Seismic Assessment of a Typical Building from Azores

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Abstract

Traditional Azorean buildings are mostly made of rubble stone masonry walls. As the masonry walls are vulnerable to seismic action, it is of utmost importance: (i) the characterization of their mechanical properties and (ii) the study of the behaviour of these constructions to this type of action. On the other hand, it is important to remember that urban buildings are significantly affected by the behaviour of the city block, thus, the adjacent buildings should be considered in numerical models to properly evaluate the seismic performance of the building under consideration.

This paper aims to contribute to the abovementioned (ii), studying the seismic behaviour of a building in the city of Angra do Heroísmo and taking into account the effect of the adjacent buildings. This study begins by characterizing the existing buildings in this city, thus, for the building selected, its seismic performance is evaluated through a tridimensional model, using the non-linear static analysis and resorting to the 3MURI/TREMURI program.

Two models were compared, concluding that the surrounding buildings affect the seismic behaviour, leading to a more ductile structure in one of the main directions and increasing the damage of the main building.

The safety verification to the limit state following the procedure proposed in EC8 and considering a multiscale approach with different verification criteria and was made. The safety was not verified for both models.

Finally, a strengthening solution to reduce seismic vulnerability of the building under consideration is proposed, based on the damage patterns obtained.

Keywords Traditional Masonry; Buildings from Azores; Seismic vulnerability; Pushover analysis

1 Introduction

The Azorean archipelago was generated by volcanic activity and is situated over the Mid Atlantic Ridge between the connections of three tectonic plates: Eurasian, North-American and African.

The archipelago seismicity is caused by volcanic or tectonic activity, being the rate of occurrence of earthquakes in the region higher than in the Portuguese mainland (Carvalho et al., 2001).

According to the database of seismicity in the region (which covers a period of 550 years) there
were 34 destructive earthquakes with intensity equal or superior to level VII (IMM) (Nunes et al., 2001).

In 1980, an earthquake of magnitude 7.2 $M_L$ caused severe damage in the buildings edified on Terceira, São Jorge and Graciosa islands, being Terceira Island the most affected (Correia Guedes & Oliveira, 1992). The most recent, high intensity earthquake, in 1998, of magnitude 5.9 $M_L$, affected the buildings on Faial and Pico Island (Senos et al., 2008).

The traditional Azorean buildings, mostly made of rubble stone masonry walls, displayed a poor behaviour when subjected to the seismic action. The resistance of these constructions is mainly limited by the mechanical properties of the elements and its connections (Correia Guedes & Oliveira, 1992). Therefore, the seismic activity in Azores is of great importance in study of buildings and the reduction of its vulnerability to this type of action. Thus, the aim of this study is to evaluate the seismic performance of an existing traditional building in the city of Angra do Heroísmo.

A characterization of the buildings in the city is made, from which a specific building will be chosen in order to study its seismic behaviour, by modelling it with the 3MURI/TREMURI program by running a static non-linear analysis that takes into account the influence of the adjacent buildings in the model and then calibrating it.

2 Angra do Heroísmo Buildings - Characterization

The evolution of the buildings over time could be divided in two major groups: traditional construction and current construction. According to Oliveira (1992), 1950 is the year that separates the traditional construction from the current construction, marking the beginning of the use of reinforced concrete. Despite this, the buildings in the city continue to be mainly traditional constructions.

2.1 Traditional Construction

In the city, the buildings are disposed in line or in block, and have around 2 to 3 floors. Three types of typologies can be identified: narrow facade (6 metres long), wide facade (12 metres long) and noble houses, with considerable dimensions (Correia Guedes & Oliveira, 1992).

The heights of the buildings vary greatly, due to the difference in the ceiling heights, number of floors and the inclination of the ground.

2.1.1 Walls

The exterior walls are composed by rubble stone masonry and are the main supporting element of the floors. Their thickness varies from 60 to 70 cm and three types of construction can be distinguished: (i) single leaf masonry with regular stone; (ii) irregular stone masonry, filled with materials of thinner granulometry; (iii) double leaf masonry, with stones slightly bigger than half of the thickness of the wall.

The corners, where the connections between orthogonal walls are made, are essential to the resistance of the structure to the seismic action. These areas are built with regular stones aligned alternately in the two directions of the orthogonal walls.

The interior walls with support function are made of stone masonry. However, the interior walls that lack this function are made of aggregate wood elements, being the most common ones known as tabique walls, and having a very low resistance to seismic action. (Correia Guedes & Oliveira, 1992)
2.1.2 Foundations

The foundations that support exterior walls are an extension of the masonry walls inside the ground with a depth of 30-40 cm.

