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SEISMIC ANALYSIS OF THE SERRA DO PILAR MONASTERY CHURCH

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ABSTRACT

This paper presents part of the work developed for the seismic behaviour study of an ancient structure, over 400 years old. The church structure was numerically modelled via the finite element method, using a three dimensional model with parameters calibrated by experimental testing. *In situ* and laboratory tests on extracted samples were performed, and the structural stiffness was calibrated by dynamic tests for modal identification. The seismic analysis was performed in two calculation phases, using artificially generated accelerograms representative of the local seismicity. Results of complete linear elastic dynamic calculations allowed the analysis of the global behaviour and provided the input for more detailed local analyses of structural parts where the non-linear behaviour was considered. The seismic performance and vulnerability of the structure is addressed and discussed.

Keywords: Ancient structures; testing; modal identification; seismic analysis; non-linear behaviour; seismic vulnerability.

INTRODUCTION

The structural behaviour and safety assessment of ancient constructions is nowadays a matter of increasing concern due to the amount and value of historical heritage to be preserved.

The devastating effects and unforeseeable nature of earthquakes make the seismic action one of the most important of all the possible loadings that may be considered. Furthermore, the special features of seismic loading render even more complex the structural analysis because it often requires the study of the whole structure.

Notwithstanding the significant development of sophisticated behaviour models and testing techniques suitable for the analysis of ancient structures, several difficulties still persist mainly related with the geometric complexity and large dimensions of the specimens, the variability of material properties, the lack of knowledge about the original construction techniques and the possible subsequent interventions in the structure.

The present paper reports on the seismic analysis of the “Serra do Pilar” monastery church located nearby Porto, Portugal, a 424 years old monument that is a good example for the application of seismic analysis models in ancient structures of large dimensions.

Due to the geometric and structural complexity of the monument, a simplified analysis methodology was adopted. A first linear elastic calculation of the whole structure allowed the assessment of the global behaviour, after which the local response of some parts (arches and bearing columns) was further analysed by recourse to more detailed calculations involving the non-linear behaviour of joints between stone blocks.

The structural model was defined on the basis of the finite element method using the general-purpose computer code CASTEM 2000 (CEA, [1]). Detailed geometric data were available and generated so as to simulate visible stone blocks by means of solid elements duly individualized in order to allow modelling joints between blocks.

Mechanical and physical properties of materials were estimated by *in situ* and laboratory tests on samples extracted from the structure. Dynamic identification tests were also performed, allowing for calibration of the stiffness used in the global linear analysis. These tests were performed under both ambient vibration and micro-explosions produced nearby the structure, the modal identification of the structure having been done using frequency domain methods applied to the measured responses.

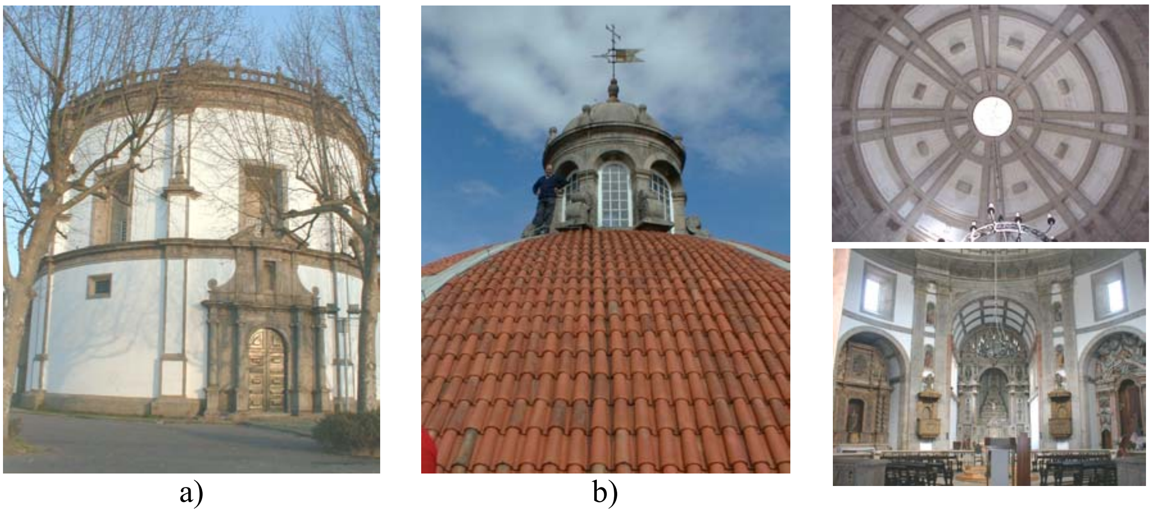
In the following sections the “Serra do Pilar” monastery church is first described, addressing both historical aspects and numerical modelling issues. The dynamic behaviour of the structure is then discussed, reporting on both dynamic test results and numerical analysis. Finally, the seismic analysis is presented, focusing on the adopted methodology and the main results obtained.

