FROM FRACTURE MECHANICS TO CASE STUDIES: THE ISSUE OF CULTURAL HERITAGE

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Resumen. La conservación y restauración de estructuras históricas continúan siendo un desafio para los profesionales a pesar de los adelantos científicos encontrados en las últimas decadas. Avances significativos han sido realizados en ensayos no destructivos, caracterización mecánica, herramientas para análisis numéricos avanzados, conocimiento en los materiales tradicionales e innovadores así como sus técnicas de aplicación. En este trabajo algunos desenvolvimientos recientes son abordados y casos de estudio que utilizan herramientas de analisis avanzado son presentados, enfocados para dar una base en la discusión de su investigación y desarrollo.

Abstract. Conservation and restoration of historical structures are still a challenge to modern practitioners even if significant research advances have occurred in the last decades. Significant advances have been made in non-destructive testing, mechanical characterization, tools for advanced numerical analysis, knowledge on traditional materials and techniques, and innovative materials and techniques. Here, some recent developments are addressed and challenging case studies that make use of advanced analysis tools are presented, aiming at providing a basis for discussion of research and development.

1. INTRODUCTION

Time shows that many historical masonry constructions collapsed due to accidental actions, like earthquakes. Nevertheless, not only exceptional events affect historical constructions. Fatigue and strength degradation, accumulated damage due to traffic, wind and temperature loads, soil settlements and the lack of structural understanding of the original constructors are high risk factors for the architectural heritage.

The analysis of historical masonry constructions is a complex task. Firstly, limited resources have been allocated to the study of the mechanical behavior of masonry, which includes non-destructive in situ testing, adequate laboratorial experimental testing and development of reliable numerical tools. Secondly, and most important, the difficulties in using the existing knowledge are inherent to the analysis of historical structures.

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. It should be expected that results of different approaches are also different, but this is not a sufficient reason to prefer one method from the other. In fact, a more complex analysis tool does not necessarily provide better results. 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 is notorious in comparison with more advanced research fields like concrete, soil, rock or composite mechanics.

2. POSSIBILITIES AND CHALLENGES

Masonry is a heterogeneous material that consists of units and joints. Units are such as bricks, blocks, ashlars, adobes, irregular stones and others. Mortar can be clay, bitumen, chalk, lime/cement based mortar, glue or other. The huge number of possible combinations generated by the geometry, nature and arrangement of units as well as the characteristics of mortars raises doubts about the accuracy of the term “masonry”. Still, the mechanical behavior of the different types of masonry has generally a common feature: a very low tensile strength. This property is so important that it has determined the shape of ancient constructions. The difficulties in performing advanced testing of ancient structures are quite large due to the innumerable variations of masonry, the variability of the masonry itself in a specific structure and the impossibility of reproducing it all in a specimen.

Accurate modeling requires a thorough experimental description of the material. The reader is referred to [1,2] for a more comprehensive discussion on these issues. A basic notion is softening, which is a gradual decrease of mechanical resistance under a continuous increase of deformation forced upon a material specimen or structure. It is a salient feature of soil, brick, mortar, ceramics, rock or concrete, which fail due to a process of progressive internal crack growth.

Masonry is a material exhibiting distinct directional properties due to the mortar joints, which act as planes of weakness. In general, the approach towards its numerical representation can focus on the micro modeling of the individual components, viz. unit (brick, block, etc.) and mortar, or the macro modeling of masonry as a composite [3].

2.1. Experimental Behavior

The properties of masonry are strongly dependent upon the properties of its constituents. Compressive strength tests are easy to perform and give a good indication of the general quality of the materials used. Recent advances are very significant due to the usage of sophisticated displacement controlled equipments. Modern possibilities include for example: (a) detailed behavior of masonry under tension and compression; (b) detailed behavior of masonry under shear, see Figs. 1-3 for details.

