Seismic Vulnerability Assessment of Lisbon's Downtown

Luís Guilherme Faria da Luz

Instituto Superior Técnico, Universidade de Lisboa

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Abstract

Pombalino buildings, built during the reconstruction of Lisbon downtown after the earthquake of 1755, are considered a historical landmark for seismic engineering and for the Portuguese heritage. However, due to inappropriate structural interventions associated to these buildings in the last decades, a weak structural behaviour during a seismic event is expected; thus, it is extremely important to study the seismic performance of the *pombalino* buildings in the current stage.

In this dissertation, the seismic performance of a *pombalino* building was evaluated using 3MURI/TREMURI program, in which pushover analyses were performed. Since these buildings are located in blocks, the analyses were performed by taking in account the influence of the adjacent buildings and after such model was compared with a model of the isolated building. Then, the main structural changes observed for this typology were modelled and their seismic performance was also evaluated. Regarding to the results obtained, it is clear that these buildings do not satisfy code safety requirements.

By means of surveys made to Lisbon downtown, geometrical and structural characteristics of the buildings were collected. This information was stored in a GIS database and represented in a three-dimensional model, using ArcGIS and CityEngine, respectively.

In order to evaluate the seismic vulnerability of the case study, buildings damage was estimated using two first level approaches, the methodology of the vulnerability index and the *Csi 3 method* (ξ_m^3). Finally, damages were assessed for the analytical model of the building studied and compared with those obtained through the simplified methods.

Keywords Pombalino Buildings; Lisbon downtown; Seismic Vulnerability; Geographic Information Systems (GIS); nonlinear analysis; Seismic performance-based assessment;

1 Introduction

The recent seismic events recorded around the world increased the concern of governmental authorities and civil protection bodies in developing seismic risk mitigation strategies. This is currently one of the priorities of the political agenda of most Mediterranean countries [1].

After the great earthquake of 1755, the Lisbon downtown was rebuilt with the aim of resisting future seismic events and *pombalino* buildings represent a historical landmark for seismic engineering and for the Portuguese heritage. In contrast with the original conception, a poor seismic performance is expected for those buildings in the current stage, as a consequence of the inappropriate structural interventions that these buildings have been exposed.

Thus, due to this, the work present herein is focused on the influence of structural changes on the seismic performance of this typology. In addition, this research will give a contribution to such topic, since there are not many studies in the literature where the influence of the structural changes was studied. Moreover, historical centres as Lisbon's downtown justify the need for the use of first level approaches on the seismic vulnerability assessment, and this will be developed in this study.

2 Characterization of *pombalino* buildings

Pombalino buildings are usually layed out in rectangular blocks, between sets orthogonal streets, having 4 floors plus mansard roofs. Buildings separation is characterized by the existence of a single side wall that rises above the roof to prevent fire from spreading between adjacent buildings. The fact that these buildings are similar and share side walls, benefits the seismic behaviour of the block as if it were a single building.

In *pombalino* buildings it is worth to highlighted, among the different structural elements, the *Gaiola Pombalina*. It is a three-dimensional wood structure which is developed from the top of the ground floor to the roof and that provides seismic resistance to the building. The *Gaiola* is constituted by a set of plane trusses filled with masonry, called *frontal* walls [2].

The façades and side walls are usually built with ordinary rubble masonry and air lime mortar and their thickness may vary along the height. The floors are made of wood planks supported on perpendicular wood joists, which are supported on the façade and frontal walls [2].

2.1 Structural Modifications

However, as the years passed the memories of the great earthquake of 1755 have been forgotten as well as the good construction practices. *Pombalino* buildings have experienced several structural interventions mostly motivated by new habits and needs of occupation. The main structural modifications and their consequences are presented below.

2.1.1 Increasing Number of Storeys

The idea of a *pombalino* block constituted by buildings with the same height nowadays is not a reality in Lisbon's downtown. Due to the irregularities in the *pombalino* block (Figure 2.1) buildings will show different dynamic characteristics which can compromise the seismic behaviour of the block.

The direct consequence of adding new floors is the increase of the weight of the structure, resulting in the increasing of the inertial horizontal forces and displacements during the seismic action. This would not represent a problem if the building was properly reinforced, but in most of the situations this does not happen [3].

Figure 2.1 – Irregularities in the pombalino block due to the increasing number of storeys

2.1.2 Interruption of Piers on the Ground Floor

In order to create wider shop windows, the vertical elements from the façade (called piers) were cut (Figure 2.2). Usually, this is performed by introducing a steel beam on the top of the piers only to transfer the loads to the adjacent elements [2].

