SAFETY ASSESSMENT OF THE JERÓNIMOS CHURCH IN LISBON

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SUMMARY

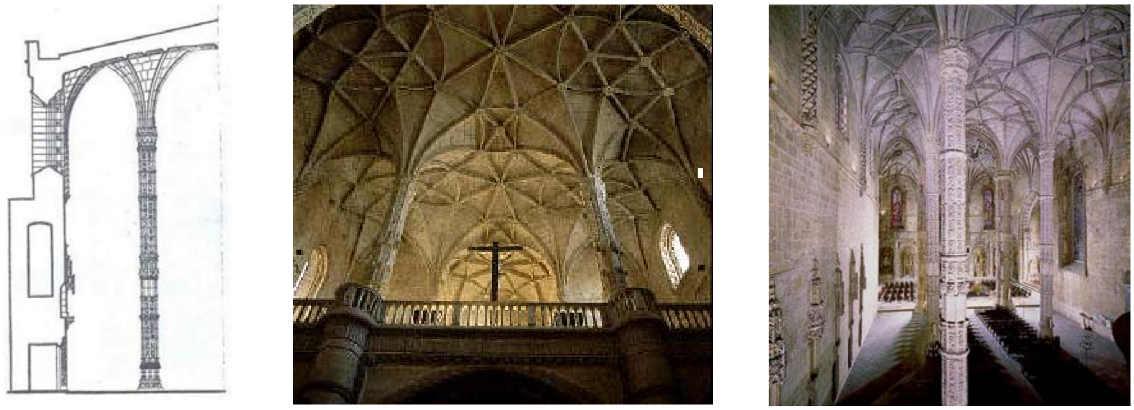
Advanced (non-linear) analysis of historical constructions represents a key contribution for the understanding of the architectural heritage. The safety of a famous Portuguese church is addressed using two different finite element models for the nave and the transept. The complexity of the ornaments of the vaults calls for a mix of volume, curved shell and beam elements. A sensitivity study and a simplified buckling analysis are also carried out to provide further insight in the results. The analysis indicates that the safety level of the structure is low, probably due to the uniqueness of the structural conception.

1. INTRODUCTION

Monastery of Jerónimos is, probably, the crown asset of Portuguese architectural heritage dating from the 16^{th} 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. Later, in December 1756, a new earthquake collapsed one column of the church that supported the vaults of the nave and resulted in partial ruin of the nave. In this occasion also the vault of the high choir of the church partially collapsed, see also [3].

The Gothic style was lately introduced in Portugal, incorporating a specific national influence. The so-called "Manueline" style (after King D. Manuel I), exhibits a large variety of architectural influences and erudite motives. An interesting aspect appears in the 16^{th} century, when the traditional three naves churches start to be replaced by a configuration with small difference in height for the naves. Here, the vault springs from one external wall to the other, supported in thin columns that divide almost imperceptibly the naves. From the traditional art, only the proportions and roof remain, being the concepts of space and structure novel. The fusion of the naves in the present Church, see Fig.1, is more obvious than in other manifestations of spatial Gothic. For this purpose, arches are not longer visible, the slightly curved vault comprises a set of ribs and the fan columns reduce effectively the free span. Additional information about the church and the vault can be found in [1,2].

The problem of safety assessment in historical constructions is quite complex. In particular, little is known about materials, variability of mechanical properties, existing damage, and constitution of the inner core of the walls, columns and vaults, among other difficulties. But one key aspect of masonry is its reduced tensile strength, which renders linear elastic analyses debatable. 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.

