Design of Seismic Retrofitting for mixed Masonry-Reinforced Concrete Buildings in Lisbon

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Abstract: The present dissertation focusses on evaluating the seismic vulnerability of an old mixed masonry-reinforced concrete (RC) building, representative of an existing typology of Lisbon building stock, and on proposing seismic retrofit solutions to improve its seismic performance. Therefore, it comprehends the following main steps: (i) seismic analysis of the unstrengthen building; (ii) identification of the critical areas; (iii) definition of adequate structural interventions; (iv) implementation of retrofitting solutions and assessment of the strengthen building for different retrofitting measures.
The study comprises the seismic analysis of the selected mixed masonry-RC building, integrated in an aggregate, using non-linear static analysis with the Tremuri software. In an earlier stage, results were analysed in terms of capacity curves and afterwards it was developed a seismic performance-based assessment method using N2 method, as suggested in Eurocode 8 (EC8-3), to determine the respective seismic displacements, the distribution of damage and to verify the seismic safety conditions.
Based on these results, a set of different retrofitting solutions were proposed: (i) improvement of the connections between walls; (ii) Improvement of floor stiffness; (iii) strengthening and confinement of the walls. These solutions were carefully chosen keeping in mind possible restrictions, such as boundaries or economic issues, intrusiveness and even protection of the built heritage. These strengthening techniques were applied in a cumulative way and based in design recommendations, in order to evaluate seismic safety and compare their influence in the building's global seismic performance.
Finally, a critical discussion is presented as well as some recommendation for this building typology.

Keywords: Mixed masonry-reinforced concrete buildings, Seismic vulnerability, Performance-based assessment, Non-linear static (pushover analysis), Tremuri software, Seismic retrofitting.

1. Introduction

1.1. Seismic risk in Portugal

Earthquakes are one of the most destructive, terrifying and unpredictable natural events, being Portugal one of the most seismic prone and susceptible region. The Portugal mainland, with singular and notable seismic background, is located in the southwest part of the Eurasian plate near the southern border of the African and North American plates, that can be subjected to both near and far seismic activity with low to large magnitudes [1].
The most recent events around the world, such as in Italy (2009, 2012, 2013, 2016, 2017), New Zealand (2010, 2011), Japan (2011), Haiti Region (2010), Indonesia (2006, 2009), China (2008) and Iran (2003), and the last big earthquakes in the mainland of Portugal in 1909 (Benavente) and 1969 (offshore), have made possible to gather an unparalleled amount of data, not only what concern seismic hazard but also to the seismic performance of different typologies of buildings. Therefore, it is undoubtedly necessary to characterize and evaluate the seismic vulnerability of the buildings' stock, identify the critical areas of the structures and possible collapse mechanisms and, then, implement some retrofitting solutions [2].

1.2. Existing Masonry Building Stock

In the earlier 30s, in Portugal, there was a revolution of the urban layout, a plan to expand the old centres. This period of development and expansion is connected to the construction of mixed masonry-RC buildings (known as "Placa" between contractors because of the introduction of RC structural elements, mainly as slabs) in a large scale, right before the introduction of the

modern and current reinforced concrete buildings.
When analysing the relatively old urban centres, this building system still represents about 30% of the Lisbon building stock [3, 4]. Thus, since this typology is representative of the current urban layout and also considering the seismic background, previously mentioned, it is crucial to perform seismic assessment of such typology. This need is in line with the new reality that the construction sector faces, where the investments have been gradually made for renovation and rehabilitation of the existing stock buildings in favour of new constructions, leading towards, once more, to the necessity to evaluate this system under study.
Therefore, old mixed masonry-RC buildings have been, in the recent years, under seismic vulnerability studies. The recent works have shown that this building system is more vulnerable to earthquake, consequently at significant risk of collapse [2, 5]. However, it is relevant to point out that the seismic performance of building structures depends on a variety of factors, such as: (i) the function of the building, which have impact on the architectural configuration; (ii) Insertion of the building on the urban layout (boundary conditions); (iii) structural details, quality of materials and execution works; (iv) the presence of past interventions.

2. Case of Study

2.1. Description of the Case Study Building

Regarding the location and the environmental aspects of the surroundings, the case study building is located at "Bairro de Alvalade", an urban area of Lisbon, which was part of urban expansion in the beginning of the 20th century. The choice of the case study building was based on the representativeness of the sample within the existing typologies of "Bairro de Alvalade", particularly in Cell I and II (see Figure 1).
In fact, based on statistics presented in [4], the buildings located in Cell I and Cell II are quite standardized in terms of material, geometry, constructive details and number of floors [3, 6]. In this paper, the block of two buildings, located in Cell I, was defined and analysed, with the aim to take into account the effect of the adjacent building.
The buildings of the block have a symmetric and elongated rectangular plan shape (17.50 m x 6.40 m) and total height of 10.13 m: two façade walls (frontal – P3 and back – P1) with openings, one shared wall (P12) and two blind side walls (P2 and P4). The building has three floors (with the total same inter storey height of 3.06 m) with two apartments per floor with an area of 154.12 m^2. At that time, the priority of the layouts was to be functional and, consequently, efficient of the spatial areas. The plan distribution of exterior and interior walls is shown in Figure 2.