Figure 2.2 – Interruption of piers on the ground floor

The effects of the inertia horizontal forces increase from top to the bottom, and by cutting piers the structure is being weakened where the seismic effects are stronger. The final result is the creation of an artificial soft-storey and history records show a bad performance for this type of structures [3].

2.1.3 Elimination of Frontal Walls

The elimination of *frontal* walls is strongly related to the need of increasing areas or adapt to a different use. This intervention is usually complemented by the introduction of steel/concrete beams supported by columns or transferring loads to adjacent walls (Figure 2.3).

Frontal walls are extremely important on withstanding the horizontal forces and contribute to the energy dissipation capacity of the system. The consequences of interrupting the *Gaiola Pombalina* will be the decrease in stiffness and strength at that level. In addition, the embrace of the façades can be compromised if the removed *frontal* walls were perpendicular to these ones.

Figure 2.3 – Frontal walls substituted by steel beams [4]

2.1.4 Replacement of Structure

There are several cases where *pombalino* buildings are completely demolished, only maintaining the façades (regulatory imposition), and usually replaced by concrete structures.

In general, these new buildings are designed in accordance to the current codes, thus presenting a good seismic response. Despite of having a better individual performance, this intervention deteriorates the seismic response of the block as it creates rigid points in the aggregate.

3 Seismic Performance of a *Pombalino* Building Inserted in a Block

3.1 Structural Modelling

In order to simulate the nonlinear behaviour of masonry, the building was modelled in 3MURI/TREMURI software [5][6] which allows the use of pushover analysis. This program is based on equivalent frame modelling [7][8] and it is recommended by some seismic codes [9][10]. According to this approach, each masonry wall is defined by macro-elements that are divided in: piers (vertical elements), spandrels (horizontal elements), linked by rigid nodes. A limitation of this program is the fact that it doesn't consider the out of-plane collapse mechanism, which is relevant for masonry buildings.

The case study is a building located in Lisbon's downtown, representative of *pombalino's* typology and it was already studied by different authors [11][12]. In one of these works, Meireles [12] developed in TREMURI a macro-element for the frontal walls behaviour that was used in the present work.

The mechanical properties of materials were selected accordingly to the Italian code [10] and Simões et al. [13]. These values are presented in Table 3.1.

Table 3.1 – Mechanical properties of masonry

	E [GPa]	G [GPa]	f_t [MPa]	f_c [MPa]	w [kN/m^3]
Stone masonry	2.8*	0.86*	0.105	7.0	22
Rubble masonry	0.5	0.167	0.039	1.07	18.35
Solid brick masonry	0.855	0.285	0.115	1.07	18

*The presented value was reduced by 50% to taking into account the cracking phenomenon.

Note: Presented values are not reduced yet by the CF.

The definition of pavements is a crucial issue when modelling masonry buildings with timber (flexible) floors.

When modelling this *pombalino* building, Meireles [12] assumed G=750 MPa. This value is proposed by LNEC for wood, but the shear modulus of the pavement is significantly lower because of the connections between wooden planks.

Accordingly to the work developed by Ponte[14], it was considered a flexible floor with G=18 MPa, calculated based on the New Zealand codes [15][16]. However, due to a lot of uncertainty when modelling flexible diaphragms, the influence of shear modulus on the pushover results was studied (Figure 3.1).

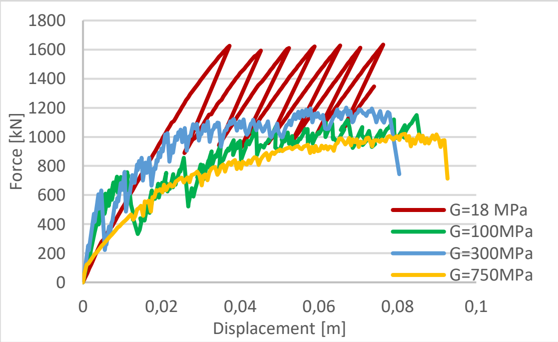

Figure 3.1 – Influence of G on the pushover results for X direction

As can be seen, the quality of the results is sensitive to the considered shear modulus since numerical problems decrease as G increases. The hypothesis of modelling the floor with G=18 MPa was excluded due to the inaccurate results obtained. In this way, it was assumed G=300 MPa for being the lowest value that guarantees some quality in the results. Table 3.2 summarizes the mechanical properties assumed for the timber pavements.

Table 3.2 – Timber pavement mechanical properties

	E [GPa]	E_{90} [GPa]	G [GPa]
Timber Pavement	12.0	0.4	0.3

In order to take into account the surrounding conditions the building was replicated to simulate the adjacent buildings as can be seen in Figure 3.2.

