Damage to masonry buildings induced by ground displacements

Study of a Pombalino building located at Lisbon downtown

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Abstract
In Lisbon, the building stock includes a large amount of masonry buildings. This raises concerns due to the masonry’s poor tensile strength and also due to structural changes during rehabilitation works that may have reduce the overall strength of the building. The progressive increase in the exploitation of the subsoil, together with a consolidation of Lisbon downtown alluvial soil are factors that, over time, may generate ground movements. This work evaluates the damages induced by the ground movement and its effect on the reduction of the building’s overall earthquake resistance.

A literature review is presented on the evolution of the damage parameters and on the threshold values proposed by the different authors. Subsequently these methods are compared with the results obtained in the numerical simulations. The numerical simulation was performed using the 3MURI/TREMURI program that is able to take into account the non-linear behaviour of structural elements.

Two types of ground movements were imposed to the building that was studied in two situations: (i) isolated building and (ii) building in interaction with adjacent buildings (city block). Comparing these two situations, it was concluded that the block effect is beneficial, particularly for the backwards of the building, and that the deformation of the structure is similar to the predicted. It can also be observed that the threshold values proposed in the empirical methods underestimates the structural damage in the case study.

Finally, the capacity curves the isolated building for various ground movements are defined and compared. These results show that the resistance and ductility of the masonry building decreases as the imposed displacement increases.

Keywords: Pombalino buildings; Structural damage; Ground displacements; Settlements; 3MURI/TREMURI; Capacity curves.

1 Introduction
Before the earthquake of 1755, Lisbon was a city with narrow and irregular streets, which was located mainly along the Tagus River and the Castle of the city. After the 1755 earthquake it was necessary to rebuild the center of Lisbon. This reconstruction, led by Marquês de Pombal, produced a structured urban area and built constructions resistant to
seismic action. At this point, the Gaiola Pombalina was introduced (Meireles, 2012; Lopes, 2010).

With the development of Lisbon, the exploration of the underground became a viable way to face traffic congestion and parking needs. For example, the construction of tunnels and underground parking lots induced ground movements, capable of generating damages in buildings.

The main objectives of this work is to assess the damage of a representative Pombalino building when subjected to soil movements and evaluate the reduction of the respective seismic resistance. This work seeks to evaluate the effect of the insertion of the building in its block and in the isolated situation as well as the type of configuration of soil deformation.

2 Assessment of damage to buildings due to ground displacements

2.1 Classification of damage

In the last 60 years, several empirical methods were proposed to assess damage to buildings induced by ground movements.

Burland and Wroth (1974) concluded that the angular distortion parameter could not always be properly related to the observed damage. They introduced the concept of critical tensile strain ($\varepsilon_{\text{crit}}$), that is the average tensile extension for the beginning of cracking. They further suggested that the critical tensile strain value varies between 0.05% and 0.10% for masonry walls and structures and between 0.03% and 0.05% for reinforced concrete structures. Burland et al. (1977) replaced the concept of critical tensile strain by limit tensile strain ($\varepsilon_{\text{lim}}$), that depends on the limit state and type of structure.

Burland et al. (1977) also proposed a more detailed damage classification that includes a description of damage and the degree of cracking. This classification also refers to the correlation between the limit tensile strain and different categories of damage proposed by Boscardin and Cording (1989).

Due to the complexity of determining the critical tensile strain in real structures, Burland and Wroth (1974) suggested that an isotropic linear elastic beam can be considered to estimate the critical tensile strain.

2.2 Risk evaluation methodology of damages

Mair et al. (1996) proposed a staged analysis (preliminary, simplified and detailed analysis) for the assessment of damages in buildings to identify cases with category equal to 2 or less, that do not require significant repairs.

2.2.1 Preliminary analysis

The first stage of analysis assumes green field conditions. The damage risk is negligible if the maximum settlement is less than $10 \text{ mm}$ and the maximum rotation is less than $1/500$. If one of the criteria is not checked, it should proceed to a simplified analysis. This preliminary analysis is conservative since it only considers superficial movements and does not account for the stiffness of the structure.

2.2.2 Simplified analysis

It assumes that the building is represented by the equivalent beam model proposed by Burland and Wroth (1974), and greenfield is applied to the beam. The analysis is conservative, as it does not take into account the stiffness of the building. After the analysis, if the building presents a damage category equal to or greater than 3, it is necessary to proceed with the detailed evaluation.

2.2.3 Detailed assessment

It should give special attention to the excavation sequence in the case of tunnel construction, structural continuity, foundations, building
orientation and displacements that the building has experienced previously (Burland, 1995). In this step the ground-structure interaction is particularly important. This consideration reduces the amplitude of the displacement of the structure. If the damage category remains high it is necessary to establish preventive measures of protection (Burland, 1995).