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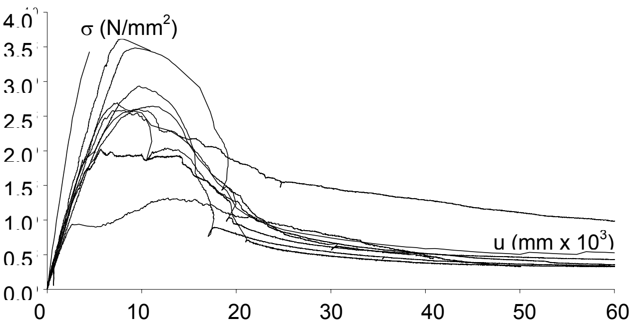
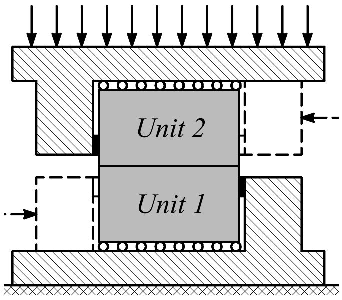


(b)

Fig. 1. Tensile test results in brick specimens: (a) crack profile; (b) stress-elongation diagrams. The thicker line is the average of all specimens [4].

Modern challenges include the cyclic behavior of units under tension, the characterization of mortar behavior (cured inside the masonry composite), irregular masonry (one leaf and multi-leaf), dry masonry, biaxial testing (displacement controlled), creep, fatigue and durability tests, and NDT in ancient materials.



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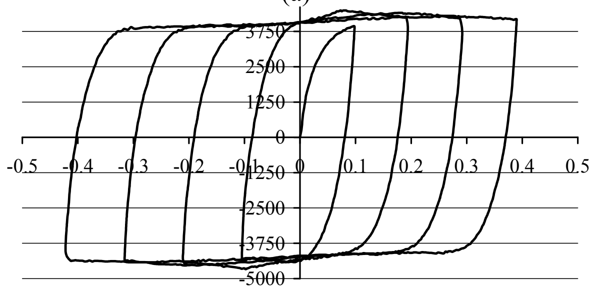


(b)

Fig. 2. Shear testing of a dry masonry joint under cyclic loads: (a) test setup; (b) stress-elongation diagrams [5].

Fig. 3. Aspects of shear wall failure under cyclic loading, for regular and irregular stone masonry [6].

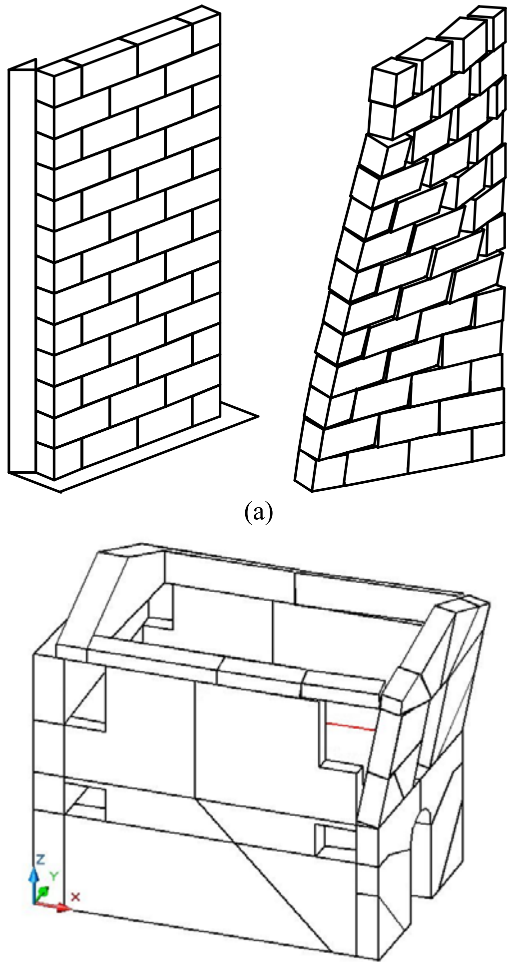
2.2. Numerical Analysis

The simplest approach to the modeling of complex historical buildings is given by the application of different structural elements resorting to truss, beam, panel, plate or shell elements to represent columns, piers, arches and vaults, with the assumption of homogeneous material behavior. This is hardly an adequate representation of masonry and a number of non-linear anisotropic models usually in the field of finite element models have been proposed. Another possibility is the use of homogenization techniques, which receives substantial attention for researchers.

