The *Brasões* Building of National Palace of Sintra Characterization and Structural Performance

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October 2019

ABSTRACT

The National Palace of Sintra obtained the Cultural Built Heritage status, due to its historical and cultural importance. The *Brasões* Room in the Palace is the most important heraldry room in Europe.

This study aims to assess the seismic performance of the *Brasões* Building and to verify its structural safety. Therefore, different numerical models were developed, a linear numerical model in SAP2000 software and a non-linear model in 3MURI software. For the geometrical characterization of the models a laser scanning survey was conducted.

Ambient vibrations tests were performed in the *Brasões* Building in order to obtain its dynamic characteristics such as frequencies and vibration modes. The post processing of the obtained data in the ARTeMIS software enabled the models' calibration.

The seismic assessment of the National Palace of Sintra, with rubble stone masonry walls and columns and timber floors and rooftop, is performed considering the in-plane and out-of-plane response of the structure. The in-plane response refers to the global behavior controlled by the in-plane capacity of walls and it was determined through non-linear static analyses. The out-of-plane response, denotes to the activation of local mechanisms, was evaluated by kinematic analyses considering the macro-block modelling approach. With the seismic assessment was possible to identify the *Brasões* Building vulnerabilities.

Seismic retrofit measures are proposed in order to verify the code seismic safety of *Brasões* Building.

Keywords: National Palace of Sintra; *Brasões* Building; Seismic Assessment; Non-linear static analysis (pushover analysis); Ambient Vibrations Tests; Seismic Retrofit.

1. Introduction

The Cultural Built Heritage identifies the uniqueness of a city, the history of a country, the testimony of a nation, thus, it is imperative its preservation. With rubble stone masonry walls, the National Palace of Sintra, has undergone several structural modifications and interventions over the centuries, having acquired an undeniable historical value, being the only medieval monument preserved.

Located at the highest point of the National Palace of Sintra, the *Brasões* Room is the greatest exponent of Manueline intervention in the building and represents almost a temple of nobility.

King Manuel I (1469-1521), at the beginning of the 16th century, between 1517 and 1518, ordered the construction of a quadrangular tower, containing in its interior the much-admired *Brasões* Room (Silva, 2003). This room exudes the patriotism of the 16th century, when the discoveries reached the maximum expression of the Portuguese expansion throughout the world (Ferreira, 2009). The decoration of the room (Figure 1) arose from the heraldic and genealogical concern of king

Manuel I, who expresses the game between intimacy and the representation of power in this unique room throughout Europe (Silva, 2003).

Figure 1 - Ceiling of Brasões Room, where is shown the majesty of the room, through the coat of arms represented, in the centre, the one of D. Manuel I reign

Considering the National Palace of Sintra location, in the historical centre of Sintra, the Palace is subjected to the seismic activity. The earthquake is a natural phenomenon whose occurrence can cause a catastrophe with harmful consequences. It is also known that the most rubble stone masonry buildings, such as the National Palace of Sintra, are not prepared to resist to seismic activity.

Hence, the preservation, conservation and protection of this built heritage is inevitable. Therefore, this study aims to assess the seismic performance of the *Brasões* Building and to verify its structural safety. It also contemplates, at a later stage, eventual seismic retrofit measures in order to reduce vulnerabilities that can be identified.

Seismic assessment of the Cultural Built Heritage requires the use of an integrated and demanding interdisciplinary approach consisting on a series of steps (Figure 2), as proposed in (Bento, 2019).

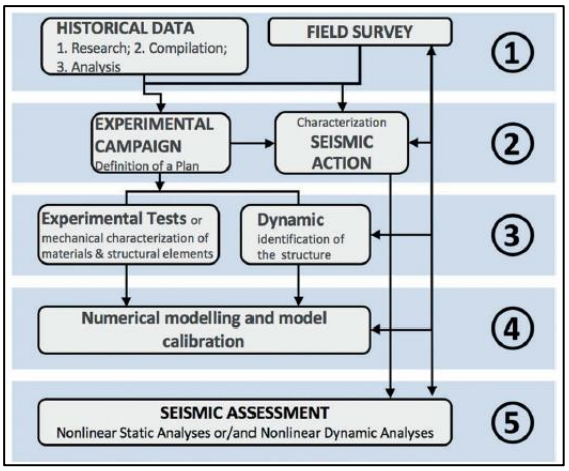

Figure 2 - Multidisciplinary approach for the seismic assessment of the Cultural Built Heritage (Ponte et al., 2019)

The initial phase of the approach mentioned above (Figure 2), focus on a compilation and analysis of historical data, as well as a field survey that complements the knowledge of the structure. Next, it is defined a plan for the experimental campaign. In this phase it is also important to characterize the seismic action. In the third part, the experimental tests are carried out. Following, numerical modelling and modal calibration are performed based on the experimental results. In the last phase, a seismic assessment is accomplished considering the overall behavior of the building and the development of partial mechanisms of the structure.

