# DYNAMIC IDENTIFICATION AND 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 behavior study of an ancient structure, over 400 years old. The church structure was numerically modeled 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 dynamic tests were carried out for modal identification and structural stiffness calibration. The seismic analysis was performed in two calculation phases, under the action of artificially generated accelerograms representative of the local seismicity. Results of linear elastic dynamic calculations allowed the global behavior to be analyzed and provided the input for more detailed local analyses of structural parts where the non-linear behavior was considered. Finally, the seismic vulnerability of the structure is briefly addressed and discussed.


## 1 INTRODUCTION

The structural behavior and safety assessment of ancient constructions is becoming a research topic of increasing concern in view of the value and amount of historical heritage to be preserved.

Among all the possible loadings that may be considered, the seismic action is, for sure, one of the most important due to its devastating effects and unforeseeable nature. Moreover, the special features of seismic loading, often requiring structural analyses of the whole structure, render even more complex the study of this type of constructions.

The fast evolution of technological and computational tools has allowed a significant development of sophisticated behavior models and testing techniques suitable for the analysis of ancient structures. Notwithstanding the considerable scientific advances in this field, several difficulties still persist mainly related with the geometric complexity and large dimensions of the problem, the variability of material properties, the lack of knowledge about the original construction techniques and the possible subsequent interventions in the structure.

In this context, the present paper deals with the dynamic characterization and seismic analysis of the "Serra do Pilar" monastery church located nearby Porto, Portugal. For its historical importance and particular features, this 424 years old building is a good example for the use of seismic analysis models in ancient structures of large dimensions.

As described later, due 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 behavior, after which the local response of some parts (arches and bearing columns) was further analyzed by recourse to more detailed calculations involving the non-linear behavior 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 [1]. Detailed geometric data were available and generated such that visible stone blocks could be simulated by means of solid elements duly individualized in order to allow the modeling of the joints between blocks.

The estimation of mechanical and physical properties of materials was supported by *in situ* and laboratory tests on samples extracted from the structure. In particular, dynamic identification tests were performed to evaluate its dynamic characteristics, 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 performed using two frequency domain methods [2] for output only modal identification applied to the measured responses.

In the following sections a brief description of the "Serra do Pilar" monastery church is first given, along with some comments on the numerical modeling. Then the dynamic tests are addressed, describing the instrumentation, the testing sequence and the analysis of test results. Next, the numerical modal analysis is presented, the corresponding results being discussed through comparison with modal properties obtained from experimental tests. Finally, the seismic analysis is addressed, mainly focusing on the adopted methodology, the results being presented and briefly discussed.

## 2 THE "SERRA DO PILAR" MONASTERY CHURCH

### 2.1 Historical background

The "Serra do Pilar" monastery was founded back in 1537 and is located in the S. Nicolau hill, in the historical center of Vila Nova de Gaia. The construction works developed slowly, such that only in 1567 the first monastery was actually built. In 1598 it was decided to replace the church by a new and larger one that finally opened in 1678.

Comparing to the current architecture by that time in the country, this cylindrical church (made out of local granite stone) was an original option, probably inspired in the Rome Pantheon as evidenced in Figure 1 where outside and inside views of the church are included.

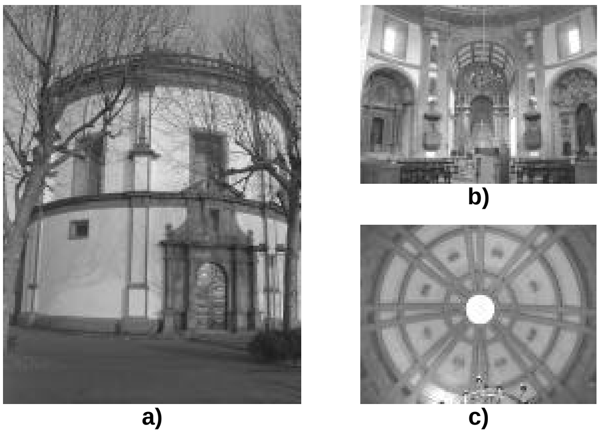


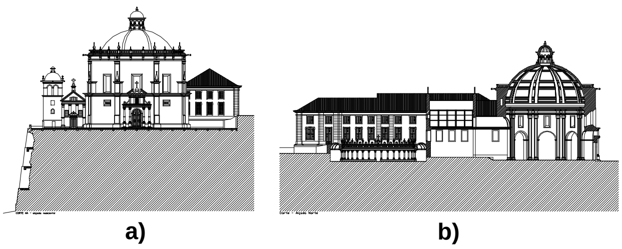
**Figure 1: The "Serra do Pilar" monastery church. (a) Outside, b) inside and c) the dome structure**