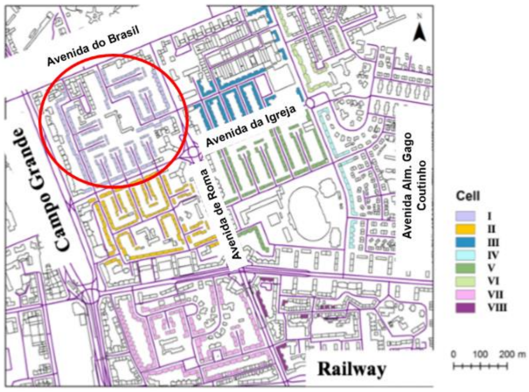

Figure 1 - Top view of "Bairro de Alvalade" and its division into cells [5]

In what regards the structural details and mechanical properties, the information gather lead to:
- Ground soil foundation at "Bairro de Alvalade" is classified as type B and C (deposits of very dense sand, gravel or very stiff clay and deep deposits of dense or medium-dense sand, gravel or stiff clay) [3].
- Foundation system was made with very stiff stone masonry and with hydraulic mortar, characterized as a thick continuous wall, which was enlarged in its base [6].
- Walls: façade walls P1 and P3 (identified in Figure 2) are made of rubble stone masonry with hydraulic mortar with a thickness of 0.5 m on the ground level, which are decreasing 0.05 m in each floor till a minimum of 0.4 m, while under the window openings, they are characterized by hollow clay brick masonry with the reduced thickness of 0.15 m; Side walls P2 (from adjacent building) and P4 are made of rubble stone and hydraulic mortar with a constant thickness of 0.5 m; interior walls are mainly made of hollow clay brick masonry and cement mortar with variable thickness: 0.15 and 0.25 m.

Additionally, the external walls were strengthened (belted) on all floors by reinforced concrete beams at the height of the window lintels with the thickness equal to that of walls and height of 0.20 m [3, 10]. It is assumed that the existence of these RC beams along the front and back façades. The concrete used on the buildings has a low to moderate resistance (C16/20) and the RC elements have lower percentages of steel reinforcement, whereas the steel corresponds to the class A235.

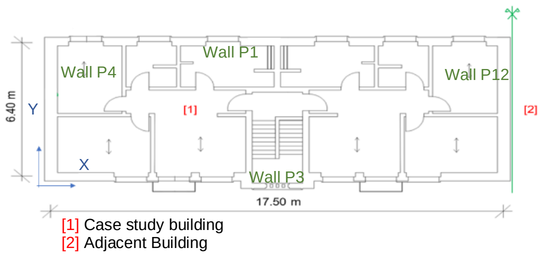

Figure 2 - Case study building layout.

- The specific information regarding the building's walls materials properties are given in Tables 1, 2 and 3. It is relevant to mention that the number of experimental tests addressed, or even the possibility to observe the construction details and perform any type of destructive or semi-destructive tests, to characterize the mechanical properties of the materials in this type of buildings was very limited. However, due to the similar type of rubble stone masonry used in buildings in Portugal and Italy, it was decided to follow, as reference, the values proposed by the Italian code [9]. Thus, those values were adopted as initial ones and then corrected by using the Bayesian approach by accounting for the available tests carried out on these buildings in Lisbon. For detailed information about this procedure, refer to [3].

Regarding the floors, the following characteristics were adopted:

- Timber floors made of *Pinho Bravo* beams, with a section of 0.08 x 0.16 m^2, covered by timber boards and placed perpendicular to the façade, with an average offset of 0.40m. This type of floor is presented mainly at the social and private areas;
- Reinforced concrete slabs, barely reinforced, are mainly presented at the service areas such as kitchens, bathrooms and balconies. Based on literature, it was found that thickness can vary between 0.07 - 0.12 m (adopted 0.12 m). In this paper, mean value was adopted and applied by changing the masses of intermediate floors (permanent and live loads).
- The configuration of roof is timber framed of Pinho Bravo, which consists of main beams mainly disposed parallel to the façade and supported by vertical or diagonal timber elements loading the main internal and side walls.
- The main stairs are located in the middle of the building for U_x direction (direction of the façade), providing symmetry along U_y axis (direction of the side walls), and made with wooden materials.

More details about the buildings are available in [3] and [11].