THE “SERRA DO PILAR” MONASTERY CHURCH

Historical background

The “Serra do Pilar” monastery was founded back in 1537 and is located in the S. Nicolau hill, in the historical centre of Vila Nova de Gaia. The construction works developed slowly, such that only in 1567 the first version of monastery was actually built. In 1598 it was decided to replace the church by a new and larger one that finally opened in 1678. Made of local granite stone, this cylindrical church, probably inspired in the Rome Pantheon as evidenced in Figure 1, was a rather innovative option in contrast with the current architecture by that time in the country.

Located right in the south bank of Douro river, in a strategic position facing the historical centre of Porto, the monastery was used as fortress during the French invasions and the liberal wars, after which it was found in a very degraded state even further aggravated during the 19th century. Since 1927, upon an almost semi-ruin state, the Portuguese institution for monument conservation (DGEMN – “Direcção Geral dos Edifícios e Monumentos Nacionais”) has been performing reconstruction and rehabilitation works in the monument, in order to preserve this remarkable piece of art from the renaissance.



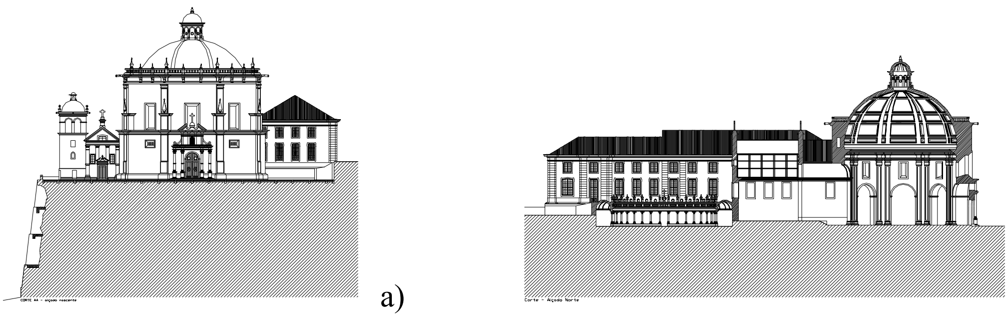
a) b) c) d)

Figure 1: The “Serra do Pilar” monastery church: a) Outside, b) the roof and the small upper dome, c) the dome structure and d) inside

Structural discretization and numerical modelling

The “Serra do Pilar” monastery church is a 29.6m high cylindrical shaped building made of stone block masonry, with an external radius varying between 15.7m and 14.6m, and covered by a 0.60m thick hemi-spherical dome of internal diameter 23.0m. On the top of the dome and laying on its inner ring, four columns 4.0m high and separated by windows give support to another small dome (Figure 1-b)) that completes the roof structure (Figure 1-c)). Inside there exist eight buttresses (16.8m high) placed in between small chapels (Figure 1-d)) defined by arches supporting the windows. In the main chapel zone there is a larger arch (13.4m high) adjacent to a 14m high vault covering the main altar zone developing along a rectangular shape area.

