Improving the seismic resistance of masonry buildings: Concepts for cultural heritage and recent developments in structural analysis

Paulo B. Lourenço, Daniel V. Oliveira
Department of Civil Engineering, University of Minho, 4800-058 Guimaraes, Portugal. pbl@civil.uminho.pt.

ABSTRACT:
Conservation and restoration of historical structures are still a challenge to modern practitioners even if considerable research advances have occurred in the last decades, namely with respect to non-destructive testing, mechanical characterization, tools for advanced numerical analysis, knowledge on traditional materials and techniques, and innovative materials and techniques. In the paper, the ICOMOS Recommendations for the Analysis, Conservation and Structural Restoration of Architectural Heritage are briefly reviewed, together with recent developments in structural analysis. The proposed methodology is applied to Monastery of Jerónimos, in Lisbon, Portugal, including the following steps: seismic action characterization, from the identification of earthquake source areas to the artificial generation of acceleration time histories, using specific theoretical models and including superficial site-effects; simple numerical modeling for a preliminary knowledge of the structural behavior; experimental mechanical characterization of materials and structural elements; installation of static and dynamic monitoring systems aiming at a better understanding of the static and dynamic behavior; development of advanced numerical models including calibration against relevant experimental data; non-linear dynamic analysis of the structure for different earthquake levels.

1 INTRODUCTION

The analysis of historical masonry constructions is a complex task that requires specific training. The continuous changes in materials and construction techniques, that swiftly moved away from traditional practice, and the challenging technical and scientific developments, which make new possibilities available for all the agents involved in the preservation of the architectural heritage, are key aspects in the division between the science of construction and the art of conservation and restoration.

The consideration of these aspects is complex and calls for qualified analysts that combine advanced knowledge in the area and engineering reasoning, as well as a careful, humble and, usually, time-consuming approach. Several methods and computational tools are available for the assessment of the mechanical behavior of historical constructions. The methods resort to different theories or approaches, resulting in: different levels of complexity (from simple graphical methods and hand calculations to complex mathematical formulations and large systems of non-linear equations), different availability for the practitioner (from readily available in any consulting engineer office to scarcely available in a few research oriented institutions and large consulting offices), different time requirements (from a few seconds of computer time to a few days of processing) and, of course, different costs.

The possibilities of structural analysis of historical constructions have been addressed e.g. in Lourenço (2002), where it is advocated that most techniques of analysis are adequate, possibly for different applications, if combined with proper engineering reasoning. It is noted that only very recently the scientific community began to show interest in modern advanced testing (under displacement control) and advanced tools of analysis for historical constructions. The lack of experience in this field was notorious in comparison with more advanced research fields like concrete, soil, rock or composite mechanics.

Recently, Recommendations for the Analysis, Conservation and Structural Restoration of Architectural Heritage have been approved. These Recommendations are intended to be useful to all those involved in conservation and restoration problems and not exclusively to the wide community of engineers. A key message, probably subliminal, is that those involved in historic preservation must recognize the contribution of the engineer. Often engineering advice seems to be regarded as something to be sought

at the end of a project when all the decisions have been made, while it is clear that better solutions might have been available with an earlier engineering contribution.

An issue related with this message is that conservation engineering requires a different approach and different skills from those employed in designing new construction. Often historic fabric has been mutilated or destroyed by engineers who do not recognize this fact, with the approval of the authorities and other experts involved. Moreover, even when conservation skills are employed, there are frequent attempts by regulating authorities and engineers to make historic structures conform to modern design codes. This is generally unacceptable because the codes were written with quite different forms of construction in mind, because it is unnecessary and because it can be very destructive of historic fabric.

The need to recognize the distinction between modern design and conservation is also of relevance in the context of engineers' fees. The usual fee calculation based on a percentage of the cost of the work specified is clearly inimical to best conservation practice, when the ideal is to avoid any structural intervention if possible. Being able to recommend taking no action might actually involve more investigative work and hence more cost to the engineer than recommending some major intervention.

Modern intervention procedures require a thorough survey of the structure and an understanding of its history. Any heritage structure is the result of the original design and construction, any deliberate changes that have been made and the ravages of time and chance. An engineer working on historical buildings must be aware that much of the effort in understanding their present state requires an attempt to understand the historical process. The engineer involved at the beginning of the process might not only have questions that can easily be answered by the archaeologist or architectural historian, but he might be also able to offer explanations for the data being uncovered.

Here, the modern approach towards structural conservation is reviewed, together with a review on recent structural analysis advances and application to an emblematic case study.

2 REVIEW OF RECOMMENDATIONS FROM ICOMOS

Structures of architectural heritage, by their very nature and history (material and assembly), present a number of challenges in conservation, diagnosis, analysis, monitoring and strengthening that limit the application of modern legal codes and building standards. Recommendations are desirable and necessary to ensure rational methods of analysis and repair methods appropriate to the cultural context.

Therefore, the International Scientific Committee for the Analysis and Restoration of Structures of Architectural Heritage (ISCARSAH) has prepared recommendations (Icomos, 2001), intended to be useful to all those involved in conservation and restoration problems. These recommendations contain Principles, where the basic concepts of conservation are presented, and Guidelines, where the rules and methodology that a designer should follow are discussed. In addition, normative and pre-normative are gradually becoming available, e.g. ISO 13822 (2003), EN 1998-3:2005 or FEMA 356 (2000), at least with respect to seismic rehabilitation, which is a major concern.

2.1 Principles and Guidelines

The principles entail: General criteria; Research and diagnosis; and Remedial measures and controls. A multi-disciplinary approach is required and the peculiarity of heritage structures, with their complex history, requires the organization of studies and analysis in steps: condition survey, identification of the causes of damage and decay, choice of the remedial measures and control of the efficiency of the interventions. Understanding of the structural behavior and material characteristics is essential for any project related to architectural heritage. Diagnosis is based on historical information and qualitative and quantitative approaches. The qualitative approach is based on direct observation of the structural damage and material decay as well as historical and archaeological research, while the quantitative approach requires material and structural tests, monitoring and structural analysis.

Often the application of the same safety levels used in the design of new buildings requires excessive, if not impossible, measures. In these cases other methods, appropriately justified, may allow different approaches to safety. Therapy should address root causes rather than symptoms. Each intervention should be in proportion to the safety objectives, keeping intervention to the minimum necessary to guarantee safety and durability and with the least damage to heritage values. The choice between "traditional" and "innovative" techniques should be determined on a case-by-case basis with preference given to those that are least invasive and most compatible with heritage values, consistent with the need for safety and durability. At times the difficulty of evaluating both the safety levels and the possible benefits of interventions may suggest "an observational method", i.e. an incremental approach, beginning with a minimum level of intervention, with the possible adoption of subsequent supplementary or corrective measures, see Figure 1.

The methodology stresses the importance of an "Explanatory Report", where all the acquired information, the diagnosis, including the safety evaluation, and any decision to intervene should be fully

detailed. This is essential for future analysis of continuous processes (such as decay processes or slow soil settlements), phenomena of cyclical nature (such as variation in temperature or moisture content) and even phenomena that can suddenly occur (such as earthquakes), and for future evaluation and understanding of the remedial measures adopted in the present. In this process, experimental and numerical techniques are of relevance to provide the necessary knowledge about materials and the structure itself.
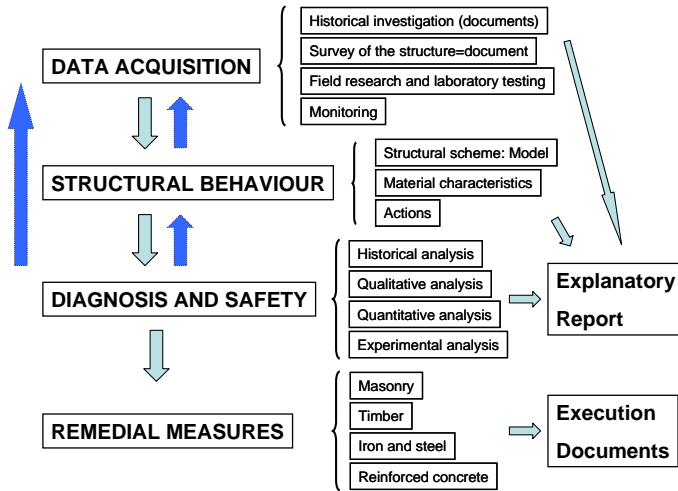

Figure 1. Possible flow-chart for ICOMOS Methodology

Next, some recent developments in numerical analysis are briefly reviewed.