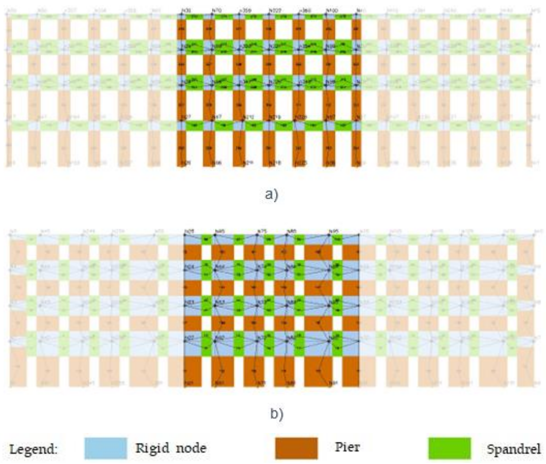

Figure 3.2 - Macro-elements mesh for the modelled building

3.2 Nonlinear Static Analysis

The nonlinear static (pushover) analysis were performed according to the EC8-3 [17]. These analyses were made in the two main directions (X is the direction parallel to the façades and Y is perpendicular to the façades).

For existing masonry buildings, the safety verification should be performed for the ultimate limit state of Significant Damage (SD).

The results below show the capacity curves obtained by applying an uniform distribution in the positive direction, since it is the most conditioning. More detailed results can be found in [18].

3.2.1 Isolated Building Model versus Building Inserted in Aggregate Model

In this section, a comparison is made between the results obtained for an isolated building model and a building inserted in an aggregate. The capacity curves are presented in Figure 3.3.

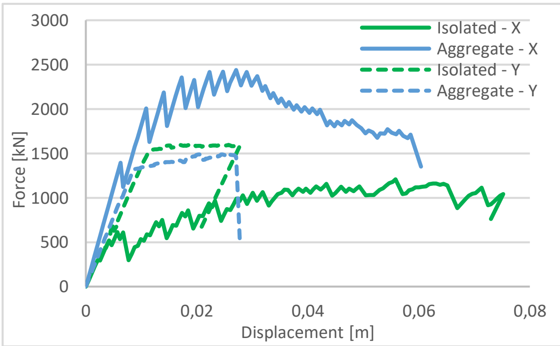

Figure 3.3 - Capacity curves for the isolated building model and the building inserted in aggregate model

Analysing the capacity curves, it is possible to conclude that X and Y direction present a flexural and shear behaviour, respectively.

For X direction, there is a large increase in the strength capacity when the building is inserted in aggregate due to the adjacent buildings which provide a better distribution of the nonlinear behaviour. On the other hand, for the orthogonal direction the results for both models are similar.

Since the side walls do not have any openings, a higher strength and stiffness for Y direction was obtained for the isolated model, what was expected. Although the same does not occur for the aggregate model. This can be explained by the replication of the façade walls, which triple its length and consequently increasing the stiffness of the building for X direction.

3.2.2 *Increasing a Storey*

As mentioned, increasing number of storeys is one of the main structural modifications observed in Lisbon's downtown; thus, it was modelled and compared with the results from the original building. The building with 5 floors is represented in Figure 3.4.

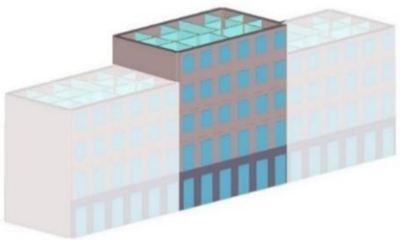

Figure 3.4 – Increasing a storey model

Figure 3.5 shows the results of the capacity curves.

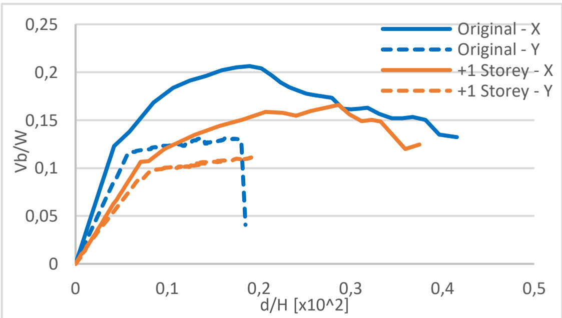

Figure 3.5 - Capacity curves for the original model and the increasing storey model

The direct consequence of this intervention is the increase of weight in the structure. Since the building was not exposed to any seismic reinforcement, it exhibits the same strength but the inertia horizontal forces will be higher. Thus far, the capacity of the building decreases for both directions, as can be seen in Figure 3.5.

3.2.3 *Cut off a Pier on the Ground Floor*

By cutting a pier on the ground floor, it can be anticipated a poor seismic response as it creates an artificial soft-storey in the structure. This was modelled and a steel beam was introduced to transfer the loads to the adjacent piers (Figure 3.6).

