SEISMIC SAFETY ASSESSMENT OF THE CHURCH OF MONASTERY OF JERÓNIMOS, PORTUGAL

Lourenço, Paulo B.[1]; Roque, João[2]; Oliveira, Daniel V.[3]

[1] PhD, Professor, University of Minho, ISISE, pbl@civil.uminho.pt

[2] PhD, Adjunct Professor, Polytechnic Institute of Bragança, ISISE, jroque@ipb.pt

[3] PhD, Assistant Professor, University of Minho, ISISE, danvco@civil.uminho.pt

Preservation of historical constructions with high cultural heritage value is an actual theme in modern societies as these constructions play an important role in the industry of tourism and culture, and consequently in the economy and in the image of countries and self-esteem of people. The seismic hazard of Portugal and, due to its vicinity, of the Mediterranean basin puts under potential risk of damage and collapse a high number of historical constructions, namely most of the old masonry constructions, particularly vulnerable to seismic actions. The seismic behaviour of the Church of Monastery of Jerónimos, Portugal, is discussed here with a numerical simulation, using artificial seismic acceleration time histories in agreement with three seismic hazard scenarios for 475, 975 and 5000 years return periods, allowing to assess its seismic safety.

Keywords: *Historical buildings, in situ investigation, seismic assessment, numerical analysis*

INTRODUCTION

An adequate methodology to assess seismic vulnerability consists of the following main steps: (i) research, compilation and analysis of relevant historical data for generic characterization of the construction; (ii) installation of static and dynamic monitoring systems; (iii) experimental tests for mechanical characterization of materials and/or structural elements; (iv) dynamic identification of the structure; (v) characterization of the seismic action, from the identification of potential source areas to the artificial generation of acceleration time histories, including local site effects; (vi) numerical modelling and model calibration against existing experimental results, as a tool to simulate the structural behaviour; (vii) non-linear static analyses (pushover analysis) and non-linear dynamic analyses, in the time domain, for different seismic hazard levels; (viii) identification of structural vulnerabilities, collapse modes and safety evaluation; (ix) recommendations to minimize the seismic vulnerability of the construction, including possible intervention solutions for its conservation/rehabilitation or strengthening.

Monastery of Jerónimos in Lisbon is, probably, the crown asset of Portuguese architectural heritage. The construction of the monastery started in 1499 or 1500. Due to the 5% tax of the gold and spices from Africa and India, the construction initially planned was of gigantic size (four times the size of the actual monastery), including four cloisters and four dormitories. In fact, only one dormitory and one cloister were completed in the early times. The monastery is built with limestone quarried locally. During the 16^{th} century, the construction of the monument was carried out in three successive phases. The works in the 17^{th} and 18^{th} centuries are merely decorative or minor. In the 19^{th} century, questionable works of re-composition or restoration were carried out and, in 1940, an attempt to correct previous mistakes and return the monastery to its original configuration was made.

The monumental set has considerable dimensions in plan, more than 300×50 m^2, and an average height of 20 m (50 m in the towers), see Figure 1. The monastery revolves around two courts. The larger court is bordered by a long arcade of two levels that hosts the Ethnographic Museum of Archaeology and the Maritime Museum. The smaller court or the Cloister is bordered by the Church, the Sacristy, the Chapter Room and the Refectory.

Figure 1: Aerial view of Monastery of Jerónimos

The proposed methodology is applied here to the Church of Santa Maria of Belém, one body of the Jerónimos Monastery compound. Based on this methodology, the most relevant results of the study are presented and discussed, and further used to support the diagnosis of the construction. The seismic behaviour of the Church was numerically simulated using artificial seismic acceleration time histories in agreement with three seismic hazard scenarios for 475, 975 and 5000 years return periods.

IN SITU INVESTIGATIONS

The church has considerable dimensions, namely a length of 70 m, a width of 40 m 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 2. In order to assess the safety of the church, A first set of in situ investigations have been carried out, see Lourenço & Krakowiak (2004) for details: (a) three-dimensional survey of the church; (b) ultrasonic tests in the columns to assess the integrity; (c) radar investigation to detect the thickness of the masonry infill in the vault and pier; (d) removal of the roof, visual inspection, bore drilling, metal detection and chemical analysis of materials.