3 RECENT DEVELOPMENTS IN NUMERICAL ANALYSIS

Masonry is a material exhibiting distinct directional properties due to the mortar joints, which act as planes of weakness. Depending on the level of accuracy and the simplicity desired, it is possible to use different modeling strategies Micro-modeling studies are necessary to give a better understanding about the local behavior of masonry structures. This type of modeling applies notably to structural details. Macro-models are applicable when the structure is composed of solid walls with sufficiently large dimensions so that the stresses across or along a macro-length will be essentially uniform. Clearly, macro modeling is more practice oriented due to the reduced time and memory requirements as well as a user-friendly mesh generation.

Linear elastic analysis can be assumed a more practical tool, even if the time requirements to construct the finite element model are the same as for non-linear analysis. But, such an analysis fails to give an idea of the structural behavior beyond the beginning of cracking. Due to the low tensile strength of masonry, linear elastic analyses seem to be unable to represent adequately the behavior of historical constructions.

3.1 Discontinuum models (Micro-modeling)

Different approaches are possible to represent heterogeneous media, namely, the discrete element method, the discontinuous finite element method and limit analysis.

The typical characteristics of discrete element methods are: (a) the consideration of rigid or deform-able blocks (in combination with FEM); (b) connection between vertices and sides / faces; (c) interpenetration is usually possible; (d) integration of the equations of motion for the blocks (explicit solution) using the real damping coefficient (dynamic solution) or artificially large (static solution). The main advantages are an adequate formulation for large displacements, including contact update, and an independent mesh for each block, in case of deformable blocks. The main disadvantages are the need of a large number of contact points required for accurate representation of interface stresses and a rather time consuming analysis, especially for 3D problems.

Discrete elements have been used for masonry e.g. in Azevedo et al. (2000). The finite element method remains the most used tool for numerical analysis in solid mechanics and an extension from standard continuum finite elements to represent discrete joints was developed in the early days of non-linear mechanics, with an early application to masonry, Page (1978). On the contrary, limit analysis received far less attention from the technical and scientific community for masonry structures, even with also an early application in Livesley (1978). Still, limit analysis has the advantage of being a simple tool, while having the disadvantages that only collapse load and collapse mechanism can be obtained and loading history can hardly be included.

The explicit representation of the joints and units in a numerical model seems a logical step towards a rigorous analysis tool. This kind of analysis is particularly adequate for small structures, subjected to states of stress and strain strongly heterogeneous, and demands the knowledge of each of the constituents of masonry (unit and mortar) as well as the interface. In terms of modeling, all the non-linear behavior can be concentrated in the joints and in straight potential vertical cracks in the centerline of all units. In general, a higher computational effort ensues, so this approach still has a wider application in research and in small models for localized analysis. Applications can be carried out using finite elements, discrete elements or limit analysis.

A complete micro-model must include all the failure mechanisms of masonry, namely, cracking of joints, sliding over one head or bed joint, cracking of the units and crushing of masonry, Lourenço and Rots (1997). By adopting appropriate evolution rules in a finite element environment, Oliveira and Lourenço (2004), it is possible to reproduce non-linear behavior during unloading. Figure 2 shows the

results of modeling a shear wall with an initial vertical pre-compression pressure. Figure 3 illustrates results using advanced solution procedures for non-linear optimization problems, with a limit analysis constitutive model that incorporates non-associated flow at the joints and a novel formulation for torsion, Orduña and Lourenço (2005).
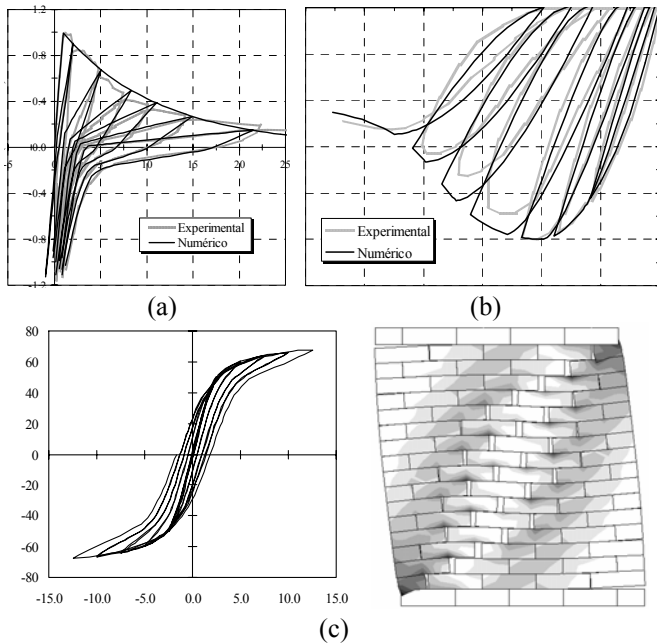

(a)

(b)

(c)

Figure 2. Behavior for an interface model extended to cyclic formulation: (a) tension-compression, (b) compression and (c) force-displacement diagram and collapse of shear walls

(a)

(b)
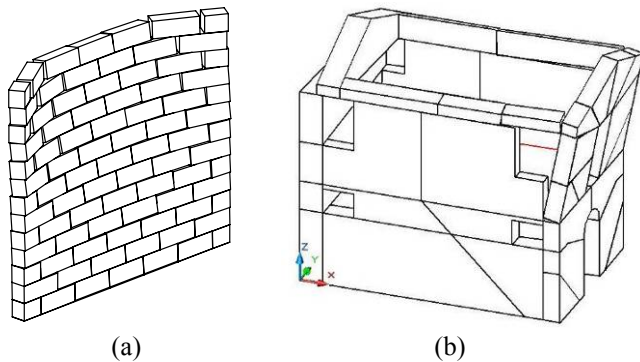
Figure 3. Results for rigid block limit analysis: (a) panel subjected to out-of-plane failure and (b) simplified analysis of a complete building with macro-blocks

3.2 Continuum models (Macro-modeling)

The finite element model seems to be the most adequate tool for the application of continuum models. Only a reduced number of authors tried to develop specific models for the analysis of masonry structures, always using the finite element method. A powerful plasticity model, Lourenço et al. (1998), combines the advantages of modern plasticity concepts with a powerful representation of anisotropic material behavior, which includes different hardening/softening behavior along each material axis. Figure 4 shows the results of modeling a shear wall with an initial vertical pre-compression pressure and a wall panel subjected to out of plane failure.

Another approach that is receiving much attention from researchers is the homogenization theory, in which the macro constitutive behavior of masonry is obtained from a mathematical process involving the geometry and the constitutive behavior of the masonry components. Figure 5 illustrates typical results obtained for homogenized failure surfaces and homogenized constitutive behavior, see Zucchini and Lourenço (2002), Zucchini and Lourenço (2004) and Milani et al. (2006).
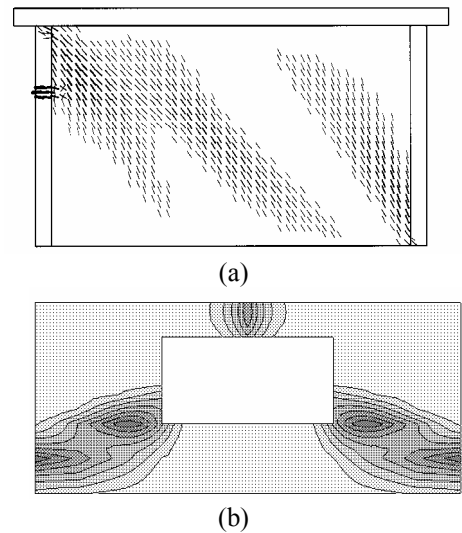

(a)

(b)