The explicit representation of the joints and units in a numerical model seems a logical step towards a rigorous analysis tool. 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. Applications can be carried out using finite elements, discrete elements or limit analysis. Naturally, a higher computational effort of the approach ensues and simulations are usually focused in structural elements or details.

Recent advances include, for example: (a) an interface model for the cyclic loading of masonry structures [7]; (b) a novel solution procedure and a three-dimensional yield surface for limit analysis of blocky masonry structures [8], Fig. 4; (c) advances in the non-linear homogenization of masonry due to the incorporation of internal deformation modes [9]; (d) understanding of the mechanics of masonry under compression using meso-scale approaches [10], Fig. 5.

Modern challenges include the extension for fatigue and creep, coupled porous-mechanics problems for deterioration and life prediction, and consideration of the modern computational techniques to solve the problem of ill-conditioning of quasi-brittle continuum mechanics.



(a)

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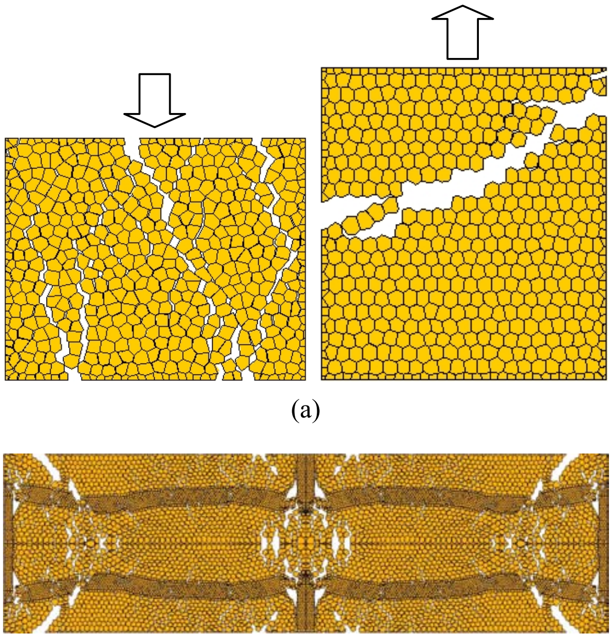
Fig. 4. Application of limit analysis of blocky masonry structures to: (a) masonry wall with out of plane failure; (b) masonry building subjected to seismic loading.



(a)

(b)

Fig. 5. Usage of a particle model to understand the mechanics of masonry under compression: (a) simulated compressive and tensile failure; (b) collapse of a masonry basic cell under vertical compression.

3. CASE STUDIES

In this section two case studies are presented in order to illustrate the engineering application of the tools presented before. This is always a difficult task and, even when an adequate diagnosis has been carried out, it is often necessary to adjust the initial design according to unexpected findings. The case studies are at different levels of remedial measures. In the first case study (Monastery in Arouca), the works have been completed. In the second case study, (Monastery of Jerónimos in Lisbon), an iterative diagnosis procedure is under way.

3.1. Chimney of Monastery of Arouca

The chimney is made of three brick masonry walls, making a tapered channel section leaning against a thick masonry wall from the Monastery envelope, supported by three stone granite lintels, see Fig. 6. The lintels are supported in stone columns and corbels. Hidden by the plaster, internal brick arches with 0.25 m thickness were found. The function of these elements is obvious, aiming at reducing the bending / tying load of the stone lintels by transmitting part of the load from the walls directly to the columns.