In the corners, foundations are composed of higher quality masonry and have a bigger area section that goes deeper into the ground. (Correia Guedes & Oliveira, 1992)

2.1.3 Floors

Wooden beams that support wooden boards (0.4 metres length and 2.2 to 2.5 metres thickness) compose the floor structure. These beams, disposed with a distance of 0.50 to 2.0 metres, are orthogonal to the facades and separate spans of 3.5 to 5.5 metres in the lower span direction. The most common wood for the pavement structure is soft pine. (Correia Guedes & Oliveira, 1992)

2.1.4 Roofs

The roofs usually have two slopes, and are composed by a wooden truss structure and covered by regional clay tile.

The wooden trusses can have different shapes, and are connected to the facade walls. (Correia Guedes & Oliveira, 1992)

2.1.5 Response to the 1980 earthquake

The damage distribution in the city was not homogeneous, varying with the geology of the ground, the topography and characteristics of the buildings (Teves-Costa et al., 2004).

As mentioned before, the most affected buildings were traditional ones, having higher levels of damage in the exterior walls (from cracks to collapse) and roof structures (dismantling of the roof-wall connection, collapse of the structure).

2.1.6 Building modifications and new constructions

The current constructions are characterized by the use of reinforced concrete structures, masonry walls of cement bricks, and floors and roof slabs of reinforced concrete.

After the 1980 earthquake, several measures were implemented to reinforce the damaged structures. These measures consisted on the use of reinforced concrete elements, restraining the slab levels and the structure. Slabs in worse conditions were replaced by concrete slabs (Borges & Ravara, 1980).

3 Case study

Since the surrounding buildings have a significant influence on the behaviour when subjected to the seismic action, the case study was chosen by picking a building in the middle of a typical block that was higher than the surrounding buildings, as this situation is more demanding. In Figure 1 the studied building can be identified as being the one at the centre.

Figure 1 Front facade of the main building (centre) and its surroundings

The characterisation of the building, mechanical properties, structural elements, etc. was defined through an in situ analysis and by consulting the existing literature on such buildings. Additional information can be found in (Fagundes, 2015).

The building contains three floors, and was not subjected to intrusive modifications on the structure, preserving its original characteristics.
The exterior walls are made of rubble stone masonry, with a thickness of 60 centimetres. On the first floor, the only interior walls made of masonry delimit a room, not supporting the above pavements. The remaining interior walls are made of wood, tabique, and have a thickness of 10 cm.

The pavement structure is supported by massive wood beams (30x20 cm) and circular cast iron columns (with a diameter $D_{\text{max}}=22$ cm), dividing the spans of the building in 5, 4 and 4.5 m.

The pavement structure itself is composed by smaller beams (30x15 cm), spaced by 130 cm, and 2.5 cm wooden boards.

Simple wooden trusses spaced by 185 cm compose the roof structure.

The building’s use varies with each floor: it has a commercial area (first floor), an office and storage (second floor); and a residential area (third floor).

The surrounding buildings have concrete reinforcements and concrete slabs. The correct modelling of these elements is very important since the position of these slabs will interfere with the seismic behaviour of the main building.

4 Structural Modelling

There are different analysis that can be made for the seismic capacity study of the building: linear or non-linear, static or dynamic (CEN, 2004a).

The non-linear analyses are generally considered the more adequate to evaluate the seismic behaviour of existing masonry structures (Lourenço, 2002). According to the Eurocode 8, part 3 (CEN, 2004b) the correct analysis for this type of masonry building is the non-linear static analysis.

3MURI/TREMURI (S.T.A. Data, 2005)/(Lagomarsino et al., 2002) was the software used to model and analyse the building in study. This program was developed specifically to study masonry structures under seismic actions, by running non-linear static analyses, while considering the in plane behaviour of the walls and their shear and flexural reactions. The program uses a macro-element method called Frame by Macro Elements (FME).

Two models were built in this study: one representing the isolated building, with all the abovementioned characteristics, and another with the surrounding buildings in aggregate. This difference permits the comparative analysis of the interaction between the building and its surroundings.

Masonry walls have a wide range of mechanical properties depending on the materials used, which makes its characterization harder.

The experimental analyses results from LREC for typical Azorean walls (Medeiros, 2011) were used in order to correctly represent the mechanical characteristics in the model.

Table 1 resumes all the material’s used and its properties.

