Study of seismic behaviour of a "Gaioleiro" building

André João Silva Ferreira

October 2019

Abstract: The present dissertation is based on the seismic study of a typology of buildings often found in the center of Lisbon, called "Gaioleiros".
The structural vulnerabilities revealed at a seismic level are typical features of this type of buildings. They are proven to be one of its main shortcomings compromising structural safety. Current regulations ask for demanding and sometimes complex analyses in order to check its safety. Often this type of procedures focuses on the use of nonlinear analyses which require advanced technical knowledge and lengthy analyses. However, a linear analysis of response spectrum will be applied considering some concepts induced by the Italian regulation NTC 2008 with an affectation of the coefficient of behaviour to consider the nonlinearity.
The main goal of this dissertation is to establish a simplified seismic safety analysis procedure for existing buildings. To achieve this, a study of a "Gaveto" building will be carried out. This building is comprised mainly of stone masonry and brick interconnected by structurally intervened floors. Furthermore, it intends to study the influence of the reduction of seismic action proposed by the Italian regulation, with the aim of calling into question equal seismic security requirements for new and existing buildings. In order to complete the study, one needs to know more about the vibration modes of the building, further looking into different floor designs in the numerical model.
The results showed that despite the improvement of structural performance at the seismic level of the building due to the rigid diaphragm behaviour of the pavement, it does not check the safety. The reinforcement of the walls should be a priority.

Keywords: "Gaioleiros"; Seismic study; Linear analysis; Structural masonry; Seismic Vulnerability.

1. Introduction

The city of Lisbon pioneered the field of anti-seismic construction, in the period following the seismic catastrophe it suffered in 1755. This earthquake was so devastating that it triggered not only genuine and widespread concern, but also the need to change certain constructive habits to improve the quality of buildings and their structural capabilities. Plans designed and structured to find a constructive typology that could improve the capacity of buildings to resist seismic actions were put in place by Marquês de Pombal. The result was the emergence of the so called "Pombalino" building. As time went by, lightly builders and entrepreneurs began to simplify buildings, to be able to increase the financial returns and the speed of the construction process while the building quality didn't keep up. This led to the emergence of "Gaioleiro" buildings. (Delgado, 2013). This dissertation aims to study the seismic behaviour of this type of buildings due to its structural vulnerabilities and its widespread presence in Lisbon's "Pombalina" downtown area. The most common structural vulnerability related to this type of building is the development of out-of-plane mechanisms. This is caused by the lack of braces of the structural main walls that do not support out of plan actions due to their inflexibility. To avoid this, it is necessary for the building to have a "box behaviour". This allows the building's walls and floors to work together leading to a significant reduction in the possibility of this kind of mechanism occurrence.

However, ensuring this behaviour the in-plane mechanisms deserve to be properly studied (Simões, et al., 2017). Figure 1 shows two types of mechanisms often found: (a) rupture of the lintel; (b) rupture of piers.

The most important contribution of this work is the characterization of the materials in the study case, as well as its application in a 3D numerical model capable of a simplified analysis of the most critical areas to be reinforced in the face of Seismic action. To this end, it is intended to develop a seismic safety verification methodology for this case (figure 2). This methodology is similar to that developed by João Luís. (Silva, 2011). In addition, a modal analysis of the numerical model will be carried out, comparing the main differences between flexible floor (often found in this typology) and the use of rigid diaphragm on floors, in order to make displacements compatible on the horizontal plane. This last hypothesis shows to be very relevant for the development of the box behaviour concept. It should be noted that, although the latter hypothesis is very relevant for improving the seismic behaviour of this type of building, it is not at all enough to comply with the regulation's requirements.

