On the seismic response of buildings in aggregate: analysis of a typical masonry building from Azores
Camila Fagundes; Rita Bento; Serena Cattari

Abstract
In 1980, an earthquake of magnitude 7.2 ML caused severe damage to the existing buildings in some islands of the Azores Archipelago, Terceira being the most affected one. The traditional Azorean buildings are mostly made of rubble stone masonry walls and are often inserted in aggregates (or blocks), including adjacent buildings with different height or/and different floor levels. This paper presents the results from the seismic performance-based assessment of a case study of such masonry building stock. The assessment was addressed to the global seismic response of the building, assumed to be governed by the in-plane capacity of the masonry walls, and taking into account the interaction with the adjacent buildings, shorter and with different floors heights. The global behavior was determined by non-linear static analyses and the effects of some relevant modeling options were considered, including the analysis of the building either as an isolated structure and inserted in its aggregate. This study has shown the relevance of modeling the structure with the adjacent buildings' floors at their actual height to assess the real expected damage.

Keywords Traditional Masonry; Buildings from Azores; buildings in aggregate; Seismic assessment; 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. Consequently, the archipelago seismicity is caused by volcanic or tectonic activity. 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) [1]. In 1980,
an earthquake of magnitude 7.2 M\textsubscript{L} caused severe damage in the existing buildings on Terceira, São Jorge and Graciosa islands, being Terceira the most affected island with 11 899 damaged buildings [2]. An unusual Mercalli Modified Intensity (MMI) distribution throughout Terceira was observed, with values varying between V and IX [3]. In Figure 1 the epicentre of this earthquake is shown.

![Epicentre of 1980 earthquake. MAR – Mid-Atlantic Ridge. Adapted from Nunes [4]](image)

The traditional Azorean buildings, mostly made of rubble stone masonry walls, are vulnerable to the seismic action being their resistance mainly limited by the mechanical properties of the structural elements and the quality of the connections [3]. The existing buildings in Angra do Heroísmo, the city in Terceira Island, are mainly traditional constructions built before 1950 (Figure 2). It is clear that the most vulnerable buildings are the oldest ones concentrated in the historical city center, a prone seismic area due to both source characteristics and site conditions [5]. The typical buildings herein located are often inserted in aggregates (or blocks) in which each structural “unit” is interacting with the adjacent one. Often, there are adjacent buildings with different height or/and different floor levels. In fact, frequently adjacent buildings do not have the floors at the same level and this can be due to the different height of each floor or to the ground’s slope.
These old traditional buildings are characterized by exterior walls composed by rubble stone masonry, that are the main supporting elements of the flexible timber floors, which are themselves composed by wooden beams and wooden boards; the interior walls are made of aggregate wood elements, being the most common ones known as *tabique* walls and having a very low resistance to seismic action.

As aforementioned, the damage due to the 1980 earthquake was not distributed homogenously through the city (Figure 3), varying with site effects (both the superficial geology and the topography), earthquake source characteristics and structural vulnerability of the buildings [6]. In the city of *Angra do Heroísmo* more than 3000 buildings were damaged [2], corresponding to about 75% of the existing buildings [7], and consequently 15 878 people were dislodged [2].
Nevertheless, relevant damages have been surveyed in stone-masonry buildings in some areas of the *Angra do Heroísmo* city. In the damaged blocks, the first and the last building were severe damaged by local collapses and large cracks (Figure 4). In the internal part of the damaged blocks different type of damage was observed: decayed floors and roofs collapse or with large continuous deformations, out of plane collapse of the walls (Figure 5) and diagonal shear cracking mainly in piers (Figure 6).

![Figure 4 Severe damage of corner buildings – 1980 Azores Earthquake [9]](image-url)
Moreover, in-plane damage of walls consequent to the pounding of two adjacent buildings with different height and different floors levels was seen (Figure 7).
This study aims to evaluate the seismic performance of an existing representative masonry traditional building in the city of Angra do Heroísmo, as preliminary step for supporting mitigation policies of their seismic vulnerability. Although, the occurrence of local mechanisms associated to the out-of-plane response of walls is recognized as typical of buildings in aggregate, the attention is herein focused only on the in-plane mechanisms, that have been proved equally remarkable and are still less examined in literature at least in such type of configurations ([10], [11], [12], [13]), by focusing the attention to the pounding effect. Accordingly, the assessment of a building, inserted in the middle of a group of buildings, was carried out considering its global response in two different approaches: isolated building and building in aggregate, taking into account the influence of the adjacent buildings.