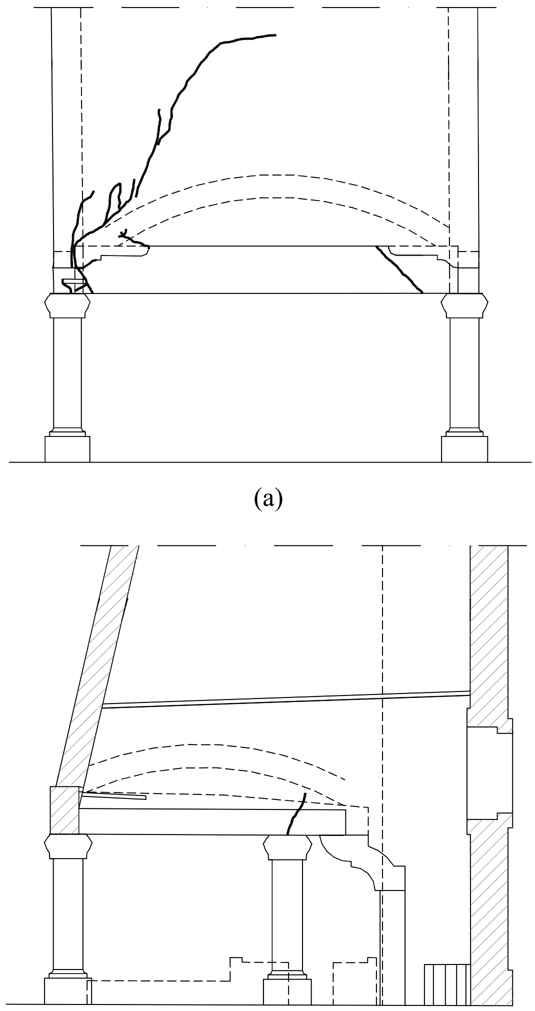
In order to uncover the hidden structure of the arches, four inspection openings were made in the plaster. It could be observed that the rubble masonry fill between the arches and the lintels was not separated from the arches, as expected from good building practice. Two other defects were found, namely: (a) the main arch is asymmetrical with respect to the lintel; (b) the cross section of the left side lintel has a tapered shape, with a height reduction towards the external wall support.

The structure of the chimney is complemented by a set of iron ties, distributed along the height and inside the chimney. These ties aim at stabilizing the main wall, which is inclined about 15º with respect to the vertical position. Finally, two iron ties are also present inside the chimney, at the column corners and aligned at 45º with the lintels. These ties are part of the original fabric and it is likely that their function was to help resisting the thrust of the system of arches / lintels. It is noted that an iron cramp was added to connect externally the main lintel and the left lintel.

The main damage exhibited by the chimney consists of a sudden diagonal crack, which appeared suddenly in the main lintel, close to the right support, and resulted in temporary propping of the structure. This crack intercepts the anchoring zone of the 45º iron tie that connects the main lintel and the right lintel, which is a singular and weaker part of the lintel. It is also noted that this tie is corroded close to the anchoring zone. The main lintel exhibits also several cracks close to the left support and in the vicinity of the iron cramp, which is severely corroded. In the masonry wall above the main lintel, a set of diagonal cracks is present. The inspection openings in the plaster indicate that the cracks do not intercept the hidden masonry arch, but they run through the arch extrados, see Fig. 6a. This set of cracks represents significant danger and a pre-collapse situation of the left support, with a failure mechanism involving rotation of the wall with a hinge forming at the right support. It is noted that an ancient crack is also present in the left side linter and wall, probably due to the reduction of height of the stone lintel in the corbel region, as discussed above, see Fig. 6b. As a result of this crack, two stone columns under each side lintel were added to the structure in an unknown date.

In order to complete the diagnosis and safety evaluation, two three-dimensional models of the chimney were prepared aiming at simulating extreme possibilities, which take into account the fact that there is not separation between the arches and the material filling the space between the arches and the lintels. The two extreme possibilities regarding the arching effect in the walls are considered here. Therefore, the first model does not include the filling material under the arches (Model 1) and the second model considers that the walls are fully supported in the lintels (Model 2). Obviously, Model 2 is more unfavorable for the stone lintels, being the most conservative approach.

The models are made of quadratic solid finite elements (bricks and wedges), with approximately 500 elements and 3805 nodes, making a total of 11415 degrees of freedom. Non-linear material properties have been considered both in tension and compression. Two loads have been considered in the analysis, one due to the self-weight of the chimney and another due to the load transmitted by the vault of the kitchen. For the numerical simulation, only the box section of the chimney has been considered.



(a)

(b)

Fig. 6. Geometry and observed cracking patterns: (a) main wall; (b) left side wall.