Due to its privileged location, right in the south bank of the Douro river and facing the historical center of Porto, the monastery was used as fortress during the French invasions and the liberal wars, leading to a very degraded state even further aggravated during the 19th century. Since 1927, upon a 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.

### 2.2 Structural discretization and numerical modeling

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 that completes the roof structure (Figure 1-c)). Inside there are eight buttresses (16.8m high) placed in between small chapels (Figure 1-b)) defined by arches supporting the windows. In the main chapel zone there is a larger arch (13.38m high) adjacent to a vault with 14m height which covers the main altar zone developing along a rectangular shaped area.

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

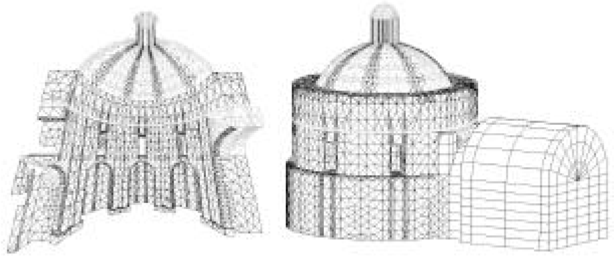


a) b)

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

The numerical modeling 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 are plastered and were found to be made of very irregular stone masonry; for that reason they are designated as filings.

Based on the geometric and topographic data, half mesh of the church structure was defined (Figure 3-a)) using the computer code CASTEM 2000 [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 elements [3].



a) b)

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

Concerning material properties, several laboratory and *in situ* experimental tests were carried out [3], allowing first estimations of mass density and deformability parameters, such as Young modulus and Poisson ratio. For the mass density, values ranging from 20 to 25 kN/m$^3$ were adopted, 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.

## 3 DYNAMIC TESTS OF THE CHURCH STRUCTURE

### 3.1 Instrumentation

The dynamic tests of the Serra do Pilar monastery church were carried out using the following equipment:

- 12 accelerometers FBA-11 from Kinemetrics;
- 3 variable capacitance accelerometers from Endevco (model 7290A-2) with corresponding power supply and signal conditioning units;
- 1 personal computer with a data acquisition card AT-MIO-16XE-10 from National Instruments, with 16 bits A/D conversion;
- 1 chassis BNC-2090 from National Instruments, with 16 channels, connected to the DAQ card by means of a suitable cable;
- cables for power and signal transmission.

The FBA-11 accelerometers are uniaxial force balance acceleration sensors from Kinematics. Due to their dynamic range of 135 dB, low noise performance and bandwidth that goes from DC to 50 Hz, they are appropriate sensors for the dynamic testing of civil engineering structures.

In order to use the FBA-11 accelerometers, an adequate system for power supply and signal conditioning was developed at LNEC. This system comprises 4 power supply and signal conditioning units. Each of these units can be connected to three accelerometers and contains two 12V batteries, analog filters and amplifiers with user selectable gain factors. The units are connected to the sensors with relatively short cables (20 m) and to the data acquisition board with long cables that, at the present, range from 50 m up to 200 m. Between these long cables and the data acquisition system there is a small box to convert the three pairs long cables to BNC terminals.

In the tests of the Serra do Pilar monastery church, seven of the FBA-11 accelerometers had a factory configured full-scale range of ±1g and the other five had ±4g. The corresponding gain factors at the power supply and signal conditioning units were set-up, respectively, to 50 and 200. The FBA-11 accelerometers have an output of ±2.5V, while the DAQ Card AT-MIO-16X-10 has an input of ±10V. With the selected range and gain factors, the minimum acceleration amplitude that could be measured during the tests was 0.0024 mg. The data acquisition was performed with software developed in LabView [4].