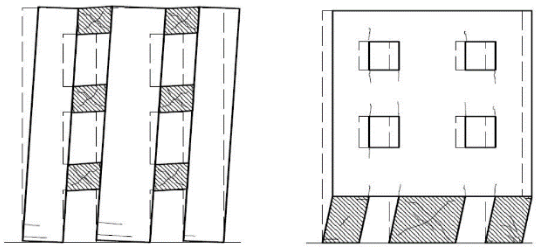


Figure 1 Plane rupture mechanism: (a) lintels(b) piers. (Candeias, 2008)

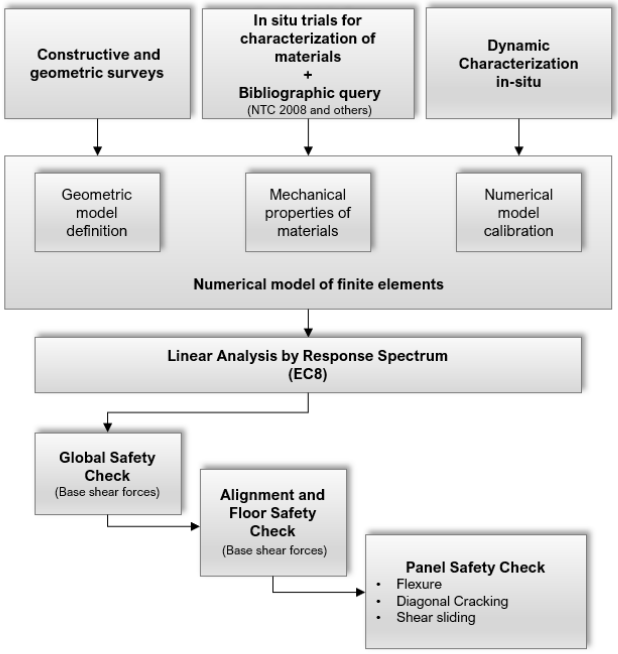


Figure 2 Applied methodology.

2. Characterization of the case study

Completed in 1906, the building under study is located in the Coração de Jesus parish. It measures approximately 22 m in length and 24 m in height (figure 3). The building has six floors: one of which consists of a semibasement, a ground floor, three high floors and an attic. On the first five floors, the right foot is constant of 3.5 m, except in the meekness that presents 3 m.

Figure 3 Main building façade

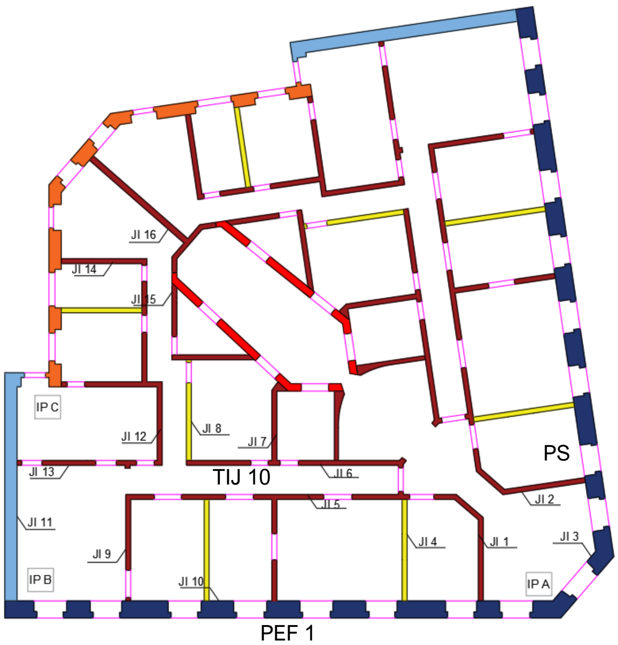
This covers an area of 550 m^2, containing two apartments per floor (each with 195 m^2). This building contains a lobby at its core so that it is possible to improve air circulation through the indoor areas.

The building includes rubble stone masonry walls on the front façade and gables; brick masonry on rear walls, inside the building and in the lobby; and partition walls of wood ("tabique"). The analysis of the different parts of the building was carried out by running an adequate bibliographic research together with viewing windows on walls and slabs on the third floor. As well as of probing wells in the foundations of the building. Figure 6 shows in the plant the location of the viewing windows performed and in Figure 4 depicts some of the samples obtained. The figures (a) and (b) represent samples of JI3 (façade wall) and JI13 (inner wall). The probing well in Figure 4 (c) was carried out on the ground floor, located in Figure 6. In this well it was possible to verify that the walls are seated on brick masonry arches. In Figure 4 (d) it is possible to observe that the floor beams seats on the wall of the PEF 1 façade.