3 Building studied

The building studied is located in Rua da Prata, 210-220. It is a Pombalino building built after the historic earthquake of 1755 and is located in the center of Lisbon.

3.1 Pombalino buildings

3.1.1 Foundation system

The soils of downtown Lisbon are of alluvial nature consisting of gross sedimentary deposits. Soils are not adequate for a shallow foundation system. Therefore, the building was founded in piers linked by masonry arches. Timber piles were headed in the piers by a wooden railing. The high density of piles produces an improved ground layer (Simões and Bento, 2012; Gomes, 2014).

3.1.2 Ground floor

The ground floor is entirely of stone masonry to prevent the spread of fire and moisture from soil and gives a great rigidity to the base of the building (Mascarenhas, 1996).

3.1.3 The Gaiola Pombalina: “frontal” walls and floors

The “frontal” walls are typically used for interior walls and are made up of a matrix of modules with horizontal, vertical and diagonal timber members. Subsequently, the modules are filled with masonry. In the event of a seismic situation the “frontal” walls are the main structural elements dissipative of energy (Simões and Bento, 2012; Catulo, 2015).

The floors are made of wooden beams with floor boards arranged perpendicularly to these (Simões and Bento, 2012).

3.1.4 External walls, stairs and roof

The exterior walls are built of rubble stone masonry with air lime mortar and have high thickness (Catulo, 2015).

On the ground floor the stairs are executed in stone masonry and the upper floors of wood. The stairwell is formed by “frontal” walls (Catulo, 2015).

The roofs are made of wood and have a triangular shape (Catulo, 2015).

3.2 Geological characterization of downtown Lisbon

The valley of downtown Lisbon is composed of Miocene formations, including Argilas e Calcários dos Prazeres, Argilas do Forno do Tijolo, Calcários de Entre-Campos and Areolas da Estefânia. The valley is filled with thick layers of alluvial deposits originating in Holocene. These deposits are mainly composed of fine sand with variable percentage of organic material, with varying levels of strength. The surface alluvium layer is composed of debris originated by the 1755 earthquake ruins (Meireles, 2012).

3.3 Structural model

The modelling of the building under study was developed by the program 3MURI/TREMURI (3Muri release 5.0.1). Its main advantage is to be able to model adequately the nonlinear behaviour of masonry, taken onto account the mixed behaviour of shear and bending of the structural elements (Magenes and Penna, 2009).

The modelling of masonry walls of the structure with openings is performed with vertical (piers) and horizontal (spandrels) macroelements connected by rigid nodes. Figure 1 shows the wall mesh.
generated by the program for the façade and backwards of the building under study.

![Figure 1: Automatic wall mesh for the isolated building: a) Façade; b) Backwards.](image)

The program presents inability to analyse spandrels with low axial stress. Normally the program assumes that these elements show damage, which does not happen in reality.

The final modelling of the building studied is shown in Figure 2 for the isolated situation. Besides the isolated situation, the building was modelled in interaction with its neighbours in order to simulate the block effect. The “frontal” walls were implemented based on the work of Meireles et al. (2011).

![Figure 2: Model isolated building: a) Ground floor; b) Full building.](image)

The mechanical properties of the materials used in the study are presented in Table 1 and Table 2 (NTC, 2008; Meireles, 2012).

4 Numerical simulation of damage to the building due to ground displacements

At this stage it is intended to make an assessment of the building in study due to imposed displacements. Two displacement configurations were analysed:

1. **Displacement configuration 1:** The façade and the backwards of the building suffer differential settlement, sidewall 1 presents a uniform settlement and sidewall 2 does not suffer any displacement (Figure 3 a) and b)).

2. **Displacement configuration 2:** the façade and sidewall 1 are subjected to differential settlement, and in turn, the backwards and sidewall 2 are not subjected to any displacement (Figure 3 c) and d)).

In order to compare the two kinds of deformation and the difference between considering the effect of a block or not, submitted patterns of damage will be to the angular distortion ($\beta$) values 1/500 (0.20%), 1/200 (0.50%) and 1/500 (0.67%) and for horizontal strain ($\varepsilon_h$) values 0%, 0.06% and 0.20%.