### 3.2 Testing sequence

The testing campaign [3] 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 locations of the accelerometers throughout the structure as represented in Figure 4.

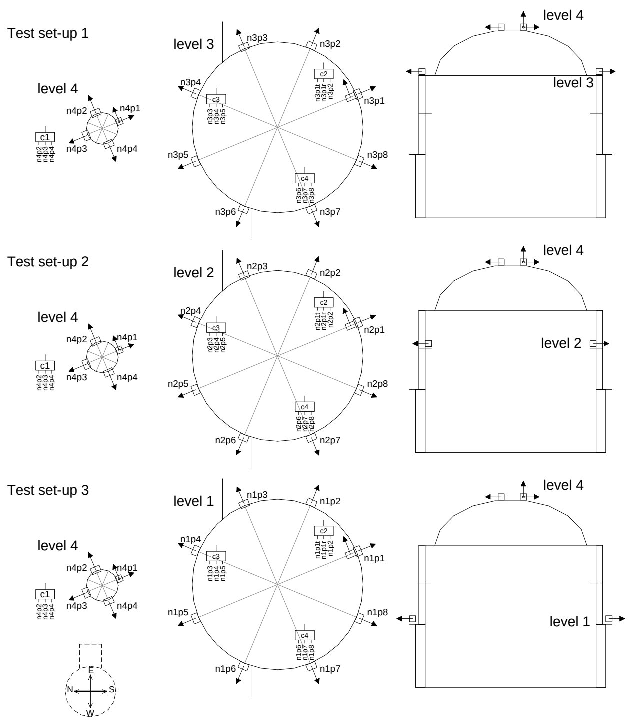



**Figure 4: Instrumented points for the three different testing configurations**

In fact, three different configurations were considered in which accelerometers were moved sequentially from the upper to the lower level of the structure as shown in Figure 4. However, for any of the three configurations, five accelerometers were kept fixed in the top of the church dome in order to have reference measurements allowing to estimate the amplitude and phase relations between the instrumented points. Four of these five accelerometers corresponded to two radial directions, whereas the remaining one was positioned perpendicularly to one of those directions.

For each test set-up, and due to the limited number of available accelerometers, mainly radial components were measured although at least one tangential component was also obtained for each level. Thus, at each stage, eight radial components were recorded, corresponding to the eight buttresses of the structure, allowing to measure both translational and ovalization modes of the church.

In each test set-up a total of about 9 minutes of ambient vibration measurements were recorded with a sampling frequency of 200 Hz. After recording the ambient vibration data, a micro-explosion test was performed, and the corresponding accelerations in the structure were recorded with a sampling frequency of 500 Hz.

### 3.3 Analysis of results

Results in terms of recorded accelerations have shown a quite stationary response under ambient vibration, as illustrated in Figure 5-a). With maximum acceleration amplitude around 0.2 mg, this type of response was somewhat unexpected for that kind of excitation.

The response for both the ambient vibration and the micro-explosions is also shown in Figure 5-b), with an enlarged time scale. It is noteworthy the larger amplitude of the signal due to the micro-explosion, and also the stationary characteristic of the recorded ambient vibrations.

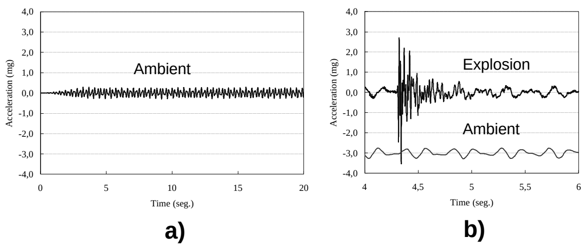



**Figure 5: Response of the n1p2 accelerometer due to (a) ambient vibration and (b) explosion**

The accelerations recorded in the tests were analyzed for modal identification purposes using the basic frequency domain method [2], implemented in a computer program using Labview [4], and the frequency domain decomposition method [2] implemented in the Artemis software package [5].