The linear elastic results of Model 1 and Model 2 indicated that the model without the internal arching system leads to the most unfavorable loading conditions, as expected. Therefore, this model has been selected for performing a non-linear analysis up to the collapse of the structure. Structural collapse was found for a load factor of 2.04, where the load factor represents the ratio between the applied loads and the original reference loads. For the ultimate load factor, the most damaged zones in the masonry walls occur close to the supports of the side lintels, see Fig. 7a. This damage is both in tension and compression. Fig. 7b illustrates the damage (measured by the maximum principal strains) for the lintels-columns set, which clearly defines the collapse mechanism. Three plastic hinges appeared in the side lintels, one hinge at mid-span with cracking at the lower face of the lintels (positive bending moments), and two hinges at the supports with cracking at the upper face of the lintels (negative bending moments).

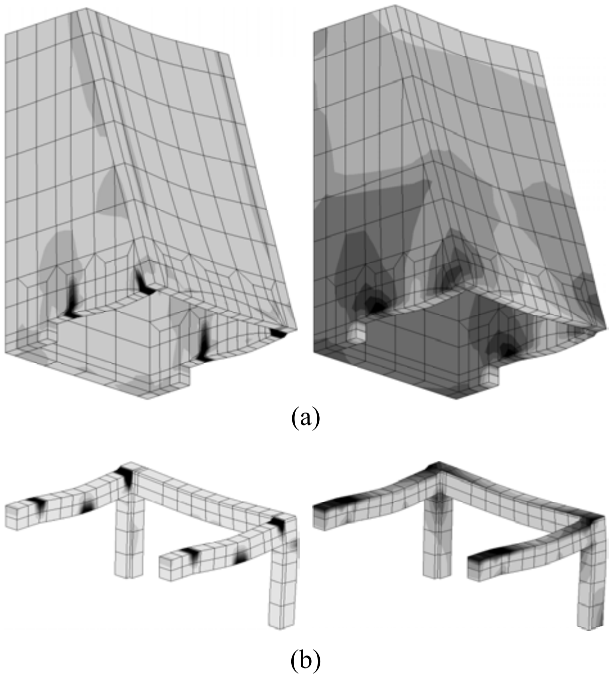


Fig. 7. Principal strains plotted in the deformed mesh configuration for Model 2, using non-linear analysis and a load factor of 2.0: (a) brick masonry part of the model; (b) stone columns and lintels. The maximum (tension) strains are plotted in the left and the minimum (compression) strains are plotted in the right. Results are dimensionless and darker color indicates higher damage.

Fig, 8 shows the force-displacement diagram for the mid-span of the side lintels, where the non-linear behavior is clearly visible. The global response is approximately linear until a load factor of 1.0. Afterwards, a progressive non-linear response dominates until collapse, followed by a descending branch (softening regime) captured only with a reduction of the applied load. Obviously, in a real physical situation a load reduction would be impossible and the chimney would just collapse in an uncontrolled manner.

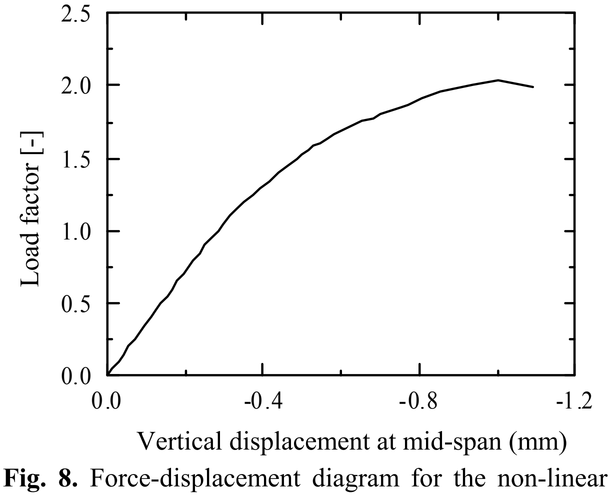
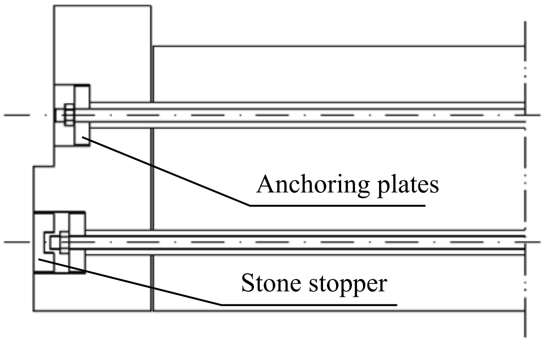


Fig. 8. Force-displacement diagram for the non-linear analysis (internal arching system ignored). Vertical displacement measured at mid-span of a side lintel.