2.2. Modelling details of the Case Study Building

The global response of the case study building was assessed using the Tremuri program [13], by analysing the results from the building modelled in the aggregate. It is worth noting that, with the aim of defining the pushover curves of the case study building, only results for one building are considered Tremuri software is based on the equivalent frame model approach, where each resistant masonry walls with openings may be discretized by a set of panels, deformable elements that are connected between themselves: (i) Piers - vertical resistant elements which support vertical and lateral loads; (ii) Spandrels - horizontal elements between two vertically aligned openings, which couple piers in the case of seismic loads and; (iii) Rigid nodes - undamaged masonry portions confined between piers and spandrels.

Following the recent formulation implemented in Tremuri program [12], the nonlinear response of masonry panels is modelled by nonlinear beams characterized by a piecewise linear constitutive law.

In what regards the reinforced concrete elements, they were modelled as nonlinear beams by assuming elasto-perfectly plastic hinges concentrated at the end sections [12]. It is worth highlighting that in the case of buildings under investigation, in which the presence of reinforced concrete elements is limited only to RC ring beams, the seismic nonlinear response is dominated more by masonry elements than RC elements (although they provide an essential role in coupling piers).

The floor was modelled with orthotropic membrane finite elements, with an equivalent thickness and characterized by modulus of elasticity in the floor warping direction ($E_{1,eq}$) and in the orthogonal direction ($E_{2,eq}$) and an equivalent shear modulus (G_{eq}). The shear modulus, which represents the most important parameters of the floor model, influences the distribution of horizontal forces amongst the walls, in both linear and non-linear phases. Therefore, the timber floors were defined as an equivalent membrane with 0.022 m thickness and characterized by $E_{1,eq}$ = 29.45 GPa , $E_{2,eq}$ = 12.00 GPa and G_{eq} = 0.00988 MPa, following Italian tests in similar cases. For the reinforced concrete slabs, the following properties were considered: 9.17 GPa for E_{eq} equal in both directions, as defined in Eurocode 2-1 and 3.82 GPa for G_{eq} (calculated assuming a Poisson coefficient of 0.2).

Table 1 - Mechanical parameters and weight adopted in the numerical model.

Mechanical and geometrical properties	Young Modulus E (GPa)	Shear Modulus G (GPa)	Comp. Strength f_m (MPa)	Shear Strength τ_0 (MPa)	Specific Weight γ (kN/m3)
Rubble stone masonry	0.820	0.274	2.330	0.077	21.0
Solid clay bick masonry	5.730	1.910	7.190	0.277	18.0
Hollow clay bick masonry	2.950	0.983	1.660	0.277	15.0
Reinforced concrete	Concrete class: C16/20 Steel class: A235				

Table 2 – Parameters which include the drift corresponding to the different levels of damage (δ_3, δ_4, δ_5) and the percentage of residual strength after collapse (β_3, β_4), different for the two possible failure modes: shear (S) and flexural (F).

Mechanical and geometrical properties		
Parameters for Piers	δ_{3_S}	0.00291
	δ_{4_S}	0.00488
	δ_{5_S}	0.00686
	δ_{3_F}	0.00583
	δ_{4_F}	0.00976
	δ_{5_F}	0.01470
	β_{3_S}	0.70
	β_{4_S}	0.40
	β_{4_F}	0.85
Parameters for Spandrels	δ_{3_S} and δ_{3_F}	0.00190
	δ_{4_S} and δ_{4_F}	0.00580
	δ_{5_S} and δ_{5_F}	0.01940
	B_{3_S} , β_{4_S} and β_{4_F}	0.60

Table 1 - a) Permanent and variable loads. b) Mechanical properties in the numerical model.

a)

Mechanical Property	Gravity Load G (variable load Q) (kN/m2)
Wooden Floor	1.10 (2.00)
RC slab	3.78 (2.00)
RC beams	3.78 (2.00)
Roof	1.15 (0.40)

b)

k_0	0.65	Value of the shear for which starts the degradation of the stiffness, normalized to the ultimate shear. Ratio between shear at the end of the elastic phase and the ultimate shear strength.
k_{el}	1.50	Ratio between initial and the secant stiffness at the point in which the maximum strength is reached.

In this study, the global seismic response of the building, which is controlled by the in-plane capacity of the masonry walls, was obtained by applying static lateral load patterns on the structure. Thus, two different distributions of lateral loads, based on the recommendations of the European EC8 [8] and Italian [9] standards, were used for the definition of pushover curves: (1) uniform, pattern of forces applied to each node of the building and proportional to its mass and (2) pseudo-triangular, pattern of forces proportional to the product between the mass of the node and its height with respect to the base. Moreover, pushover curves for two different directions, namely the longitudinal (U_x) and the transverse (U_y) direction, have been examined in the presented study, for the case study building, considering also positive and negative direction.