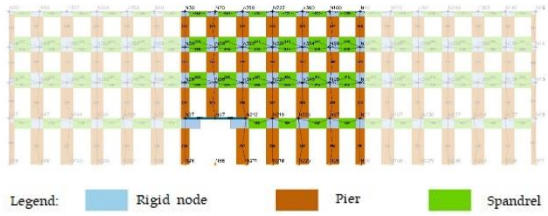

Figure 3.6 – Front façade mesh after cutting a pier

The comparison between the capacity curves is shown in Figure 3.7.

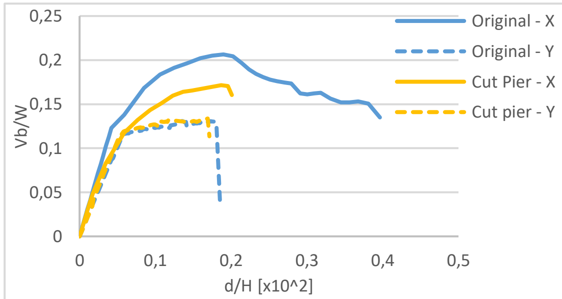

Figure 3.7 – Capacity curves for the original model and the cut of a pier model

This intervention mainly affects the X direction, since the ultimate displacement and strength decrease. This is due to the stress redistribution that is affected by this structural change. Nothing should be addressed for Y direction, as the results are quite similar.

3.2.4 *Removal of Frontal Walls*

Specific *frontal* walls were removed creating a wider space and steel beams were also introduced, as represented in Figure 3.8.

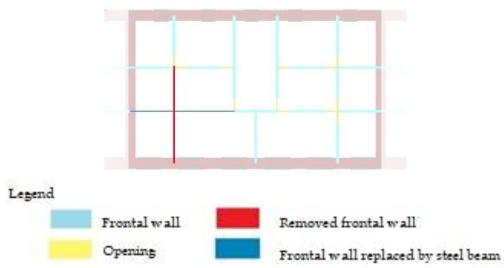

Figure 3.8 – Structural modifications on the upper floors

Figure 3.9 shows the capacity curves for the compared models. The façade walls are the main responsible for the flexural behaviour in the X direction and their strength capacity is affected by this intervention, since that the removal of *frontal* walls affects the correct transmission of loads. Once again, these changes do not affect the results on Y direction.

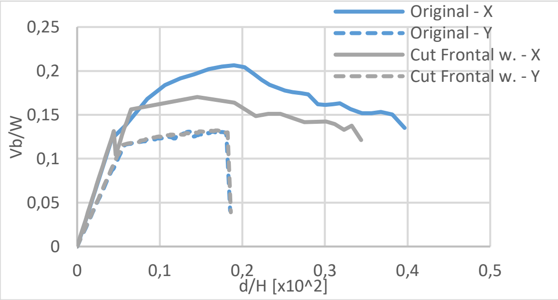

Figure 3.9 – Capacity curves for the original model and the and the removed frontal walls model

3.3 N2 Method

The seismic performance of the structure is evaluated through the target displacement according to the N2 method [19] as suggested by the EC8-1 [20].

Since seismic action is quantified by a response spectrum, the capacity curves were transformed in a single degree of freedom (SDOF) equivalent curve. The seismic response of the SDOF is quantified by the target displacement (d_t^*) which can be obtained by a graphical procedure based on the period (T^*) of the equivalent structure.

For the safety verification, the displacement imposed by the earthquake must not exceed the ultimate displacement of the SDOF structure ($d_u^*/d_t^* > 1$). According to NP EN 1998-3 [21], d_u^* must be reduced to 3/4 of its value. The results shown below are for seismic action type 1.3 for being the most demanding, as concluded in [18].

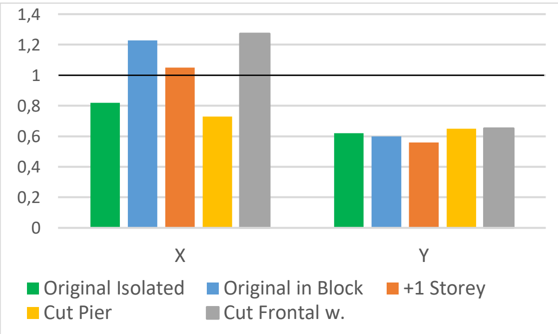

Figure 3.10 - d_u^*/d_t^* criterion for the analysed models

Analysing Figure 3.10, it can be concluded that none of the cases satisfies the safety criterion, particularly in the Y direction, where the ratio is always less than 1. For this reason, the seismic assessment is conditioned by the Y direction that evidences the worst behaviour due to the shear failure modes associated to the side walls. Nevertheless, the X direction is more influenced by the structural interventions modelled.