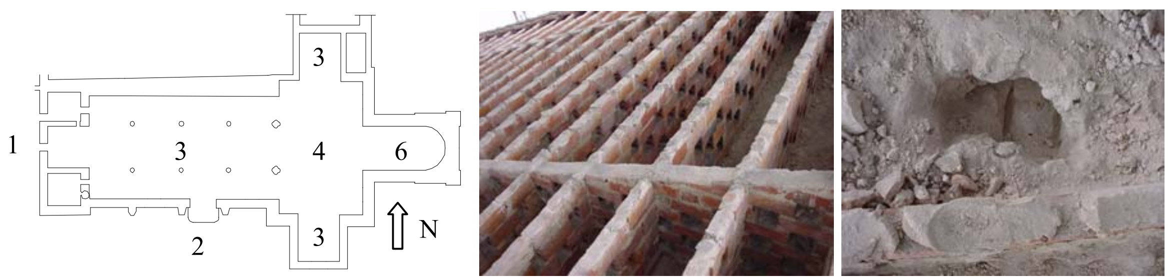


(a) (b) (c)

Figure 1. Church of Monastery of Jerónimos: (a) transversal cross-section; (b) vault on top of the choir; (c) aspect of the three naves.

2. PRELIMINARY INVESTIGATION

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 Fig. 2. In order to assess the safety of the church, the following preliminary tasks have been carried out: (a) three-dimensional survey of the church; (b) ultrasonic tests in the columns to assess the integrity [1]; (c) radar investigation to detect the thickness of the masonry infill in the vault and pier [5,6]; (d) removal of the roof, visual inspection, bore drilling, metal detection and chemical analysis of materials [6].



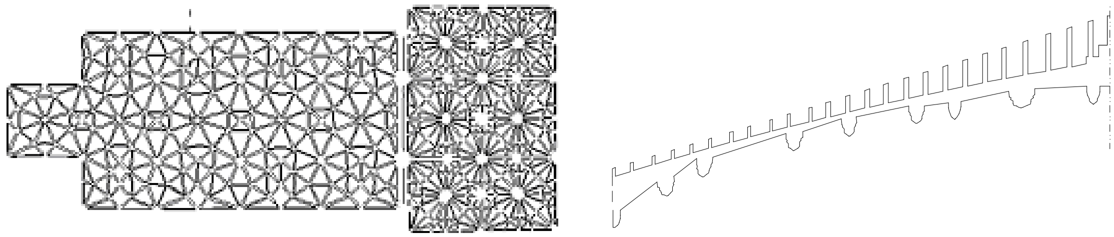
(a) (b) (c)

Figure 2. Survey: (a) Plan, with 1 – Axial doorway, 2 – Lateral doorway, 3 – Nave, 4 – Transept, 5 – Side chapels, 6 – Chancel; (b) removal of the roof and existing system to support the roofing tiles; (c) visual inspection of the infill and rib.

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 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, see Fig. 3. 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 30's to provide support for the roofing tiles, see [6] for details.



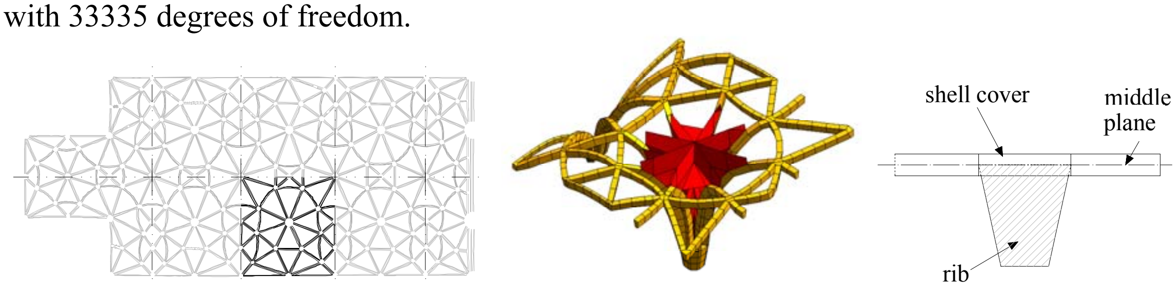
(a) (b)

Figure 3. Aspects of the vaults: (a) plan and (b) transversal cross-section of the nave.