2. Field Survey

The development of a numerical model requires the geometric characterization of the existing construction, as close to the reality as possible, being necessary to carry out an exhaustive study of the building.

Firstly, technical drawings of National Palace of Sintra, made available by *Parques de Sintra-Monte da Lua, S.A.,* were analyzed. However, there were inconsistencies in them. Hence, a field survey was performed in order to correct the plans provided.

The field survey began with numerous visits to the Palace, where a visual inspection and a manual survey of measurements were carried out, such as wall thicknesses and width of windows/doors. Subsequently, a geographic

survey campaign was carried out using a laser scanner. Laser scanning is a recent technique of 3D (three dimensions) surveys of high precision. This type of survey, both inside and outside the structures, allows a large set of data to be obtained in a short period of time with high precision. As well as the complete and dense three-dimensional reconstruction of the raised elements, which allows the swift creation of architectural and structural models (Bento, 2019).

The point cloud data obtained for *Brasões* Building (Figure 3a)), endorsed the correction of all previous measures mentioned (Figure 3b)) and development of the final geometrical model.

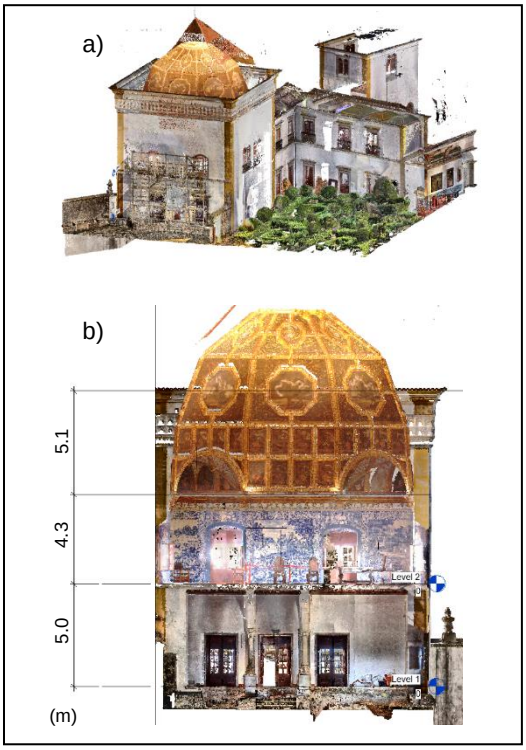

Figure 3 - Brasões Building point cloud data from laser scanning technique

3. Experimental Tests Campaign

A campaign of ambient vibration tests was completed in *Brasões* Building in order to obtain its man dynamic characteristics: the frequencies and the vibration modes.

The dynamic characterization tests consist on the measurement of the structure's response to environmental vibrations caused by wind, road traffic, pedestrian traffic, among other sources.

The post-processing of the collected data makes it possible to identify the dynamic features of the structure: frequencies, damping coefficients and configurations of the main vibration modes of the structure (Milosevic, et al., 2015).

Dynamic characterization tests were carried out on the *Brasões* Building and later the modelling and data processing were performed in the ARTeMIS Modal 4.0 software (SVIBS, 2015). The obtained results were used to calibrate the linear (SAP2000) and non-linear (3MURI) *Brasões* Building models.

3.1. Linear Numerical Model in SAP2000 Software

A linear numerical model was initially developed in the SAP2000 software (CSI, 2017), which allowed modal analysis to be executed and an estimation of the main frequencies and vibration modes were defined. These results helped to define the positions of the sensors and the number of setups to be used for the ambient vibration tests.

The National Palace of Sintra is composed by rubble stone masonry walls and columns. Since it is an extremely heterogenous material, it hinders the model calibration. The values of the materials mechanical properties were based on the values proposed in (MIT, 2009) and on the studies carried out on (LNEC, 1997). The values obtained are presented in Table 1. It is important to mention that for floor materials (timber), the density was not considered (Table 1) as they were defined as loads in the model. Additionally, in this specific case, the value of distortional modulus (G) is not the usual one, presented in the study conducted by the Civil Engineering National Laboratory (LNEC, 1997). In this situation, the timber boards are cut and distributed, applied to the floor, reducing its strength. Thus, according to the study conducted in (Giongo et al., 2014) and the detailed concepts of the New Zealand standard in (Brignola et al., 2012), the value of the timber distortional modulus of 18 MPa was determined and adopted.

