Seismic Assessment of a mixed masonry and reinforced concrete building

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May 2022

ABSTRACT

Masonry buildings with reinforced concrete elements emerged in the 30s, with the introduction of reinforced concrete in construction. These buildings are usually called "placa" buildings or mixed masonry and reinforced concrete buildings and represent about one third of the buildings in Portugal. Many of these buildings are in poor condition or have undergone major structural changes, which cause a decrease in the strength of the structure.

Regarding this and considering the Ordinance Nº302/2019 (Portaria, 2019) that have been implemented, it is necessary to perform a seismic evaluation of the global behaviour of existing buildings, whenever they will be subject to interventions. It was in this context that the interest in studying a typical "placa" building in the city of Lisbon arose.

The seismic evaluation was performed using the 3Muri program, and several analyses were developed, namely: (i) modal dynamic analysis; (ii) non-linear static or pushover analyses; (iii) sensitivity analyses; (iv) local out-of-plane mechanism analyses. A seismic evaluation was also performed using the expedite methods developed for buildings with flexible or rigid floors, and several reinforcement techniques were studied, considering the failures presented in the seismic safety.

This dissertation also aims to fill gaps regarding the modelling of the roof in the 3Muri programme, allowing us to understand if there are differences in the results obtained if it is structurally modelled or if its effects on the modelled structural elements are considered, and thus to give indications about the best modelling approach.

Keywords: Mixed masonry-reinforced concrete buildings, Seismic vulnerability assessment, Pushover analysis, Sensitivity analysis, Local mechanisms analysis, Reinforcement solutions.

1 Introduction

After the catastrophic earthquake of 1755, the building reconstruction of the city of Lisbon took special care, not only with the resistance of the structures to gravitational forces, but also with the resistance to possible horizontal forces that an earthquake cause.

Some of the buildings constructed after the earthquake, may have some deterioration and may have suffered structural interventions that altered their seismic response. According to Ordinance Nº 302/2019 (Portaria, 2019), it is now required that a seismic evaluation of the global behaviour of existing buildings is carried out whenever they are subject to interventions.

The aim of this dissertation is to evaluate the seismic behaviour of a mixed masonry and reinforced concrete building and, based on the results, to develop ways to make the structure more resistant, thus studying different reinforcement techniques.

The building under study is part of Bairro de Alvalade, which was built based on the programme of Economic Rent Houses, introduced with the implementation of the social public policy. The urbanization plan of Bairro de Alvalade is divided into 8 cells, numbered I and VIII, with functions of residence, leisure and commerce.

The building is located at Rua Fernando Caldeira, in cell I of the Alvalade neighbourhood, in Lisbon, and is representative of a set of 230 buildings, located in cells I and II, as represented in figure 1. Their construction began in December 1946 and ended in September 1948.

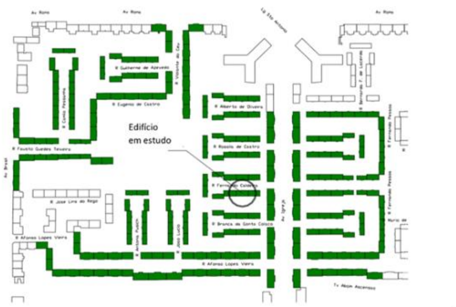


Figure 1 - Location of the building under study and similar buildings (in green) in cell I and II of Bairro de Alvalade (Lamego, 2014).

The building consists of three floors, each with two dwellings. The main facade is shown in figure 2.

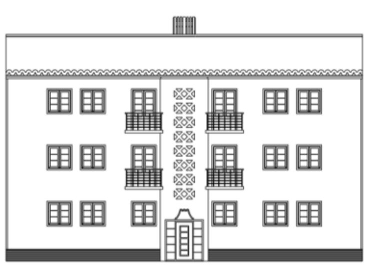


Figure 2 - Main elevation (left) and rear elevation (right) of the building under study (Lamego, 2014).

Its foundations are direct and work as a thicker extension of the walls, which are made of ordinary stone masonry, solid brick or hollow brick

There are also concrete beams on the facade walls, above the windows and on the interior walls in the doorways. On the floors, in the wet areas, there is a concrete slab reinforced in both directions and in the remaining areas the floor is made of wood.

A pine wood's structure with "Lusa" tiles characterize the roof of the building.

2 Structural modelling

Based on the information gathered in the previous chapter and according to Part 3 of Eurocode 8 (NP EN1998-3, 2017), it was decided to apply a non-linear static analysis to assess the seismic behaviour of the building.

The building was modelled three-dimensionally using the calculation program 3Muri (S.T.A DATA, 2020), a commercial version. This program uses a discretization with "macro-elements", for the simulation of masonry structural elements, in three-dimensional structures, forming an equivalent frame model (EFM).