The basic frequency domain method [2] involves, mainly, the computation of auto-spectra, coherence between different measurement points and $H_1$ estimates of the frequency response functions, also between different measurement points. The frequency domain decomposition method [2] consists, basically, in a singular value decomposition of the spectral densities matrix.

Before the application of both modal identification methods, the acceleration records were pre-processed with the following operations: trend removal; band-pass filtering between 0.25 Hz and 12.5 Hz, with a 8 poles Butterworth filter; decimation of the signals from the sampling rates of 200 Hz and 500 Hz to 25 Hz. The decimation of the signals implied a Nyquist frequency of 12.5 Hz, which, according to a preliminary finite element analysis, would be enough to include the main vibration modes of the church in the modal identification analysis.

The average normalized power spectral density (ANPSD) was computed taking into account the whole set of records and is depicted in Figure 6, with logarithmic scale in the ordinates. In the ANPSD presented in Figure 6, there are two predominant peaks of the spectral density, for frequencies around 4.5 Hz and 6.8 Hz, but there are also other less evident peaks that were carefully analyzed in terms of the coherence and $H_1$ frequency response function estimates of the different measurement points in relation to the reference ones.

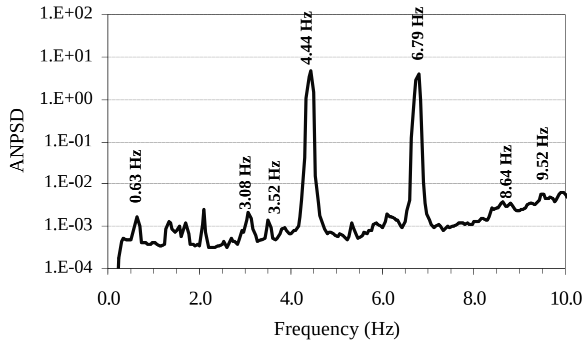



**Figure 6: Average normalized power spectral density**

Figure 7 shows the coherence between the response 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 it is also possible to see that, for frequencies of about 3.1 Hz and 3.5 Hz, there are also high values of the coherence, although lower than 1.

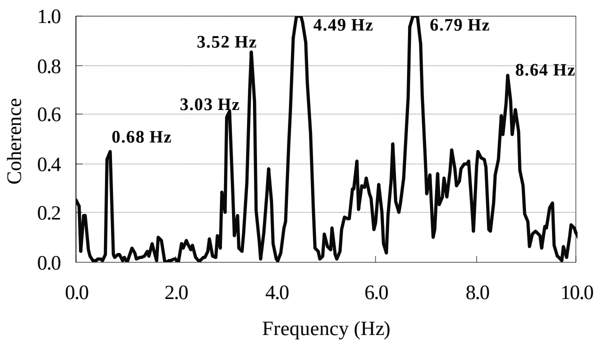



**Figure 7: Coherence between records at points n4p3 (level 4) and n1p5 (level 1)**

For a more conclusive interpretation of the test results in terms of modal identification, it was quite important to obtain a graphical representation of the vibration mode shapes (amplitude and phase relations between the different measurement points). This was done using both methods and coincident results were obtained.

Figure 8 shows the modal configurations that were obtained using the Artemis software package [5], for the frequencies of 3.08, 3.52, 4.44 and 6.79 Hz, the first two corresponding to translational modes in the N-S and E-W directions, whereas the last two appear more difficult to understand. However, a more detailed inspection of mode shape animations suggests an ovalization trend in those configurations.