Table 1- Material properties which characterize the numerical linear model

Material	**Elasticity Modulus E (*GPa*)**	**Distorsion Modulus G (*MPa*)**	**Density (*KN/m³*)**	***Poisson* Coeficient**	Reference
Walls Rubble Stone Masonry	1.2	500	18	0.2	(MIT, 2009)
Timber	12	18	0	0.3	(LNEC, 1997)
Timber and Tile	12	4615	0	0.3	(LNEC, 1997)
Columns Stone Masonry	3.2	1333	22	0.2	(MIT, 2009)

3.2. Ambient Vibration Tests

The structure response to environmental vibrations is measured by accelerometers (sensors). Six highly sensitive accelerometers were used in this project, one of them being an EpiSensor ES-T, triaxial, and the other five sensors were EpiSensor ES-U2, uniaxials.

An adequate dynamic characterization of the structure implies a conscious and an appropriate placing of the accelerometers. For this reason, the results obtained in the modal analysis carried out in the SAP2000 program were used. It was identified in the first fundamental vibration modes, the points with the greatest displacement and, consequently, where the highest vibrations are expected to occur. These are the appropriate places for the sensors to be located. As only five uniaxial sensors were available, it was necessary to change their position during several phases, making a total of four test campaigns (setups). The sensors location in all four phases of the tests are identified in the Figure 4.

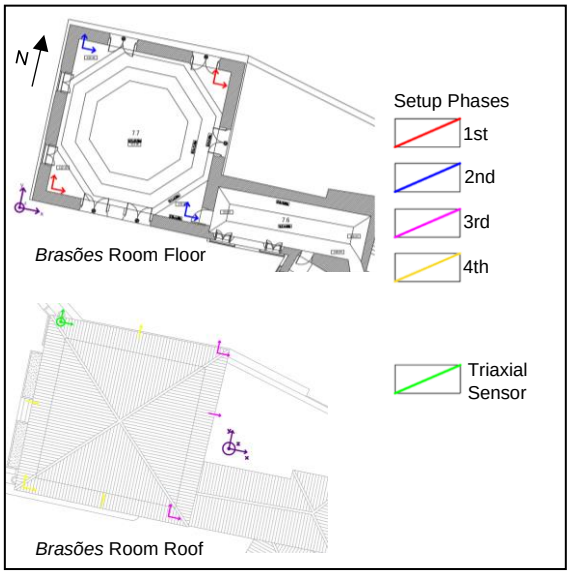

Figure 4 - Accelerometers position in the several phases of the vibration tests

3.3. Results

The introduction of the fieldwork data in a geometric model in the ARTeMIS software allows, after processing, to obtain the main dynamic characteristics of the building.

Modal identification was carried out through the application of the Frequency Domain Decomposition method (FDD). Subsequently, the results were improved using the Enhanced Frequency Domain Decomposition method (EFDD). It should be noted that both procedures, implemented in the ARTeMIS software, are also called peak selection methods, since the natural frequencies of vibration of the system are identified through the amplitude peaks of the functions of spectral density response of the system (Ventura et al., 2002). In this work, it was chosen the results from EFDD method for the calibration of numerical models, since this is an improved process and for this reason more reliable.

As evidenced in Figure 5, the translation mode in the Y direction, transverse (North-South) direction of *Brasões* Building is the first to develop. Expected result, since the structure is restrained by the adjacent buildings of *Brasões* Room in the longitudinal direction (X). Thus, it was predictable that the translation mode in the longitudinal (X) direction of the building would only occur after the mode in Y, being the second most expressive peak of the graphs that were analyzed.

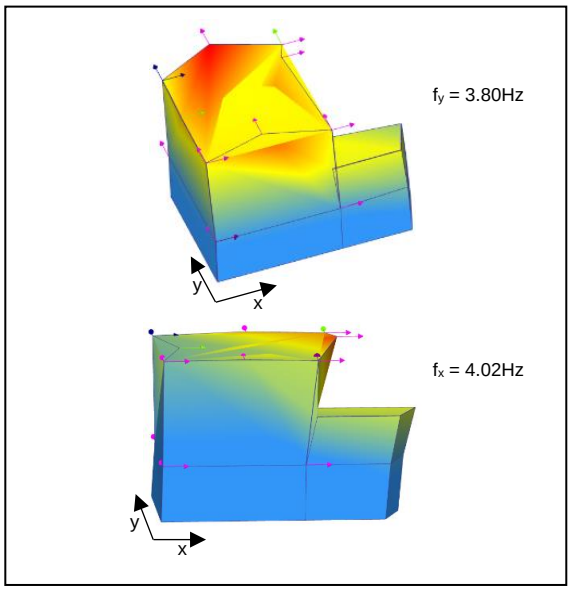

Figure 5 - Vibration modes and frequencies of the ARTeMIS model

4. Numerical Modelling and Model Calibration

4.1. Non-Linear Numerical Model in 3MURI software

The non-linear analysis model of the *Brasões* Building was developed using the 3MURI software (S.T.A. DATA, 2018). This program models the masonry building through the equivalent frames' method. It consists on dividing a wall, with openings, into several deformable elements, where the non-linear response and deformation is concentrated, and into rigid fragments that connect the different deformable elements. The macro-elements that models the structural elements of masonry, are piers (vertical elements) and spandrels (horizontal elements), which are the deformable elements, and rigid nodes, which are the rigid elements (Lagomarsino et al., 2013).