<table>
<thead>
<tr>
<th>Material properties</th>
<th>Masonry</th>
<th>Wood C18</th>
<th>Steel S235</th>
<th>Concrete C16/20</th>
</tr>
</thead>
<tbody>
<tr>
<td>$w$ [kN/m$^2$]</td>
<td>19</td>
<td>4</td>
<td>79</td>
<td>25</td>
</tr>
<tr>
<td>$E$ [GPa]</td>
<td>0.741</td>
<td>9</td>
<td>210</td>
<td>29</td>
</tr>
<tr>
<td>$G$ [GPa]</td>
<td>0.222</td>
<td>0.56</td>
<td>80.769</td>
<td>12.083</td>
</tr>
<tr>
<td>$\nu$</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>$f_m$ [MPa]</td>
<td>118.52</td>
<td>19.3</td>
<td>171.1</td>
<td>32.4</td>
</tr>
<tr>
<td>$t$ [MPa]</td>
<td>3.21</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

4.1 Numerical modelling and analysis

4.1.1 Masonry walls

In 3MURI, the wall characteristics are first defined, and only then the existing openings inserted.
The process of modelling the openings is very important, as it will define the geometry of the macro-element mesh in the computational discretization of the FME method.

The FME method admits that the behaviour of the masonry wall can be described as a set of three distinct elements (macro-elements): (i) Pier, vertical element that supports the gravity and seismic load; (ii) Spandrel, horizontal element defined between two vertically aligned openings; (iii) Rigid node, element of non-damaged masonry, confined between piers and spandrels.

This modelling is based on the damage patterns observed on masonry walls subjected to an earthquake, where the cracks are located in these specific elements (Lagomarsino et al., 2013).

The piers and spandrels have a non-linear behaviour and are modelled as a 2D element, defined between two end nodes. The static and kinematic variables are transferred between nodes and other elements, by the rigid nodes (Lagomarsino et al., 2013).

In 3MURI/TREMURI the non-linear behaviour, involving shear and bending, is activated when a generalized force reaches the maximum value of the respective strength criteria of each in plane collapse mechanism: flexural-rocking, shear-sliding, diagonal-cracking shear (Lagomarsino et al., 2008).

4.1.2 Floors

The flexible floors, composed by wooden structures, are modelled by the program as an orthotropic membrane finite element defined by 3 to 4 nodes with plane stress and two displacement degrees of freedom in each node. The Young modulus represents the stiffness of the membrane in the two directions and accounts for the degree of connections between this element and the walls. The shear modulus influences the horizontal force that is transferred from the floor to the walls (Lagomarsino et al., 2013).

4.2 Mesh edition

3MURI does not permit to differentiate the heights of walls in one same level. However, since this variation is very important in the definition of the damage pattern of the walls, in particular the facade walls, the modelling of the macro-elements with the correct geometry and position is of great importance. Consequently, an alternative method was adopted in order to correctly define the walls in their position. This method consists on the definition of every baseline necessary to define the top and bottom of a wall.

With this distinct approach, the automatically generated mesh required further edition. The mesh automatically generated by the program and the final edited mesh ready for the analysis process of the facade wall are represented in the Figure 2.

![Figure 2 a) Automatic wall mesh, b) edited wall mesh](image)

To calibrate the numerical model, some dynamic in situ vibration tests were used as reference. These experimental dynamic values were taken from buildings in Azores, with similar characteristics and obtained from (Neves et al., 2004) and (Oliveira & Navarro, 2010).
5 Seismic Assessment

The seismic action was defined regarding the National Norm NP EN 1998-1 (CEN, 2010).

More comparisons have been made with this assessment, presented on (Fagundes, 2015).

5.1 Non-linear pushover analysis

A non-linear static analysis or pushover analysis, begins with the definition of the capacity curve, imposing a lateral horizontal distributed load and obtaining the displacements reached while loading in order to the base shear force.

In this work, the analyses were performed for each main direction of the building and considering two load cases: (i) pseudo-triangular, proportional to the product between the mass and height and (ii) uniform, proportional to the mass.

The ultimate displacement was defined by three different criteria: (i) development of a collapse mechanism, (ii) reduction of 80% of the maximum base shear force, as proposed in EC8, (iii) maximum value of inter storey drift (Simões et al., 2014).

According to the Norm NP EN 1998-3 (CEN, n.d.), existing masonry buildings, must be evaluated to the Limit State of Significant Damage.

5.1.1 Results – isolated building

Comparing the capacity curves obtained in Figure 3, it can be concluded that the uniform load has a higher value of the base shear force for every case, meaning that the pseudo-triangular load is the most demanding load distribution case.

Being the X direction parallel to the facade walls and the Y direction perpendicular to the X direction, the behaviour on the Y direction is more rigid and resistant than in the X direction, which presents a more ductile behaviour. This behaviour is due to the gable walls that resist to the action in the Y direction that have no openings; on the other hand, the facade walls that have openings have a higher capacity to redistribute stresses.

In this case the ultimate displacement is conditioned by the criterion (iii), being presented on Figure 3 as a red cross. Despite the criterion
(iii), this building also has a collapse mechanism that can be shown on Figure 3 by the final decrease on the pushover curve, defining an ultimate displacement by the criterion (ii), presented as a red circle.