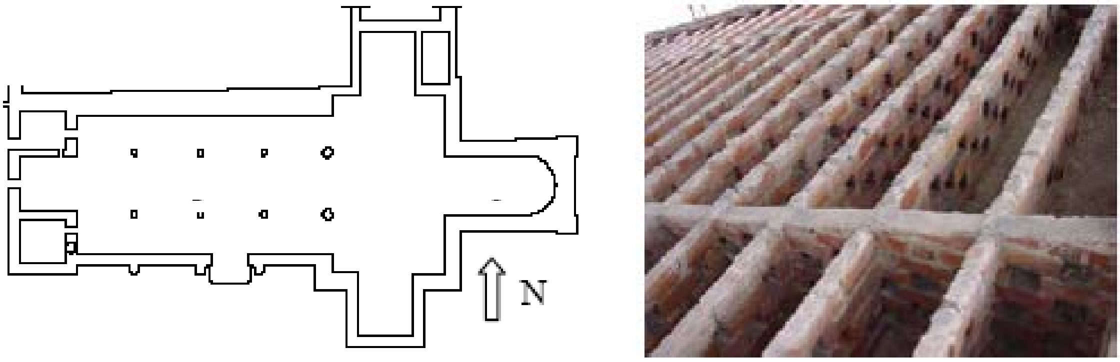


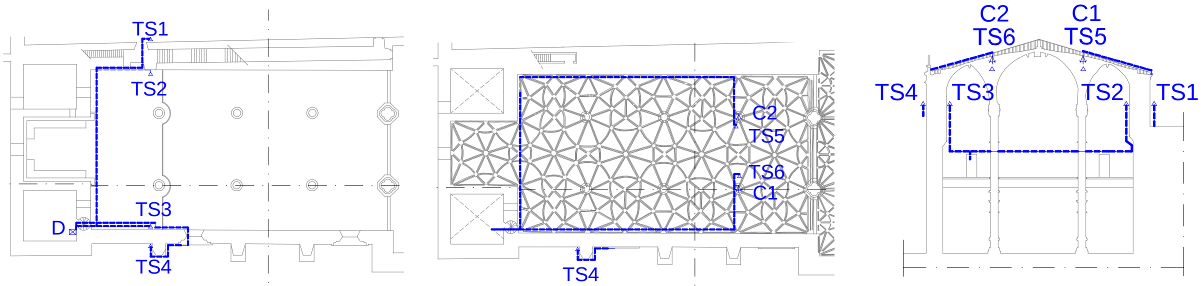
Figure 2: Survey: (a) Plan; (b) removal of the roof and existing system to support the roofing tiles

After gathering this information, simplified seismic assessment methods based on the geometry were used, according to Lourenço & Roque (2006). These methods indicated a very high slenderness of the columns and larger vulnerability in the transversal direction of the nave. Even if the value obtained for the percentage of masonry walls in each direction (12% and 17%) was rather high, due to the much relevant cultural value of the building, it was decided to make a more thorough investigation about the church safety. The additional experimental tests carried out included the following: (a) laboratory testing of the materials; (b) identification of modal parameters; (c) installation of a static and dynamic monitoring system.