<table>
<thead>
<tr>
<th>Material</th>
<th>$E$ (GPa)</th>
<th>$G$ (GPa)</th>
<th>$f_c$ (MPa)</th>
<th>$f_{t,90}$ (MPa)</th>
<th>$\gamma$ (kN.m$^{-2}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rubble masonry</td>
<td>1.23</td>
<td>0.41</td>
<td>2.50</td>
<td>0.043</td>
<td>20</td>
</tr>
<tr>
<td>Stone masonry</td>
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<td>0.86*</td>
<td>7.00</td>
<td>0.105</td>
<td>22</td>
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<tr>
<td>Brick masonry</td>
<td>1.50*</td>
<td>0.50*</td>
<td>3.20</td>
<td>0.076</td>
<td>18</td>
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</table>

<table>
<thead>
<tr>
<th>Material</th>
<th>$E$ (GPa)</th>
<th>$G$ (GPa)</th>
<th>$f_c$ (MPa)</th>
<th>$E_{90}$ (MPa)</th>
<th>$f_{c,90}$ (MPa)</th>
<th>$\gamma$ (kg.m$^{-2}$)</th>
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<tbody>
<tr>
<td>Wood</td>
<td>12.00</td>
<td>0.75</td>
<td>18.00</td>
<td>0.40</td>
<td>6.90</td>
<td>580</td>
</tr>
</tbody>
</table>

Table 1: Properties of different masonry (*reduced by a factor of 2).

Table 2: Properties adopted for wood.
In Figure 4 are shown the cases analysed for the combination of different $\beta$ and $\varepsilon_h$ values in order to respond the different categories of damage according to Boscardin and Cording's (1989) proposal.

Knowing that the building’s length ($L$) is equal to 17.3 m and replacing the values of $\beta$ and $\varepsilon_h$ on equations (1) and (2), the values of displacement $\Delta s$ and $\Delta h$ to be applied to the structure are obtained.

$$
\beta = \frac{\Delta s}{L} \quad (1) \quad \varepsilon_h = \frac{\Delta h}{L} \quad (2)
$$

### 4.1 Results – Isolated building

The pattern of damage due to gravity action is presented in Figure 5. The spandrels are the only elements with damage.

Figure 5 shows the damage to the façade and backwards of the isolated building when subjected to displacement configurations 1 and 2.

#### 4.1.1 Displacement configuration 1

The spandrels are the most damaged due to the imposition of vertical displacements. With an angular distortion of $1/200$ all spandrels of the façade and backwards collapse by bending. With the imposition of horizontal displacements there is the appearance of shear yielding damage in the spandrels and bending yielding damage in the piers. The collapse of the largest number of structural elements begins for $\beta = 1/200$ and $\varepsilon_h = 0.2\%$.

#### 4.1.2 Displacement configuration 2

The façade has collapsed by bending and even by shear in the upper floors. For higher values of $\beta$ the piers presents bending yielding damage. For lower values of $\beta$ the introduction of horizontal...
displacements becomes beneficial in the upper spandrels of the façade. The backwards shows bending damage in the spandrels and piers for higher values of \( \beta \) compared to the displacement configuration 1 due to interaction with the interior structural elements.

4.2 Results – Building in city stock
The damage due to gravitational actions is identical to that of the isolated building situation (Figure 5).

4.2.1 Displacement configuration 1
Figure 7 shows the damage to the façade and backwards of the building when it is included in a city block. The façade damages with increasing \( \beta \) and \( \varepsilon_h \) is clear. The collapse begins to values of \( \beta \) equal to 1/200 and \( \varepsilon_h \) equal to 0.2\%. The backwards has collapsed by bending in the spandrels for low values of angular distortion and horizontal strain. The piers are quite conditioned by the existence of horizontal displacements, mainly emerging shear damage.

4.2.2 Displacement configuration 2
The façade has an increase of bending yielding damage in some piers of the higher floors compared to displacement configuration 1. The collapse occurs for angular distortion values close to 1/200, which is close to the angular distortion limit structural collapse (1/150). Regarding the building backwards, it does not present any damage due to the imposition of displacements in the structure.

4.2.3 Comparison of results
In the case of displacement configuration 1, the façade and backwards do not show a large difference in damage between the isolated building and the block effect.

Regarding the displacement configuration 2 the differences are high between the two models analysed. Concerning the façade, the isolated building situation influences the structural behaviour and early development of plastic damage. There is a great heterogeneity of damage to the façade of the building. On the contrary, considering the block effect, it translates into a more homogeneous distribution of damage and is less dependent on the angular distortion. The building backwards is the wall that shows a greater disparity in results between the two models. In the case of consideration of the block effect, the increase in displacement is totally indifferent to the structural behaviour of this wall. For the isolated building, the backwards presented increments of damage with increased horizontal strain and angular distortion.