Thus, a case study was chosen and the seismic global behaviour of this building (the isolated one and block-inserted) was assessed with a nonlinear static analysis and adopting, as modelling approach, the equivalent frame one by using the Tremuri program (the commercial release to generate the model [10] and then the scientific version [11]). The nonlinear static (pushover) analysis was performed to obtain capacity curves of the structures under examination followed by the application of the N2 method [12] to assess their seismic performance. The safety was checked according to the performance limit states and based on the criteria recommended by the Eurocode 8 [13] and on the multi scale approach proposed in [14].

2 Case study

The building studied in this work is located in an area of the city (identified in Figure 2 trough a blue dot) where no severe damage or collapse occurred (see Figure 3). The tallest building from three adjacent buildings in the middle of a typical block of masonry buildings was chosen. In Figure 8 it is possible to observe the difference of height between buildings and the difference of the pavement elevation as highlighted by the windows position. The adjacent buildings have suffered some structural modifications, in particular the traditional timber floors were replaced by reinforced concrete (RC) slabs. Thus, the study of such typical as-built old masonry Azorean building, inserted in a traditional block, was considered quite interesting as potentially prone to
pounding effects. Therefore, more damage is expected than the one it suffered in the 1980 Azorean earthquake, even if the same seismic intensity was adopted as at that time all the buildings had timber floors.

![Figure 8](image)

*Figure 8* Front facade of the main building (centre) and its surroundings (on the left, original drawing and on the right an actual photo)

The characterisation of the building, geometrical and mechanical properties and the definition of structural elements, was defined through an *in situ* analysis and by consulting the available literature on such buildings. The building contains three floors, with the following ceiling heights: 3.85 m, 3.95 m and 3.95 m. This building 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 0.60 m. On the first floor, the only interior walls made of stone masonry (equal to the exterior walls) delimitate a room, not supporting the upper pavements. The remaining interior walls, *tabique*, are frame composed by vertical timber boards and horizontal laths, and have a thickness of about 0.10 m.

The timber pavement’s structure is supported by massive wood beams, with a height of 0.30 m per 0.20 m width, and circular cast iron columns (with a maximum diameter of 0.22 m), dividing the spans of the building in 5, 4 and 4.5 m. The floors’ structure itself is composed by a set of smaller beams (with a height of 0.30 m per 0.15 m width), perpendicular to the façade walls and spaced by 1.30 m. The beams are covered by timber boards, with 2.5 cm thickness, as it can be seen in Figure 9 a).
The roof has two slopes and is composed by a frame timber structure (Figure 9 b) made of several wooden trusses spaced by 1.85 m, covered by regional clay roof tiles.

The building’s use varies with each floor namely a commercial area at first floor, an office and storage space at the second floor and a residential area at the third one.

After the 1980 earthquake, several measures as the insertion of reinforced concrete elements and, in several cases, the replacement of deteriorated timber floors by RC slabs [19], were implemented in the damaged structures. This structural modification was made to the adjacent buildings in study, too.

Figure 10 a) presents the plan view of the three buildings, being the building in study the one in the centre, where the asymmetry of the global plan and the difference in the geometry of each building can be observed. In Figure 10 b) the geometrical characteristics and the location of the interior vertical elements are identified. Additional information related to the characterization of the building can be found in [20].
3 Structural Modelling

In order to assess the seismic global response, a three-dimensional model of the isolated building and of the building together with the two adjacent buildings was developed in the 3Muri program [10,11]. It is based on the equivalent frame approach [21], assuming that the in-plane behaviour of the masonry wall with openings can be described as a set of three distinct elements: (i) Pier, a vertical element that supports the gravity and seismic load; (ii) Spandrel, a horizontal element defined between two vertically aligned openings; (iii) Rigid node, the masonry portion at the intersection between piers and spandrels. The nonlinearity is concentrated only in piers and spandrels, being the nodes assumed as rigid: this simplified assumption is testified for instance by Figure 6, in buildings of Angra do Heroísmo, due to the 1980 earthquake, where a prevailing shear damage in such elements is shown. The nonlinear
The behaviour of the piers and spandrels has been herein described by adopting a simplified formulation based on non-linear beam elements [21].