Comparing both models for the original *pombalino* building, it can be seen that the aggregate effect positively affects its flexural behaviour and increases significantly the d_u^*/d_t^* criterion.

As expected, the interruption of a pier is the most extreme situation since the ductility of the structure is highly affected.

Contrary to what was expected, the removed frontal walls model presents good results due to an increase of ductility on X direction. This can be justified by the fact that the program assumes a perfect transmission of forces between the steel beam and the façade nodes. For this reason, these results deserve a careful interpretation as they happen because of a modelling limitation. It must be highlighted that this intervention is always unfavourable to the structure.

4 Geometric and Structural Characterization of Lisbon's Downtown

According to Catulo [22], a GIS database was created and geometric and structural information about *pombalino* buildings based on file records from municipal archives were stored. By means of regular visits, in the present work this database was updated by collecting and storing data about 298 buildings. The studied zone is represented in Figure 4.1.

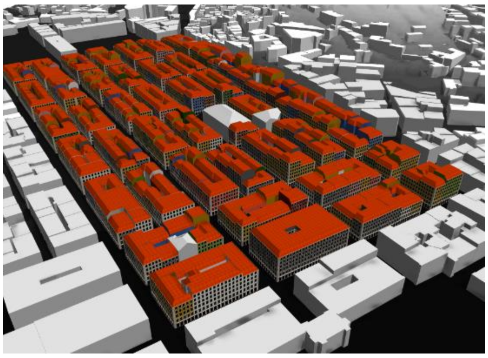

Figure 4.1 – Definition of the studied zone

Beyond of being extremely useful in storing and managing information, GIS tools are appropriate to represent the information by using thematic maps of a certain attribute. The storage and management of the information were made using ArcGIS [22], and the attributes were represented in a three-dimensional model developed by Ildefonso et al. [23] using CityEngine [24].

4.1 Buildings Typology

Figure 4.2 shows the typology of each building from the studied area.

As mentioned above, introducing concrete structures in *pombalino* blocks is equivalent to the creation of extremely rigid points. Thus, from the 36 *pombalino* blocks, only 9 are integrally composed of *pombalino* buildings. In them, 233 buildings are *pombalino* buildings (78.19%) and 65 (21.81%) are concrete structures.

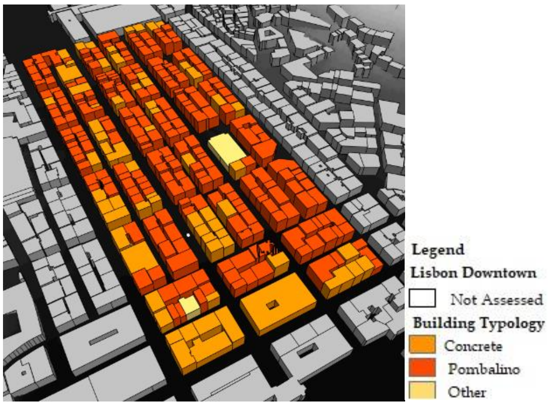

Figure 4.2 – Typological classification

4.2 Number of Storeys

According to the reconstruction plan of Lisbon's downtown, *pombalino* buildings should be composed of 4 floors plus the mansard roof. As shown in Figure 4.3, this is not a reality for the current *pombalino* buildings.

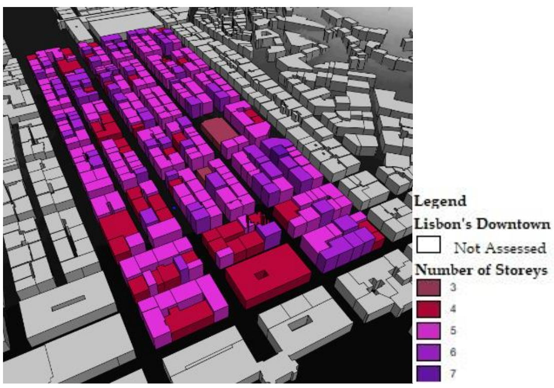

Figure 4.3 – Number of Storeys

Table 4.1 shows the distribution of the number of floors for *pombalino* buildings. As can be seen, only 13.30% of buildings respect the original conception. Thus, 63.95% have 5 floors and there is 21.89% with 6 floors, adding the consequences mentioned in 2.1.1.