Figure 4 Viewing windows carried out: (a) JI 3 - Irregular stone masonry; (b) JI 13 - Brick masonry; (c) Probing well (PS); (d) Wood floor (IP B)

Often one can also find in this type of buildings brick masonry arches in the upper area of the of windows designed to forward the loads to the adjacent zone (figure 5). Usually this type of buildings has undergone severe changes in the original layout of structural elements (Simões, et al., 2017). In this case of study, there were two major interventions that undermined the seismic behaviour of the building. The first took place somewhere between the conclusion of the building and its first intervention (1944), the lobby underwent a significant intervention as it doubled its original size. Later, between 1944 and 1955, to improve the commercial space in the semibasement, it was decided to open large spans both on the right and left side of the building (in relation to the axis of symmetry).

Figure 5 Masonry arch over window (orange)



Legend:
Blue – Rubble stone masonry wall (0.70 m)
Light Blue – Rubble Stone masonry wall (0.50 m)
Orange - Brick masonry wall (0.50 m)
Red - Brick masonry wall (0.30 m)
Dark Red - Brick masonry wall (0.20 m)
Yellow - Tabique wall (0.10 m)

Figure 6 Map location of: viewing windows on walls (JI) and floor (IP) held on floor 3; probing well on r/c (PS); identification of two alignments TIJ 10 and PEF 1.

3. Development of the numerical model

The design of a numerical model of an existing structure involves a high degree of complexity as it needs to ensure a correct definition of structural elements and their actions in order to achieve results realistically consistent with what is intended to be analysed. It is, therefore, important to simplify the model in order to select the important structural elements for the analysis of the way the structure works. This simplification process is also important to try to bring the model closer to reality as possible, namely by performing the calibration of the model according to the results of dynamic tests carried out *in situ*. *Sap2000 v20* software was used to perform this study (Computers and Structures Inc, 1995). It should be noted that the choice of the programme is decisive when it comes to the results and conclusions.

3.1. Definition of Materials

The materials observed in the work were brick masonry, stone masonry, wood, steel and concrete. Table 1Table 1 shows the modulus of elasticity and the volume weight considered. Using a simplified approach one can state that the damping coefficient used for all materials was 5% and Poisson coefficient of 0.2. Since the modulus of masonry elasticity was the most impactful parameter in the fundamental frequency of the building, it suffered changes after performing in situ characterization, as shown on table 1.

Table 1 Summary of the properties of the materials defined in the numeric model (Farinha & Reis, 1993) e (Simões, et al., 2017).

	E (GPa)	• (kN/m^3)
Rubble Stone Masonry	0.8 -> **1.0**	19
Solid Brick Masonry	1.1 -> **1.5**	18
Timber	8	-
Concrete C16/20	30	25
Steel 275	200	78

Designation	Symbol	Brick	Stone
Shear strength [MPa]	C_u	0.092	0.032
Tensile strength [MPa]	f_t	0.138	0.039
Compression strength [MPa]	f_m	2.40	1.50
Friction coefficient	$\tan \varphi'$	0.40	0.40

3.2. Definition of Elements

The elements created in the model were considering their main resistant characteristics. Therefore, the masonry walls were developed through Shell elements, while the floors and "tabique" partition walls were developed through frame elements. The main beams of the floors were guided according to the provisions previously spotted in the building. These dowels ("tarugos") are smaller wood elements perpendicular to the main beams and are arranged by meter. All wood elements were assumed to be biarticulated. The tabique walls were considered as a braced portico with a stiffness equivalent to that obtained by a test performed by Azevedo and Lopes (Azevedo & Lopes, 1995). Due to the low bending stiffness of the masonry walls the the m_{11}, m_{12}, m_{22}, v_{13} and v_{23} was reduced to 10% of the initial value (Freitas, 2009).

3.3. Definition of actions

Permanent, variable and seismic character actions were considered. First, the top floor was not modelled and is considered in the support alignments. The weight of the balconies, copings, stairs, pavement, partition walls and roofing were also considered using linear loads. The variable actions were obtained through EC1-1 and seismic actions through EC8-1, EC8-3 and NTC 2008 (EC8-1, 2005; EC1-1, 2009; EC8-3, 2017; NTC, 2008).

For seismic safety verification it is predicted the adoption of the seismic combination introduced by EC0 (EC0, 2009), in which seismic action will be obtained through a linear analysis by response spectrum. The value recommended by EC8 for the behaviour factor is 1.5.

