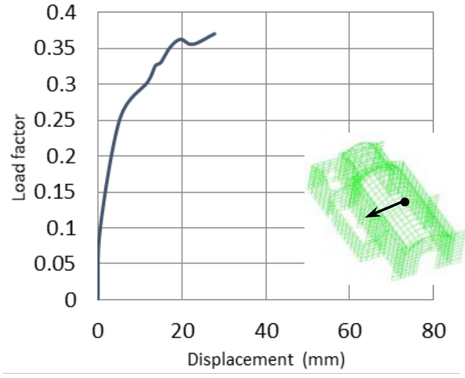


Figure 9. Capacity curves in the longitudinal direction: (a) - X; (b) + X.



(a)



(b)

Figure 10. Capacity curves in the transversal direction: (a) - Y; (b) + Y.

6 CONCLUSIONS

The paper presents a study on the Our Lady of Conception church located in the village of Monforte (Portalegre, Portugal). The work involved the survey of the pathologies, dynamic identification tests and numerical analysis. The survey of the structural pathologies allowed to conclude that the arches of the vault and the masonry transversal walls present severe cracks. Furthermore, the foundation of the church presents settlements. The dynamic identification tests allowed to estimate the experimental dynamic properties of the structures, which were used for the calibration of the numerical model. After calibration of the numerical model pushover analysis with a load pattern proportional to the mass were carried out. The results showed that the transversal direction of the structure is the most vulnerable one and the load factor is equal to 0.35.

As a future works, pushover analysis with load pattern proportional to the first mode shape and dynamic non-linear analysis with time integration should be carried out, aiming at concluding on the safety of the structure and influence of the existing damage.

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