Unlike Finite Element based programs, Tremuri program does not allow the user neither to select nor create a fictional control node located in the centre of gravity at the roof level of each structure, as suggested in the Eurocode for the definition of the pushover curves (described as base shear versus top displacement). To overcome this issue, the choice was based on the recommendations from Tremuri team, which states to adopt the node above the wall which first reaches damage. This step was then validated by observing the damage distribution for each pushover analysis and by evaluating the damage of the macro-elements near to the selected control node. For U_x direction, the control node adopted was N20, while for U_y direction, analyses were performed for the control node N99 (Figure 3).

Three additional remarks must be stated: (i) For the purpose of this rehabilitation study, the seismic performance-based assessment was conducted by evaluating safety indicators recommended for the Severe Damage limit state [8]. Moreover, the seismic action and the ultimate displacements were reduced by a factor of 0.75 as it is proposed in [8]. (ii) Based on previous non-linear dynamic analysis study [3], for the buildings with rectangular shape in plan, the most appropriate load pattern that defines the behaviour of the building is: triangular load pattern for the direction of the façades (U_x direction), and the uniform load pattern for the direction of the side walls (U_y direction). The results presented along the paper follow this observation; (iii) Case study building before applying any retrofitting technique was considered and referred in the following text as stage "0". At this phase, it was considered and adopted bad connections between external – external walls and external – interior walls, to approximate to the real condition of the building. In order to model this detail, the values for area and inertia were 0.0002828 m^2 and 0.0001414 m^4, respectively, adopted for the equivalent link beam in connections. These values were obtained by an iterative process, by a continuous decrease until the pushover curves do not show a significant difference of stiffness, strength and displacement.

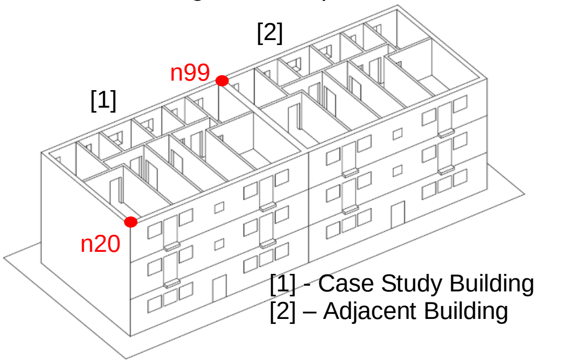

Figure 3 - Three-dimensional representation of the building with the control nodes indicated.

3. Seismic Performance - Based Assessment

3.1. Pushover analysis

The pushover curves provide essential information about the behaviour of the building in terms of: (1) stiffness; (2) overall strength and; (3) ultimate displacement capacity. These curves plot the base shear capacity, V_b, versus the average displacement of nodes located at the roof level, d_n, and the analyses were only stopped for 20% decay of the maximum base shear capacity [8, 9] or when a sharp drop in pushover curves (consequence of the possible existence of a collapse mechanism) became clear. The pushover curves obtained for the case study building, for the most demanding cases are plotted in Figure 4.

From Figure 4, one may observe that the most vulnerable direction is the U_x direction for both load patterns, exhibiting less strength, ductility and energy dissipation on structural elements in that direction. This result can be explained by the fact that a greater number of walls are in the U_y direction, where the most of them are characterized as blind walls (no openings) in contrast with the higher number of openings in the U_x direction. For U_x direction, the façade walls P1 and P3 (designated on Figure 3) face more demand while the interior walls have a more significant role to redistribution of forces than support. For U_y direction, it is observed a higher redistribution of load among a higher number of blind walls, being the side and shared walls, P4 and P12 respectively, more demanded. As expected, the peripheral and shared walls are crucial to describe the behaviour of the building, because of their position on the layout and cross section. Additionally, it is relevant to highlight the importance of analysing the building in all directions and of considering the building inserted as part of an aggregated block, when doing a seismic performance-based assessment.

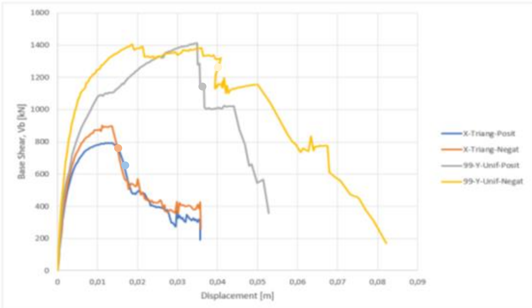

Figure 4 - Pushover curves for the case study building at its original state for both directions and considering the conditional cases of load pattern (dot mark the ultimate displacement considered).

The damage pattern for the ultimate displacement (d_u) of the building for back façade wall P1 (pseudo-triangular load pattern) is shown in Figure 5. The caption displays the type of failure for each structural element, whereas "DL" represents the possible damage level until collapse. For U_x direction, the behaviour of spandrels was mainly characterized by shear failure, due to their geometrical characteristics and to the presence of reinforced concrete lintel beams of the building façade, while the piers were mainly characterized by flexural damage. Once comparing the damage maps, one may expect shear damage around the openings. The interior brick walls present flexural and insignificant damage in the piers.