The characterization of the mechanical properties of masonry walls is a difficult task mainly due to the fact that the number of experimental campaigns specifically addressed to this type of buildings is very limited. In this study, the experimental data adopted result from LREC for typical Azorean walls [22]; they have been then divided by a confidence factor of 1.35 (“Limited Knowledge”), as proposed in Eurocode 8 [23]. The latter choice is justified by the fact that the quality on-site inspections was limited as it was not possible to observe the construction details on the buildings, neither to perform any type of destructive or semi-destructive tests.

Table 1 summarizes the values of the masonry mechanical properties adopted, together with the properties of the iron columns and the gravity and live loads considered.

<table>
<thead>
<tr>
<th>Material</th>
<th>Specific weight</th>
<th>Young modulus E</th>
<th>Shear modulus G</th>
<th>Compressive strength f_m</th>
<th>Shear strength τ_0</th>
<th>Gravity loads (Variable loads)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rubble Stone Masonry</td>
<td>19</td>
<td>0.741</td>
<td>0.222</td>
<td>1.185</td>
<td>0.032</td>
<td>2nd Floor 0.72 (3 and 7.5) 3rd Floor 0.72 (2)</td>
</tr>
<tr>
<td>Iron Columns</td>
<td>79</td>
<td>210</td>
<td>80.769</td>
<td>1.171</td>
<td>-</td>
<td>Roof 1.7 Balcony 1.97 (2.5)</td>
</tr>
</tbody>
</table>

The floors were modelled with orthotropic membrane finite elements, characterized by an equivalent thickness and by modulus of elasticity $E_{1,eq}$ in the floor warping direction, $E_{2,eq}$ in the orthogonal direction and an equivalent shear modulus $G_{eq}$. In this study, the timber floors were defined as an equivalent membrane with a quite flexible behavior with 0.025 m thickness, as suggested in [24], and characterized by $E_{1,eq} = 0.3$ GPa, $E_{2,eq} = 9$ GPa and $G_{eq} = 0.56$ GPa (structural timber with C18 strength class [25]).

The two models, one representing the isolated building and the other with the surrounding buildings in aggregate, are depicted in Figure 11. Only by comparing the two models is it possible to show the importance of considering the adjacent buildings. In fact, the structural
damage in piers due to the impact of floors at different elevations of adjacent buildings (Figure 7) is impossible to be taken into account in the isolated model.

Moreover, it should be noted that the model with the buildings in aggregate suffered some differences in the modelling process of the gable walls. In fact, when only the isolated building is modelled, the gable walls are defined as one wall element per floor (see Figure 10 a) – no perpendicular walls exist in the interior). Nevertheless, since these walls of the main building are intercepted perpendicularly by the walls of the adjacent buildings, the gable wall should be divided on its length in three parts, instead of one wall element per floor.

![Figure 11 a) Isolated building model, b) buildings in aggregate model](image)

### 4 Seismic Assessment

The seismic evaluation of the global behaviour was developed based on a pushover analysis and followed by the N2 method, originally proposed by Fajfar [16] and adopted also in Eurocode 8 [23][26]. Thus, the seismic performance-based assessment consisted in determining the performance point, from the intersection of the structure capacity curve with the seismic demand, and the verification of the fulfilment of the performance level (or limit state) requirement according to the code. Based on the National Annex of Part 3 of Eurocode 8 [26], existing buildings must be seismic assessed considering the Significant Damage limit state (SD), the one herein considered to verify the seismic safety. This SD limit state is directly defined on the capacity curve on basis of the conventional limit displacement equal to $\frac{3}{4} d_u$, where $d_u$ is the ultimate displacement corresponding to the 20% decay of the maximum base shear [13].
In buildings with rigid floors in plane, the definition of this limit state is straightforward as a significant number of walls reach the same state almost at the same time. Nevertheless, in structures with flexible floors, such as the case study (timber floors) under examination, the limited load transfer provided by the pavements to the walls leads to a more independent behaviour of the walls and the real damage imposed in a wall may not be evident by the analysis of the building capacity curve. Therefore, for the evaluation of the seismic performance of the structure, the heterogeneous damage distribution and the possible concentration of damage in certain walls should be taken into account [27].