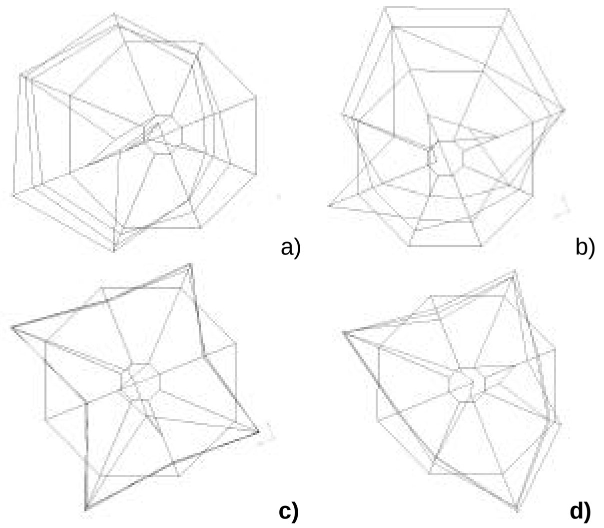



**Figure 8: Plan views of modal configurations.**
**a) 3.08 Hz, translation N-S; b) 3.52 Hz, translation E-W; c) 4.44 Hz, ovalization; d) 6.79 Hz, ovalization**

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

## 4. NUMERICAL MODAL ANALYSIS

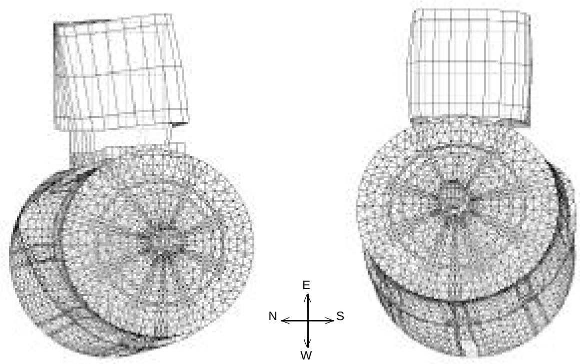
The numerical analysis for the computation of frequencies and vibration modes was performed by recourse to the computer code CASTEM 2000, using the mesh and values of material properties referred in section 2.

A first trial with those properties led to calculated values of the two first frequencies not far from those experimentally obtained. After little adjustments of the Young modulus, in particular in the infill zones where it is prone to significant variability, good agreement was obtained between numerical and experimental results, at least for the first two modes. Table I includes the frequency values for the first five modes of vibration, as obtained from the numerical analysis, and also the experimentally estimated values.

| 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 | 6.36 (Ovalization) | ---- |

**Table 1: Natural frequencies: numerical and experimental**

Accordingly, the first two vibration modes obtained from the analysis are illustrated in Figure 9, where a good agreement can be confirmed with the corresponding modes experimentally recorded and presented in Figures 8-a) and 8-b).




a) 3.13 Hz b) 3.54 Hz

**Figure 9: 1st and 2nd vibration modes, numerically obtained (plan view)**

The other three upper modes (not illustrated herein) were found to correspond essentially to torsion motion (3rd mode) and to ovalization configurations (4th and 5th modes), which seems to be in qualitative agreement with the type of the 3rd and 4th modes estimated from the experiment.

## 5. SEISMIC ANALYSIS

### 5.1 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 artificial accelerograms (per direction of excitation) generated to comply with the local seismicity of Porto region.

The Portuguese seismicity is mainly influenced by two types of strong-ground motions, namely moderate magnitude with short focal distance earthquakes (seismic action type I, associated with intra-plate events) and high magnitude with large focal distance earthquakes (seismic action type II, corresponding to inter-plate events). This is reflected in the design response spectra (and also power spectra) prescribed in the national standards, which are currently being adapted to fit the forthcoming European Standard for seismic design, the Eurocode 8 (EC8) [6], by means of the National Application Document (DNA) [7].

Artificial accelerograms were obtained with 10s duration, in compliance with the basic shape of the response spectra prescribed in the referred document (DNA) for the seismic action type I (more demanding for the present structure). The peak ground acceleration of 0.1g (for hard soil) was chosen according to previous studies [8] for a return period of 10000 years, which was considered appropriate for an ancient (over 400 years old) structure that must stand there for the next generations.

**5.2 Analysis methodology**

For the envisaged analysis and due to the large dimensions and complexity of the church structure, a simplified analysis methodology was adopted. This strategy involved two separate but dependent calculation phases in order to assess both the global structural response and the local behavior of some parts of the structure. According to this objectives, the two calculation phases developed as follows:

- 1st Phase – Linear Elastic Global Calculation
  The whole structure was elastically analyzed under the action of self-weight and of three orthogonal direction accelerograms. The Newmark method was used 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 in the substructure of 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 behavior [3].