Figure 4. Results for macro-modeling analysis: (a) shear wall and (b) panel subjected to out-of-plane failure
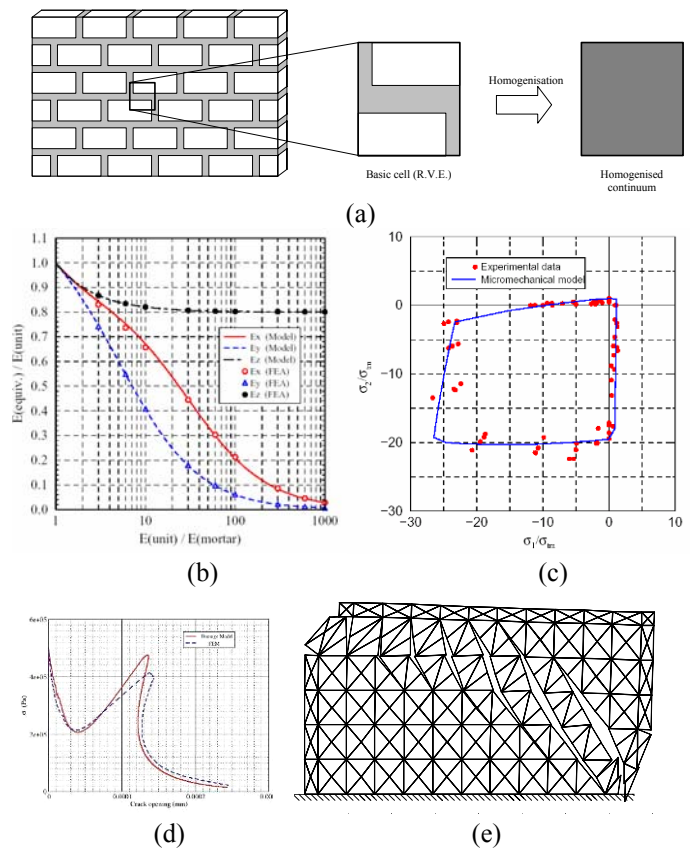

(a)

(b)

(c)

(d)

(e)

Figure 5. Results for homogenization (macro) analysis: (a) basic cell and process; (b) Young's modulus; (c) failure surface; (d) constitutive behavior in tension; (e) results of shear wall using limit analysis finite elements.

4 APPLICATION TO MONASTERY OF JERÓNIMOS, LISBON

Monastery of Jerónimos is, probably, the crown asset of Portuguese architectural heritage dating from the 16th century. The monumental compound has considerable dimensions in plan, more than 300×50 m^2, and an average height of 20 m (50 m in the towers). The monastery evolves around two courts and is located in the right shore of Tagus river, in Lisbon. The construction started in 1502 and ended in 1604. Its original plan is now missing. It was built in limestone that has been removed mainly from its implantation place. One court is composed by the Church and the cloister of the monastery. The Church has considerable dimensions, namely a length of 70 m, a width of 23 m (main nave) and 40 m (transept) and a height of 24 m. The plan includes a single bell tower (South side), a single nave, a transept, the chancel and two lateral chapels, see Figure 6.


(a)
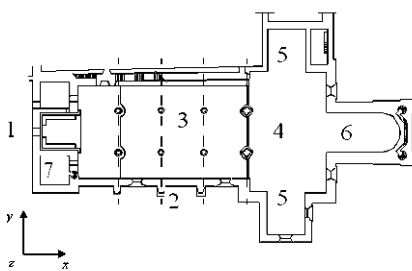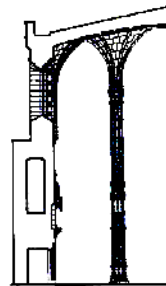
(b)   (c)

Figure 6. Monastery of Jerónimos: (a) general view; (b) plan (1-axial doorway, 2-lateral doorway, 3-nave, 4-transept, 5-side chapels, 6-chancel; 7-South bell-tower); (c) half of transversal cross-section

The main nave is divided by two rows of slender columns, with a free height of about 16.0 m. Each column possesses large bases and fan capitals. The transverse sections of the octagonal columns have a radius of 1.04 m (nave) and 1.88 m (nave-transept). The South wall has a thickness of around 1.9 m, possesses very large openings and its stability is ensured by three large trapezoidal buttresses. The North wall, with an average thickness of 3.5 m, includes an internal staircase that provides access to the cloister. A slightly curved vault ceiling comprises a net of ribs that support the stone slabs. The fan capitals reduce effectively the free span from one external wall to the other, see Figure 6c. The chancel walls are also rather thick (around 2.5-2.65 m). Additional information about the church and the vault can be found in Genin (1995) and Genin (2001).

The construction resisted quite well to the earthquake of November 1, 1755. Later, in December 1756, a new earthquake caused the collapse of one column of the church that supported the vaults of the nave, which resulted in the partial ruin of the nave. By this occasion, also the vault of the high choir of the church partially collapsed, see also Mourão (2001). Also, in 1887-1888 the bell-tower was modified and elevated. In 1947-1949 the church cover was restored and brick masonry walls were built at the extrados of the vault nave to provide support for tiles (see Figure 7). In 1963, minor consolidation works were done including the vault bed joints refill. In 1999-2001 a study of stone pathology was conducted. Since 1949, several historical documents have referred stone fragments falls from the vaults of the church. These successive happenings illustrate clearly the need for a reliable seismic assessment of the monument.
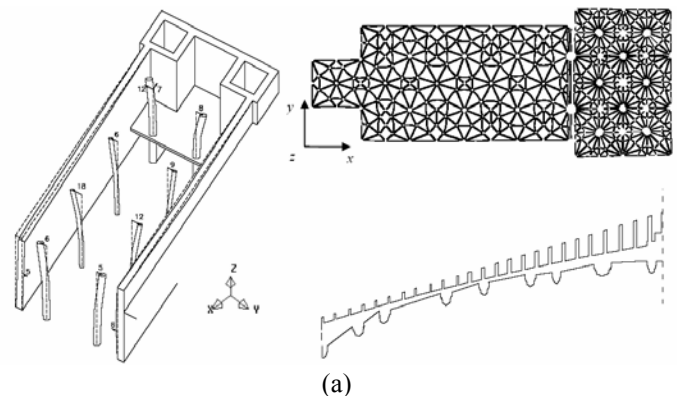
(a)

(b)

Figure 7. Geometry and survey of the nave: (a) survey of the columns and external walls, together with vault plan and transversal cross-section; (b) removal of the roof and existing brick wall system to support the tiles, together with GPR inspection of the columns

The analysis of previous existing works allows concluding that the geometrical survey of the main nave demonstrates a vertical non-alignment for all

the columns and the external walls (Figure 7, Genin 1995). Also, the radar investigation and ultrasonic tests carried out show that the columns of the nave seem to be of good quality and made of a single block or two blocks, see Genin (1995) and Lourenço et al. (2007), and a variable thickness mortar layer seems to exist on the extrados of the vault. On the other hand, a concrete-like material with stones and clay mortar fills the fan capitals (Oliveira, 2002).

Finally, an existing geotechnical report shows that the bed rock is located a few meters below the surface and that direct foundations were found in the monastery.

Using available geometric data, a set of simplified in-plane and out-of-plane indexes were computed. The results are summarized in Table 1. It is stressed the high slenderness of the columns (γ_4) and the apparent vulnerability of Church in the transversal direction ($\gamma_{3,Y}$). Detailed information on these indexes can be found in Lourenço and Roque (2006).

Table 1. Simplified indexes based on geometric data.

In-plan area ratio (γ_1)		Area to weight ratio (γ_2)		Base shear ratio (γ_3)	
X	Y	X	Y	X	Y
0.17	0.12	1.2	0.8	1.4	0.95

Slenderness ratio of columns (γ_4)	Thickness to height ratio of columns (γ_5)	Thickness to height ratio of perimeter walls (γ_6)
70	0.06	0.08

Considering that available data on structural parameters is quite scarce, it was decided to promote a campaign of experimental tests aiming at (a) mechanical characterization of the materials; (b) dynamic modal identification; (c) long term monitoring through the installation of static and dynamic monitoring systems.

As it was neither allowed to collect samples nor to use flat-jacks, the mechanical characterization of the masonry was performed in laboratory by carrying out uniaxial compressive tests on limestone prisms, as similar as possible to those employed in the monument. A compressive strength of 10 N/mm^2 and a Young's modulus within the range of 20-50 kN/mm^2 were found.