4.2. Modal Analysis

The first analysis to be performed in the 3MURI model was a modal analysis, which allowed the calibration of the model. Therefore, it was possible to determine the frequencies and vibration modes and compare them with the experimental values (considered in this study as the reference values) and with the values obtained in the developed model in SAP2000 program (already calibrated).

The calibration of 3MURI model consisted of an iterative and conscious process where mostly the elasticity modulus (E) of the rubble stone masonry was adjusted, achieving the values that define the materials' characteristic of the *Brasões* Building already presented in Table 1. However, as it is a non-linear analysis, other parameters must be defined, which are shown in Table 2. The values presented were taken from the italian regulation (MIT, 2009). The values of the elasticity and distortion modulus (E and G) were reduced in order to consider the cracking phenomenon. According to Part 1 of Eurocode 8 (EC8-1) (CEN, 2010), in the absence of more precise assessments, the two contributions to stiffness (E and G) can be considered equal to half of the respective unsplit values (0.5E). However, according to the study conducted in (Simões A. G., 2018), the adoption of this value leads to very conservative estimations of masonry non-linear behavior. Therefore, the value of 0.66E was adopted, as suggested. Additionally, in Table 2 there is a solid brick masonry material, which characterizes the vault floor in the building.

Table 2 – Material properties with the cracked resistance which characterize the non-linear model

Material	**Elasticity Modulus E (*GPa*)**	**Distortion Modulus G (*GPa*)**	**Density (*KN/m³*)**	**Compression Strength f_m (*MPa*)**	**Shear Strength τ (*MPa*)**
Walls Rubble Stone Masonry	0.792	0.264	18	0.235	0.041
Solid Brick Masonry	0.631	0.210	16	0.118	0.088
Columns Stone Masonry	2.112	0.704	22	0.800	0.120

4.3. Results Comparison

Table 3 provides the frequencies of the vibration modes obtained in the 3MURI, in the ARTeMIS (experimental test values) and in SAP2000 models. The different values are compared and the error percentage in relation to the results obtained by the environmental vibration tests are presented. The first two vibration modes obtained are illustrated in Figure 6.

Table 3 - Mode vibrations frequencies comparison

Vibration Mode	ARTeMIS Model	SAP2000 Model		3MURI Model	
	f (Hz)	f (Hz)	Error (%)	f (Hz)	Error (%)
1st Mode in Y direction	3.795	3.520	7.24	3.700	2.51
1st Mode in X Direction	4.016	4.064	-1.20	4.097	-2.02

It must be pointed out that the obtained results are quite reasonable since the error percentage is always lower than 10%, for any vibration mode analyzed (Table 3).

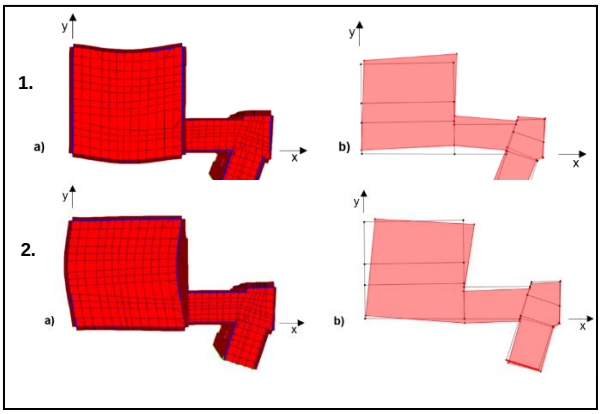

Figure 6 - Brasões Building vibration modes. Where pictures a) refers to the Sap2000 model and pictures b) concerns the 3MURI model. 1 refers to the translation mode in X direction and 2 to the translation mode in Y direction

5. Seismic Assessment

For the seismic assessment of *Brasões* Building two different approaches are embraced, taking into account the in-plane and out-of-plane behavior of the walls. In both approaches the 3MURI program (S.T.A. DATA 2018) is used. For the evaluation of the global behavior, non-linear static analyses are conducted, while the local behavior of the out-of-plane collapse mechanisms are evaluated through kinematic analysis according to the macro-block approach.