Consequently, in this work, the limit states were defined based on [13] and integrating the multiscale approach proposed in [14], which aims to monitor through proper variables (as the interstory drift) the possible concentration of damage in parts of the building (such as single walls), that may not relate with the significant strength decay of the base shear force on the global capacity curve.

4.1 Non-linear static analysis

In this work, the non-linear static (pushover) analyses were performed for each main direction of the building considering two load patterns: (i) pseudo-triangular, proportional to the product between the mass and height and (ii) uniform, proportional to the mass. To perform the pushover analyses, the control node between the left gable wall and the back façade was chosen. This node influences both main directions of the building and allows a better convergence, in a nonlinear phase, than other ones. In particular, the node chosen allows to follow the mechanism generated in the back façade that is the dominant in determining the seismic response and the attainment of limit states in the building. Despite such choice on the control node, the displacement plotted in the capacity curves is the average displacement on the top of the building, considered more representative of the whole response of the building than a single node (in particular in this case where diaphragms are flexible).

The ultimate displacement was defined as the minimum value of the displacements defined by three different criteria: (i) reduction of 80% of the maximum base shear force, as proposed in Eurocode 8 [17], (ii) development of the collapse of a certain group of elements (e.g. all the piers in one floor), (iii) attainment of the maximum inter storey drift [14], [27]. It is worth to
highlight that in this study the inter storey drift was determined from the contribution of both horizontal displacement and rotation components being in presence of flexible floors and weak spandrels that provide a moderate coupling effects among piers and guarantee a moderate restrain to rotation [14].

4.1.1 Capacity curves and limit state definition

The capacity curves obtained from the pushover analyses are presented in the following figures for each model. In particular, Figure 12 shows those obtained for the isolated building.

![Capacity curves for the isolated building](image)

**Figure 12** Capacity curves for the isolated building

The two load distributions describe the envelope of the building’s behaviour. It can be concluded that the uniform load has a higher value of the base shear force, meaning that the pseudo-triangular load is the most demanding load distribution case for this model.

Being the X direction parallel to the façade wall and the Y direction parallel to the gable walls, the behaviour on the Y direction is more rigid and resistant than in the X direction. This is due to the gable walls that resist to the action in the Y direction without openings. Regarding the ductility behaviour, one can observe that the behaviour in Y is clearly less ductile than the behaviour in the X direction. The ultimate displacement is identified on the capacity curves in Figure 12 as a black dot and it characterises the development of a collapse mechanism on the third and second floor as shown in Figure 14. The wall identification is provided in Figure 13.
Thus for the isolated building the ultimate displacement is conditioning by criterion (ii) defined in 4.1.

In Figure 15 the capacity curves for the aggregate are presented, for X and Y direction and for both lateral load distributions. In these curves, the walls of the adjacent buildings contribute for the computation of the base shear force, because the main goal of this study is to analyse the global behaviour of a group of buildings.
The results in Figure 15, with both load distributions, describe the expected behaviour of the building in aggregate: the higher limit is defined by uniform load distribution and the lower by a pseudo-triangular load with a significant variation of strength specially in the Y direction. In addition, it is noteworthy that the X direction continues to have more ductility and less resistance than the Y direction.

It is worth noting that the great variation between the uniform and pseudo-triangular load patterns for building in aggregate in Y direction (much more significant than in case of isolating building) is mainly due to torsional effects due to the adjacent buildings. In fact, the pounding effect of the adjacent RC slabs induce a quicker damage for the pseudo triangular load, especially in the back facade of the building, as shown in Figure 16.