Regarding the analysis in the U_y direction, the pattern of damage is conditioned by the type of load distribution. With respect to the U_y direction with the uniform load pattern (concerning the maximum value of base shear), side walls have shown high damage leading to mechanism at the ground level, being relevant to highlight the relation thickness – height of the building. Piers in this story reached the shear failure, being an indicator of progress of in-plane failure mechanism (see Figure 6, damage map for two walls in U_y direction for uniform load pattern). Additionally, the pseudo-triangular load distribution leads to an identical collapse mechanism, but on the top of the walls (see Figure 6, damage map for two walls in U_y direction for pseudo-triangular load pattern). The previous observations can be explained by the differences of seismic demand simulated by the two force distributions adopted [14].

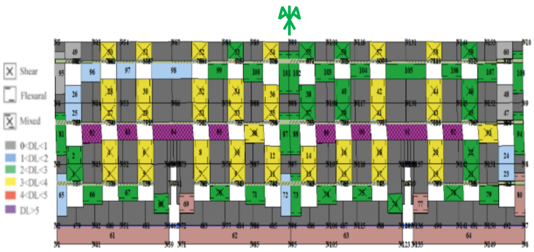

Figure 5 - Damage map of the back load-bearing façade wall P1 for the ultimate displacement along U_x direction.

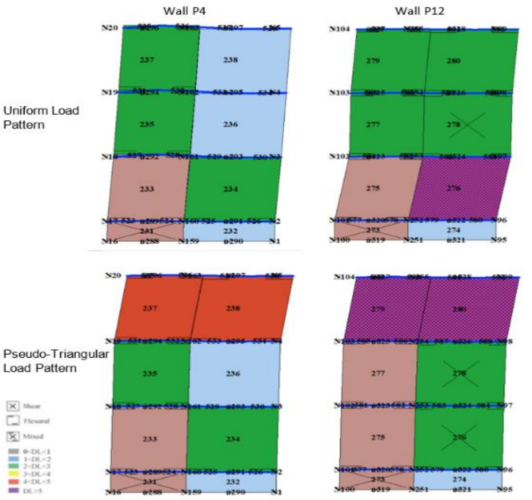

Figure 6 - Damage maps of the side wall P4 and shared wall P12 for the ultimate displacement along U_y direction.

3.2. N2 Method and Discussion of Results

Following the recommendations of structural code EC8-1 [7], originally proposed by Fajfar [15], the N2 Method was adopted, to evaluate the performance of the structure and to determine the structure's performance point (d^*_t) which is the intersection between the capacity curve of the structure and the seismic demand in terms of response spectrum.

In this procedure, an elasto-perfectly plastic force-displacement relationship was assumed to define the SDOF bi-linear capacity curve: (1) the initial stiffness was determined by the point corresponding to 70% of the maximum base shear reached on the first branch of the curve as suggested by NTC [9]; (2) the yield force was determined in such way that the areas under the pushover and the elasto-perfectly plastic curves are equal.

Moreover, the seismic performance-based assessment was then conducted by evaluating the following indicators, recommended both by EN 1998-3 [8] and the NTC [9] for the Severe Damage limit state safety verification. The latter recommends: (i) the ratio q^*, computed by dividing the acceleration in the structure with the unlimited elastic behaviour, Se (T^*), for the limited structural strength F^*_y/ m^*, to be lower than 3, aiming to limit the overall acceptable ductility of the building; (ii) the ratio between ultimate and target displacement d^*_u / d^*_t is larger than 1; (iii) the ratio between the maximum admissible ground acceleration, $a_{g,max}$ compatible with the fulfilment of the ultimate limit state and the reference ground acceleration, a_{gR} (considered equal to 1.50 ms^{-2}) is larger than 1. The results from safety verification can be seen in Table 4a. The other results used in the calculation are presented in Table 4b.

This final step of the assessment consists of checking if the building withstands the seismic demand defined according to the national Annex of Part 3 of EC8 [8] (correspondent to a return period of 308 years). The response spectrum was initially defined for 5% viscous damping and for soil type C, earthquake type 1.3. The buildings' importance factors are equal to 1. For the non-linear static analysis processed, to evaluate the seismic performance, it was considered only the earthquake type 1.3, which is more demanding for the structure, as it was proven at [3]; thus, the peak ground acceleration defined in EC8-1 [7] was multiplied by 0.75.