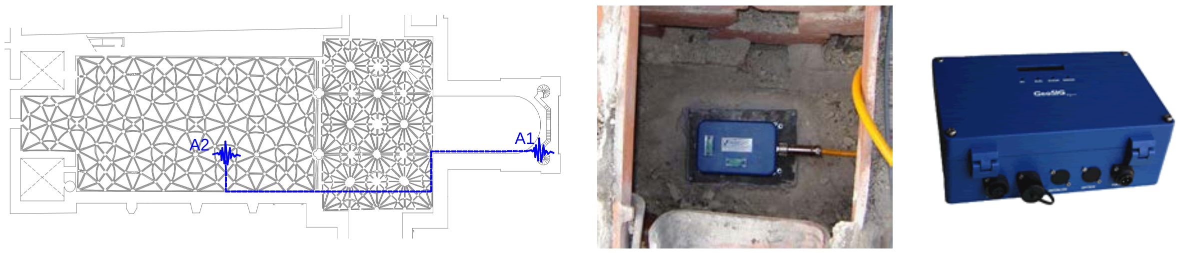
It was not possible to carry out flat-jack tests or to remove any samples from the church. Therefore, an experimental program was set-up in stone masonry replicates with different joint interposition material, see Roque (2009), subjected to monotonic and cyclic uniaxial compressive loading. The value obtained for the compressive strength of masonry was 10 N/mm^2, with considerable dispersion, as expected by using stone as the material for the masonry unit material (the coefficient of variation was about 30%). The values for the elasticity modulus ranged between 20 and 50×10^3 N/mm^2.

The static monitoring system is shown in Figure 3a and is composed of six temperature sensors, two tilt meters, one anemometer, one humidity sensor and one data logger. The sampling rate is one reading per hour in order to observe the temperature variation during the daily cycle. The temperature sensors are distributed in the structure to evaluate the effect of the temperature gradient in the response of the structure. The static monitoring system was installed in the end of May 2005 and is active until today. The dynamic monitoring system is composed by two triaxial strong motion recorders, see Figure 3b. The two recorders are connected by an enhanced interconnection network, which allows a common trigger and time programmed records. Each recorder works independently and the data are stored locally. The dynamic system is in operation since April 2005 until today. Details on the system and results are given in Oliveira et al. (2005) and Ramos et al. (2010). The movements seem to be stabilized and the small earthquakes recorded so far seem not to have provoked damage to the structure.

The dynamic identification tests were carried out using ambient vibration and allowed to identify the frequencies and modes of the structure. Thirty points in the extrados of the vault were used, see Figure 4a. Ten points are localized on the top of the external walls with the purpose to measure the nave boundaries constraints and also the global dynamic response of the church. The other points are located either on the top of the columns or on the top of the vaults keys. To ensure adequate selection of the reference point for all measurements, preliminary signal measurements were done and it could be concluded that point P1 was the one with more significant signal vibration amplitudes. For every measured point, the roof tiles were removed and the signal acquisition was done directly on the top of the nave (extrados) to avoid any possible noise signals from the roof structure. Eight mode shapes and frequencies were estimated. The first mode identified is transverse, involving the nave, the external walls, columns and buttresses. The second mode identified is local for the vault. The values of the frequencies found for the first four modes were 3.6, 5.1, 6.3 and 7.3 Hz. One additional identification of the columns allowed to estimate the first local mode of the columns for a frequency of 7.0 Hz.

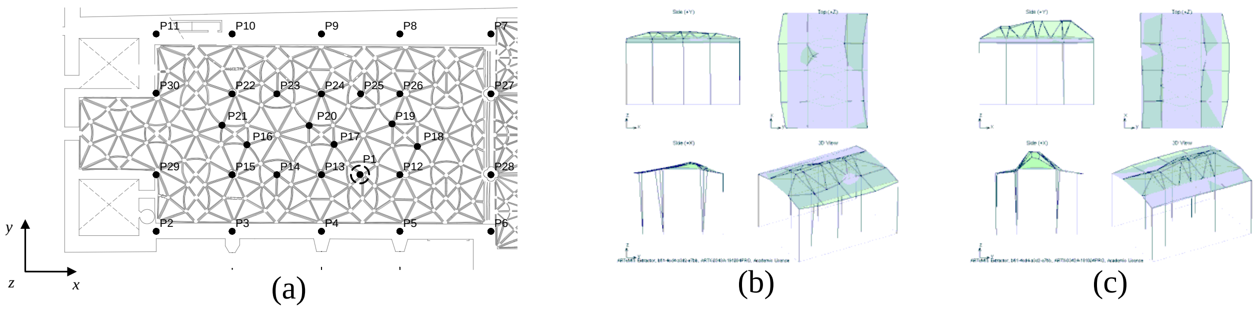


(a)



(b)