5.1. Global Response

Non-linear static analyses made possible to assess the global behaviour of *Brasões* Building. The response of the structure in terms of top displacements and base shear and in terms of the relative displacements (drift) and shear force of the masonry walls, is evaluated for different limit states. According to Part 3 of Eurocode 8 (EC8-3) (CEN, 2017), for existing buildings of importance class III (as the National Palace of Sintra) the seismic assessment requires the verification of the near collapse (NC), significant damage (SD) and damage limitation (DL) limit states, for both seismic actions (type 1 and type 2).

5.1.1. Capacity Curves

The pushover curve, or base shear-displacement force curve, is able to describe the non-linear response of the building when subjected to horizontal seismic forces and provide essential information to predict its behavior in terms of stiffness, resistance capacity, ductility and ultimate displacement capacity (Lagomarsino et al., 2013). Lateral loads were applied in the positive (+) and negative (-) directions for each main direction of the building, meaning, X (longitudinal) and Y (transverse). The lateral loads were applied according to a uniform distribution (uniform), proportional to the mass, and a pseudo-triangular distribution (static forces), proportional to the product between mass and height.

In order to determine the ultimate displacements of capacity curves, two different criteria were set. On the one hand, criterion 1, proposed by EC8-3 (CEN, 2017), states that the ultimate displacement corresponds to the point where the maximum shear force shows a decay of 20%. On the other hand, in criterion 2 the

ultimate displacement consists on the displacement value when a partial collapse mechanism is formed, which in most cases causes the interruption of the analysis before reaching the 20% reduction in the shear force. This occurrence can be identified in the capacity curve when in the non-linear phase there is a sudden loss of force and/or a large increase in displacement. The capacity curves of the *Brasões* Building are represented in Figure 7 .

The analysis of the figure states that the pseudo-triangular loading is the most conditioning for the evaluation of seismic performance, i.e., the structure presents a lower resistant capacity to this type of load, when compared to the distribution of uniform loads. Additionally, it is verified that the X direction (longitudinal direction), is the one that presents the highest strength and stiffness, since the maximum base shear force and the slope of the curve are always higher in this direction, in comparison with the Y direction, in all cases. This fact was expected, taking into account that the structure is longitudinally restrained by the adjacent building. It is also possible to assess that the structure is more ductile in the longitudinal direction (X), since, as expected, there is a greater distribution of non-linear behavior by the different structural elements in this direction before the collapse.

5.1.2. N2 Method

The performance of the structure was assessed using the N2 method proposed by EC8-1 (CEN, 2010). This method is based on the identification of the performance point, or objective displacement, which corresponds to the displacement of the structure when subjected to a seismic action. This point is calculated through the intersection between the capacity curve of the structure and the response spectrum of the seismic action (Cattari & Lagomarsino, 2013). In order to verify the safety criterion, the objective displacement (d_t^*) cannot be higher than the ultimate displacement of the structure (d_u^*), in other words, the displacement imposed by the earthquake cannot exceed the ultimate displacement of the structure, defined for the limit state under analysis.

It is important to note that according to EC8-3 (CEN, 2017), the safety verification for the significant damage limit state (SD), the ultimate displacement, should be multiplied by 3/4. However, for the case of the damage limitation limit state (DL), the capacity for a global evaluation should be defined as the maximum displacement corresponding to the yielding displacement (d_y^*).