3. ANALYSIS OF THE MAIN NAVE

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 and more unfavourable, see Fig. 4a. Appropriate symmetry boundary conditions have been incorporated. 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 Fig. 4b,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 behaviour assumed. All elements have quadratic interpolation, resulting in a mesh with 33335 degrees of freedom.



(a) (b)

Figure 4. 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 = 3.0 / **6.0** / infinite N/mm^2) and another type for the rubble infill (E = 1000 N/mm^2 and f_c = 0.5 / **1.0** / 2.0 N/mm^2). Given the uncertainty about the mechanical properties, a sensitive analysis was carried out, assuming the bold values as the reference values. 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 [7] for details.

The results for the reference analysis ($f_{c,stone}$ = 6.0 N/mm^2 and $f_{c,rubble}$ = 1.0 N/mm^2) are shown in Fig. 5, 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 [4].

Fig. 5a 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, meaning that the ultimate load factor is equivalent to the safety factor of the structure. It is possible to observe that the response of the structure is severely non-linear 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, using a linear elastic model. The collapse of the columns is due to the normal and flexural action. The safety factor is 2.0, which is rather low for this type of structures.

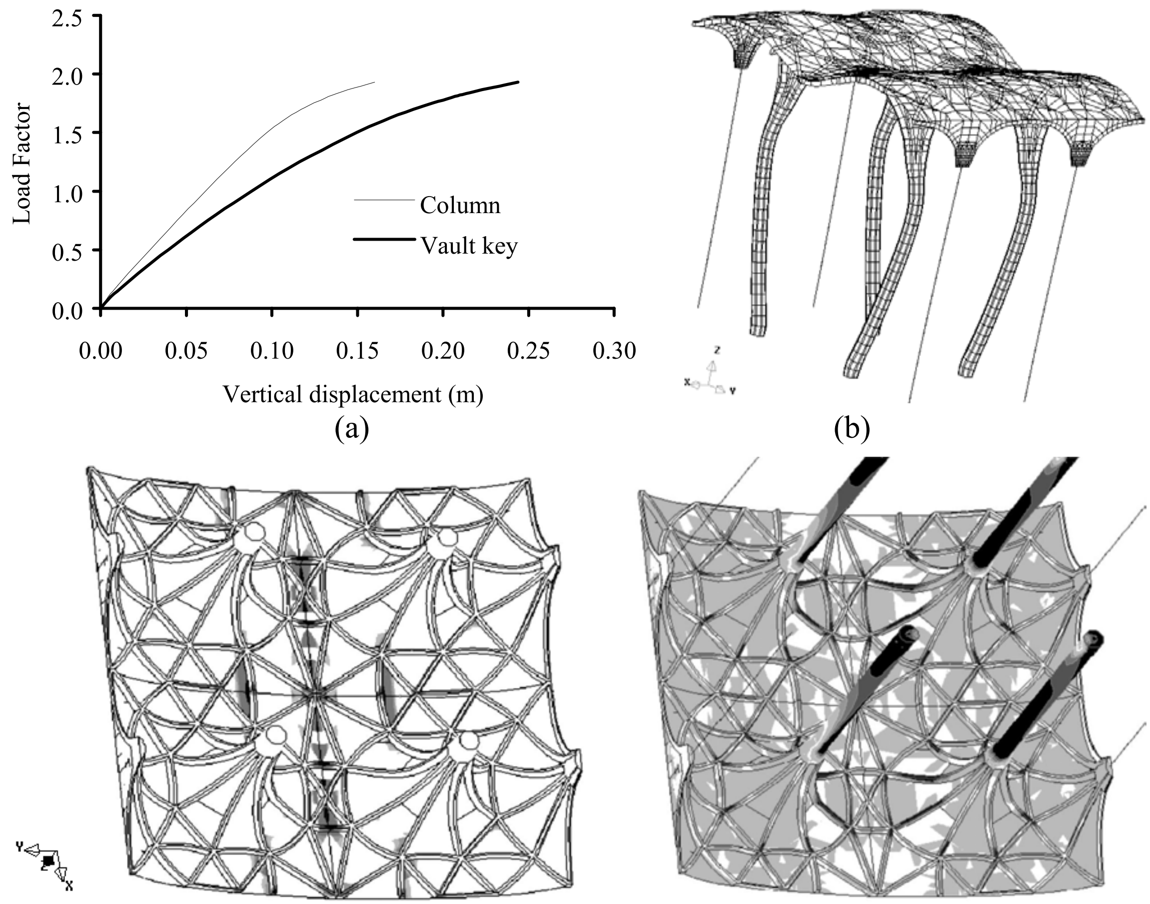