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.

**5.3 Results**

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

The linear elastic calculation allowed a first insight into the stress state under the prescribed seismic load. The average peak values of maximum principal tensile and compression stresses were found to be 2.30 MPa and 3.33 MPa, respectively, occurring very localized near the window openings. In the remaining parts of the structure significantly lower principal tensile stresses are 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 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.

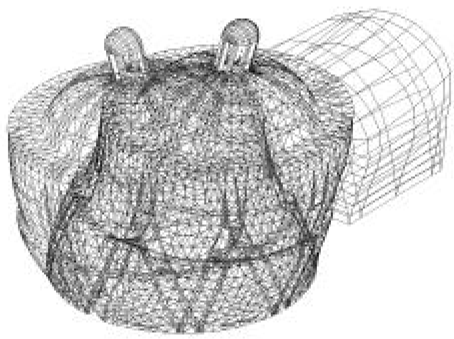


**Figure 10: Peak displacement envelopes from linear elastic analysis**

From the non-linear calculations, a more detailed analysis was carried out for the arches and respective columns, particularly in terms of principal stresses and displacements. Although calculations were made for five increasing intensities, 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; Figure 11 refers to the tensile stress distribution for the higher intensity. 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 compressions stay far bellow the stone strength (actually, strength values above 70 MPa were found in compression tests) but tensile stresses in the blocks, although still 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.

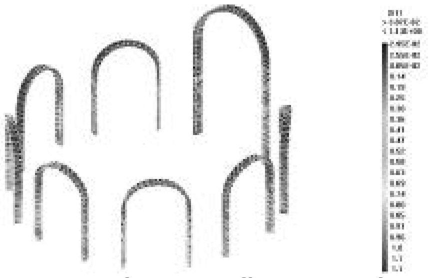


**Figure 11: Maximum tensile stresses (1.15 MPa) in blocks of arches and columns (intensity 2.4)**

The above mentioned non-linearity was confirmed by the inspection of deformed shapes of the arches as the seismic intensity is increasing, as shown in Figure 12-a).

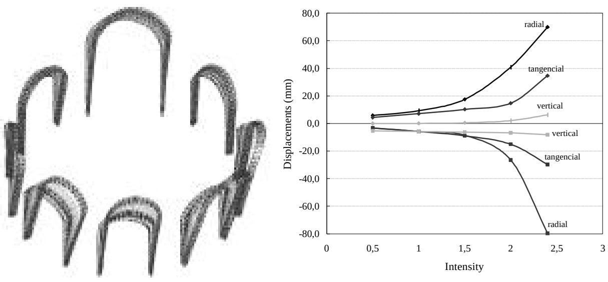



**Figure 12: a) Maximum deformed shape of arches and columns; b) Vulnerability functions of arch displacements**

Vulnerability functions of arches displacements were plotted for radial, tangential and vertical directions, leading to the curves shown in Figure 12-b). These relations between peak response values and the seismic intensity clearly show a pronounced non-linear structural response for intensity values above 1.5. 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 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.

### 6. CONCLUSIONS

The present work has shown the possibilities of combining numerical analysis based on parameters calibrated by cross information from laboratory and/or *in situ* testing and from dynamic identification tests in large and old structures.

Starting from a carefully chosen numerical model where distinct behavior zones are duly individualized and using material properties adequately selected from local testing, it is possible to compare computed modal characteristics with those obtained in dynamic testing. The comparison allows stiffness parameter tuning which, for the present case, led to very good agreement of numerical and experimental predictions.

Despite the high stiffness of the structure, dynamic tests under ambient vibration and micro-explosions allowed the first frequencies and mode shapes to be determined through both the basic frequency domain method and the frequency domain decomposition technique. 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 behavior 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 behavior of the church structure was assessed for the site seismic action and a more detailed local analysis allowed to confirm its robustness and capacity to withstand strong ground motions.


### ACKNOWLEDGEMENTS

The authors wish to acknowledge the Portuguese institution for monuments and national buildings (DGEMN) for all the support given during this work.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