The static and dynamic monitoring systems were installed in the main nave of the church. These systems are targeted on the structural behavior of the columns, because these elements control the structural behavior of the nave. The static monitoring system is composed by, see also Figure 8: (a) six temperature sensors (TS1 to TS6); (b) two uniaxial tilt meters (C1 and C2); (c) one data logger (D) for the data acquisition and data record. A wind sensor and a hygrometer are currently being added to the system, with the purpose of completing the study of environmental influences.

The dynamic monitoring system is composed by two strong motion recorders (with converter ana-lyzer) connected each one to one triaxial accelerometer. Two points were selected to install the accelerometer sensors as it can be seen in Figure 9. The two recorders are connected by an enhanced interconnection network, which allows a common trigger and time programmed records.
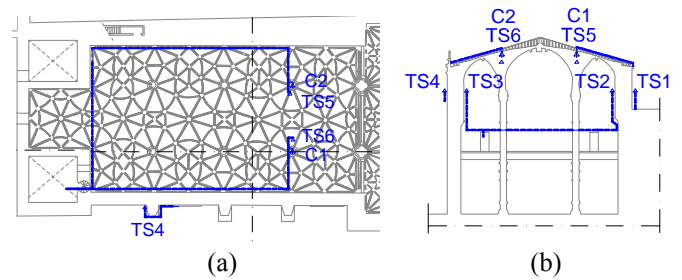

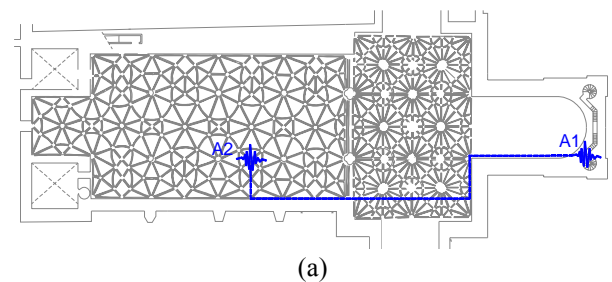Figure 8. Static monitoring system: (a) vault plan; (b) nave cross-section

Figure 9. Dynamic monitoring system: (a) location of the accelerometers (A1 on the base and A2 on the top of the vault nave); (b) sensor A2 directly on the vault extrados; (c) strong motion recorder (analyzer)

The main nave (vault and columns) of the church was dynamically identified by resorting to two experimental techniques (EFDD and SSI). Thirty points on the extrados of the vault were selected to measure the acceleration response, see Figure 10a. The first and second mode shapes are presented in Figure 10b,c. They are not global modes of the church, as only its main nave vibrates in y (North-South) and z (vertical) directions.

Table 2 summarizes the four estimated resonant frequencies, damping coefficients and Modal Assurance Criteria (MAC) for both techniques. Modal identification of the nave columns identified typical first mode shape configurations with 7.0 Hz resonant frequency.
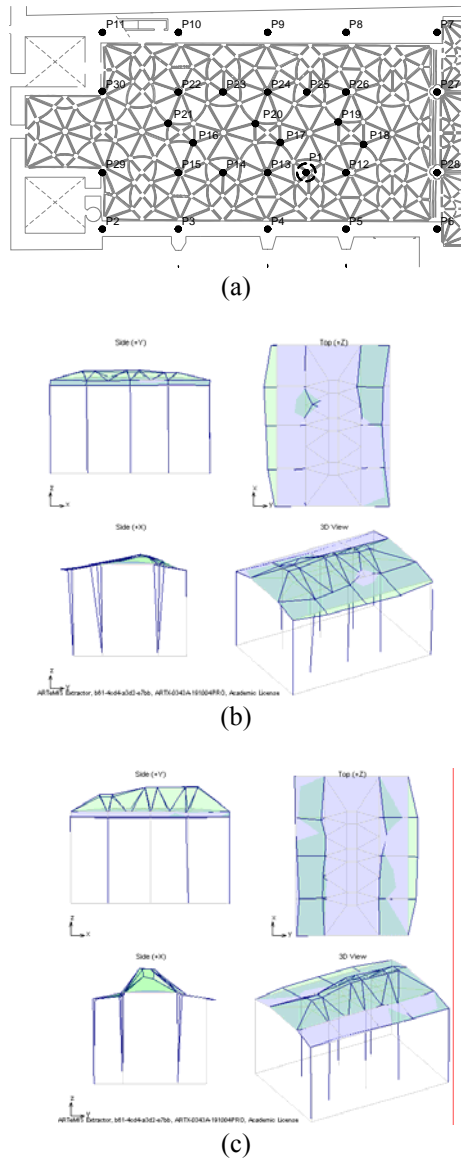
(a)

(b)

(c)

Figure 10. Experimental modal identification of the nave vault: (a) in-plan measurement points location; (b) first mode shape result at 3.7 Hz; (c) second mode shape result at 5.1 Hz

Table 2. Measured resonant frequencies and damping coefficients on the vault.

Mode	Frequency [Hz]		Damping [%]		MAC
	EFDD	SSI	EFDD	SSI	
1	3.69	3.68	2.34	1.26	0.99
2	5.12	5.04	1.11	2.68	0.92
3	6.29	6.30	1.00	0.82	0.67
4	7.23	7.29	0.77	1.44	0.67

A seismic study, compiling seismicity and tectonic data, between 1900 and 1990 in the Western zone of the Iberian Peninsula has revealed that Lisbon region has two major source zones: (a) the Atlantic adjacent zone and (b) the Continental zone. Supported by seismic hazard studies conducted for mainland Portugal (see Sousa, 2006), three stochastic hazard scenarios were used with return periods of 475, 975 and 5000 years, respectively. In the absence of available seismic records, three acceleration time-histories were artificially generated for each seismic scenario (see Table 3) using advanced theoretical models, see Carvalho et al. (2004).

Table 3. Seismic scenarios and main features of the correspondent acceleration time-histories generated.

Return Period (years)	Magnitude (M_w)	Source Distance (km)	PGA (g)	Duration (sec.)	Name of the accelerogram
475	7.4	204.7	0.10	10.2	475_M74_1
		204.7	0.09	10.6	475_M74_2
		204.7	0.12	7.70	475_M74_3
975	7.8	204.7	0.17	14.9	975_M78_1
		204.7	0.17	13.9	975_M78_2
		204.7	0.14	15.0	975_M78_3
5000	8.2	204.7	0.21	20.29	5000_M82_1
		204.7	0.23	20.71	5000_M82_2
		204.7	0.21	20.75	5000_M82_3

No site effects were considered for local seismic action. In fact, based on an existing geological-geotechnical report, on recent excavations performed for an adjacent construction and in the absence of visible settlements signs on the structure, it is expected that Jerónimos Monastery is founded on the bed rock.

4.1 Global structural analysis of the compound

As reported above, several changes were made in the structure, namely additions connecting previously separated bodies, and changes in the structures of two towers and in the roofs. The effect of these changes in the seismic performance of the structure remained an open issue. For this purpose, a preliminary pushover analysis was adopted. Pushover analysis is a non-linear static analysis carried out under conditions of constant gravity loads and monotonically increasing horizontal loads.

Aiming at validating the adopted modeling approach, a first model of the Refectory using three-dimensional volume elements and a refined geometry, the so-called refined model, was compared with a second model using shell elements and a very simplified geometry, the so-called simplified model. The refined model included the openings with larger size and the actual thickness of the walls. Vaults were represented by curved shell elements located at the center line of the elements. The simplified model did not include any openings, and the vaults were replaced by flat slabs. The slabs were located at the upper vault level because the vaults have a low curvature and it was observed that better results could be obtained by placing the flat slabs at this level, instead of placing the elements at the mass center of the vault. Additionally, in the simplified model, the vaults of the two compartments that form the entrance of the cloister were considered leveled and the staircase was substituted by a flat slab at medium height.