Outside and laterally, there are some other small buildings of minor importance for the present study. Figure 2 gives overall and schematic views of the church and surrounding constructions.

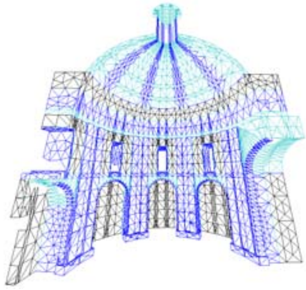


a) b)

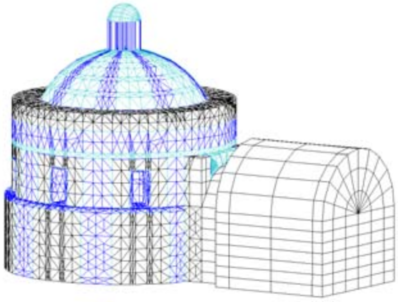
Figure 2: Schematic representations of the monastery. (a) Frontal elevation and (b) longitudinal section

The numerical modelling involved a first stage for geometric definition of the structural elements, based on available data from DGEMN and on additional topographic survey. These elements consisted of the buttresses, the arches, the ring above the arches and the dome, where the stone block pattern was clearly visible so as to allow a more rigorous discretization in order to respect the existing joints between blocks. The remaining zones were considered as filings, since they are plastered and made of very irregular stone masonry.

Based on the geometric and topographic data, half mesh of the church structure was defined (Figure 3-a)) using the computer code CASTEM 2000 (CEA, [1]), assuming structural symmetry relative to the vertical plan crossing the entrance and the main altar zone through the central axis of the dome. Then, using the symmetry operator available in the code, the complete mesh was obtained as illustrated in Figure 3-b), where both the stone blocks and the filling zones were simulated by four node solid elements, with due care to isolate each block elements. This procedure allowed joints between and above blocks in the arches to be discretized using joint elements based on three node triangular finite elements (Almeida, [2]).



a)



b)

Figure 3: Numerical modelling of the church. (a) Half-mesh (inside view) and (b) total mesh (outside view)

Material properties such as the mass density, Young modulus and Poisson ratio were first estimated by means of several laboratory and *in situ* experimental tests (Almeida, [3]). Values ranging from 20 to 25 kN/m^3 were adopted for the mass density, while for the Young modulus a wide range of values between 2 GPa (in the filling zones) and 20 GPa (in the visible stone blocks) was considered along with an average constant Poisson ratio of 0.2.

DYNAMIC BEHAVIOUR OF THE CHURCH STRUCTURE

Further calibration of the global structure stiffness was pursued by recourse to dynamic tests on the "Serra do Pilar" monastery church, from which modal parameters were identified and compared with numerically obtained ones.

Dynamic tests

The dynamic tests of the "Serra do Pilar" monastery church were performed with the key collaboration of the LNEC (National Laboratory of Civil Engineering). The testing campaign (Almeida, [2]) consisted in the measurement of accelerations induced in the church structure by ambient sources of excitation (like wind and traffic in nearby roads) and by explosions of low power detonators buried in the area surrounding the church.

Three tests were carried out, corresponding to different configurations of accelerometers located throughout the structure. Accelerometers were moved sequentially from the upper to lower levels of the structure, though keeping always some of them fixed in order to provide reference measurements. Amplitude and phase relations were obtained between signals recorded in the instrumented points, mainly referring to radial directions and therefore leading essentially to translation and ovalization modes of vibration.

For each test set-up, after recording the ambient vibration data, a micro-explosion test was performed, and the corresponding accelerations in the structure were recorded. An example of such results is included in Figure 4-a) for the ambient vibration and, with an enlarged time scale, in Figure 4-b) for the explosion and also the ambient vibration.