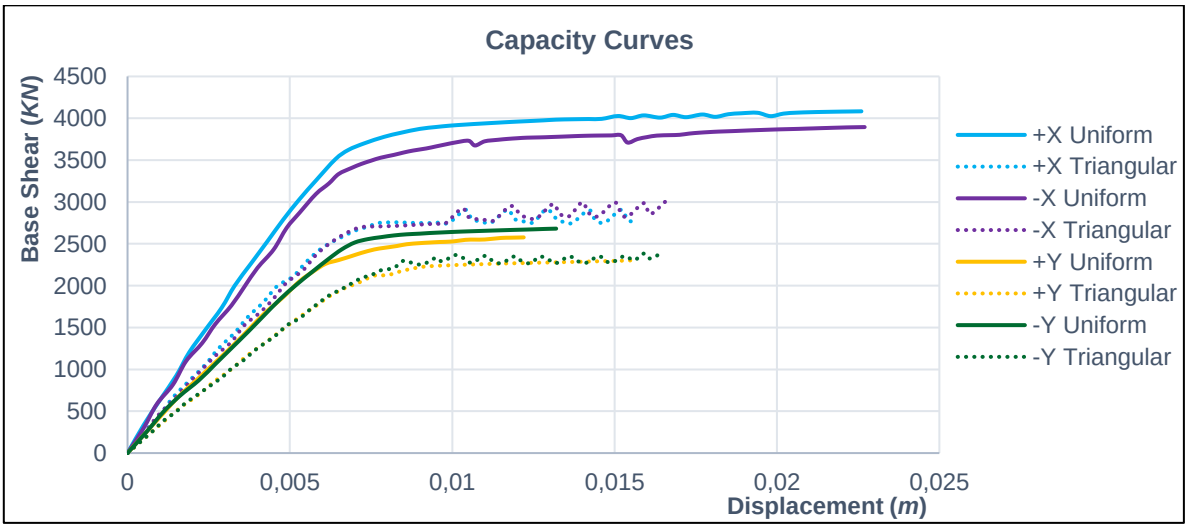

Figure 7 - Capacity curves of the Brasões Building, for X and Y directions up to the ultimate displacement

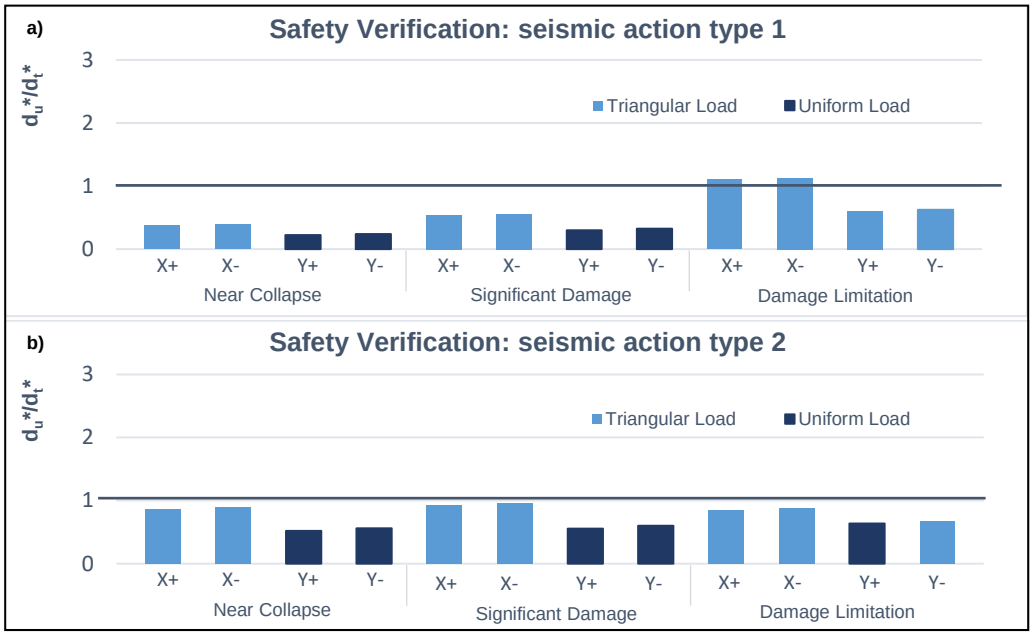

Figure 8 - Safety Verification for seismic action type 1 and 2

Figure 8 a) and b) illustrate the safety criterion by representing the ratios, $\frac{d_u^*}{d_t^*}$, for seismic action type 1 and type 2, respectively. It is shown the most conditioning load distribution for each direction and limit state. It should be noted that the safety of the structure is satisfied if $\frac{d_u^*}{d_t^*} > 1$.

The safety verification is not satisfied, in most cases, either for seismic action type 1 (Figure 8 a)) or for seismic action type 2 (Figure 8 b)). Only for the positive and negative X direction of the damage limitation limit state (DL), for seismic action type 1, the safety verification is satisfied. However, as the safety verification must be fulfilled for both directions of the building (X and Y), *Brasões* Building does not meet the performance requirements for the two types of seismic actions and for the three types of limit states recommended in EC8-3. It was found that for the limit state of near collapse (NC) and significant damage (SD), seismic action type 1 is the most demanding for the structure, when compared with the seismic action type 2. Nevertheless, seismic action type 2 becomes a conditioning factor for the limit state of damage limitation (DL) for the X direction. In fact, for this limit state the structure is still in a linear regime (at the limit of this behavior) and it was expected that earthquake type 2 had more demanding requirements.

5.2. Local Response

This section outlines the study of *Brasões* Building for the local response. Kinematic analyses were performed according to the macro-block modeling approach for the study of out-of-plane local behavior of rubble stone masonry walls, for seismic action type 1 and type 2 through 3MURI software. The safety verification of near collapse limit state (NC) as defined in (MIT, 2009), is satisfied if the spectral seismic acceleration of activation of the local mechanism (a_0^*) is higher than the spectral acceleration of the response spectrum evaluated for T=0 (a_{0-min}^*). The a_0^* is adequately amplified to consider the portion of the structure involved in the mechanism. Thus, the local collapse mechanisms with the highest probability of occurrence were identified considering the geometry of *Brasões* Building, its construction details and restrictions given by the structure. The seven collapse mechanisms identified were modeled according to the macro-block modeling approach (Figure 9).