With regards to the comparison of results and classification of damage suggested by Burland et al. (1977) and Boscardin and Cording (1989) their relationship is not straight. For \( \beta = 1/150 \) the building is in a high level of damage (category 4 or 5) according to Figure 4 and the results show that for both displacement configurations. However, for \( \beta = 1/200 \) the category of damage should be lower in accordance with Figure 4 and it is not verified with the analytical study. The results demonstrate that the building presents a degree of damage identical to \( \beta = 1/150 \). For \( \beta = 1/500 \) the elements collapse mainly due to flexure behaviour and can hardly enter an average degree of slight damage (category 3 and 2).
Figure 6: Damage pattern in isolated building for $\beta$ values equal to 1/500, 1/200 and 1/150 combined with $\varepsilon_h$ values to 0%, 0.006% e 0.2%. Scale factor equal to 10.
### Building in city stock

#### Façade

<table>
<thead>
<tr>
<th>Angular distortion, $\beta$</th>
<th>Displacement configuration 1</th>
<th>Displacement configuration 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/500</td>
<td><img src="image1" alt="Image" /></td>
<td><img src="image2" alt="Image" /></td>
</tr>
<tr>
<td>1/200</td>
<td><img src="image3" alt="Image" /></td>
<td><img src="image4" alt="Image" /></td>
</tr>
<tr>
<td>1/150</td>
<td><img src="image5" alt="Image" /></td>
<td><img src="image6" alt="Image" /></td>
</tr>
</tbody>
</table>

#### Backwards

<table>
<thead>
<tr>
<th>Angular distortion, $\beta$</th>
<th>Displacement configuration 1</th>
<th>Displacement configuration 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/500</td>
<td><img src="image7" alt="Image" /></td>
<td><img src="image8" alt="Image" /></td>
</tr>
<tr>
<td>1/200</td>
<td><img src="image9" alt="Image" /></td>
<td><img src="image10" alt="Image" /></td>
</tr>
<tr>
<td>1/150</td>
<td><img src="image11" alt="Image" /></td>
<td><img src="image12" alt="Image" /></td>
</tr>
</tbody>
</table>

**Legend:**
- Rigid node
- Shear yielding damage
- Shear collapse
- Bending yielding damage
- Bending collapse
- No damage
- Tension

**Figure 7:** Damage pattern in building in city stock for $\beta$ values equal to 1/500, 1/200, and 1/150 combined with $\varepsilon_h$ values to 0%, 0.006%, and 0.2%. Scale factor equal to 10.
4.3 Influence of damage induced by ground movement on the building capacity curve

In general, the initial deformation of the building due to differential settlements of the foundation soil is not taken into account in the seismic analysis. The main objective of this analysis is to compare the capacity curves after having applied deformation on the foundation corresponding to different values of angular distortion. These capacity curves will be obtained through a pushover analysis in the x direction with uniform horizontal loads distributed in height and the control node located on the top floor near the centre of gravity.

Figure 8 shows the results of the capacity curves of the isolated building under the action of displacement configuration 1 (Figure 3 a)).

As is possible to visualize in Figure 8, from an initial deformation \( \beta = 1/400 \) the structural behaviour is different, the ultimate shear force is much lower and so is the deformability capacity. The structure is far less ductile compared to \( \beta \) equal to 0 and 1/500. For values close to \( \beta = 1/200 \) the structure cannot resist a high intensity seismic action, as the ultimate shear force and the deformability capacity are very low.

5 Conclusions

This paper presents a brief review of empirical methods for assessing damage to the structure due to ground movements. Thereafter, a Pombalino masonry building built after the 1755 earthquake was analysed when subjected to imposed displacement at its foundation. The influence of vertical and horizontal movements of the ground in the distribution of damage in the building were evaluated. Subsequently the capacity curves of the structure with damage due to movements in the base, were evaluated; for this, horizontal loads, reproducing the inertial forces that are generated when an earthquake occurs, were applied at the level of the floors.

The Pombalino buildings are characterized by stiff and heavy exterior walls. The wooden floors, with flexible behaviour in the plane, do not allow for the behaviour of the building as a whole. Moreover, the horizontal forces that are generated in a seismic situation are distributed by vertical elements according to their area of influence on the floor and not due to its stiffness. Hence, one can have interior walls subjected to forces greater than desired (exceeding their load capacity) and the outer walls with lower forces/tension than their resistant values.

The isolated building and the building in city stock were considered in the evaluation of the building due to the imposed displacements in the base.

Relating the angular distortion values recommended by different authors and Eurocode 7 and that obtained for the building studied, it is concluded that the damage structures begin at angular distortion values inferior to 1/150.

As shown in the capacity curves, it is clearly important to consider the effect of the initial deformation of a building more accurately by taking into consideration the foundation soil.
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