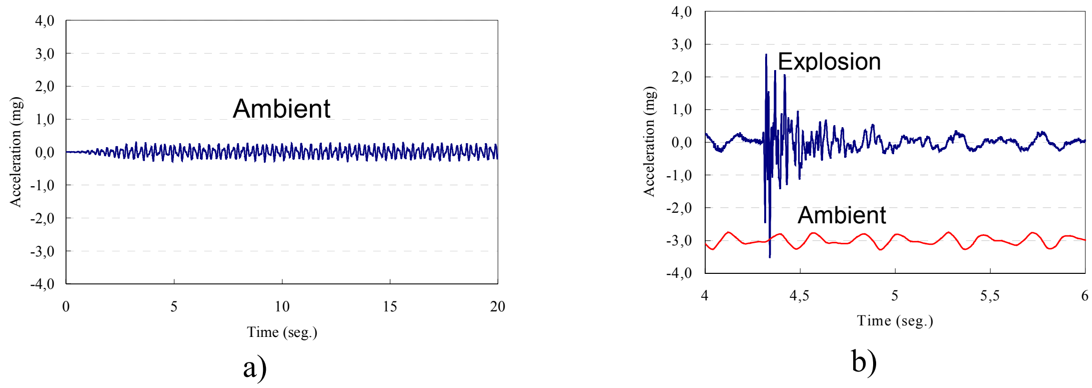


Figure 4: Dynamic tests. Response due to (a) ambient vibration and (b) explosion.

A quite stationary response was observed under ambient vibration (Figures 4a and 4b), which was somewhat unexpected for that kind of excitation. The accelerations recorded in the tests were analysed for modal identification purposes using two frequency domain methods as described with more detail in Arede *et al* [3]. Auto-spectra, coherence between different measurement points and H_1 estimates of the frequency response functions were obtained, as well as the average normalized power spectral density (ANPSD) that was computed taking into account the whole set of records.

Figure 5a shows the ANPSD plot in logarithmic scale where two predominant peaks were found for frequencies around 4.5 Hz and 6.8 Hz, together with other less evident peaks that were carefully analysed in terms of the coherence. In fact, Figure 5b shows the coherence of the H_1 function between the accelerations recorded at points in the same vertical plan, but in two different levels, namely at the top of the dome (level 4) and approximately at half-height of the buttresses (level 1). From these results it is evident that the coherence values are very close to 1 for frequencies around 4.5 Hz and 6.8 Hz, but also that, for frequencies of about 3.1 Hz and 3.5 Hz, there are still high values of the coherence, although lower than 1.

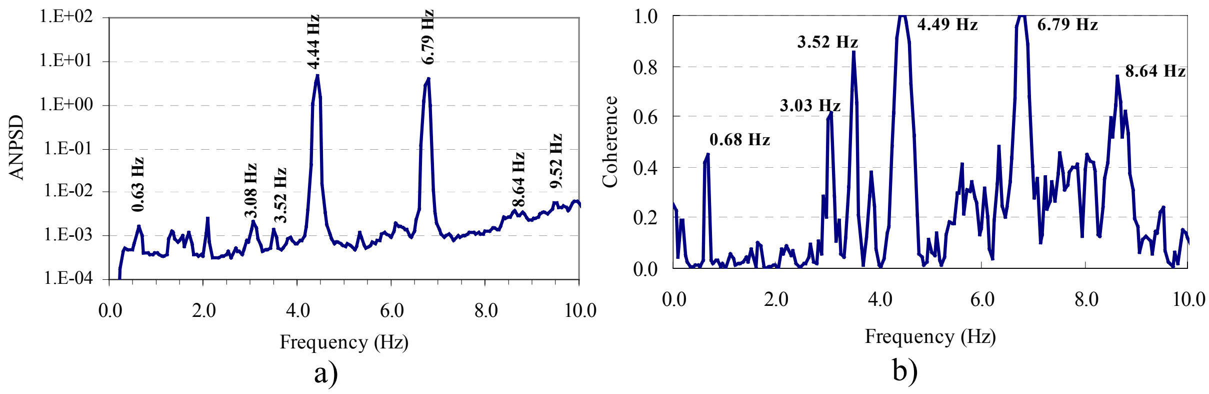
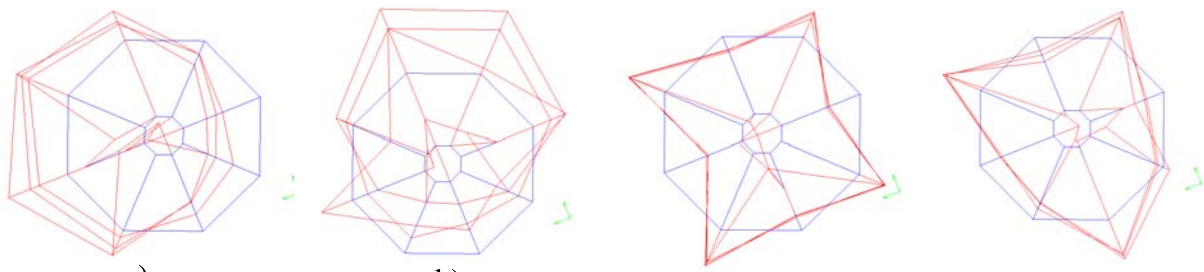