Figure 3: Monitoring systems installed in the church: (a) static system with nave plan, vault plan and cross section (TSi for temperature sensor, Ci for tilt meter and D for data logger); (b) dynamic system with location and type of strong motion recorders (A1 is at the church base and A2 is in the vault)



(a) (b) (c)

Figure 4: Experimental identification of the modal parameters of the church: (a) location of measuring points; (b) first mode; (c) second mode

GLOBAL STRUCTURAL ANALYSIS OF THE COMPOUND

Several different analyses have been carried out in this monastery, taking into consideration the data available, the sought information and the available budget for structural assessment. The first analysis carried out focused on the behaviour of the full monastery compound under seismic loading. For this purpose, non-linear static (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. 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. 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 safety assessment, five independent pushover non-linear analyses were carried out, namely for vertical loads and for seismic loading along two directions (with positive and negative sign). Figure 5 shows the deformed mesh for seismic loading along the transversal direction to the church nave. It can be seen that the towers of the Museum are the critical structural elements featuring displacements of around 0.10 m in each case and a maximum crack width of around 0.01 m. The 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 exceptional loading condition and that the stresses are localized, it is assumed that the structure is not at risk. The average maximum values for the compressive stresses are around 2.0-2.5 N/mm^2, which seem rather acceptable.

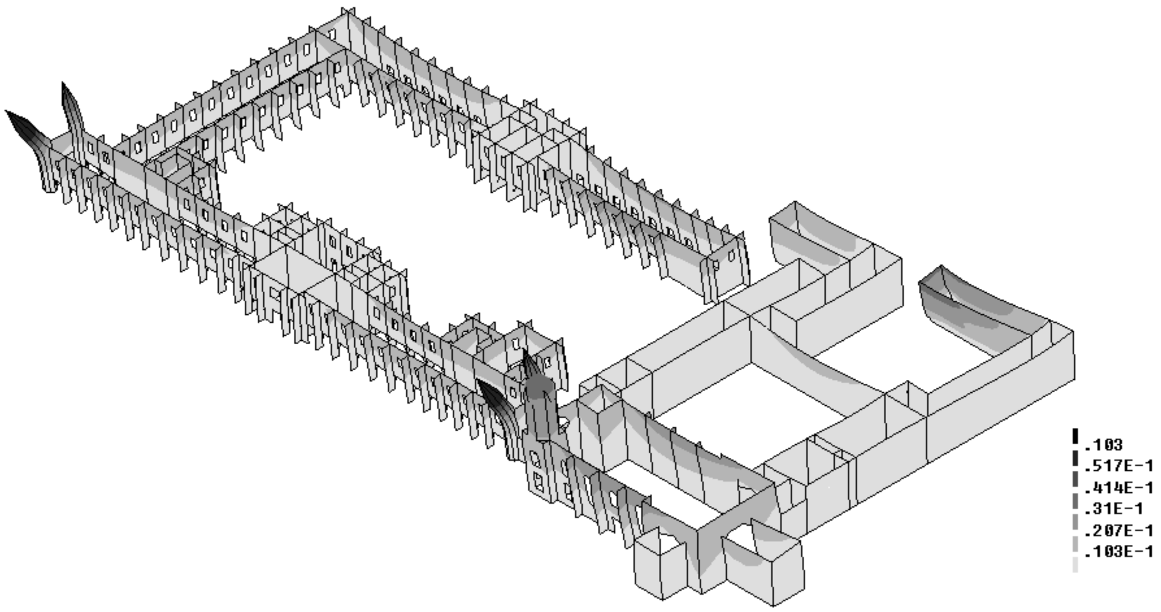


Figure 5: Deformed meshes and contour of maximum displacements for seismic load along the transversal Z axis of the model

DETAILED STRUCTURAL ANALYSIS OF THE CHURCH (GRAVITY LOAD)