Figure 5. Results of nave analysis: (a) load-displacement diagram, (b) incremental deformed mesh at failure, (c) maximum principal strains (equivalent to cracks) and (d) minimum principal stress (compression).

The deformed mesh at failure, see Fig. 5b, 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. 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. Fig. 5c 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 behaviour of the vault. Finally, Fig. 5d 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.

Fig. 6 presents the load-displacement diagrams for different compressive strengths of the stone masonry and infill. The influence of the compressive strength of the stone masonry is very significant, as shown in Fig. 6a. The safety factor of the structure is reduced to 1.0, for a compressive strength of 3.0 N/mm^2, and increased to a value larger than 5.0, for an infinite compressive strength. One the contrary, the influence of the compressive strength of the infill is marginal, as shown in Fig. 6b. The safety factor of the structure is kept constant and only minor changes of stiffness can be observed. The collapse mechanism remains unchanged in all the analysis.

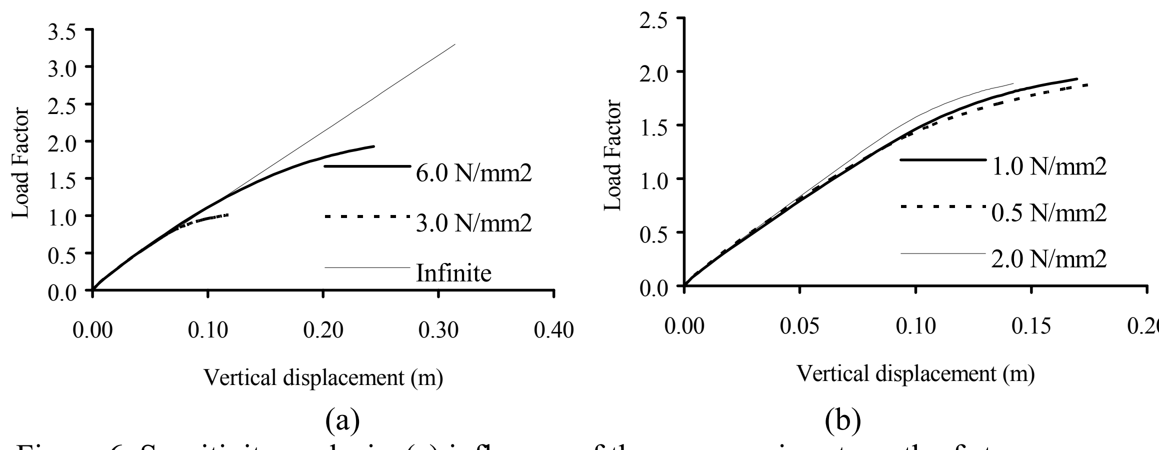


Figure 6. Sensitivity analysis: (a) influence of the compressive strength of stone masonry and (b) influence of the compressive strength of the infill.

Finally, Fig. 7 illustrates the influence of the geometrical non-linear behaviour in the analysis. It can be observed that this additional non-linear behaviour, in the reference analysis, reduces the safety factor to a value of 1.4, which is rather low. Again, the collapse mechanism remains unchanged.