Figure 5: a) Average normalized power spectral density, b) Coherence of H_1 functions between records of points at level 4 and at level 1

For more conclusive interpretations of the test results in terms of modal identification, graphical representations of the vibration mode shapes (amplitude and phase relations between the different measurement points) were obtained (Arede *et al*, [3]) as shown in

Figure 6 for the modal configurations corresponding to the frequencies of 3.08, 3.52, 4.44 and 6.79 Hz. The first two modes refer to the N-S and E-W directions, whereas the last two, though appearing more difficult to understand, suggest ovalization trends.



a) b) c) d)

Figure 6: Plan views of modal configurations. a) 3.08 Hz, translation N-S; b) 3.52 Hz, translation E-W; c) 4.44 Hz, ovalization (?); d) 6.79 Hz, ovalization (?)

Comparison with numerical modal analysis

Although with some topics requiring further investigation, and possibly additional testing to confirm the conclusions that were drawn from the tests, the results above described show a reasonable agreement with those obtained by numerical modal analysis.

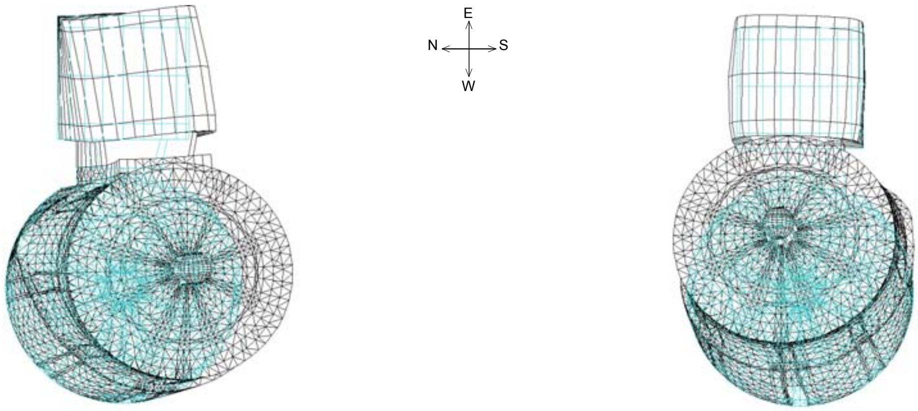
The numerical computation of frequencies and vibration modes was done using the computer code CASTEM 2000, with the mesh and the values of material properties above referred, leading to calculated values of the two first frequencies not far from those experimentally obtained. After little adjustments of the Young modulus, mainly in the infill zones, good agreement was obtained between numerical and experimental results, at least for the two first modes. Table 1 includes the frequency values for the five first modes of vibration obtained from the numerical analysis, and also the experimentally estimated values.