Table 4 shows the results of the accelerations calculated by the 3MURI program, for the seismic action type 1, the more conditioning one, which enables to assess the safety level of the structure.

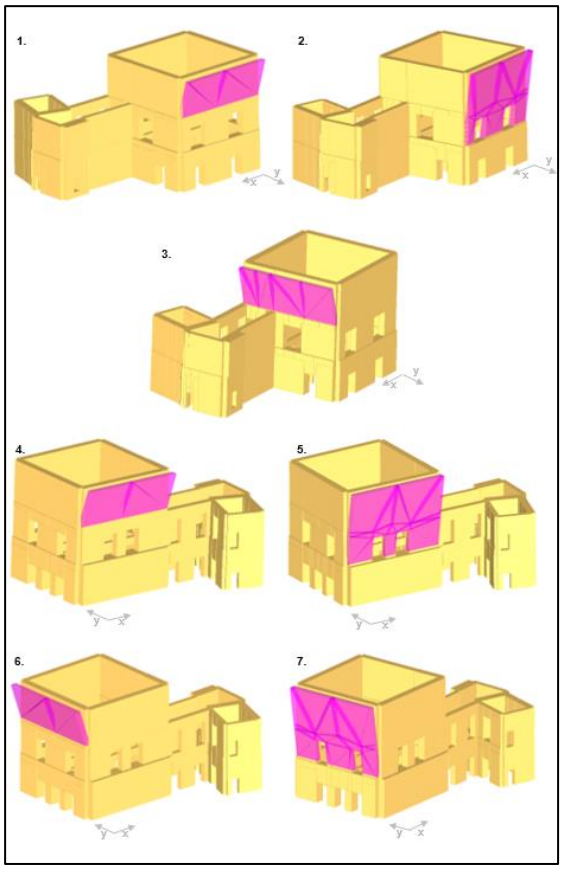

Figure 9 - Local collapse mechanisms of Brasões Building

Table 4 - Out-of-plan security verification

Mechanism	Horizontal Impulses (*KN*)	a_0^* (*m/s²*)	a_{0-min}^* (*m/s²*)	**Verify?**
1	0	1.35	3.87	**No**
2	0	**0.67**	208	**No**
3	0	1.35	3.87	**No**
4	0	1.52	3.87	**No**
5	0	0.79	2.08	**No**
6	0	1.52	3.87	**No**
7	0	0.80	2.08	**No**

The analysis of Table 4 permits to verify that even without any horizontal load applied, that is, without counting the horizontal impulses of the roof trusses, the safety of the structure is not verified. Logically, with the horizontal loads coming from the roof would worsen the situation. It should be noted that the most conditioning mechanism, i.e. the first to collapse, is mechanism 2, since it has the lowest value of spectral seismic acceleration to activate the mechanism. This result would be expected, since the collapse mechanism consists of two floors of the wall, in the direction of the first fundamental vibration mode of the structure (translation mode in the Y direction).

6. Seismic Retrofit

The analyses presented previously led to the conclusion that the *Brasões* Building of the National Palace of Sintra needs structural reinforcement in order to improve its seismic performance and to satisfy the safety verifications defined in the regulations. In this chapter, a structural strengthening solution is proposed to reduce seismic vulnerability by improving the behaviour of the structure in its global and local response.

6.1. Global Response

In order to enhance the global performance of *Brasões* Building, it is proposed to improve the quality of the rubble stone masonry walls through the injection of mortar and the introduction of reinforced plaster in the vulnerable external masonry walls. According to (MIT, 2009), to take into account the effect of the proposed reinforcement solutions, the mechanical properties of the building materials should be affected (increased) by multiplicative coefficients presented in table C8.5.II of (MIT, 2009). The multiplicative coefficient value is dependent on the type of reinforcement chosen. The reinforced walls are illustrated in the Figure 10.