3.4. Global Characterization

After the definition of all previous guidelines, the creation of the numerical model of finite elements is followed. The model was created using *Autocad* and *SAP2000*. It is represented in Figure 9. Figure 7 depicts metal beams located under the upper alignments.

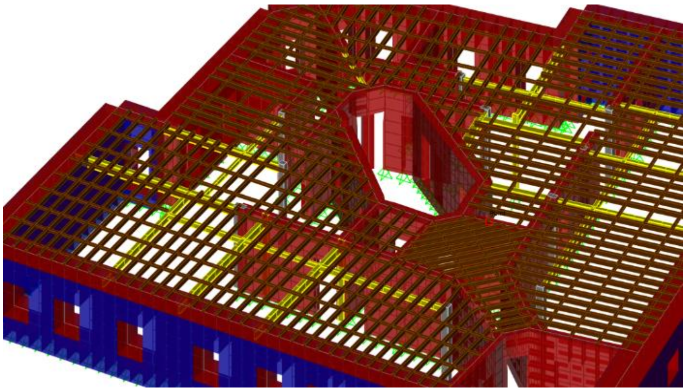


Figure 7 Cut above the ceiling of the floor 0.

4. Modal analysis

This is one of the most relevant analysis to understand how the structure works when exposed to regulated seismic action. That is the reason underlying the study of its main dynamic characteristics. It is customary in this type of analysis, combining visual and numerical results to better justify the modal features of the model. One of the main requirements for performing an efficient modal analysis is to ensure a high mass sum (at least 90% of the total mass of the structure). It turns out that for this type of buildings the mass of the building is mostly on the walls, making a lot of local modes pop up. It was necessary to use a sample of 400 modes to obtain the 90% figure required.

To analyse the influence of the behaviour of the rigid diaphragm, two distinct models were analysed: (a) Floor with rigid behaviour; (b) Floor with flexible behaviour (Figure 8). This is due to several reasons. One of which is the orientation of the walls of the lobby which turn out to be one of the preponderant reasons for this first mode to be displayed in illustrated mode, obtaining greater inertia parallel to the axis of symmetry rather than the perpendicular axis. In addition, it is possible to observe the reduced number of walls still being able to resist in the observed direction. Table 2 refers to the modal participation factors and frequencies obtained in both models.

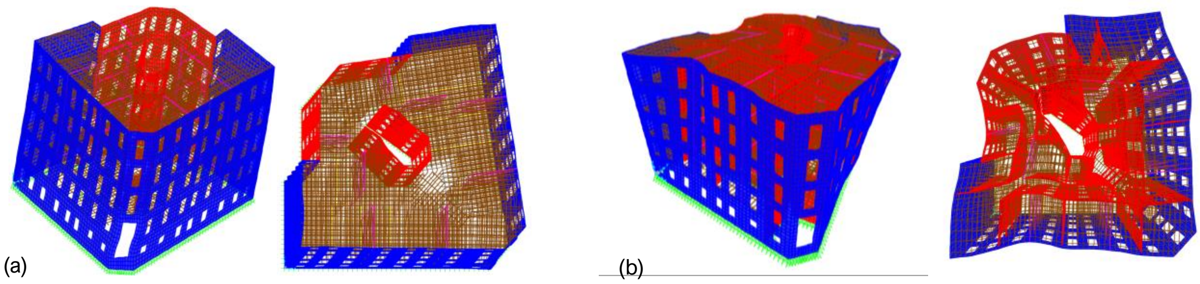


Figure 8 3D representation of the 1st vibration mode: (a) Rigid floor (b) Flexible Floor

Table 2 Modal participation factors and frequencies of the 3 most relevant modes.

Modes	Rigid				Flexible			
	f	Ux	Uy	Rz	f	Ux	Uy	Rz
1	2.50	**0.34**	**0.45**	0.00	1.47	**0.20**	**0.25**	0.02
2	2.63	**0.44**	**0.32**	0.00	1.96	0.00	0.03	**0.23**
3*	3.33	0.00	0.00	**0.70**	2.04	**0.36**	**0.20**	0.00

*The third mode of the Rigid Model appeared at 27º position on the modal analysis.