Table 4.1 – Number of storeys for pombalino buildings

	Total	%
3 Storeys	1	0.43
4 Storeys (Original)	31	13.30
5 Storeys	149	63.95
6 Storeys	51	21.89
7 Storeys	1	0.43

4.3 Ground Floor Piers

During the on-site visits, a detailed study was made in order to classify the ground floor piers of *pombalino* buildings. Figure 4.4 shows the classification followed.

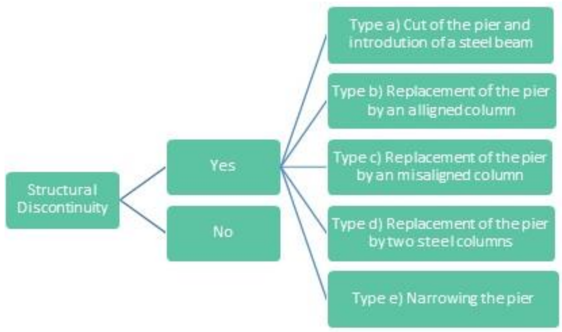

Figure 4.4 – Classification of the ground floor piers

Figure 4.5 illustrates the types of discontinuities observed for the ground floor piers in Lisbon's downtown.

Figure 4.5 – Types of discontinuities for ground floor piers

Table 4.2 summarizes the results for the percentage of discontinuities in *pombalino* buildings.

It is concluded that 50.6% of *pombalino* buildings have pier discontinuities and it is highlighted the fact that 14.2% of the buildings have more than a half of the piers with discontinuities.

Table 4.2 – Piers discontinuities in pombalino buildings

% Piers Discontinuities	Total Buildings	% Buildings
0	115	49.4
0 a 25	43	18.5
25 a 50	42	18.0
50 a 75	21	9.0
75 a 100	12	5.2

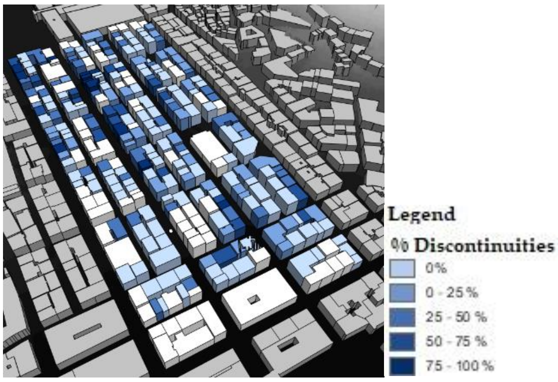

Figure 4.6 – Percentage of discontinuous piers

According to the types of discontinuities observed, Figure 4.7 shows the distribution of each case by the analysed buildings.

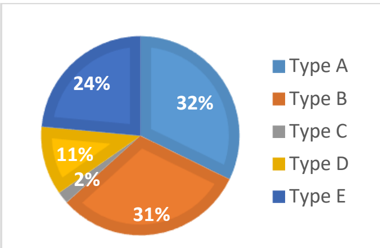

Figure 4.7 – Types of discontinuities

The most frequent discontinuities are Type A and Type B, with 32% and 31% respectively. As already mentioned, interrupting a pier on the ground floor (Type A) is possibly the most vulnerable situation, thus it is represented in Figure 4.8.

As shown, the interruption of piers on the ground floor is more evident in commercial streets, as *Rua Augusta*, *Rua da Áurea* and *Rua da Prata.*

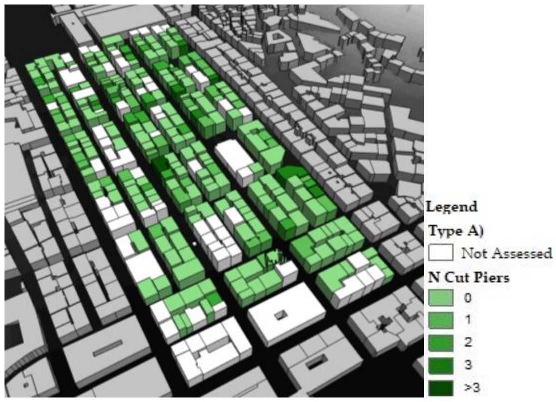

Figure 4.8 – Type A) discontinuity

5 Vulnerability Assessment of Lisbon's Downtown

According to the scale of the studied area, two different (first level) approaches were used to assess vulnerability of *pombalino* buildings from Lisbon's downtown: (i) Vulnerability Index method; (ii) ξ_m^3 method.

It should be noted that the comparisons made for these methods are performed for the same seismic intensity. Detailed procedures can be found in [18].

5.1 Vulnerability Index Method

Adapted by Vicente [25] for Portuguese buildings, this method is based on the GNDT II method [26] and the macroseismic method [27]. Vulnerability index method was applied to Lisbon's downtown by Falcão et al. [28], but due to the updating of the GIS database in this work it justifies a new assessment. When applied by Vicente [25] to the Coimbra's downtown, a detailed inspection was made to the interior of the buildings, however the same was not possible in the present study.