Table 4 – a) Verification of safety condition parameters, for both direction and conditional cases of load pattern. b) Auxiliary values.

a)

Case Study Building: "0"			q* ratio	du* / dt*	ag_max / agr
Control Node	Direction	Load	[<3]	[>1]	[>1]
N20	X+	Triangular	2,3578	0,7347	0,5986
	X-	Triangular	2,0987	0,7169	0,5956
N99	Y+	Uniform	1,3515	1,5235	1,0009
	Y-	Uniform	1,3252	2,8257	1,4720

b)

Case Study Building: "0"			Equiv.	Yielding	Yielding Force	Ultimate	Target Displ.
Control Node	Direction	Load	Period [s]	Displ. Dy [m]	Fy [kN]	Displ. Du [m]	Dt [m]
N20	X+	Triangular	0,2619	0,00416	757,5	0,01676	0,01711
	X-	Triangular	0,2566	0,00449	851,0	0,01531	0,01602
N99	Y+	Uniform	0,3073	0,01069	1312,0	0,03662	0,01803
	Y-	Uniform	0,2241	0,00580	1338,0	0,04086	0,01085

These two last indicators (d^*_u/ d^*_t and $a_{g,max}$/ a_{gR}) did not meet the safety requirements just in U_x direction, leading to the conclusion that the possible retrofitting solutions should be applied and have more impact along that axis. These results obtained from the application of the N2 Method procedure show that practically the studied analyses failed to verify the previously mentioned safety requirements, which has driven the need for seismic retrofitting, detailed in the next chapter 4.

4. Seismic Retrofitting of mixed Masonry-RC Buildings

4.1. Basic Principles of Buildings Design

When looking at the case study building, the structural system, the choice of materials and the adopted bad connections between elements lead to the existence of unclear trajectories of seismic forces because, which allowed to a low reliable prediction of seismic behaviour.

The building under study only presents significant geometric variation in elevation, not in plant, between floors, since the thickness of the façade walls decrease in each floor, which affects the behaviour of the structure. It is relevant to state that: (i) the principle of regularity fails and (ii) the geometry of the building in height is a preponderant factor of the seismic behaviour of a structure.

In what regards the symmetry, it is not always easy to impose, due to the irregularity of the structures and the compatibility with the architecture. For case under study, the symmetry is only observed on one axis and the existence of elements of great rigidity orientated in the perpendicular direction. Therefore, there was the need to make the structure capable to accommodate some torsion, since this behaviour is inconvenient in view of the seismic action response. The structural elements further away from the center of rotation/ stiffness would suffer an increase of stress and displacements, which may lead to the collapse of those elements. Nevertheless, it is worth to mention that the torsion of buildings (due to asymmetries of the vertical structural elements or masses) is attenuated in buildings inserted in blocks.

Therefore, for the purpose of this study, to improve the integrity of structure, by increasing the in-plane and out-of-plane walls strength capacity and redistribute seismic loads between the load-bearing walls, the use of strengthening through traditional techniques are suggested.

4.2. Solutions 1.1 and 1.2

The retrofitting solutions 1.1 and 1.2 are related to the improvement of the wall-to-wall connections by means of effectively tying walls together with steel tie-rods. The solution 1.1, referred as “1.1”, only takes into consideration the improvement of the connections between perimeter walls, as the solution 1.2, referred as “1.2”, adds the connections between the interior and exterior walls. A possibility could be the use of threaded steel tie-rods with a diameter of 16 mm, which are usually installed horizontally beneath floors and roofs on both sides of the wall and restrained at the ends by steel anchor plates.

This improvement is an old provision and regularly used in rehabilitations, to enhance the building integrity. The advantages of the use of these solutions are: (i) prevent out-of-plane collapse during an earthquake, leading to a like box behaviour; (ii) enhancing the connections providing a better load distribution and distributing diaphragm loads along the length of the wall [2, 16].

In order to simulate this technique, the equivalent beam modelled elements were considered rigid, thus adopting the following new values for area and inertia were 10.0 m^2 and 5.0 m^4, respectively (new geometrical values were based on the default values considering as good connection given by the program).

4.3. Solutions 2.1 and 2.2

The solutions 2.1 and 2.2 adopted in this study are referred for retrofitting the floors, avoiding a high intrusiveness technique and increasing the stiffness of the flexible floor in-plane. The solution 2.1, referred as “2.1”, was only implemented on the building case of study, while the solution 2.2, referred as “2.2”, includes the retrofitting of the adjacent building’s floor. These solutions were added to solution 1.2, where the case study building already has improved connections.

These two solutions require the replacing of all deteriorated structural timber elements by new elements adequately connected, restoring their original resistant capacity. Additionally, a possible solution could be: the installation of diagonal steel braces at both floors and roof level between timber joists, anchored with galvanised steel threaded rods and galvanised steel angle brackets as well as a new layer of timber sheathing, laid perpendicular to the existing one and adequately nailed to the floor [19].

In order to simulate this technique and taking into consideration the Tremuri program limitation on designing such a solution, it was adopted a stiffness multiplier for the above strengthen described timber floor. This decision was based on the NZSSE guidelines, after ASCE (2014) [17, 18]. Therefore, it was considered a double straight sheathing membrane typology, with diagonal sheathing chorded, using a fair rating condition to the new timber floor provision. Hence, a multiplier of 9.0 was applied to the G_d values both for the load direction, acting parallel and perpendicular to timber joists. Consequently, the new equivalent shear modulus G_{eq} was then obtained by dividing G_d by the thickness of the new membrane (double straight sheathing), which was considered equal to 4.2 cm (2.2 cm from original single straight sheathing typology assigned plus 2.0 cm of the new timber sheathing layer).