Two models were elaborated, where the difference between the two lies in the roof. In Model A it is not structurally modelled, i.e., the effect of the roof is considered by equivalent loads representative of its weight. In Model B the roof is modelled using the Roof tool of 3Muri. Figure 4 presents the two three-dimensional models. In terms of results, it was concluded that very similar values are obtained and, for this reason, only the development of Model A is presented herein. For more details, please refer to (Coelho, 2022).

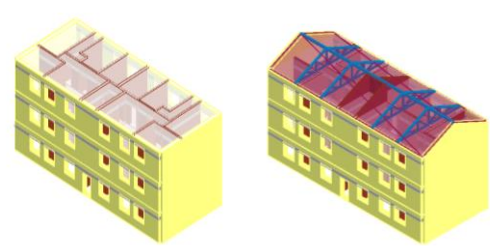


Figure 3 - Three-dimensional model of Model A (left) and Model B (right).

2.1 Material properties and loads

For the definition of the walls of the building it was necessary to define the properties of ordinary stone masonry, solid brick and hollow brick. For the first two, it was considered the average values of the properties referred in the Italian Regulation (NTC, 2018), and for the last one, the values from the reference work (Miloševic, 2019). For the definition of the properties of the reinforced concrete beams present in the walls, Part 1-1 of Eurocode 2 (EN 1992-1-1, 2004) was used.

As indicated in Part 3 of Eurocode 8 (NP EN1998-3, 2017) the average values of the material properties have to be decreased by a confidence factor, which is defined as a function of the level of knowledge of the structure in question. As no in situ tests were performed on the building structure, the most conditioning level was considered - limited knowledge (KL1). Therefore, the values of the properties were decreased by a confidence factor of 1.35. Table 1, 2 and 3 summarise the properties admitted for the constituent materials of the walls and beams modelled in 3Muri.

Table 1 - Masonry properties.

Material	f_c [Mpa]	τ [MPa]	E [GPa]	G [GPa]	ω [KN/m³]	v (-)
Rubble Stone	1.50	0.025	0.87	0.29	19.0	0,2
Solid Brick	0.345	0.09	1,50	0,50	18	0,2
Hollow Brick	0,166	0,028	2,96	0,98	18	0,2

E -young modulus; G - shear modulus; v – Poisson coefficient; ω- specific weight; f_c – average compressive strength; τ - shear strength.

Table 2 - Concrete properties.

Material	f_c [Mpa]	f_k [Mpa]	E [GPa]	G [GPa]	ω [KN/m³]	v (-)
Reinforced concrete	32,40	16,00	29	12	25	0,2

E -young modulus; G - shear modulus; v – Poisson coefficient; ω - specific weight; fc – average compressive strength; fk - characteristic compressive tension.

Table 3 - Steel properties.

Material	f_{ym} [Mpa]	E [GPa]	G [GPa]	ω [KN/m³]	v (-)
Steel	235	210	81	79	0,2

E -Young modulus; G - Shear modulus; v – Poisson coefficient; ω - Specific weight; f_{ym} – yield stress

For the definition of the concrete slabs it was used the Part 1-1 of Eurocode 2 (EN 1992-1-1, 2010, which are illustrated in Table 4. The same table also shows the properties defined for the pine wood, proposed by (LNEC, 1997) for the elasticity module and by the New Zealand Regulation (New Zealand Society of Earthquake, 2017) for the distortion modulus, and the rest was defined according to the work (Miloševic, 2019).

Table 4 - Material properties of the pavement constituent materials.

Material	e (cm)	E (GPa)	G (GPa)	ν (-)	w (kN/m^3)	f_c (MPa)	f_k (MPa)
Pine	2,2	12	0,012	-	5,8	18,0	18,0
Concrete	10,0	29	12	0,2	25,0	32,4	16,0

e – thickness; E -young modulus; G - shear modulus; v – Poisson coefficient; ω - specific weight; fc – average compressive strength; fk -characteristic compressive tension.

2.2 Applied loads and masses

For wooden floors, reinforced concrete slabs, balconies and roofs, the permanent load of the respective elements was insert into the programme. However, for reinforced concrete walls and beams, 3Muri automatically calculates the load from the volume weight and its geometry.

The stairs were not modelled in the program, but the corresponding overload was applied to the floors. The same happens in the case of the balconies, where a corresponding linear load was applied in the window area.

For the roof, linear loads along the facade walls representing the weight of the roof structure and linear loads corresponding to the weight of the gable walls and stairwells, which support the roof.

Table 5 summarises the loads applied to the building.

Table 5 - Applied loads.

Field of application	Permanent load (kN/m^2)	Overload (kN/m^2)
Reinforced concrete slab	3,3	2
Wood flooring	1,1	2
Balcony	3,3	3
Stairs	1,1	4
Roof	1,1	0,4

3 Seismic analysis

As recommended in Part 3 of Eurocode 8 (NP EN1998-3, 2017) a global evaluation of the building was performed using 3Muri. More specifically a modal analysis, where the fundamental vibration modes and the model's vibration frequencies are obtained, and non-linear static analyses (pushover), where the resistant capacity curves of the structure and its ultimate displacement are obtained. A sensitivity analysis to the properties of the building's constituent materials was also studied. Local mechanism analyses were performed, where collapse mechanisms were evaluated through a non-linear kinematic analysis.