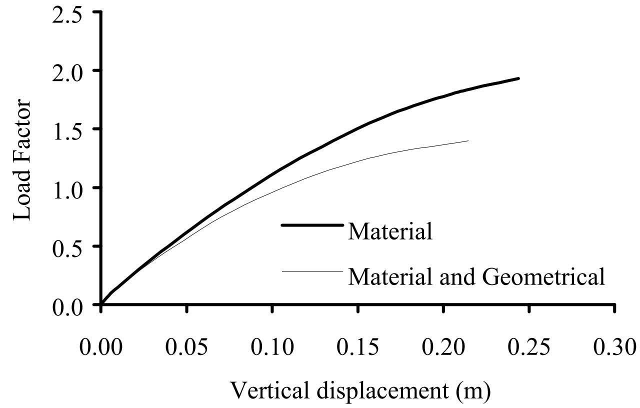
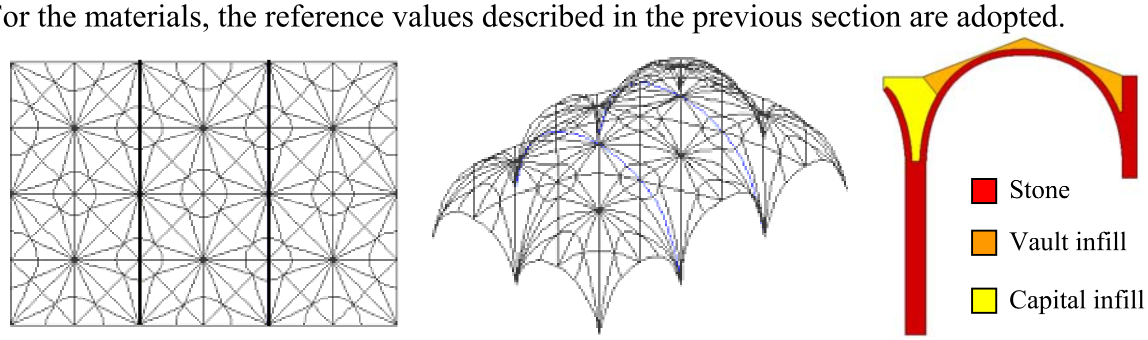


Figure 7. Influence of geometrical non-linear behaviour.

4. ANALYSIS OF THE TRANSEPT

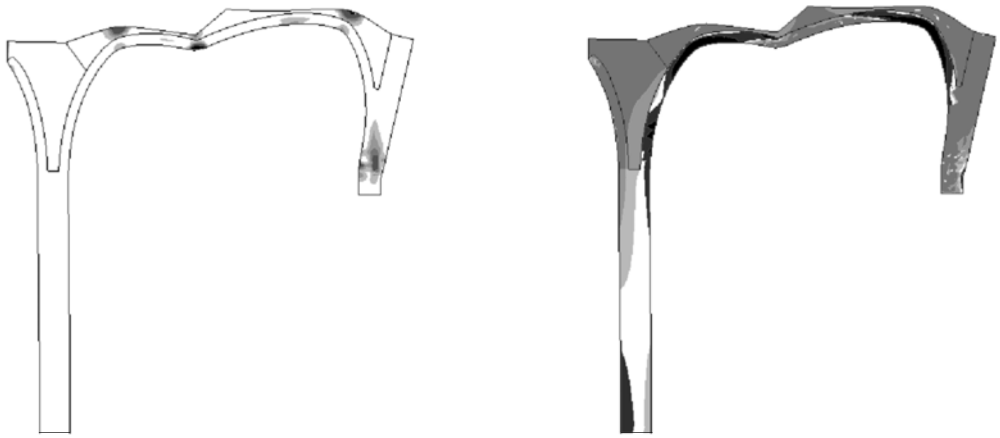
The second part of the structural assessment of the church focus on the transept vault. This vault has a geometry and structural scheme different from the nave. In plan, the vault forms a rectangle with $18.8 \times 28.0\ m^2$, using a basic square with a side of 4.7 m repeated 24 times. The vault exhibits, in plan, straight and circular ribs, together with keys at the intersection, ser Fig. 8. The transverse section of the ribs is equal to 0.4 m (height) by 0.28 m (width), with the exception of the two central ribs (arches) indicated with a thick line in Fig. 8a, showing a cross section of $0.60 \times 0.40\ m^2$. Therefore, the two arches are the most significant structural elements of the transept 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. Fig. 8c 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, see [4] for a complete description. 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. For the materials, the reference values described in the previous section are adopted.