The columns of the church are very slender and exhibit moderate out-of-plumb deviation. As the model previously adopted for the church was very simplified and the vaults were not adequately represented, more refined models have been adopted for a new study of the church under vertical and earthquake loading. The south wall has a thickness of around 1.9 m and possesses very large openings. Three large trapezoidal buttresses ensure the stability of the wall. The north wall is extremely robust (with an average thickness of around 3.5 m). This wall includes an internal staircase that provides access to the cloister. The chancel walls are also rather thick (around 2.5-2.65 m). The nave is divided by two rows of columns, with a free height of around 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 column drums seem to be made by a single block or two blocks, for the nave, and four blocks, for the transept. The vaults are ribbed and are connected to the columns by large fan capitals. Cross section of the nave vault is, mostly, a slightly curved barrel vault, even if supported at the columns. Thin stone slabs are placed on top of the stone ribs. On top of the slabs, a variable thickness mortar layer exists. The part of the slab inside the capital is filled with a concrete-like material with stones and clay mortar. On top of the vaults, brick masonry walls were built during the 1930’s to provide support for the roofing tiles.

The adopted model for the main nave includes the structural detail representative of the vault under the most unfavourable possibility, see Figure 6a, 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 6b. 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 behaviour 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.

The actions considered in the analysis include only the self-weight of the structure. Figure 6c 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 behaviour of the nave is justified by the rather high tensile stresses found in the ribs. The collapse of the columns is due to the normal and flexural action. The deformed mesh at failure, see Figure 6d, indicates that the structural behaviour 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). The compressive strength of the columns controls the safety of the church for vertical loading. Nevertheless, there is some vault effect with slightly larger displacements at the central octagon, formed between the four capitals. The stresses are bounded in tension and compression, meaning that cracking and crushing occurs. Note that the results are shown for a conservative compressive strength of 6 N/mm^2. If the value found experimentally is used, the safety factor of the structure is larger than five, meaning that no damage due to the vertical loading should be expected in the structure.