TABLE 1

NATURAL FREQUENCIES: NUMERICAL AND EXPERIMENTAL

Mode	Frequencies (Hz)	
	Numerical	Experimental
1	3.13 (Translation N-S)	3.08 (Translation N-S)
2	3.54 (Translation E-W)	3.52 (Translation E-W)
3	4.21 (Torsion)	4.44 (Ovalization)
4	6.07 (Ovalization)	6.79 (Ovalization?)
5	6.33 (Ovalization)	----

The two first vibration modes obtained from the analysis are illustrated in Figure 7, where a good agreement can be confirmed with the corresponding modes experimentally recorded and presented in Figures 6a and 6b. The other three upper modes (not illustrated herein) were found to correspond essentially to torsion (3^{rd} mode) and to ovalization motions (4^{th} and 5^{th} modes), which seem to be in qualitative compliance with the type of the 3^{rd} and 4^{th} modes estimated from the tests.



a) 3.13 Hz b) 3.54 Hz

Figure 7: 1st and 2nd vibration modes, numerically obtained (plan view)

SEISMIC ANALYSIS

Seismic action

The seismic analysis of the “Serra do Pilar” monastery church was performed in the time domain, using as input action a set of five synthetic accelerograms (per direction of excitation) generated to comply with the local seismicity of Porto region.

According to previous studies by Campos Costa [4], a peak ground acceleration of 0.1g (for hard soil) should be adopted for a return period of 10000 years which was deemed appropriate for such an ancient (over 400 years old) structure that must be preserved for the next generations. The Eurocode 8 (EC8) [5] response spectrum was adopted together with the provisional prescriptions of the National Application Document (NAD) [6], in order to generate synthetic accelerograms of 10s duration and compliant with that spectrum particularized for the Portuguese seismic action type I (moderate magnitude with short focal distance earthquakes) that is more demanding for the present structure.

Analysis methodology

Due to the large dimensions and complexity of the church structure, a simplified analysis methodology was adopted, comprising two separate though dependent calculation phases allowing to analyse both the global structural response and the local behaviour of restricted parts of the structure. Therefore, the adopted approximate technique developed as follows:

1st Phase – Linear Elastic Global Calculation

Considering the self-weight action and three orthogonal direction accelerograms, the complete church structure was elastically analysed using the Newmark method to integrate the dynamic equilibrium equations. Stresses in all structural elements were obtained and, particularly for the blocks of the arches and their columns, the equivalent nodal forces were calculated and stored as a time-varying vector field.

2nd Phase – Non-Linear Local Analysis

The so-obtained force vector field was then applied to a substructure comprising the arches and columns, in order to perform non-linear static calculations for each step of load variation. The solid elements of this reduced mesh were still assumed behaving linear elastically, whereas the interfaces between stone blocks were simulated by zero thickness joint elements with non-linear behaviour (Almeida, [2]). The approximation subjacent in this strategy is that the remaining structural parts are little affected by the non-linear behaviour of the arches and columns substructure, which seems an acceptable assumption for not very intense non-linearity.

For seismic vulnerability assessment purposes, several non-linear static calculations were performed in the 2nd phase corresponding to increasing intensities of the seismic action. This was done by multiplying the time-varying vector field obtained from the 1st calculation phase by the following sequence of factors 0.5, 1.0, 1.5, 2.0 and 2.4, the later having been conditioned by convergence problems possibly reflecting loss of structural equilibrium.

Results

Several results were obtained, some of which are briefly referred next in terms of average peak values of principal stresses (tensile and compressive) and of radial, tangential and vertical displacements. These average values were taken from the peak values corresponding to the five accelerograms considered.

The linear elastic calculation allowed a first insight into the stress state under the prescribed seismic load. The peak values of principal tensile and compressive stress fields are illustrated in Figure 10, from which it was possible to obtain the maximum values of 2.30 MPa (tension) and 3.33 MPa (compression) occurring very localized near the window openings. In the remaining parts of the structure, significantly lower principal tensile stresses were found, not exceeding 0.37 MPa in the substructure of the arches and columns (approximately at mid-height of the columns), whereas the principal compressive stresses show peak values of 2.52 MPa in the arches and columns.