The modal analysis presented shows in a simplified way the impact of the rigid diaphragm behaviour in this type of buildings. Even with very rigid walls on the periphery, a high percentage of openings on the façade wall and with poor out-of-plane behaviour, it was shown an improved behaviour of the periphery walls with the introduction of the rigid floor. They worked efficiently and collaboratively with the inner walls of the building, promoting the removal of the 1st torsional mode and the reduction of deformations located out-of-plane of masonry walls. Notwithstanding, it will be important to mention that there is an increase of base shear forces on the periphery walls in relation to the walls of the interior with the introduction of rigid behaviour on the floors. This promotes particularly conditioning situations for the seismic assessment.

5. Seismic Assessment

For the seismic study, it will be adopted a modal response spectrum analysis with q-factor approach using two distinct cases: one related to the Italian regulation (NTC, 2008) that refers to use 65% of seismic action used in new buildings; and a second case using 100% of the seismic action recommended by EC8-3 (EC8-3, 2017). The use of the partial coefficient of materials suggested by EC6-1 (EC6, 2005) with a value of 1.5 was also considered. The level of knowledge assumed was level one corresponding to a value of 1.35. The last two were used to reduce resistance.

This methodology introduces increasingly more detailed checks, starting with a global verification by evaluating the total shear force for each of the main directions. Subsequently, each alignment by floor will be analysed and finally a more detailed check defined by panels (piers and lintels).

5.1. Global Analysis

This analysis is aimed at carrying out a simplified check to determine the global resistant capacity of the building. This analysis will be performed through the *mohr-coloumb* equation, $\tau = Cu + \sigma \cdot tan\ \varphi'$, which evaluates a possible global shear rupture. Table 3 shows the following parameters: Total vertical force for quasi permanent combination (F_v); Base Shear force (V_b); Shear resistance (V_{Rd}); Security check (VS).

Table 3 Global analysis of Seismic Action at 100% and 65%.

			Seismic action 100%			Seismic action 65%		
Designation	$F_{v\ (cqp)}$	V_{rd}	V_{bx}	$V_{bx\ 65}$	VS	$V_{bx\ 65}$	$V_{by\ 65}$	VS
Global	28 425	12 200	7965	7994	Yes	5145	5164	Yes

It should be noted the importance that the weight of the structure and its friction coefficient have for analysing the shear resistance in this type of buildings. It turns out that 90% of the V_{Rd} is related to the weight of the building and the friction coefficient. This means that even if the cohesion of the material is changed, it will not bring about major implications in this type of verifiction. By comparing the results, safety is verified in both cases.

5.2. Analysis by alignment and floor

To match a global and a more detailed analysis (per panel), an alignment and floor verification will be put forward in order to understand which ones are the most conditioning. This analysis will be performed for the shear rupture with the same method on 5.1. Figure 9 shows the seismic analysis performed in the load-bearing walls, PEF1 and TIJ10.