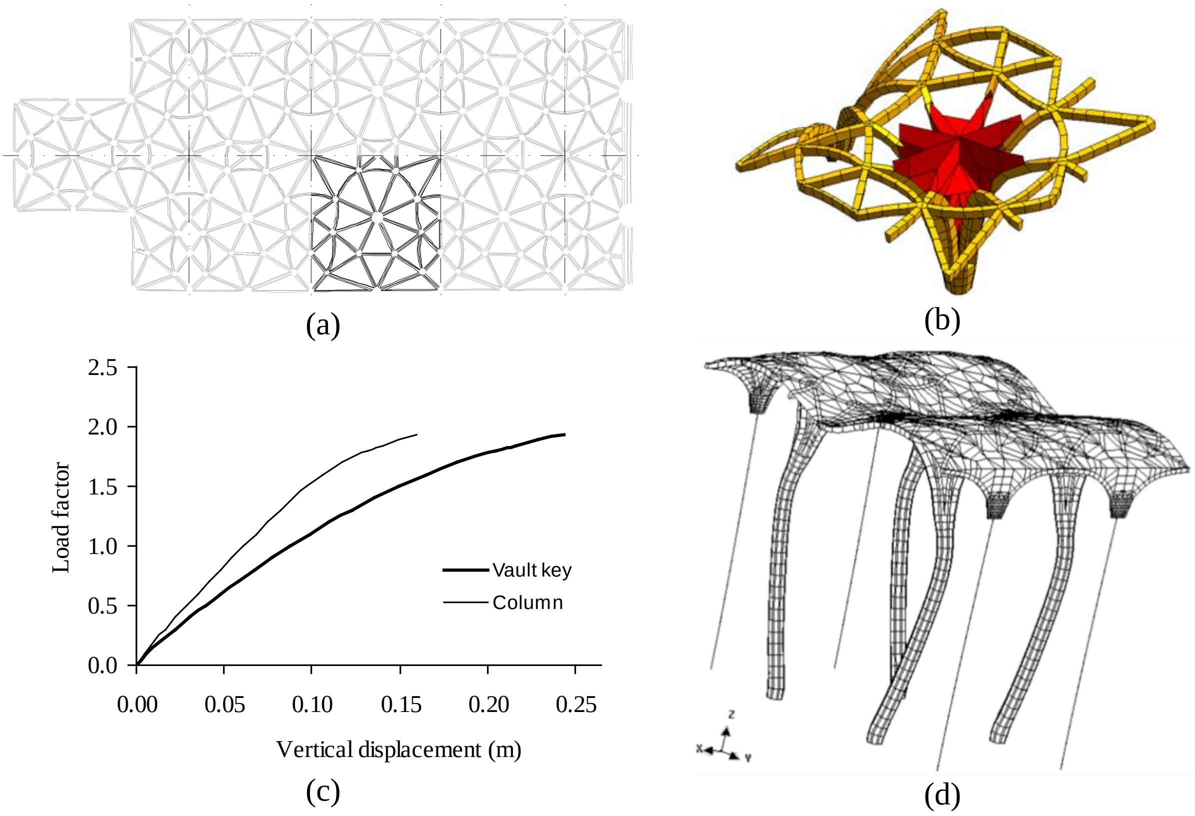


Figure 6: Detailed model of the church for vertical loading: (a) church plan and basic plan; (b) details around capital; (c) load-displacement diagram; (d) incremental deformed mesh at failure

DETAILED STRUCTURAL ANALYSIS OF THE CHURCH (EARTHQUAKE LOAD)

For the seismic analysis of the church, a simplified model using beam elements was used, with a total of 25452 degrees of freedom, see Figure 7a. The time-effort necessary for total mesh generation, including definition of supports, loads and thicknesses, can be estimated in three man-months. This model was subjected to pushover analyses and time integration. The time for analysis and testing different retrofitting techniques can be estimated in nine man-months.

The model elastic properties were defined in order to replicate the modal identification results, with an additional validation with the temperature data from the static monitoring system and a comparison in the non-linear range with the previous detailed model for gravity loading. Subsequently, pushover analyses parallel and perpendicular to the nave of the church were carried out. Figure 7b-f shows results in terms of seismic load vs. selected horizontal displacements, failure mechanism in the finite element model and virtual collapse mechanisms. The results indicated that the columns are again much relevant for the collapse of the nave, with eccentric compression and cracking. The weakest mechanism for the seismic loading is the transverse one involving collapse for the non-constrained side of the nave, but only with 10% difference with respect to the collapse of the cloister side. Still, adequate global strength seems to be found with the push-over analysis.