4.4. Solutions 3.1 and 3.2

The retrofitting Solution 3.1 and 3.2 involve the shear strengthening and confinement of masonry structural walls by the implementation of reinforced steel mesh, in the building already with the connections improved (1.2) and stiffening of the floors and the roof (2.1). The solution 3.1, referred as "3.1", was designed to reinforce only the two load-bearing walls in U_x direction (front and back façades), while the solution 3.2, referred as "3.2", adds reinforcement on the side (P4) and shared (P12) walls to the solution 3.1, in other words, reinforcement on both directions (see Figure 2 for the identification of the walls). A possible solution could be: firstly, the application of a first layer of filling mortar in the proportion of 1:3 (sand, Portland cement and water) for voids and surface regularisation, introduction afterwards of a welded steel mesh made of S275 steel and spaced ribs, then fixed on both sides of the masonry wall through a system composed of galvanised screws, galvanised steel threaded rods and steel anchor plates equally spaced. Finally, the application of a second layer of fine sand-blasted finishing mortar [20, 21].

This retrofitting solution was simulated in the model by considering an improvement of the parameters of the masonry, which comprises an increase of the mechanical parameters adopted for the stone masonry typology assigned to the original, before retrofitting, condition of the building. The multiplier factor was based on previous study [2] and then validated by Italian code [22]. Thus, a multiplicative factor of 2.5 was adopted to the values for the mechanical properties, including the drift values of the masonry walls, represented on Tables 1 and 2, without significant change in the unit weight.

4.5. Discussion of the results

The final results obtained for safety verifications are presented in Table 5. The safety condition regarding the q* ratio was verified in all studied cases.

Table 5 – Verification of the safety conditions, for both direction and considering the appropriate load patterns. a) Ratio between ultimate and target displacements; b) Ratio $a_{g,max}$/ a_{gR} values.

du* / dt* [>1]				
Control Node	N20		N99	
Direction	X+	X-	Y+	Y-
Load	Triangular	Triangular	Uniform	Uniform
0	0,7347	0,7169	1,5235	2,8257
1.1	0,8399	0,5474	3,7052	5,1598
1.2	0,6991	0,5411	5,8501	4,5070
2.1	0,6678	0,5234	7,9499	4,1820
2.2	0,6521	0,5220	7,0528	4,5568
3.1	2,6969	1,5945	8,3352	5,5338
3.2	1,9516	0,8735	14,5923	11,9488

ag_max / agr [>1]				
Control Node	N20		N99	
Direction	X+	X-	Y+	Y-
Load	Triangular	Triangular	Uniform	Uniform
0	0,5986	0,5956	1,0009	1,4720
1.1	0,6616	0,4996	1,8175	2,2026
1.2	0,5883	0,4892	1,6079	1,6355
2.1	0,5694	0,4743	1,8981	1,5308
2.2	0,5601	0,4733	1,7750	1,6745
3.1	1,4913	1,0417	2,7314	2,3413
3.2	1,1826	0,6875	2,7651	2,8300

- It was observed that only Solution 3.1, which involves strengthening and confinement of masonry structural walls along U_x axis, as well as includes the improvement of the connections between walls (1.2) and the increase of the floor and roof stiffness (2.1), a highly intrusive intervention, verifies the safety conditions. Figure 7 shows the overall impact of each retrofitting solutions, mainly solution 3.1, in terms of stiffness, strength and ultimate displacement.
- Regarding the safety condition $q^* < 3$, it was observed that this indicator was always verified, for both directions and load patterns, even before applying retrofitting techniques. The values were decreasing as implementing retrofitting solutions, as expected.
- In what regards the safety condition $d^*_u / d^*_t > 1$, it is possible to observe that, only for U_x direction, safety parameter was not satisfied, exception made for Solution 3.1. The values related to the solutions involving strengthening and confinement of masonry structural walls present a meaningful increase, related to a more significant increase of stiffness, resistance strength and bearing higher displacements.
- For safety condition $a_{g,max}/ a_{gR} > 1$, the obtained values follow the same trend as the previous parameter. It is possible to observe

that, only for U_x direction, safety parameter was not satisfied, once more, exception made for Solution 3.1.

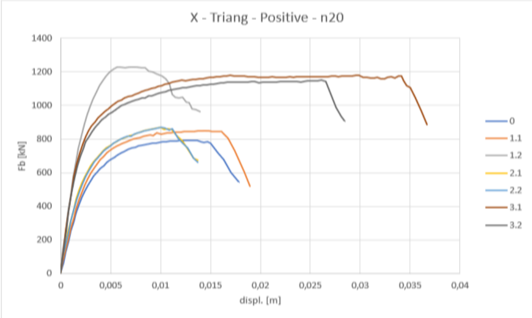

Figure 7 – Pushover curves from before retrofitting to solution 3.2, for the most demanding directions and corresponded load pattern.