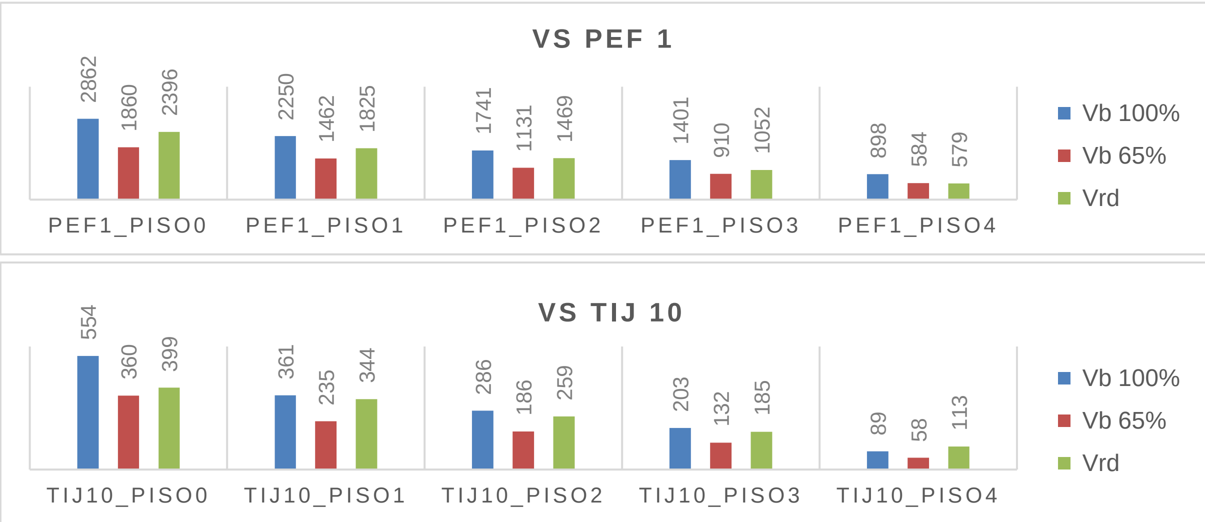


Figure 9 Representative graph of the safety check of the PEF 1 and TIJ 10 alignments (100% and 65% of seismic action).

Among all checks carried out in each alignment of the building, the safety check on the ground floor was achieved on the 65% mark, in opposition to the 100% mark when it didn't occur. It is also possible to understand that on the upper floors, both for the 65% and 100% of seismic action, the probability of this type of collapse mechanism to take place is higher. This is mainly due to the existence of lower compression and shear.

Compared to the analysis on 5.1, there is some inconsistency in the overall security check of the structure. This is due to the sensitivity that this mode of rupture presents in the face of the weight of the building introduced in the second portion of the shear resistance (V_{Rd}). In the verification by alignment and floor, it is shown a greater coherence with reality due to better detailing of the attributed forces, as well as the consideration of mechanical properties.

5.3. Panel analysis

The following study will focus on the analysis of 3 rupture modes on piers: (a) flexure; (b) shear sliding and (c) diagonal cracking (figure 10). The shear forces (V_b) are obtained at the base of each panel. The equations provided by the Italian regulation will be used to calculate the shear resistance (V_{Rd}) for each panel. The results obtained for the two variations of seismic action for piers will be mentioned for two alignments: an interior (TIJ 10) and a peripheral one (PEF1).

For the lintels PEF1 alignment a more simplified analysis will be applied by checking tensions (σ_{12}) (figure 12), with the intensity of the seismic action both at 100% as 65%, establishing a comparison with the shear resistance (cohesion).

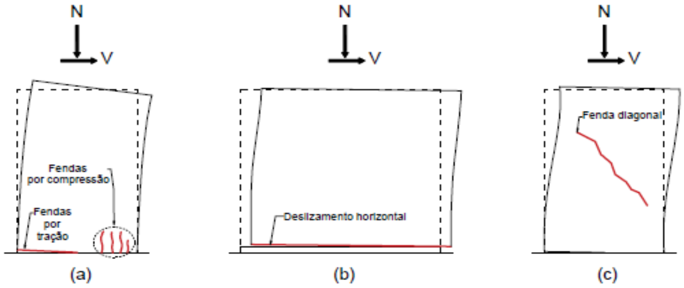
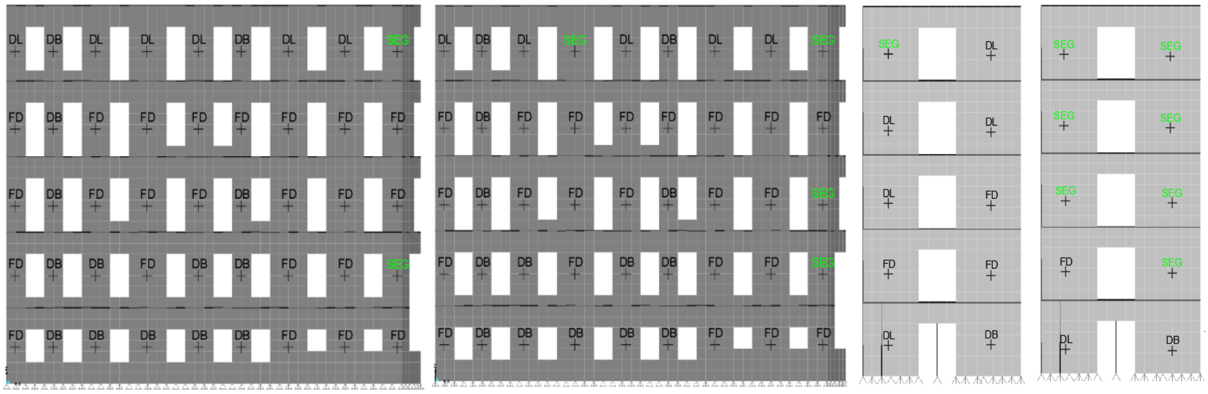


Figure 10 Mode of collapse in masonry panels: (a) flexure; (b) shear sliding; (c) diagonal cracking (Calderini, 2009).