![Figure 16](Image)

**Figure 16** Collapse mechanism in Y direction for the aggregate model – pseudo-triangular load case

The ultimate displacement is also marked as a black dot in Figure 15. In this model the global behaviour is different but the ultimate displacement is also defined by the occurrence of a collapse mechanism (criterion (ii)). However, the collapse mechanism on the X direction occurs for high values of top displacement, which do not correspond to a realistic or conservative assumption for the evaluation of the seismic behaviour since, for this value, the structure has already collapsed due to large inter storey drift values. Therefore, the possibility of collapse through criterion (iii) has to be evaluated. In Figure 20 the collapse mechanism in the X direction can be seen, on phase 4 (step 73).

As previously referred, a multiscale approach, which correlates the damage on the building at different levels, is of utmost importance since it allows monitoring the occurrence of significant damage in parts of the building, which may not necessarily correspond to significant strength decay of the overall base shear represented in the pushover curves. In this study, the type of buildings in analysis would certainly lead to a non-uniform distribution of internal forces and
stresses, leading to a concentration of damage in specific masonry walls and consequently collapse of the structure due to the formation of a partial mechanism.

Thus, to better understand the distribution of damage in the main vertical structural elements the evolution of the inter storey drift of these elements with the top displacement is presented in the following figures (Figure 17 and Figure 18) for the pseudo-triangular load case, for each storey level ($\delta Li$ stands for the inter storey drift of the wall at the floor level $i$). These graphics show the drift for each wall with the same direction as the load.

![Graphs showing inter-storey drift](image)

*Figure 17* Inter storey drift by displacement, for the X direction *a)* isolated building, *b)* building in aggregate
It is worth to mention that the maximum value considered in the graphs is 0.5%, that is the interstory drift limit threshold assumed as reference for the SD limit state.

For the isolated building model, the maximum inter storey drift is only reached on the third floor: in the back facade (wall 4) for the X direction and on the gable wall (wall 1) for the Y direction. Comparing the values of displacement corresponding to the attainment of the maximum drift, with the value of the top displacement associated to the formation of the collapse mechanism of the global analysis, it can be concluded that the drift criterion is more demanding, being the correspondent displacement lower, and therefore the ultimate displacement to be considered for the seismic assessment.
For the model with the buildings in aggregate, for the X direction, the maximum drift is reached at the third floors’ level, for the same wall as in the isolated building model (wall 4, back facade). This behaviour is justified due to the asymmetry of the buildings in plane (Figure 10), which generates a torsional effect that aggravates the damage in the back facade. On the Y direction the maximum drift is only reached at the third level, and also for the gable wall 1, in the same walls as the ones in the isolated building model. The correspondent values of top displacements are similarly lower than the values of displacement for the global collapse mechanism, which means that for the aggregate model the drift criterion is also the most demanding. That support the decision that the ultimate displacement value is conditioned by the criterion (iii), the one chosen for the final seismic assessment.

In Figure 19 the capacity curves of the two models under examination are presented together, facilitating the comparison of essential information about the behaviour of the two models (building isolated and in aggregate) in terms of stiffness, overall strength and ductility.

Comparing the two models, the behaviour on the X direction for the building in aggregate is more ductile than the behaviour of the isolated building. This is mainly due to the fact that, along to the X direction, the façades of the buildings being arranged in a band, presenting continuity, thus exhibiting greater energy dissipation on structural elements. In the Y direction, the behaviour is similar on both models, in particular in terms of initial stiffness. This behaviour was expected since the walls aligned in the Y direction have the same length and properties as the
isolated building model. Only a clear decrease of maximum strength in the Y direction is observed for the building in aggregate (with three wall elements per storey) and the uniform load distribution; this may be due to a variation in the main failure mode which occurred for this model and this load pattern distribution (e.g. caused by the torsional effect induces some panels in tension).

Finally, it should be referred that different approaches for the modelling of gable walls (one single wall element or three wall elements per storey) affect the redistribution of stresses but, in terms of global response, the effect should be quite limited (i.e. it is expected small variation in terms of total base shear). Moreover, the mechanism generated in the back facade, that is due to torsional effects, is the dominant in the seismic response of the building and is not affected, in a relevant way, by such variation in the mesh. In this specific case, this difference is not clear in the pushover curves because of this mechanism also affects the gable wall.