The maximum deformed shape was mainly influenced by the 1st and 2nd modes of vibration as evidenced by the peak displacement envelopes also shown in Figure 10. The maximum displacements of the top of the dome (not including the small upper dome) were found to be about 1.0 cm in both radial and tangential directions and 0.7 cm downwards in the vertical direction. These are rather small displacements, which correspond to a total horizontal drift of 0.034% and reflect the very large stiffness of this structure.

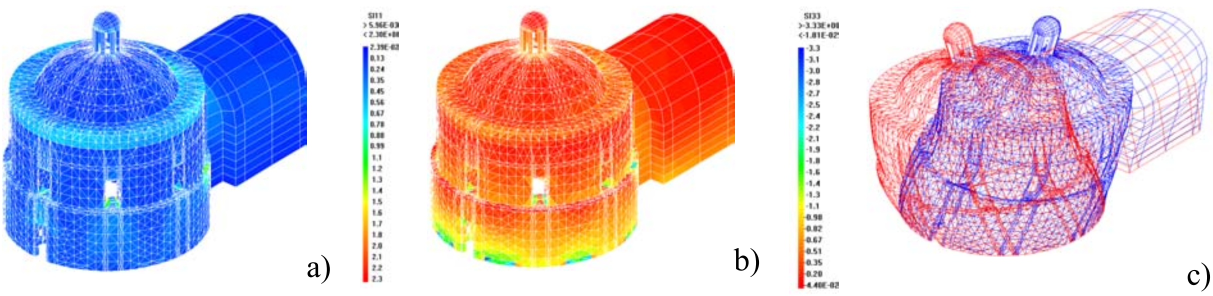
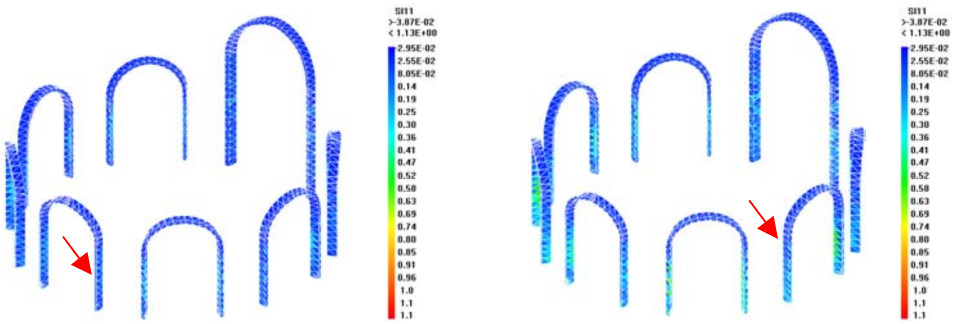


Figure 10: Maximum tensile (a) and compressive (b) stresses distributions and peak displacement (c) envelopes from linear elastic analysis

More detailed analyses were made using the non-linear results obtained for the arches and respective columns, particularly in terms of principal stresses and displacements. Calculations were done for five increasing intensities, but results just focus on intensity 1.0 and 2.4.

Concerning principal stresses, the maximum tensions of 0.38 MPa and 1.15 MPa were found in the columns, respectively for 1.0 and 2.4 intensity, as obtained from the stress distributions shown in Figure 11. Similarly, compressive stresses, occurring near the column bases, range from 2.52 MPa and 4.42 MPa, for the same seismic intensities. The stone blocks of the arches exhibit much lower tensile and compressive stresses.