DB - Flexure; **DL** - Shear sliding; **FD** - Diagonal cracking; **SEG** - Verified security



PEF 1

TIJ 10

Figure 11 Safety check of PEF 1 alignment piers and TIJ 10 (left 100% and right 65% of seismic action).

Regarding PEF 1 alignment, one can perceive that overall security is not verified. The predominant rupture modes on the first 4 floors are diagonal cracking and flexure mechanisms, while on the top floor the breakage by shear sliding and flexure mechanisms are more predominant (figure 11). The low cohesion of stone masonry, in correlation with the high shear and compression stresses on the lower floors, promotes smaller resistant efforts for the rupture modes previously stated. Regarding the percentage variation of seismic action, it shifted from 2 to 4 panels on the security check. Three of those are located in the last column where a lower V_b can be found. The internal alignment TIJ 10 proved to be the most sensitive to reducing seismic action by significantly changing from one to seven panels with verified security.

For the security check of the lintels a resistant force (V_{Rd}) was admitted with an average value of 50 kPa for brick masonry, and 20 kPa for stone masonry, already including affectation of the confidence and partial coefficients. Through figure 12, it is possible to visualize very high shear tension (σ_{12}), with values greater than 50 kPa thus leading to a failure in the safety check.

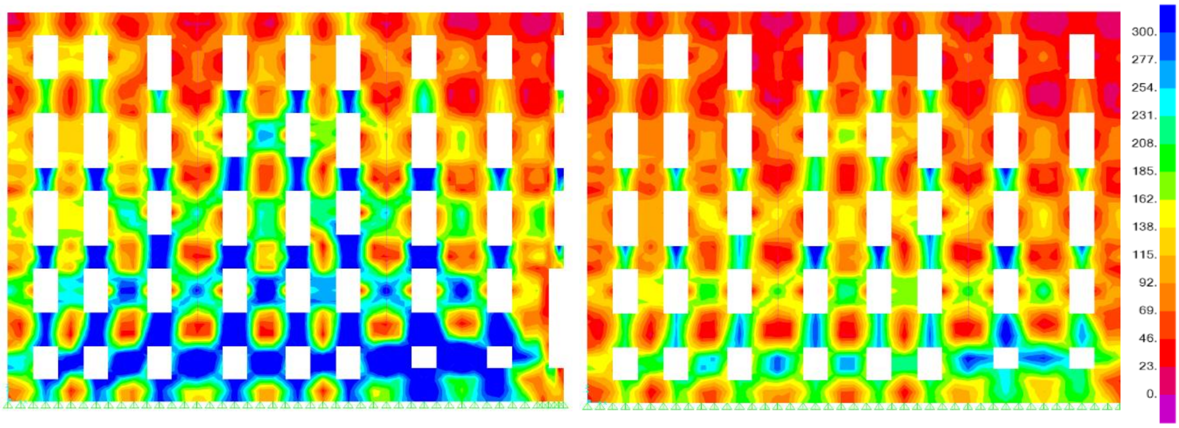


Figure 12 Analysis of σ_{12} on the wall PEF1 (left 100% and right 65% of seismic action)

6. Conclusion

Overall, security is not confirmed regarding regulatory seismic action (EC8 and NTC 2008). Although there are some alignments where the consideration of seismic reduction to 65% led to security verification, the lack of resistant capacity of the load-bearing walls to resist earthquakes is questioned.

The low cohesion of the stone masonry with a high ratio between compression tension (σ_0) and cohesion (C_u), common on the lower walls, promotes the premature occurrence of diagonal cracking. This situation is particularly noticeable in the PEF1 alignment.

As expected, the shear sliding mode proved to be the most likely to succeed on the upper floors. However, for these floors the flexure mode is also a possibility if it takes place in more slender elements.

The relevance of cohesion in this type of analysis is high. The results show the need to strengthen masonry in order to be able to support horizontal actions.