4.1.2 Damage pattern analysis

With the modelling of the adjacent buildings, alongside the floors at the right position, the analysis of the damage pattern of the walls is of great interest. In fact, it would be possible to observe the damage due to the impact of the floors at different elevations on the masonry wall elements of the adjacent building.

In this section the evolution of the damage pattern of the main facades of the three adjacent buildings with the increase of the loading in the X direction, for the pseudo-triangular load distribution, is presented and discussed. In Figure 20 the various phases of the damage pattern are shown, identified chronologically by step of the incremental analysis as shown in Figure 15; the last phase (phase 4) corresponds to the value of ultimate top displacement defined according to criterion (ii) (i.e. corresponds to the formation of a partial mechanism).
On phase 2 the first pier element to have damage due to building pounding is identified. It is important to note that this element was the first to reach the nonlinear behavior at this story level. On the next phase (phase 3) a pier on the second floor (identified with a black circle) also reaches the nonlinear behavior. Finally, on phase 4 a more advanced stage of the damage distribution is reached, where it can be seen that the pier elements are the most affected elements in comparison with the spandrels, which means that these elements have more visible damage (confirm the damage observed due to 1980 earthquake in Terceira, as illustrated in Figure 6). This distribution is also coherent with the damage imposed in similar buildings on past earthquakes, as referred in [28].

4.2 N2 Method and safety verification

For the evaluation of the seismic performance the N2 method, proposed by Fajfar [16] as described in the Eurocode 8 [23] is adopted.

4.2.1 Seismic action

The seismic demand was defined taking into account the National Annexes ([23] and [26]), with a peak ground acceleration (PGA) of 2.23 m/s² (that is a reference PGA of 2.5 m/s² reduced by 89% according to [26]). The soil type was determined according to the characterisation of soil
profiles in *Angra do Heroísmo* in [29] (soil type A was adopted with a soil factor S equal to 1).

The elastic response spectrum considered is shown in Figure 21.

![Figure 21 Acceleration Displacement Response Spectrum (ADRS)](image)

### 4.2.2 Performance points

The performance displacement was computed from the intersection between the capacity curve and the inelastic response spectrum; the capacity curve was obtained by properly converting the pushover curve representative of the nonlinear response of the original structure (multi degree of freedom - MDOF) into an equivalent single degree of freedom (SDOF) system through the participant factor concept introduced in [16] and adopted also in Eurocode 8 [23].

The seismic performance-based assessment is analysed in terms of the ratio between the ultimate \(d_u^*\) and the performance displacement (or target displacement \(d_t^*\)) for the equivalent SDOF system (* represents the values for the SDOF system). According to the displacement performance based assessment criterion, safety is verified if \(d_u^* / d_t^* > 1\).

### 4.2.3 Seismic performance assessment

#### 4.2.3.1 Comparison of the isolated and aggregate models

The seismic performance was assessed for the two models under examination (Table 2), by adopting criterion (iii) and the pseudo-triangular load pattern, being the worst condition.
Table 2 Seismic performance-based assessment results

<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>$d_u^*$ [m]</td>
<td>0.0321</td>
<td>0.0034</td>
</tr>
<tr>
<td>$d_t^*$ [m]</td>
<td>0.0522</td>
<td>0.0088</td>
</tr>
<tr>
<td>$d_u$ [m]</td>
<td>0.039</td>
<td>0.004</td>
</tr>
<tr>
<td>$d_t$ [m]</td>
<td>0.063</td>
<td>0.012</td>
</tr>
</tbody>
</table>

The results are depicted on Figure 22. It can be concluded that, for the seismic demand defined according the national Annexes of Parts 1 [23] and 3 [26] of Eurocode 8, both models do not verify the safety criterion, on the X or Y direction, being the values of the $d_u^*/d_t^*$ ratio significantly below the threshold.

Besides this, comparing both models it can be concluded that the building in aggregate has the worst behaviour for the studied seismic action, with a lower value of the $d_u^*/d_t^*$ ratio in both directions. It should be expected an improvement in the seismic behaviour for the building in aggregate due to the introduction of more wall elements, that would confine the main building and give it more ductility.