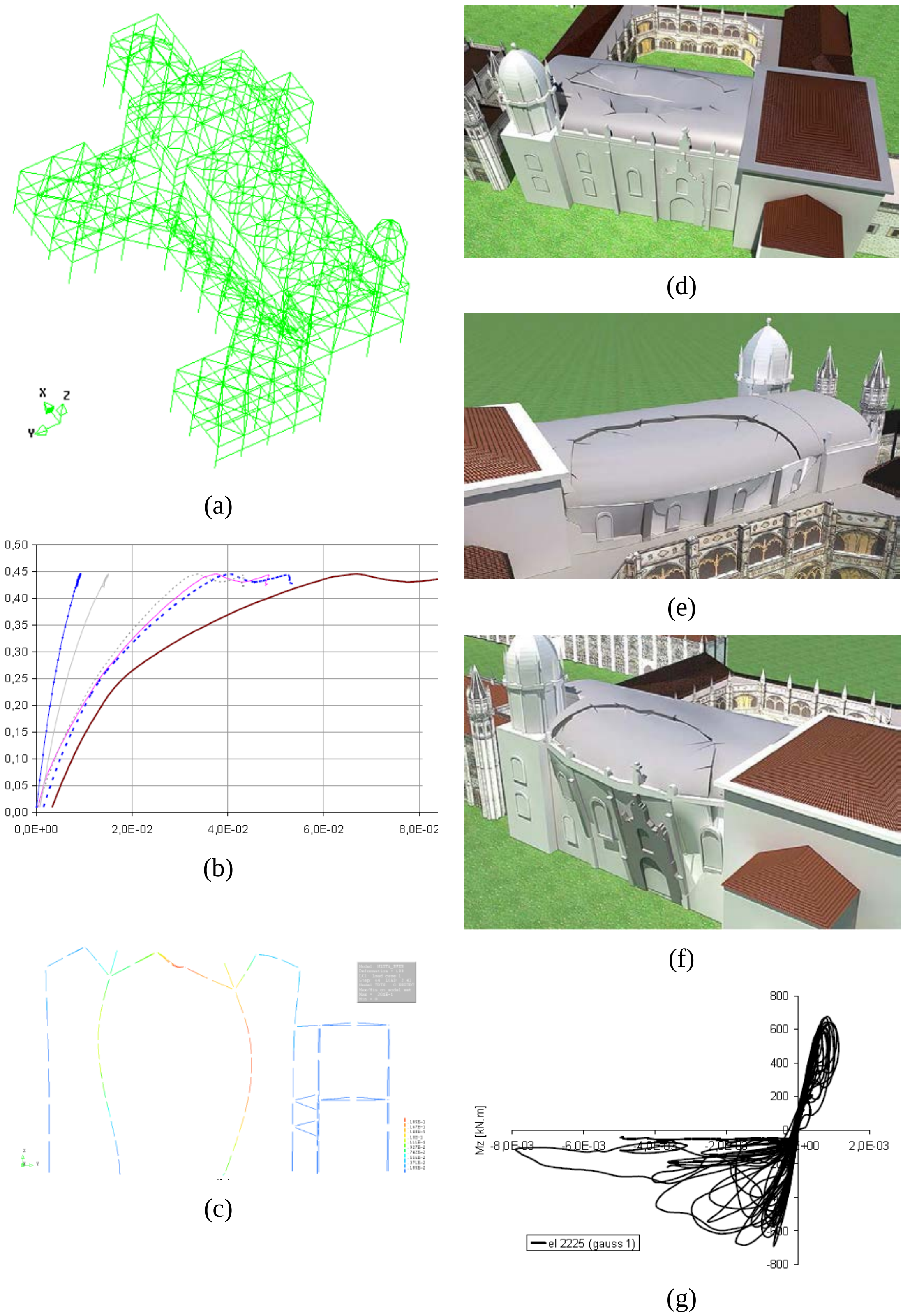
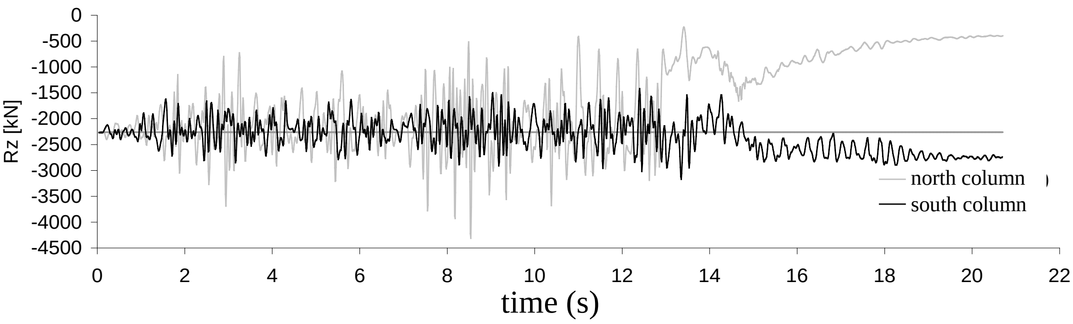