The models will be compared via a modal analysis because the main concern of the work is the performance under seismic action, see Lourenço et al. (2007) for details. In order to obtain similar results between the two models, the thickness of the walls in the simplified model had to be increased so that

the bending stiffness of the walls includes the additional restraint effect of the nodes (associated with transverse-walls). This effect is usually neglected in modern buildings because the height to length ratio of the structural elements (typically in the range of 1/20 in walls and 1/30 in slabs) is much lower than the values observed in ancient buildings (in the range of 1/7 for walls). The increase in thickness is up to 12%, which results in an increase of stiffness of around 40%, due to the cubic power of the thickness in the stiffness value. Without this correction the difference in the results between the refined and simplified models is large. Of course, it is debatable to adopt such a correction in the thickness for non-linear finite element analysis, due to the increase of strength of the corrected stiffness walls, associated to the square of thickness of a wall in case of a bending failure.

A modal analysis of the structure has been carried out and reasonable agreement is found between the refined and the simplified refined model, see Figure 11 and Table 4. Nevertheless it is stressed that the simplified model exhibits a significantly large number of local modes due to the vertical modes associated with the flat slabs. Local modes affect mostly the element selected and hardly the entire structure. It seems that it is possible to conclude that the corrected simplified model allows an adequate representation of the dynamic behavior of the construction. Note that free vibration of the flat slabs was precluded in the analysis.
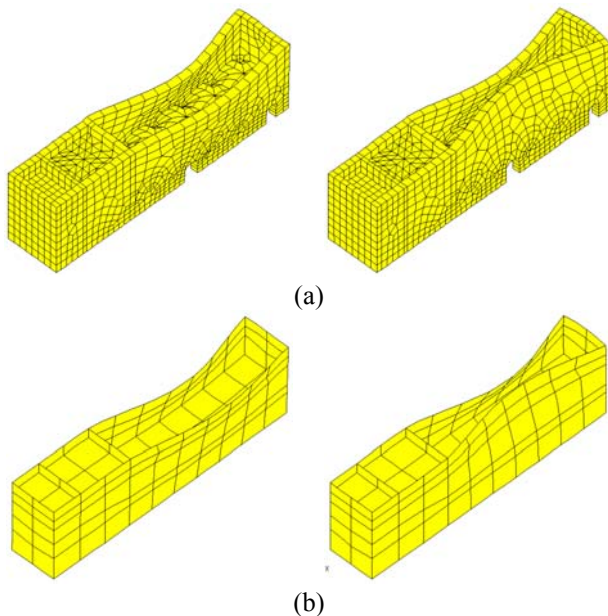

(a)

(b)

Figure 11. Shape of the first two global vibration modes: (a) refined model; (b) simplified model

Table 4. Natural frequencies f associated with the first six global modal shapes (Hz).

Refined model	$f_1 =$ 1.79	$f_2 =$ 2.26	$f_4 =$ 3.34	$f_5 =$ 3.78	$f_7 =$ 4.70	$f_8 =$ 5.41
Simplified model	$f_1 =$ 1.61	$f_2 =$ 2.41	$f_4 =$ 3.25	$f_7 =$ 3.98	$f_9 =$ 4.39	$f_{12} =$ 5.31

In the complete model only the very large openings were considered. The geometry of the model was referred to the average surfaces of the elements. All the walls, columns, buttresses, vaults and towers were included in the model, with the exception of a few minor elements. The vaults were represented as a flat slab with variable thickness due to their geometric complexity. The finite element mesh is predominantly rectangular and structured, but, for the towers and local refinements, triangular finite elements are also adopted. All elements possess quadratic displacement fields. The mesh includes around 8000 elements, 23500 nodes and 135000 degrees of freedom. The time-effort necessary for total mesh generation, including definition of supports, loads and thicknesses, can be estimated in three man-months.

For the simplified safety assessment, five independent non-linear analyses were carried out, namely for vertical loads and for seismic loading along two directions (with positive and negative sign). According to the Portuguese Code (RSA, 1993), it was assumed that the horizontal loads equivalent to the seismic action are 22% of the vertical loads, magnified by a loading safety factor of 1.5. For the non-linear analyses, a tensile strength of 0.1 N/mm^2 was adopted. Detailed information on the analyses can be found in Mourão (2001). All analyses were carried out with the software DIANA (TNO, 2005).

Figure 12 shows selected results of the analysis. The towers of the Museum are the critical structural elements featuring displacements of around 0.10 m in each case and cracks of around 0.01 m. Other cracks are visible in the church.

Maximum compressive stresses reach values up to 4.0 N/mm^2. These values are much localized in the buttresses, in one of the bodies adjacent to the monument and in the arcade. Given the fact that this is an accidental loading condition and that the stresses are much localized, it is assumed that the structure is not at risk. The average maximum values are around 2.0-2.5 N/mm^2, which seem acceptable. The force-displacement diagram for the critical seismic loading is shown in Figure 12c. Here, the load factor represents the ratio between the design loads and the applied load and the displacement is the measured horizontal component at the tower top. The analysis was continued further until collapse of the tower, which occurred for a load 25% higher than the applied design load, at a displacement larger than 0.25 m.

The non-linear analyses using the simplified model seem to demonstrate that Monastery of Jerónimos is a reasonably safe construction in what concerns the wall behavior under seismic loading. As the vaults have not been properly considered in the model, a conclusion regarding the safety of the vaults is not possible.
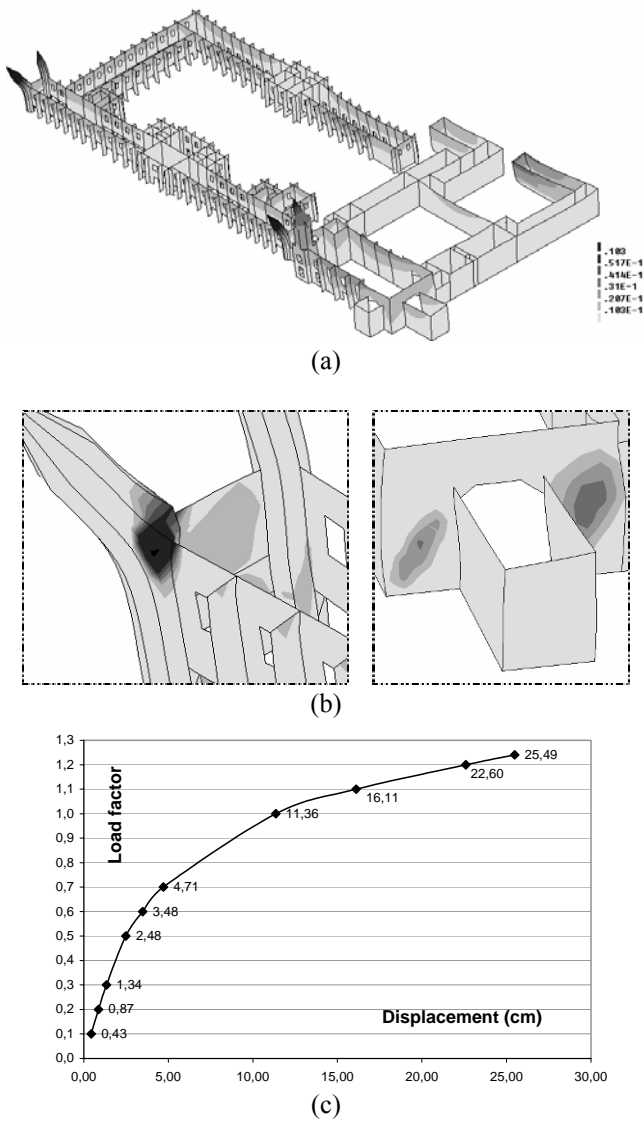
(a)

(b)

(c)

Figure 12. Selected results of the analysis for transverse loading: (a) deformed meshes and contour of maximum displacements (maximum displacement is around 0.1 m); (b) examples of cracking for tower and wall in the transept (maximum crack width is around 0.01 m); (c) force-displacement diagram

4.2 Detailed structural analysis of the church for vertical loading

The columns of the church are very slender and exhibit out-of-plumbness. As the model previously adopted for the church was very simplified and the vaults were not adequately represented, a more refined model has been adopted for a new study of the church under vertical loading.

In historical constructions, the borderline between architectural details and structural elements is not always clear. The complexity of the structure addressed in the previous section increases the difficulty in defining a finite element model appropriate for structural analysis. The lack of historical information, and the scarcity of mechanical data, limits the quality of analysis and the interpretation of data. Therefore, the adopted model should not be excessively complex.