However, the presence of the floors at different heights induces significant damage in the structure leading to a more vulnerable seismic behaviour of the building. On the other side, it is important to remember that the adjacent buildings have RC slabs that will generate significant inertial forces in the level of the floors of these buildings, worsening the pounding effect of the adjacent building.

4.2.3.2 Comparison of aggregate models with even floors

To quantify the effect of the pounding of floors at different elevations a new three-dimensional model was generated, assuming that all the buildings have the floors at the same level. In
Figure 23 the original model (Model A) and the new model (Model B) are presented and identified. The corresponding capacity curves of these models are illustrated in Figure 24.

Model B presents a very different behaviour of the original one. First of all, it is worth to note the significant increase of strength in the Y direction. This difference is easily justified after analysing the damage pattern distribution of the collapse mechanism for both models in the Y direction. As shown in Figure 25, the collapse mechanism in Model B is entirely conditioned by the gable walls, instead of being dependent of the back facade behaviour, as in Model A. In fact, in the latter the impact of the RC slabs of the adjacent buildings, leads in some walls to more pronounced torsional behaviour and to the premature collapse of some vertical with a consequent lower global resistance.

![Figure 23](image1.png)

**Figure 23** Three-dimensional models with three buildings: Model A – the original; Model B – all floors at the same level.

![Figure 24](image2.png)

**Figure 24** Capacity curves of model A and B
The capacity curves in the X direction show only a slightly increase of the resistance for Model B. The results in terms of ratio between the performance point and the ultimate displacement are shown in Figure 26 (the same criterion (iii) was used in both models, in order to be comparable).

It can be noticed that in the Y direction the ratio $d_u/d_t$ ratio is very similar, despite the huge difference in the building’s behaviour. This situation is justified by the dominance of the criterion (iii) for the definition of the ultimate displacement. In the X direction there is a very significant change in the $d_u/d_t$ ratio for model B, which leads to the safety verification of the structure behaviour. These results show again the effect of the floors of adjacent buildings with different elevations which worsen the seismic behaviour of the building. The results of model B prove that the adjacent buildings can reduce the seismic vulnerability, as clearly shown in the work of Ferrito et al. [30].
4.3 Damage patterns for the reference model

To analyse the existing structural damage, the damage patterns of the target displacement were analysed in more detail for the reference model of the building, in aggregate (model A), for the target displacement, previously defined and presented in Table 2 - Figure 27 and Figure 28.

From Figure 27 the interaction due to the floors of adjacent buildings at different elevations is clear, as referred by Cattari et al. [28]. The piers on the top floor of the front facade are significantly affected by the collision with the floor of the adjacent building; for the target displacement these structural elements are in the non-linear phase and are conditioned, due to its geometry, by flexural behaviour. In Figure 28 the torsional effect induced by the gable walls on the back facade is observed, emphasising the damage on the top floor of this facade.

Based on this damage distribution, to improve the seismic performance of this building it is suggested to include simultaneously two strengthening solutions: (i) add vertical resistant elements in some specific locations to reduce the torsional effect and to mitigate the concentration of damage in wall 4; and (ii) increase the stiffness of the floors in their own plane to improve the distribution of internal forces by the walls.
5 Conclusions

The seismic vulnerability assessment of buildings inserted in aggregates, in which each one is interacting with the adjacent, should not be studied as if dealing with isolated units. In particular, if adjacent buildings have different height or/and different floor levels, the impact of the floors in adjacent buildings would worsen the seismic performance of the buildings, as expected. Thus, it is crucial to take into account this effect in the adopted models, which is only possible if the whole aggregate is considered in the seismic analysis. It is important to note that the structural problem analyzed is also typical of many European cities located in earthquake prone areas.

In the present work, the importance of the abovementioned issue was shown through the study of seismic behaviour of a typical Azorean building, inserted in an aggregate, taller than the adjacent buildings whose floors have different elevations. Non-linear static analyses were performed using the Tremuri program. Different models were developed to compare the influence of the surrounding buildings over the studied building’s seismic performance. The damage caused by 1980 Azores earthquake was shown, proving the damage frequently observed in historical town centres due to the structural contiguity within the aggregates.

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