The results of the structural analysis clearly indicate that the sudden collapse of the main lintel is not due exclusively for structural reasons, being probably triggered by corrosion of the tie that connects the main lintel and the right side lintel (here, it is noted that the environment is aggressive due to rising damp, salt and organic materials from the old kitchen activity). Certainly the local discontinuity associated with the hole for anchoring the iron tie also contributes to weaken the main lintel. The non-linear analysis carried out indicated that, in the most unfavorable conditions (without the internal arching action), the safety factor of the structure is 2.0 and, even then, possible collapse would occur in the side lintels and not in the main lintel. The structural analysis results justify the ancient damage in the left side lintel. In fact, the numerical simulation does not take into account the cross section reduction in the left side lintel (the height varies linearly from 0.46 m to 0.30 m), which would reduce the safety factor significantly. This justifies the remedial measures adopted in the past by adding new stone columns close to the back supports of the side lintels. Finally, the cracks and damage observed in the main lintel close to the left support are due to the corrosion of the iron cramp and also to the collapse of the right support, as the rotation of the main lintel was responsible for the long crack along the extrados of the internal arch in the main wall.

The solution for repair consists of strengthening and repairing the main lintel including: (a) reconstitution of the original stone integrity by injection of epoxy resins; (b) hole drilling of the stone along its full length (4.70 m); (c) insertion of bars and injection of the hole. Fig. 9 illustrates various details of the solution, which includes two stainless steel rods with a diameter of 25 mm as internal ties / reinforcement of the granite lintel. The ties were designed after the integration of the tensile stresses of the linear elastic results for the numerical model without arching action, which is conservative. These rods are inserted in drilled holes of 50 mm and are provided with anchoring plates of 120 mm. After adjustment of the bolts, the drilled holes are injected with fluid lime mortar (Albaria Iniezione 200). Stone stoppers at both ends of the bottom tie are also included so that the anchoring plates are not visible. The stoppers are glued with epoxy resin and are made from the actual core removed from the lintel, after cutting. For the top tie, this operation is not needed because the surface finishing is plaster. It is noted that the usage of stone stoppers in both ends of the ties requires the drilling to be executed from both sides, which requires precision and qualified workers.



(a)

(b)

Fig. 9. Aspects of the works carried out in the chimney: (a) detail of anchoring zone; (b) conclusion of the works.

3.2. Monastery of Jerónimos

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. The construction resisted well to the earthquake of November 1, 1755. One year later, a new earthquake lead to partial ruin of the nave. In this occasion also the vault of the high choir of the church partially collapsed.