The adopted model for the main nave includes the structural detail representative of the vault under the most unfavorable possibility, see Figure 13a, using

symmetric boundary conditions. Therefore, the model represents adequately the collapse of the central-south part of the nave. The model includes three-dimensional volume elements, for the ribs and columns, and curved shell elements, for the infill and stones slabs, see Figure 13b,c. The external (south) wall was represented by beam elements, properly tied to the volume elements. The supports are fully restrained, being rotations possible given the non-linear material behavior assumed. All elements have quadratic interpolation, resulting in a mesh with 33335 degrees of freedom. The time-effort necessary for total mesh generation, including definition of supports, loads and thicknesses, can be estimated in three man-months.
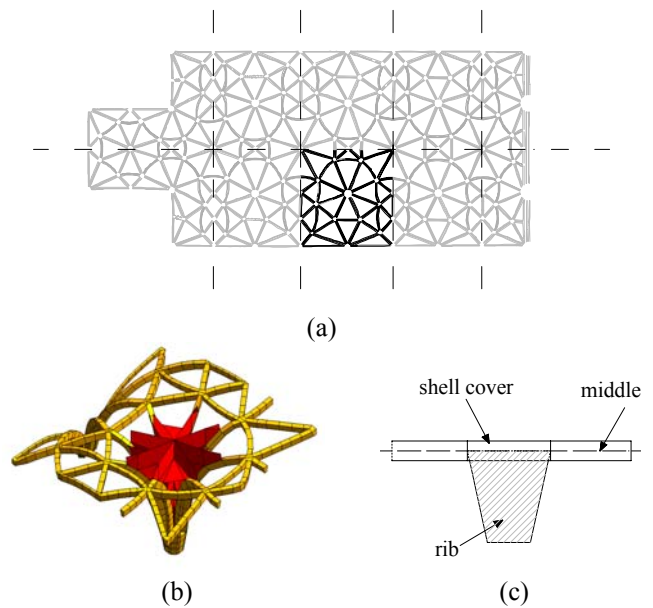

(a)

(b)

(c)

Figure 13. Aspects of the model: (a) basic pattern, (b) details around capital and (c) detail of the connection between the rib and infill

The actions considered in the analysis include only the self-weight of the structure. Two different types of materials have been considered, one type for the stone masonry (Young modulus $E = 3000$ N/mm^2 and compressive strength $f_c = 6.0$ N/mm^2) and another type for the rubble infill ($E = 1000$ N/mm^2 and $f_c = 1.0$ N/mm^2). The tensile strength has been assumed equal to zero for both materials. The material model adopted in the analysis was a total strain crack model with an ideal plastic compression limiter, please consult TNO (2005) for details.

The results are shown in Figure 14, in terms of load-displacement diagrams, deformed mesh, maximum principal strain (equivalent to tensile damage) and minimum principal stresses (compression). Further discussion on the results can be found in Lourenço and Krakowiak (2003). Figure 14a illustrates the load-displacement diagrams for the vault key and top of the column. Here, the load factor represents the ratio between the self-weight of the

structure and the applied load. It is possible to observe that the response of the structure is severely nonlinear from the beginning of loading, for the nave, and from a load factor of 1.5, for the column. The behavior of the nave is justified by the rather high tensile stresses found in the ribs, using a linear elastic model. The collapse of the columns is due to the normal and flexural action. The ultimate load factor is 2.0, which is low for this type of structures. Adding geometrical non-linear behavior in the analysis, it can be observed that this reduces the ultimate load factor to a value of 1.4, which is rather low. Nevertheless, if the compressive strength assumes the experimental value of 10 N/mm^2, the value of the ultimate load factor increases over two folds, see Lourenço and Krakowiak (2003).
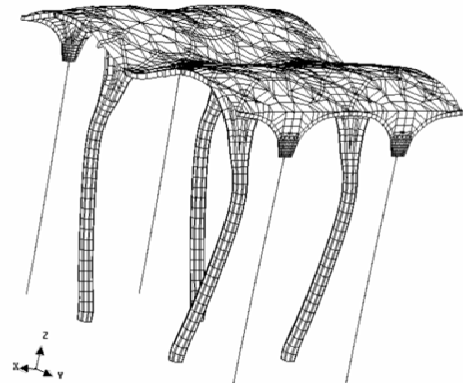
The deformed mesh at failure, see Figure 14b, indicates that the structural behavior is similar to a two-dimensional frame, with a collapse mechanism of five hinges (four hinges at the top and base of the columns and one at the key of the vault. Nevertheless, there is some vault effect with slightly larger displacements at the central octagon, formed between the four capitals. The stresses are bound in tension and compression, meaning that cracking and crushing occurs. Figure 14c illustrates the maximum principal strains, which are related to cracking of the structure. The pairs of transverse ribs that connect the columns (in the central part of the structure) exhibit significant cracking, as well as the infill in the same area. Additional cracking, less exuberant and more diffused, appears in the central octagon defined by the capitals of the four columns. Such cracking occurs at the key of the octagon and in the longitudinal ribs, which confirms the larger displacements of the vault and the bi-directional behavior of the vault. Finally, Figure 14d illustrates the minimum principal stresses at failure. It can be observed that very high compressive stresses are found in the capital ribs, particularly in the transversal area that connects a pair of columns. The columns exhibit also very high compressive stresses, which lead to the collapse mechanism described before.

## 4.3 Detailed structural analysis of the church for seismic loading

For the numerical analysis of the structure under seismic loading, a global model of the church and adjacent structures was developed (see Figure 15). Despite the high complexity of the structure, a simplified 3D model composed of beam elements (3 nodes, isoparametric formulation, axial and transversal integration) was adopted because step-by-step non-linear dynamic analysis with very refined meshes is not recommended due to time constraints. An ideal plastic material model was adopted with a compressive strength of $f_c = 10$ N/mm^2 and a tensile strength of $f_t = 0.01$ N/mm^2.
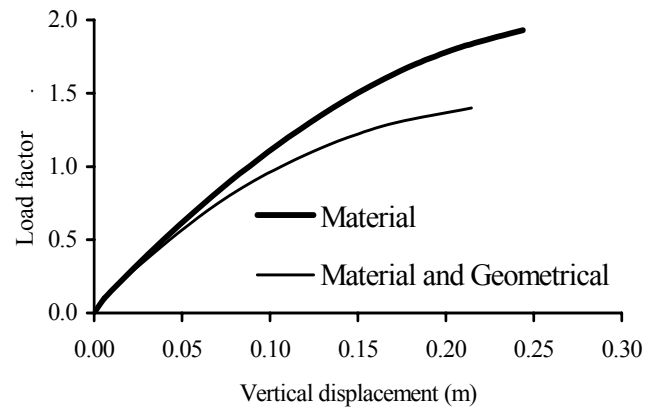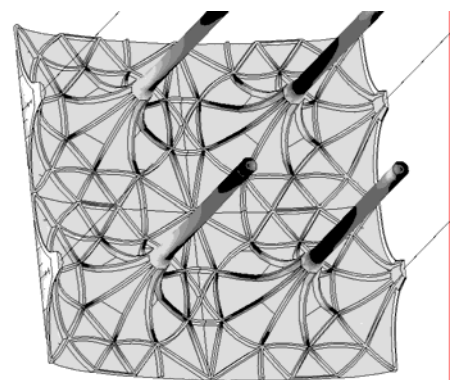

(a)

(b)
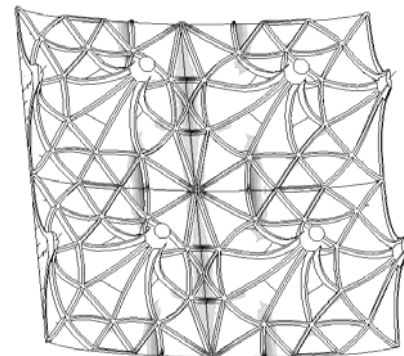
(c)

(d)