This methodology is based on the calculus of a vulnerability index (I_V^*), obtained as the weighted sum of the 14 parameters. Each parameter can be classified in 4 vulnerability classes (A, B, C and D) and a weight is assigned ranging from 0.5 to 1.5. The vulnerability index ranges between 0 and 650 and can be normalized (I_{Vf}), assuming a value between 0 and 100.

Figure 5.1 shows a histogram and an adjusted normal distribution for the assessment results. Vulnerability index (normalized) ranges between 20 and 60, resulting in a mean vulnerability index, $I_{V,médio} = 40.6$, a moderately high value. The standard deviation is low ($\sigma_{IV} =$

6.38), reflecting the homogeneity associated to these buildings.

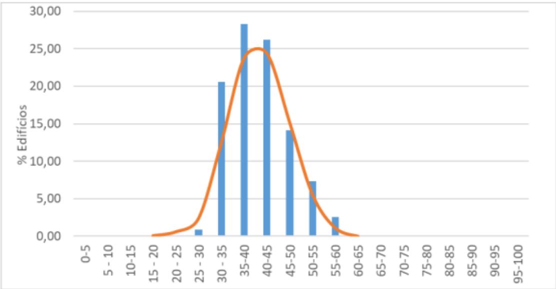

Figure 5.1 – Histogram and adjusted normal distribution

Figure 5.2 plots the variation of the vulnerability index (normalized) for the studied buildings.

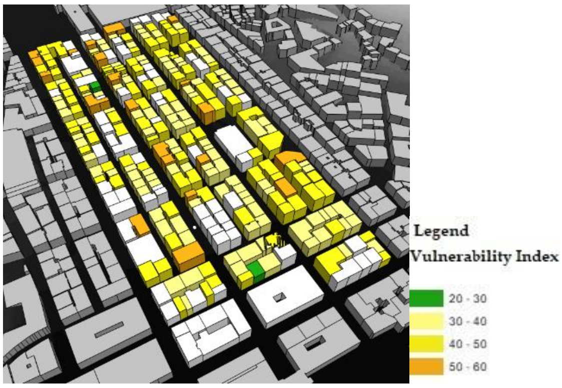

Figure 5.2 – Vulnerability index for Lisbon's downtown

After calculating the vulnerability index, mean damage grades, μ_d, can be estimated by the following expression:

$$\mu_d = 2.5 \times \left[1 + tanh\left(\frac{I_{EMS-98} + 6.25V - 13.1}{Q}\right)\right] \qquad (5.1)$$

where I-EMS98 is the macroseismic intensity described in EMS-98 scale [29], Q is a ductility factor (it was assumed Q = 2.3 as proposed in [27]) and V is a vulnerability index for the macroseismic method ranging from 0 to 1[27].

For being the most demanding to the case study, it was considered seismic action 1.3, and according to EC8-3 [21], a_{gR} must be multiplied by 0.75. A soil type C was considered for Lisbon's downtown. Thus, the considered seismic intensity is shown in Equation 5.2.

$$I\ [PGA] = a_{gR} \times S \times 0.75 = 0.18\ g \qquad (5.2)$$

In order to relate seismic intensity for both EMS-98 and PGA scales, Barbat et al. [30] proposes the following expression:

$$I_{EMS-98} = 5 + \frac{\ln(PGA) - \ln(0.03)}{\ln(1.8)} \qquad (5.3)$$

The relation between vulnerability index (I_{Vf}) and macroseismic vulnerability (V) is defined according to correlation below, proposed by Falcão et al. [28]:

$$V = 0.0068 \times I_{Vf} + 0.521 \qquad (5.4)$$

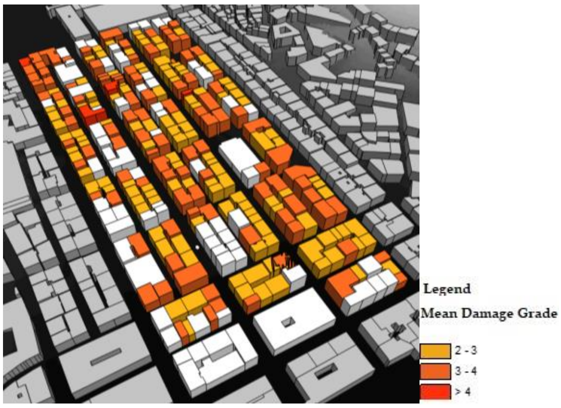

Figure 5.3 -Mean damage grade for Lisbon's downtown

For the results presented above, 54.1% of buildings have moderate damage (μ_d between 2 and 3), 44.2% show extensive damage (μ_d between 3 and 4) and 1.7 collapses (μ_d >4). According to Vicente et al. [31], buildings with μ_d $\geq$ 3 should be subject to a more detailed re-assessment.