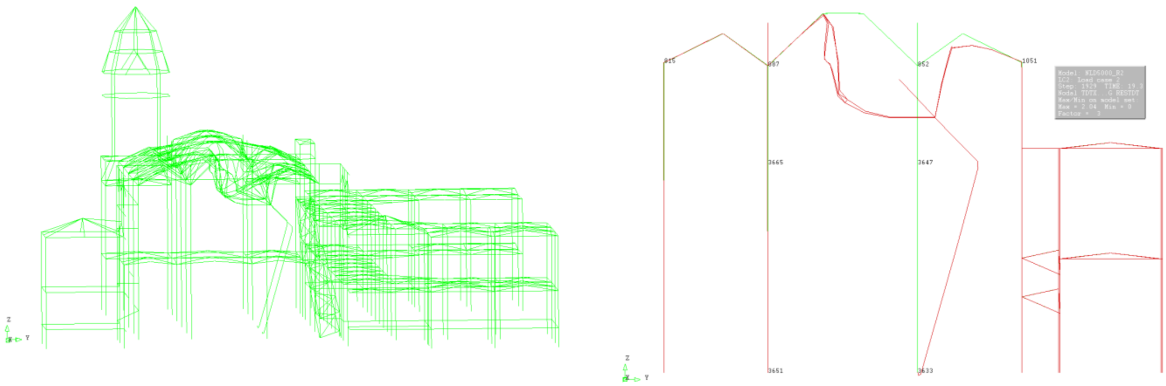
Figure 7: Detailed model of the church for horizontal loading: (a) finite element mesh; (b) pushover results, seismic action vs. displacement at given nodes; (c) example of failure mechanism; (d-f) virtual collapse mechanisms for different pushover analyses; (g) example of column response for time integration analysis

In order to further validate these observations about safety and discuss the damage introduced by an earthquake, non-linear time integration analyses were carried out, see Figure 7g for the typical results involving collapse of one of the columns in terms of moment-curvature relation. The results indicate that no local or total collapse is to be expected for a 475 or 975 years return period (yrp) earthquake, even if severe stress states and cracking occur in the vault and nave columns.

With respect to the seismic scenarios with 5000 yrp, the behaviour of the church is changed and partial collapse is found, as shown by Figure 8a. Figure 8b,c presents the failure mechanisms, which clearly involve the north columns of the nave. It is noted that this is a rather severe earthquake and a demand much higher than the one normally used for new buildings. Still, in Roque (2009) possible strengthening solutions are proposed and their seismic safety is assessed.



(a)



(a) (b)

Figure 8: Structural response for a 5000 year return-period: (a) vertical reaction in columns; (b) general view of the failure mechanism; (c) failure mode for failure mechanism

CONCLUSIONS

Dating from the 16th century, the Monastery of Jerónimos is, probably, the crown asset of Portuguese architectural heritage. Given the cultural importance of the construction, the safety of the users, the seismic hazard and the accumulation of physical, chemical and mechanical damage, a large safety assessment program of the structure was carried out.

This paper briefly review the in situ investigations carried out and the monitoring systems installed. Then, a more detailed review of the structural analyses carried out was presented, including a global analysis of the monastery compound, a detailed analysis of the church for vertical loading and a detailed analysis of the church for seismic loading. The results indicate that the safety level of the structure is adequate for vertical and horizontal loadings. Still, the monitoring system installed allows the structural health of the church to be monitored, particularly in case of future earthquakes, providing excellent reference information for future analysis of damage.

ACKNOWLEDGEMENTS

This work was partly funded by project NIKER, Grant Agreement 244123 of the 7th Framework Programme of the European Commission Project.

REFERENCES

Lourenço, P.B., Krakowiak, K.J. "Safety assessment of the Jerónimos church in Lisbon", Computer Methods in Structural Masonry – 6, Rome, Italy, 2004, pp. 75-81.

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

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

Oliveira, D.V., Ramos, L.F., Lourenço, P.B., Roque, J.A. "Structural monitoring of the Monastery of Jerónimos", International Conference – 250th Anniversary of the 1755 Lisbon Earthquake, Lisbon, Portugal, 2005, CD-ROM.

Ramos, L.F., Marques, L., Lourenço, P.B., De Roeck, G., Campos-Costa, A., Roque, J., Monitoring of Historical Masonry Structures with Operational Modal Analysis: Two Case Studies, Mechanical Systems and Signal Processing, 24(5), 2010, pp. 1291-1305.

Roque, J.A. Metodologia integrada para avaliação e mitigação da vulnerabilidade sísmica das construções históricas em alvenaria: A Igreja dos Jerónimos como caso de estudo. PhD thesis, University of Minho, Portugal, 2009, 231pp (in Portuguese). Available from www.civil.uminho.pt/masonry.