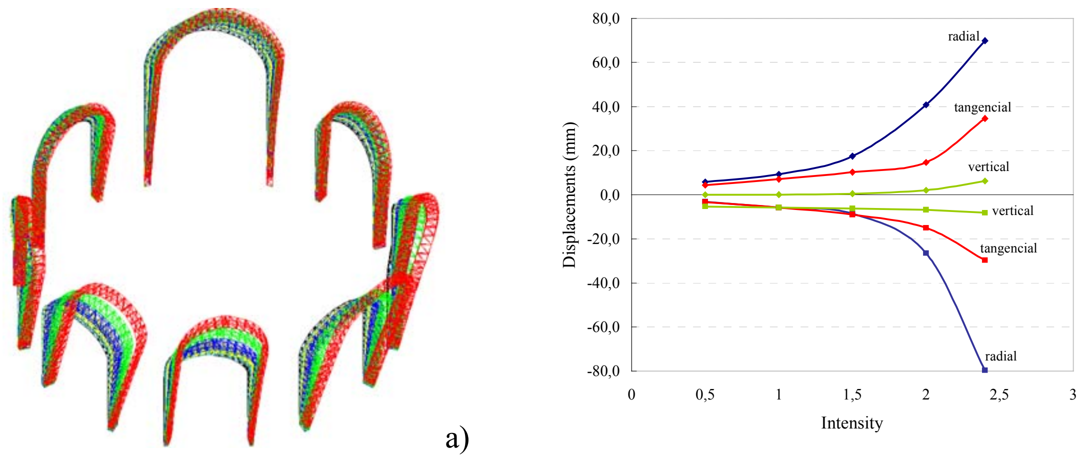
It is worth mentioning that, as usual in this kind of structures, compressions stay far bellow the stone strength, since strength values above 70 MPa were found in compression tests. However, tensile stresses in the blocks, although compatible with the stone tensile strength (about 3 MPa), suggest that increasing non-linearity occurred in the joints between blocks as the seismic intensity was increased. Actually, the 0.38 MPa tensile stress in the blocks means that slight non-linearity occurs in the adjacent joints but, for the seismic intensity 2.4, the block tensile stress of 1.15 MPa requires intense non-linearity and opening of joints.



(a) *(b)*

Figure 11: Maximum tensile stresses in blocks of arches and columns for (a) intensity 1.0 (0.38 MPa) and for (b) intensity 2.4 (1.15 MPa)

The above-mentioned non-linearity was confirmed by the evolution of peak displacements. Figure 12a shows deformed shapes of the arches as the seismic intensity is increasing, and the peak displacement (radial, tangential and vertical) of an arch keystone is plotted against that intensity in Figure 12b. The resulting curves are the vulnerability functions that show pronounced non-linear structural response for intensity values above 1.5.



a) b)

Figure 12: a) Maximum deformed shape of arches and columns; b) Vulnerability functions of arch displacements

Moreover, the maximum intensity (2.4) that was allowed until convergence problems arose (most likely due to loss of equilibrium), led to a peak radial displacement of 8 cm of the arch key stone. This means about 0.98% radial drift, which is already a considerable value for this kind of structures, but it should be noted that it was produced by a seismic action more than twice the intensity of seismic events of 10.000 years return period!!

Therefore the seismic safety of this structure seems to be quite assured, which probably might have been already proved by history. Actually, it has survived, apparently without significant damages, the strong Lisbon earthquake in 1755 that was clearly felt and produced heavy damages in the far north of Portugal.

CONCLUSIONS

The present work reported on numerical analysis based on structural parameters calibrated by cross information from laboratory and/or *in situ* testing and from dynamic identification tests in a large and old structure. Using an appropriate numerical model with distinct behaviour zones and adopting material properties adequately selected from local testing, it was possible to compare computed modal characteristics with those obtained in dynamic testing and to refine stiffness parameters which, for the present case, led to very good agreement of numerical and experimental predictions.

Despite the large stiffness of the structure, dynamic tests under ambient vibration and micro-explosions allowed the first frequencies and mode shapes to be determined through frequency domain techniques. However, some topics remain to be further clarified, particularly related with two higher and very excited frequencies.

The adopted methodology for seismic analysis is particularly appropriate as a simplified and approximate technique for large structures where full control of the non-linear behaviour remains a quite heavy (may be still impossible!!) task to be pursued. A reasonable compromise is therefore obtained between computational effort and level of results achieved. In this context, the global behaviour of the church structure was assessed for the site seismic action and a more detailed local analysis allowed confirming its robustness and capacity to withstand strong ground motions.

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