5.2 ξ_m^3 Method

This simplified method was developed by collecting a sample of 128 capacity curves of masonry buildings. The main advantage of ξ_m^3 method [32] is the possibility of defining a capacity curve for each building through empirical correlations without the need for a detailed assessment. The period of the structure (T_y), drift (θ_y) and ductility (μ) define the curve, and the following expressions were established for *pombalino* buildings:

$$T_y = 0.13 \cdot n^{0.6} \qquad (5.5)$$

$$\theta_y = 0.0007 \qquad (5.6)$$

$$\mu = 105.2 \cdot (T_y^{-0.01} - 1) \qquad (5.7)$$

In order to take into account structural modifications, performance modifiers (Δ) are introduced, applied by multiplication to S_{ay}.

As in the analytical model studied, the seismic action is defined by a response spectrum and seismic assessment also by applying N2 method. Damage estimation can be done by comparing target displacement with the spectral displacement limits defined by Risk-UE [33]. The damage distribution is shown in Figure 5.4. As can be seen, this method is more burdensome, since 82.8% of the buildings present very heavy damage/collapses.

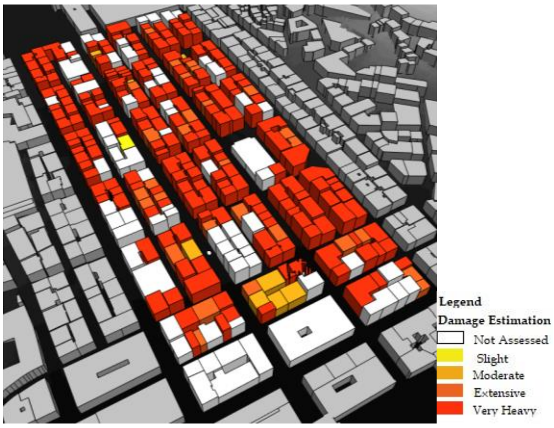

Figure 5.4 – Damage estimation for Lisbon's Downtown

5.3 Comparison Between Both Methods

A characteristic of these methods is the fact that for a building typology the results are very homogeneous, so for each method the buildings tend to be in a certain class of damage.

Vulnerability index method presents good results when its application results from a detailed inspection of the buildings which was not possible in this case.

On the other hand, ξ_m^3 method correlations for *pombalino* buildings are proposed based only in two cases for this typology, not being a representative sample. Furthermore, a limitation of this method is that performance modifiers are only applied to S_{ay}, but as already seen, cutting a pier influences mainly ductility instead of strength. Other limitation of performance modifiers is that when applied reduce onerously building's strength.

6 Conclusions

Seismic vulnerability of Lisbon's downtown was the main focus of this work, and this can be divided into the two following points.

6.1 Seismic Assessment of a *Pombalino* Building

The software used allowed the application of nonlinear static analysis and the representation of the nonlinear behaviour of masonry without needing a high computational effort. However, a limitation of the program was the fact that does not allow to consider the out-of-plane behaviour of the masonry walls.

While modelling the building, one crucial issue was the definition of flexible pavements. A sensitivity analysis was made to understand the influence of shear modulus on the numerical problems obtained.

In order to consider the aggregate effect, a *pombalino* inserted in the block was modelled and compared with an isolated building model. It was observed a large difference in the flexural behaviour of the X direction, due to an increase in strength and stiffness. On the other hand, Y direction shows, as expected, similar results for both models.

Then, structural modifications were modelled and these mainly affect the X direction. It should be highlighted that interrupting a pier model leads to the weakest seismic behaviour. Finally, it is said that safety criterion is not satisfied for all analysed models.

6.2 Vulnerability Assessment of Lisbon's Downtown

The use of GIS tools has proved to be very helpful in storage and managing data, as well as visualizing and analysing results. These tools can be useful in supporting the definition of mitigation strategies and reducing seismic risk. Through the thematic maps presented it is concluded that Lisbon's downtown in current stage is quite heterogeneous, in opposition to the original conception.

Both simplified methods applied to the case study allowed to conclude that Lisbon's downtown buildings are extremely vulnerable due to the structural modifications subjected in the last decades. Nonetheless, these simplified methods should only be a first approach in the vulnerability assessment, defining the priorities to be defined. The most vulnerable buildings should then be re-assessed with more detailed procedures.

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