(a) (b) (c)

Figure 8. Transept vault: (a) plan, (b) perspective and (c) finite element model.

The results for the transept analysis are shown in Fig. 9, in terms of maximum principal strain (equivalent to tensile damage) and minimum principal stresses (compression), depicted on the deformed mesh. The safety factor is 1.7, which is again rather low for this type of structures, even if the model is simplified and conservative. Further discussion on the results can be found in [4].



(a) (b)

Figure 9. Results of transept analysis at collapse: (a) maximum principal strains (equivalent to cracks) and (b) minimum principal stress (compression), depicted on deformed meshes.

Collapse occurs with a typical four hinges mechanism, being three hinges located in the vault and one hinge located in the right support, see Fig. 9. The collapse involving the right wall occurs due to the consideration that the nave prevents inwards movement of the (left) column. Fig. 9a indicates that the arch is cracked at the key (intrados) and both quarter spans (extrados). Significant cracking is also present in the right support. Fig. 9b demonstrates that high compressive stresses are found in the arch and in the base of the right wall. The compressive stresses in the left column are moderate and do not govern collapse.

5. SIMPLIFIED ANALYSIS OF THE COLUMNS

A simple hand calculation can provide further insight about the stability of columns, which seem to be a key issue for the nave. The axial load in the columns of the nave $N_{applied}$ is around 2455 kN. For a Young's modulus E equal to 3000 N/mm^2, a compressive strength f_c equal to 6 N/mm^2, a diameter ϕ equal to 1.04 m, a length of the column l equal to 15.75 m and an assumed buckling length l_0 equal to 1.5l, it is possible to calculate the ultimate load in the column N_{max} and a material safety factor SF_m, given by

$$N_{\max} = f_c \times \frac{\pi \times \phi^2}{4} = 5097\,kN \qquad \Rightarrow SF_m = \frac{N_{\max}}{N_{applied}} = \frac{5097}{2455} = 2.08 \tag{1}$$

The buckling load in the column N_{Euler} and a geometrical safety factor SF_g are given by

$$N_{Euler} = \frac{\pi^2 \times E \dfrac{\pi \times \phi^4}{64}}{l_0^2} = 3046\,kN \qquad \Rightarrow SF_g = \frac{N_{\max}}{N_{applied}} = \frac{3046}{2455} = 1.24\,. \tag{2}$$

These values are rather close to the values found in Section 3 and confirm that the columns of the nave are too slender. Therefore, more insight is required regarding the mechanical properties of the masonry constituting the columns. It is stressed that the value adopted for the buckling length is a mere assumption and that the ultimate load factor of 1.4 in Section 3 incorporates both the material and geometrical effects.

For the columns between the nave and the transept, with a diameter ϕ equals to1.88 m and a length l equals to 16.38 m, similar calculations yield a material safety factor SF_m

equal to 3.45 and a geometrical safety factor SF_g equal to 6.23. These values confirm that the collapse is not related with the columns.

6. CONCLUSIONS

The present paper presents a study regarding the safety of one of the most important monuments in Portugal, the Church of Monastery of Jerónimos. The limited information about the mechanical properties of the materials and the limitations of the analysis, given the enormous complexity of the construction, mean that the obtained results should be considered as the best approximation to a likely response of the structure.

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 too 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 implementation of a monitoring program.

ACKNOWLEDGEMENTS

The work was partly supported by project POCTI-ECM-38071-2001 funded by Fundação para a Ciência e Tecnologia of Portugal.

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