- The retrofitting Solutions 1.1 to 1.2, of low-to-moderate intrusiveness, which are known to be effective on enhancing the box-like behaviour of stone masonry buildings, proved to be not so effective for the in-plane global capacity of the structure, especially for U_x direction. This observation is related to the fact that are more connections to be improved along U_y direction. Therefore, the results showed that was still necessary to enhance the behaviour of the building, requiring more retrofitting solutions, by increasing the intrusiveness of those.
- The results from Solution 2.1 and 2.2 showed great influence over the global shear strength capacity and ductility, consequently, improving the building global behaviour, but still insufficient to satisfy all the safety conditions. For U_x direction, excessive diaphragm, imposing larger displacements on the load-bearing walls and, consequently, an earlier collapse can explain the results. On the other side, for U_y direction, the increase of stiffness and strength of the case study building led to higher displacements on the adjacent building. The centre of stiffness moved towards the centre of the case study building, where the retrofitting techniques were implemented, leading to higher forces on the building under study, in an earlier moment, and consequent significant damages on the sharing wall.
- The enhancement through Solution 2.2 showed to be less effective, especially for U_y direction. This observation can be explained by the fact that the connections between walls were not improved on the adjacent building, leading the poor reduction of the torsion effect. One may conclude that an increase of horizontal strength and stiffness at the floor diaphragms is favourable, especially in cases with proper connections between vertical-vertical and horizontal-vertical elements.
- When observing the damage maps from the retrofitting Solution 3.1 (Figure 8) and pushover curves for U_x direction, the load-bearing walls faced higher demands. The back façade P1 suffered more damage, mainly flexural damage, especially at mid height in piers associated to the staircase, contributing to the development of in-plane mechanism. It can be related to the existance of piers with a low shear ratio and more openings in that façade than wall P3.

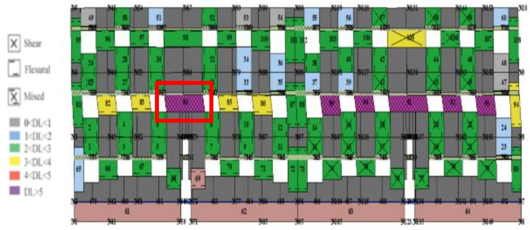

Figure 8 – Damage map of the back load-bearing façade wall P1 for the ultimate displacement along U_x direction.

- The retrofitting solution 3.2 showed to be effective for both directions. This solution presented better results comparing to Solution 3.1 for U_y direction, but not for U_x direction. This observation can be explained by significant increase of stiffness and strength in both directions on the case study building, leading to earlier collapse of the load-bearing walls at mid height of the adjacent building. Afterwards, the forces were redistributed to the case study building, and later a consequent collapse of the load-bearing walls.

5. Conclusion

The main objectives of this paper are to assess the seismic vulnerability of a masonry-reinforced concrete residential building in Lisbon, by identifying the main vulnerabilities, investigating and developing appropriate strengthening techniques in order to improve its seismic performance.
The approach to assess the seismic response of the chosen traditional stone masonry-RC building included in an aggregate was the one proposed in Eurocode 8, by performing non-linear static (pushover) analysis, using the Tremuri software and aiming to fulfil the safety requirements.
Regarding the seismic performance-based assessment analysis before the implementation of the strengthen solutions, it was concluded that safety requirements for the Severe Damage Limit State defined in EC8-3 (CEN 2017) and Italian code (NTC

2008) were not satisfied. Additionally, it was observed that the direction of the façades (U_x direction) was more demanding, being necessary to improve the capacity along this direction, while for U_y direction, the torsion effect was the main issue.

The study comprises the implementation of retrofit solutions, in a cumulative way, and with the following sequence: (i) improvement of the connections between walls; (ii) increase of the wooden floor and roof stiffness in plan and; (iii) retrofit of the most vulnerable walls (the external ones) with reinforced plaster, leaving the most intrusive technique for last.

Based on the results obtained from the global seismic analyses of the building, the solution which includes the improvement the connections between walls, the increase of the floor and roof stiffness in plan and the retrofit of the external walls in the direction of the façades (U_x direction) fulfil the Severe Damage limit states' safety requirements. It was found that the most critical structural elements regarding the safety to the SD Limit State are those surrounding openings for U_x direction, mainly the pier associated to the central staircase at mid height.

Following the work developed and presented, it is recommended to perform further studies of more buildings of this typology, in order to gather more information, in quantity and quality, about their seismic vulnerability, so that suitable and capable retrofitting interventions might be proposed and implemented.

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