Figure 14. Results of nave analysis: (a) load-displacement diagram, (b) incremental deformed mesh at failure, (c) maximum principal strains (equivalent to cracks) and (d) minimum principal stress (compression)
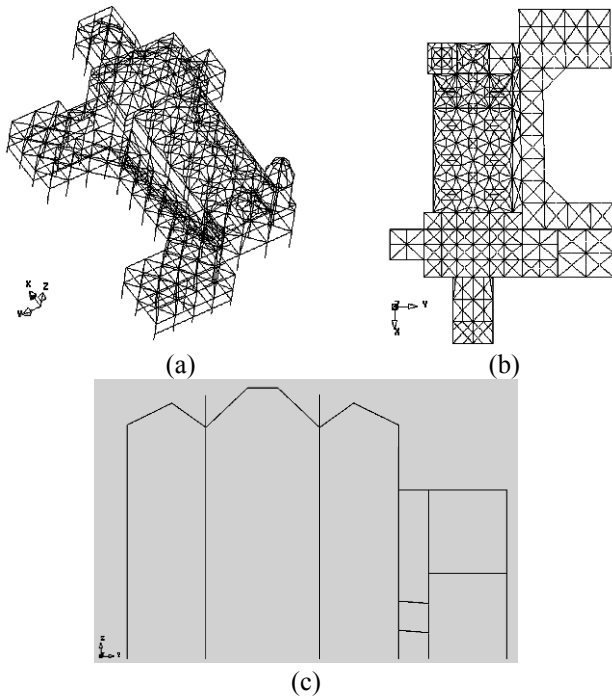
Figure 15. Finite element mesh: (a) perspective view; (b) plan view; (c) nave transversal cross section

The calibration of the model was performed in two phases. First, a preliminary comparison against the detailed numerical analysis of the vault under its self-weight shown above was performed. A second calibration was based on experimental existing results obtained from the dynamic identification and laboratory tests, presented above. In this way, the Young modulus was assumed to be equal to 30 kN/mm^2 for the columns and 12 kN/mm^2 for the other structural elements. The foundation boundaries were kept fixed. The first and fourth numerical mode shapes are exhibited in Figure 16, being similar to the measured response.
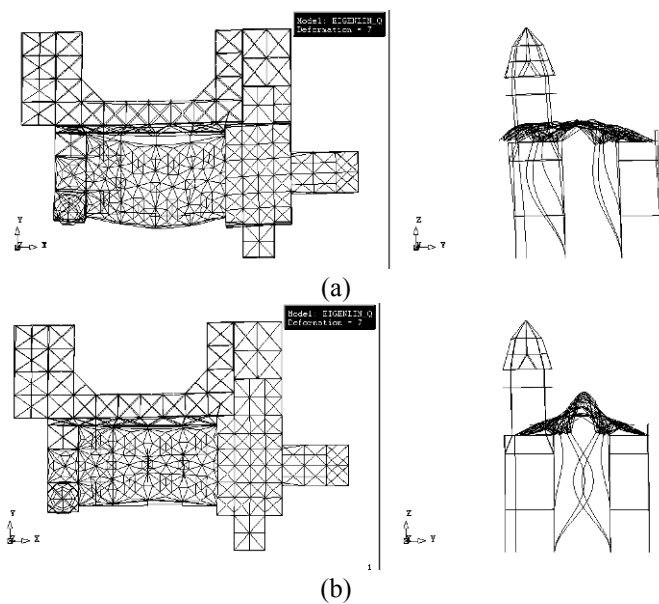

Figure 16. Numerical mode shapes: (a) 1st mode at 3.79 Hz; (b) 4th mode at 5.34 Hz

A preliminary linear static analysis under vertical and horizontal loads confirms that the transversal (y) direction of the nave controls the behaviour of the structure. Therefore, it was decided to perform non-linear static analyses for both vertical and horizontal (y) directions under an increasing gravity load factor until the development of collapse mechanisms. Collapse load factors and respective mechanisms patterns are briefly presented in Figure 17. These results show the need for a carefully numerical analysis against earthquakes, and the lack of a symmetric response.
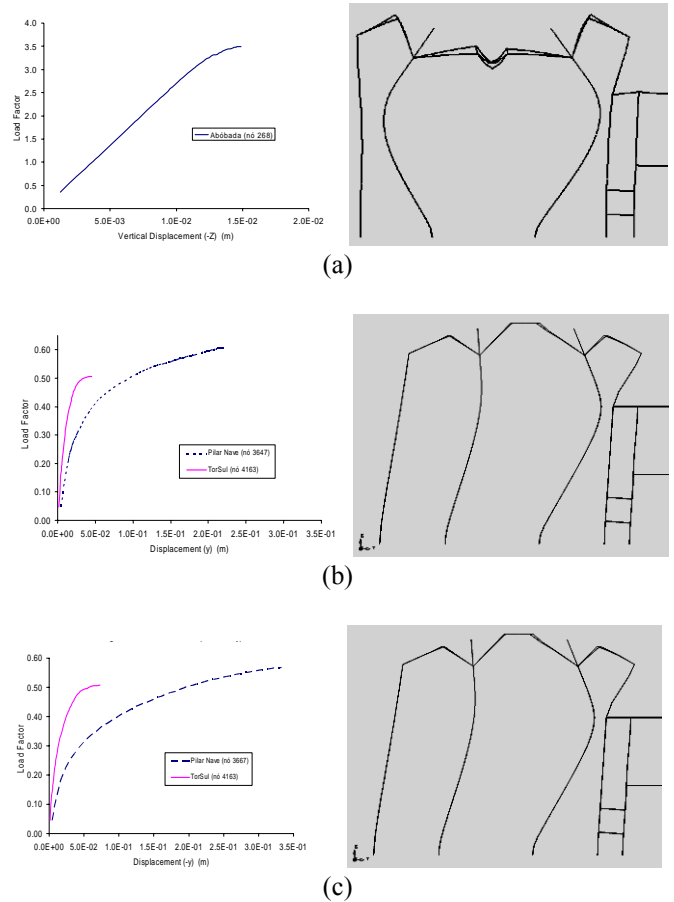

Figure 17. Results of non-linear static analysis in terms of load-displacement diagrams and transversal cross view of the nave at collapse: (a) Vertical direction (z), with $\lambda_{ult} = 3.50$; (b) Horizontal direction (+y), with $\lambda_{ult} = 0.60$ (0.50 for the bell-tower); (c) Horizontal direction (–y), with $\lambda_{ult} = 055$ (0.50 for the bell-tower). The collapse load factor λ equals the applied load / self-weight.

Following the methodology presented above, non-linear dynamic analyses were performed for the transversal (y) direction using the HHT time integration method with steps of 0.01 seconds. A damping coefficient of 2% was adopted for the computation of the Rayleigh matrix. Only results from the first hazard scenario are available at this stage (475 yrp). The numerical results obtained for this scenario, see Figure 18, show that:

- Maxima "drift" values are below 0.3% and are observed in the columns;
- The global average shear base ratio in Y direction is about 0.10 (with 0.08 for the minimum and 0.13 for the maximum);

- The maximum variation of the vertical reactions of the columns with regard to self-weight is 65%;
- The compressive stress in columns is within the elastic domain;
- The maximum compressive stress in the North and South walls (about 7.0 N/mm^2) is nearly 10 times greater than the stress caused by self-weight;
- The vaults are subjected to important compressive stresses due to the earthquake;
- The collapse of the South belfry tower by overturning is nearly to happen;
- After the earthquake, the remaining global stiffness of the structure is about 60% of the original value.
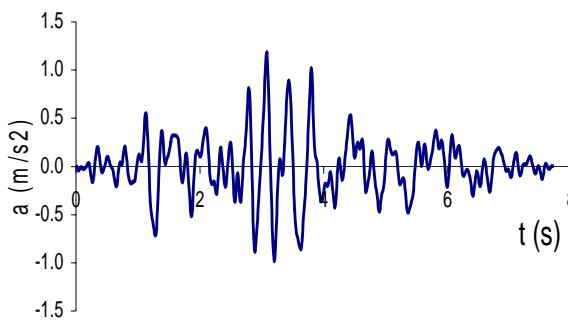
(a)
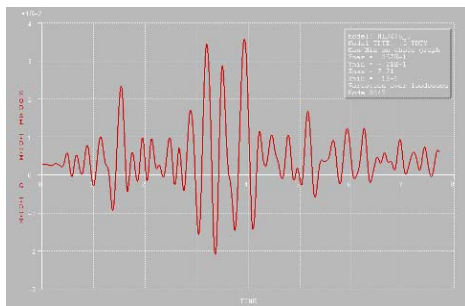
(b)
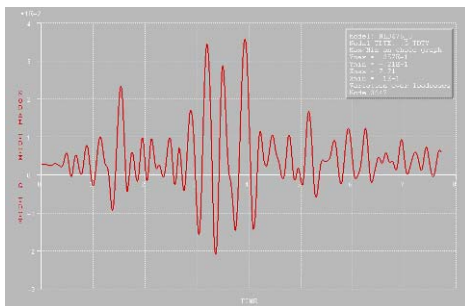
(c)

Figure 18. Seismic action and response time histories (475_M74_3): (a) synthetic accelerogram for 475 years return period; (b) vertical North column reaction; (c) transversal displacements (y) on the column

According to these results, the church will be under an important stress state against an earthquake as strong as M = 7.4 (T = 475 yrp) that will cause cracking but neither local nor global collapse is expected. The remaining two and more severe seismic hazard scenarios are currently being analyzed and, therefore, no results are available so far. These analyses are of relevance to assess the seismic safety of the monument.