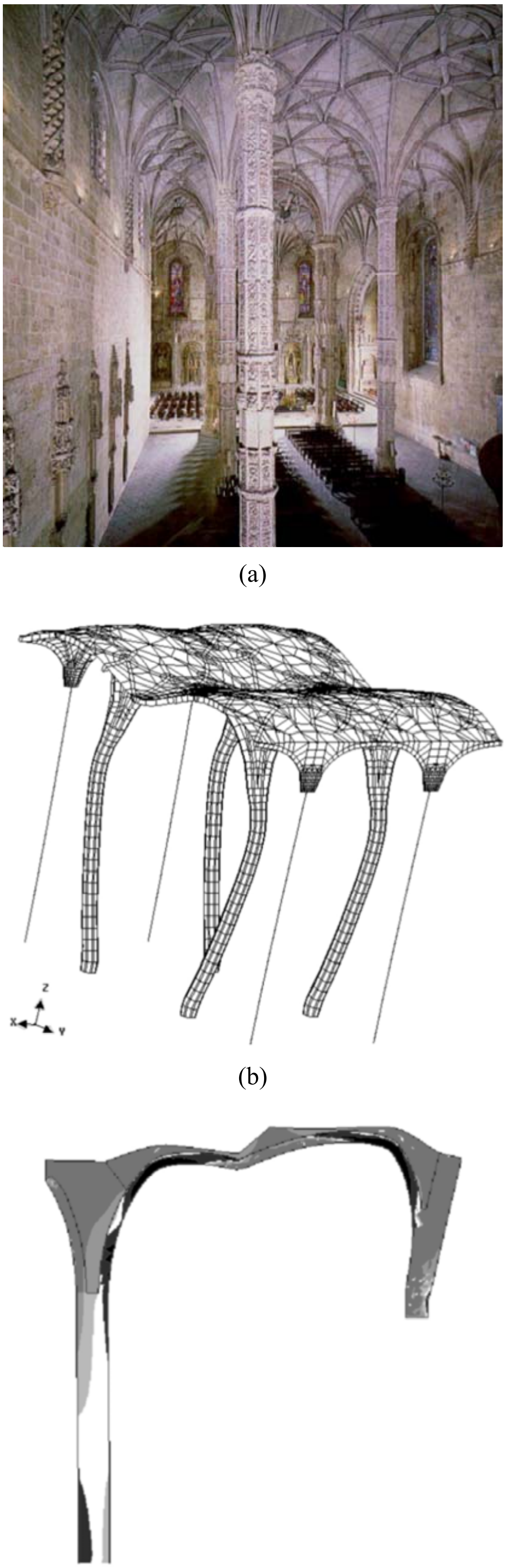
For the purpose of assessing the safety of the Church of Monastery of Jerónimos under vertical loading, two finite element models have been developed for the nave and the transept. A preliminary in situ investigation has also been carried out including geometrical survey, visual inspection, ultrasonic testing and radar testing.

The church has considerable dimensions, namely a length of 70 m, a width of 40 m and a height of 24 m, see Fig. 10a. 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 columns seem to be made of 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 the 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 wallets 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 and more unfavorable, see Fig 10b. Appropriate symmetry boundary conditions have been incorporated. The model includes three-dimensional volume elements, for the ribs and columns, and curved shell elements, for the infill and stones slabs. 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 deformed mesh at failure 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.

Given the complexity of the vault and the time consumed in the model of the nave, a simplified two-dimensional model of these arches was adopted for the structural analysis. Figure 10c illustrates the conservative adopted model, which includes the arch, the infill, the nave column and the external wall, with appropriate stiffness values and boundary conditions. All elements have quadratic interpolation, resulting in a mesh with 3530 degrees of freedom. Again, the actions considered in the analysis include only the self-weight of the structure. Collapse occurs with a typical four hinges mechanism, being three hinges located in the vault and one hinge located in the right support.

The analysis carried out allowed to conclude that: (a) collapse of the nave occurs with a failure mechanism involving the columns and the vault; (b) collapse of the transept occurs with a failure mechanism involving the external walls and the vault; (c) the compressive strength of masonry is a key factor for the response; (d) the safety of the structure seems low, when compared with similar constructions; (e) the columns of the nave are rather slender. It is stressed that the Church has been in use for some hundred years with moderate damaged ribs, and moderate tilting of the columns and sidewalls. Given the cultural importance of the construction, the safety of the users, the seismic risk and the accumulation of physical, chemical and mechanical damage, complementary NDT was proposed. The analyses carried out and the new proposed NDT results are fundamental for the definition of further action and the possible implementation of a monitoring program.



(a)

(b)

(c)

Fig. 10. Monastery of Jerónimos: (a) aspect of the nave; (b) model of the nave, with deformed mesh at collapse; (c) model of the transept, with deformed mesh at collapse and minimum principal stresses (compression). The darker color indicates higher stresses.

4. 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 it is less effective for designing strengthening. On the other hand, simplified modeling, such as limit analysis, is a great tool for everyday constructions, such as the buildings in historical centers.

Even tough large advances in the field of structural analysis of historical constructions have been observed in the last decades, a successful and cost-effective intervention remains a true challenge. Numerical models can 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. Once the fundamental mechanics are fully understood, design of the intervention can be carried out with simplified analysis tools.

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