5.1.2 Results – building in aggregate

From the results depicted in Figure 4, it is clear that the pseudo-triangular load is the most demanding load distribution, having always a lower value of the base shear force than the uniform load.

The behaviour on the X direction for the building in aggregate is more ductile than the behaviour of the isolated building. This is due to the addition of the facade of the adjacent buildings that increase the ductility on the X direction. The ultimate displacement is also conditioned by the criterion (iii) on both directions (red cross). However, the collapse mechanism in X direction occurs for high values of displacement, which is not adequate to consider, and the curve does not suffer a decrease on the maximum shear force, being constant till the building generates the mechanism. The ultimate displacement for criterion (iii) is presented as a red circle.

5.2 N2 Method and safety verification

The evaluation of the seismic performance is developed by the definition of the target displacement by means of the N2 method, proposed by Fajfar (1988) as described in the Norm EN 1998-1 (CEN, 2004a).

The performance point, defined by the intersection of the seismic response spectrum and the capacity curve, will define the target displacement that the building reaches, when subjected to the defined seismic action (Bento & Rodrigues, 2004). After the transformation of the multiple degrees of freedom system and the bi linearization of the capacity curve, the performance point was obtained. The safety verification is made by comparing the obtained displacements. Thus, the verification
criterion is based on the ratio between the ultimate and target displacements: \( \frac{d_u^*}{d_t^*} > 1 \) (Simões et al., 2015).

The seismic performance was assessed for the two models in study, the isolated building and the building in aggregate, for the criterion (iii) and the pseudo-triangular load case, for being the most conditioning.

On Table 2 the values obtained in the N2 method for the seismic performance are presented.

<table>
<thead>
<tr>
<th></th>
<th>Isolated</th>
<th>Aggregate</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>X</td>
<td>Y</td>
</tr>
<tr>
<td>( T^* ) [s]</td>
<td>1.48</td>
<td>0.76</td>
</tr>
<tr>
<td>( F_y^<em>/m^</em> ) [m/s²]</td>
<td>0.38</td>
<td>0.33</td>
</tr>
<tr>
<td>( \mu^* )</td>
<td>1.51</td>
<td>1.21</td>
</tr>
<tr>
<td>( d_u^* ) [m]</td>
<td>0.032</td>
<td>0.006</td>
</tr>
<tr>
<td>( d_t^* ) [m]</td>
<td>0.052</td>
<td>0.027</td>
</tr>
<tr>
<td></td>
<td>0.063</td>
<td>0.031</td>
</tr>
</tbody>
</table>

The results of this verification are depicted on Figure 5.

Both models do not verify the safety, on the X or Y direction, according to the criterion (iii). Besides this, from the analysis of Figure 5 it can be concluded that the building in aggregate has the worst behaviour for the studied seismic action. This result is justified by the influence of the adjacent buildings with the different floor heights.

5.3 Strengthening of the structure

The damage pattern of the target displacement was studied for the model of the building in aggregate, represented in Figure 6 for the X direction and in Figure 7 for the Y direction.

![Figure 6](image)

**Figure 6 Damage pattern of the target displacement - X direction. a) Front facade, b) back facade**

![Figure 7](image)

**Figure 7 Damage pattern of the target displacement - Y direction. a) left gable wall, b) right gable wall, c) back facade**

With the analysis of these figures the interaction on the front facade of the adjacent buildings is verified, as referred by Cattari et al. (2012), and the torsional effect induced by the gable walls on the back facade is observed.

The insertion of vertical resistant elements to reduce the torsional effect by their plan positioning, the change of flexible to rigid floors and, eventually, the strengthening of the weak masonry of some walls (Vicente et al., 2008), are proposed. These measures must be modelled and evaluated again, to verify their influence on the seismic behaviour of the building.

6 Conclusions

The Azorean buildings dating from before 1950 are structurally composed of rubble stone masonry walls and flexible wooden floors.
In the present work, the seismic behaviour of a typical Azorean building was analysed. The results of the building’s seismic performance were obtained from a non-linear analysis using the 3MURI/TREMURI software.

Two different models were developed to compare the influence of the surrounding buildings over the studied building’s seismic performance.

The ultimate displacement was defined using multiple criteria. The seismic assessment was done for the most conditioning ultimate displacement (drift limitation criterion) and load case (pseudo-triangular).

The results of the seismic performance-based assessment, given by the safety verification of the Significant Damage Limit State, led to the conclusion that both models collapse with the defined seismic action. However, the building in aggregate presented a behaviour worse than the isolated one, due to the influence of the adjacent buildings with the floors at different levels.

Analysing the damage pattern for the target displacement, the insertion of vertical resistant elements to reduce torsional effects, the change of the flexible to rigid floor and, eventually, the strengthening of the masonry walls of some walls, were proposed.

References


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Japão.