5   CONCLUSIONS

The results obtained from advanced numerical simulations in historical structures are usually important for understanding their structural behavior. As a rule, advanced modeling is a necessary means for understanding the behavior and damage of (complex) historical constructions but this requires specialized consulting engineers and extensive information about the material. Numerical models can also be used as a numerical laboratory, where the sensitivity of the results to input material parameters, boundary conditions and actions is studied, and may be invaluable in the conception and understanding of in situ testing and monitoring. This has been demonstrated using a case study in the crown asset of the Portuguese architectural heritage: Monastery of Jerónimos, in Lisbon.

The methodology presented in this paper aims at the mitigation of the consequences of the seismic risk of historical structures and can be used towards the development of management policies for the cultural heritage.

For the numerical analysis, a full 3D mesh of the compound was prepared. The validity of the model was assessed by a comparison of modal analysis between a simplified model and a refined model. The difficulties inherent to the adoption of simplified models were addressed. Namely, special care seems necessary when (a) using shell elements in ancient buildings, as the out-of-plane bending stiffness of walls seems to become incorrect, and (b) using flat shells to represent complex vaults, as erroneous bending deformation of the walls seem to occur. Nevertheless, non-linear analyses using the simplified model seem to demonstrate that Monastery of Jerónimos is a safe construction in what concerns the wall behavior under seismic loading.

In addition, a complex 3D model of the church for vertical loading allowed to conclude that: (a) collapse of the nave occurs with a failure mechanism involving the columns and the vault; (b) the compressive strength of masonry is a key factor for the response of the nave; (c) the slenderness of the nave columns play a major role in the response.

Finally, a 3D mesh with beam elements considering both non-linear material and geometric behavior was developed and calibrated against experimental results. The numerical results concerning the step-by-step seismic analysis for a 475 yrp scenario show that the monument is submitted to a significant stress state that causes cracking, but neither local nor global collapse is reached. However, the collapse of

the South bell-tower by overturning is nearly to happen. Two more severe seismic scenarios are currently under analysis.

REFERENCES

Azevedo, J., Sincraian, G., Lemos, J.V., 2000. Seismic behavior of blocky masonry structures, Earthquake Spectra, 16(2), pp 337-365.

Binda, L., Saisi, A., Tiraboshi,C., 2000. Investigation procedures for the diagnosis of historic masonries, Construction and Building Materials, 14, pp 199-233.

BSSC, 2000. Prestandard and commentary for seismic rehabilitation of buildings, FEMA 356.

Carvalho, A.; Costa, A. C., 2004. Stochastic models with progressive fault rupture for seismic action characterization in Portugal, Report 240/04 NESDE, National Laboratory of Civil Engineering, Lisbon, Portugal,

CEN, 2005. Eurocode 8: Design of structures for earthquake resistance. Part 3: Strengthening and repair of buildings, EN 1998-3:2005.

Genin, S., 1995. Descriptive study of the vault in the Church of the Monastery of Santa Maria de Belém in Lisbon (in French), MSc. Thesis, Katholiek Universiteit Leuven, Belgium.

Genin, S., 2001. The nave vault of the Jeronimos Monastery Church in Lisbon, Proceedings of Historical Constructions 2001, Eds. P. Lourenço and P. Roca, Universidade do Minho, Guimarães, pp 293-302.

ICOMOS, 2001. Recommendations for the analysis, conservation and structural restoration of architectural heritage, International Scientific Committee for Analysis and Restoration of Structures of Architectural Heritage.

ISO, 2003. Basis for design of structure – Assessment of existing structures, ISO 13822.

Livesley, R.K., 1978. Limit analysis of structures formed from rigid blocks. Int. J. Num. Meth. Engrg., 12, pp 1853-1871.

Lourenço, P.B., 2002. Computations of historical masonry constructions, Progress in Structural Engineering and Materials, 4(3), p. 301-319.

Lourenço, P.B., Cóias e Silva, V., 1998. Survey and assessment of Portuguese heritage using non-destructive methods, in view of its seismic rehabilitation, Monument-98, Workshop on seismic performance of monuments, pp 157-166, Lisbon, Portugal.

Lourenço, P.B., Krakowiak, K.J., 2003. Stability assessment of the vaults from the Church of Monastery of Jerónimos (in Portuguese), Report 03-DEC/E-02, University of Minho, Portugal.

Lourenço, P.B., Krakowiak, K.J., Fernandes, F.M., Ramos, L.F., 2007. Failure analysis of Monastery of Jerónimos, Lisbon: How to learn from sophisticated numerical models?, Engineering Failure Analysis, 14(2), pp. 280-300.

Lourenço, P.B., Roque, J.A., 2006. Simplified indexes for seismic vulnerability of ancient masonry buildings, Construction and Building Materials, 20(4), pp 200-208.

Lourenço, P.B., Rots, J.G., 1997. Multisurface interface model for the analysis of masonry structures, J. Engrg. Mech., ASCE, 123(7), pp 660-668.

Lourenço, P.B., Rots J.G., Blaauwendraad J., 1998. Continuum model for masonry: Parameter estimation and validation, J. of Structural Engineering, 124(6), pp 642-652.

Milani, G., Lourenço, P.B., Tralli, A., 2006. Homogenised limit analysis of masonry walls. Part I: Failure surfaces, Computers & Structures, 84(3-4), pp 181-195.

Mourão, S., 2001. "Study of the seismic behaviour of the Jeronimos Monastery compound (in portuguese), MSc Thesis, University of Minho, Portugal (available from www.civil.uminho.pt/masonry).

Oliveira, D.V., Lourenço, P.B., 2004. Implementation and validation of a constitutive model for the cyclic behavior of interface elements, Comp. & Struct., 82 (17-19), pp 1451-1461.

Oliveira, M.M.P., 2002. Radar investigation of the Church of Jerónimos (in Portuguese), Report 177/2002, LNEC, Portugal.

Orduña, A., Lourenço, P.B., 2005. Three-dimensional limit analysis of rigid blocks assemblages. Part II: Load-path following solution procedure and validation, Int. J. Solids and Structures, 42(18-19), p. 5161-5180.

Page, A.W., 1978. Finite element model for masonry, J. Struc. Div., ASCE, 104(8), pp 1267-1285.

Roque, J.A., 2002. Strengthening and structural rehabilitation of ancient masonry walls (in Portuguese), MSc Thesis, University of Minho (available from www.civil.uminho.pt/masonry).

RSA (M.H.O.P.T), 1983. Safety and actions code for building and bridges structures (in Portuguese), D.L. nº.235/83, Lisbon, INCM, Portugal.

Sousa, M.L., 2006. Seismic hazard in mainland Portugal, PhD Thesis, Instituto Superior Técnico, Lisboa.

TNO, 2005. Building and Construction Research, DIANA finite element analysis, User's Manual, Release 9.1

Zucchini, A., Lourenço, P.B. (2004), A coupled homogenisation-damage model for masonry cracking, Computers & Structures, 82, pp 917-929.

Zucchini, A., Lourenço, P.B., 2002. A micromechanical model for the homogenisation of masonry, Int. J. Solids and Structures, 39(12), pp 3233-3255.