One of the main conclusions that is drawn from the percentage variation of seismic action is the fact that the safety check of stone masonry walls does not show such expressive improvement rates as it does on brick masonry walls. This is due to the high shear values on the periphery walls, as well as the fact that the brick masonry wall contains a cohesion three times higher than stone masonry.

Despite the use of the rigid diaphragm, it is understood that this renovation measure does not solve all identified problems. It should also be considered an improvement in the cohesion of existing materials.

Overall, there is no safety in the lintels, and the PEF1 alignment is mainly highlighted due to its reduced shear resistant capacity. The same goes for brick masonry walls. Despite containing a superior shear resistance, the lack of safety covers all exterior panels.

Comparing the assessment by alignment and by panel, it is understood that there is a need to evaluate safety for the 3 mentioned rupture modes. This is because the limitation of uniquely using the shear sliding mode mistakenly indicates a significant improvement in structure safety for the reduction of 65% of seismic action when it comes to the second method mentioned. In fact, with the aid of a more detailed analysis (analysis per panel), evidence of "premature" occurrence of more conditioning rupture modes of flexure and diagonal cracking on the lower floors was found, i.e., V_{rd} lower than shear sliding mode. This occurred on PEF1, TIJ 10 and other alignments not mentioned here.

In relation to the global mechanisms addressed in the introduction, it will be possible to anticipate the occurrence of the two types of collapse (mechanism of figure 1). The combination of high shear forces in piers and lintels with the weak resistant capacity of the structure, leads to a strong possibility of collapse of panels prematurely on the first floor To take this possible problem into account, performing an incremental analysis, such as the nonlinear Pushover analysis, will be appropriate to understand where the first mechanisms will come up.

However, the assumed methodology allows an adequate qualitative analysis of the level of seismic safety of the structure and it is therefore possible to identify the zones with the highest susceptibility, thus reaching the goals initially proposed.

7. Bibliography

Azevedo, J., & Lopes, M. (1995). *Assessment of the Seismic Performance of a Traditional Masonry Building in Lisbon-Relatório CMEST AI 2/95.* IST.

Calderini, C. C. (2009). *In-Plane strength of unreinforced masonry piers, International Association for Earthquake Engineering, 44 (Novembre 2008).*

Candeias, P. (2008). *Avaliação da Vulnerabilidade Sísmica de Edificios de Alvenaria - Tese de Doutoramento.* Minho: Universidade do Minho.

Computers and Structures Inc. (1995). *SAP 2000 Analysis Reference.* USA.

Delgado, J. (2013, Outubro). Dissertação para obtenção do Grau de Mestre em Engenharia Civil. *Avaliação sísmica de um edíficio crítico em alvenaria*. Lisboa: Instituto Superior Técnico.

EC0. (2009). *Eurocódigo 0: Bases para o projeto de estruturas.*

EC1-1. (2009). Eurocode 1: Actions on structures. Bruxelas: CEN.

EC6. (2005). *Eurocódigo 6: Projeto de Estruturas de Alvenaria - Parte 1-1: Regras gerais para estruturas de alvenaria armada e não armada.*

EC8-1. (2005). Eurocode 8: Design of structrures for Earthquake Resistance - Part 1: General Rules, Seismic Actions and Rules for Buildings. Bruxelas: CEN.

EC8-3. (2017). *Eurocódigo 8: Projecto de estruturas para resistência aos sismos - Parte 3: Avaliação e reabilitação de edifícios.*

Farinha, B., & Reis, A. (1993). *Tabelas Técnicas.* P.O.B.

Freitas, J. (2009). *Introdução ao métodos dos Elementos Finitos:Elasticidade Plana e Tridimensional".* Lisboa: IST.

NTC. (2008). "Norme Tecniche per le Costruzioni". Official Bulletin.

Silva, J. (2011). Dissertação para obtenção do Grau de Mestre em Engenharia Civil. *Avaliação e reforço sísmico de edifícios escolares*. Lisboa: Instituto Superior Técnico.

Simões, A., Appleton, J., Bento, R., Caldas, J., Lourenço, P., & Lagomarsino, S. (2017, Fev 02). Architectural and Structural Characteristics of Masonry Buildings between the 19th and 20th Centuries in Lisbon. *International Journal of Architectural Heritage*, 12-14.