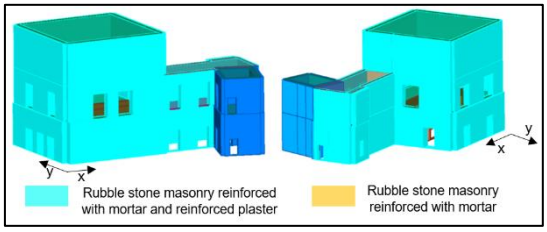

Figure 10 – Reinforced walls of Brasões Building

The performance evaluation of Brasões Building after the structural strengthening intervention was analyzed using the N2 method. Figure 11 a) and b) shows the $\frac{d_u^*}{d_t^*}$ ratio for the structure capacity curves for seismic action type 1 and type 2, respectively, and for the three limit states under study. It is concluded that the resistant capacity of the structure has increased

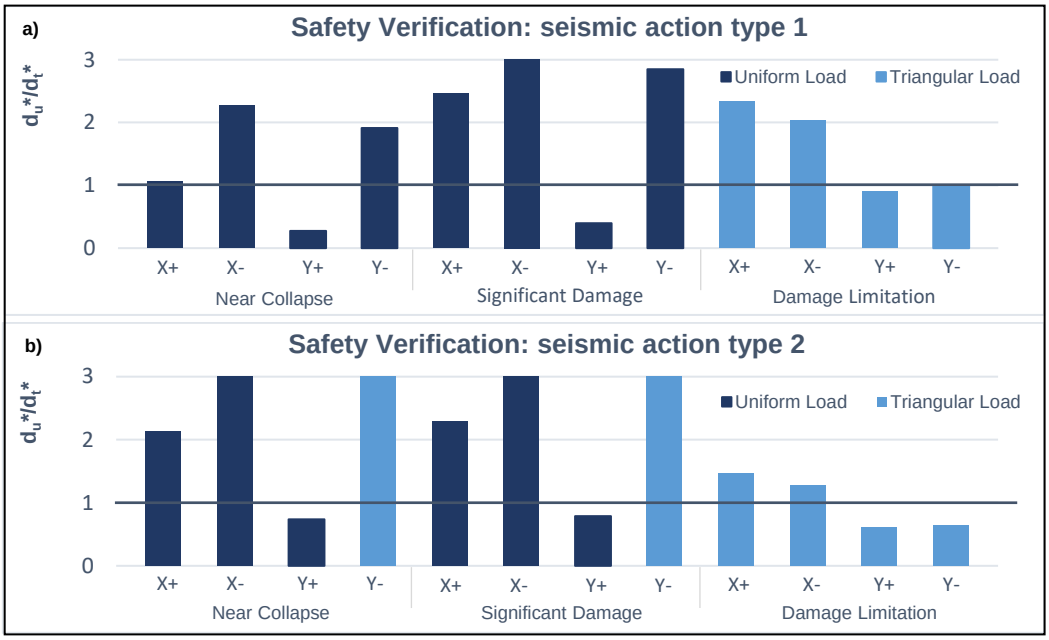

Figure 11 - Safety Verification for seismic action type 1 and 2

and the safety verification is ensured in its generality. However, it is verified that the Y direction of the structure is the most conditioning one, more precisely, the curve corresponding to the load in the positive Y direction, in which the performance of the structure is never assured, for any seismic action or limit state. It occurs since it happens to collapse the columns of the Columns Room, which belongs to *Brasões* Building.

6.2. Local Response

In order to improve the local response of *Brasões* Building it is proposed a seismic retrofit solution characterized by the introduction of prestressed cables in order to avoid the out-of-plane collapse mechanism of the walls. The retrofit solution proposed consists on four cables applied at the roof level, on the four sides of *Brasões* Room roof, to counteract the out-of-plane movement of the walls. The value of the prestressing cable force required for ensure the safety of the structure was determined through an iterative process, according to the macro-block modeling approach, where the same collapse mechanisms previously defined (Figure 9) were used. The safety analysis is satisfied for seismic action type 1, near collapse limit state, for a horizontal force of at least 70KN (Table 5).

Table 5 - Out-of-plan security verification

Mechanism	Horizontal Impulses (*KN*)	a_0^* (*m/s²*)	a_{0-min}^* (*m/s²*)	Verify?
1	70	4.00	3.84	**Yes**
2	70	2.31	2.10	**Yes**
3	70	3.92	3.84	**Yes**
4	70	3.87	3.84	**Yes**
5	70	2.25	2.10	**Yes**
6	70	3.94	3.84	**Yes**
7	70	2.31	2.10	**Yes**

7. Conclusions

This study assessed the seismic performance of *Brasões* Building to verify its structural safety. First, ambient vibration tests were performed in the building in order to obtain its dynamic characteristics to allow the calibration of the numerical models developed. For the global seismic assessment non-linear static analyses were conducted. It was concluded that the longitudinal direction of *Brasões* Building corresponds to the highest strength, stiffness and ductile direction of the structure. Additionally, it showed that the global safety verification of the structure, considering the behavior of the walls on its plan, is not assured for any type of seismic action or limit state studied. On the other hand, the out-of-plane behavior of the most vulnerable walls analyses confirm that the safety was not fulfil. In order to mitigate the vulnerabilities identified, some retrofit solutions were proposed and assessed.

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