3.1 Modal Analysis

From the dynamic modal analysis, the vibration modes and frequencies of the structure presented in table 6 were obtained and are, illustrated in figure 4. The choice of the main modes is made based on the mass participation factors, so the modes with the highest percentage of mass in each direction (X and Y) are the most representative of the building behaviour.

Table 6 - Periods, frequencies and mass participation factors (M_x and M_y), relative to the main vibration modes

Vibration mode	Period [s]	Frequency [Hz]	M_x [%]	M_y [%]
1	0,24	4,1	83,2	0
3	0,19	5,2	0,02	79,4

It is possible to conclude that the first vibration mode corresponds to the fundamental mode of the structure in the X direction (direction of the facade walls) with a mass participation in this direction of 83,2%. It presents a frequency of 4,1 Hz and represents a pure translation mode in the X direction, due to the mass factors in the Y direction having null participation.

The third vibration mode corresponds to a pure translation in Y direction (direction of the gable walls), being negligible the value of mass participation in X direction, compared to the participation of 79.4% in Y direction. This mode presents a frequency of 5.3 Hz.

As expected, the building presents a lower frequency in the direction of lower stiffness (X direction), this is because the walls are longer in the X direction and there are several openings.

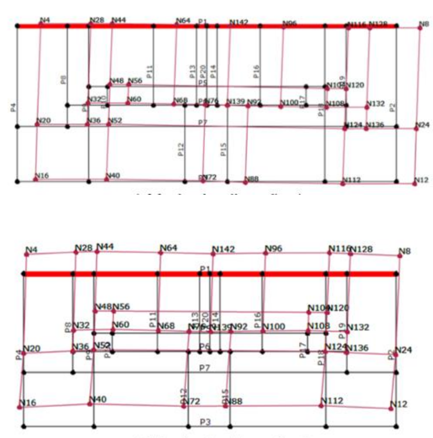


Figure 4 - Translation mode in X direction (up) and in Y direction (down).

3.2 Non-linear static analyses (pushover)

In pushover analysis, static lateral forces are applied, representative of the seismic inertia forces, which are successively increased, maintaining the same loading pattern, until the maximum base shear force is reached and, and progressively reduced to determine the ultimate displacement of the structure (Simões et al., 2014).

For the definition of the ultimate displacement (du) of the structure, associated with imminent collapse (NC) limit state, two criteria are considered. The first detailed in Part 3 of Eurocode 8 (NP EN1998-3, 2017) and in the Italian standard (NTC, 2018), which considers the value of the ultimate displacement when the basal shear force (Fb) reaches a reduction of the order of 20%, this happens in structures in which there is some ductility and a progressive degradation of strength. The second criteria applies when there is the formation of a partial mechanism, the fall of the basal shear force is abrupt, and in these cases the ultimate displacement assumes the value just before the abrupt fall.

The value of the displacement considered for the safety verification is influenced by the limit state considered, which, in the case of the building under study with importance class II, the state of severe damage (SD) is considered. Thus, the value obtained through the criteria described above, associated to the limit state of imminent collapse (NC), is reduced to ¾ of the value obtained.

Two different loads were applied to the structure, the uniform load, as indicated in Part 1 of Eurocode 8 (NP 1998-1, 2010), whose load is proportional to the mass of the building and independent of the height and the pseudo-triangular load, proportional to the product between the mass and the height of the building, recommended by several researchers studying the behaviour of masonry walls (Lagomarsino et al, 2013).

3.2.1 Capacity curves

Through 3Muri it was possible to obtain the displacement values for the severe damage limit state (d_m) and the objective displacement (d_t) in order to calculate the ratio between them and verify if it is less than one, which dictates that the seismic safety of the building is verified. Also through this division it is possible to access what is the best control node to perform the analysis in each direction. With the help of the damage maps of the walls, it was defined node 4 and node 12 as the control node of the X and Y directions respectively. In any case, the pushover curves considered for the seismic assessment are defined in function of the average displacement of all nodes, of the last floor, weighted by the mass of each node, as this is a better option for buildings with flexible floors

The graph in Figure 5 shows the resistant capacity curves obtained for Model A, from which it can be concluded that for any type of loading, the Y direction (curves in green tones) presents higher stiffness and base shear force (and consequently a higher resistance) compared to the X direction. This can be justified by the fact that the resistant walls in the Y direction do not present openings and have shorter lengths than in the orthogonal direction. Thus, it is possible to state that it is in this direction that the building presents greater resistant capacity.

Although there is not a very expressive difference, the Y direction is also the one that presents the highest ductility; this is perceptible through the comparison of the capacity curves, with the same applied loads, where the plastic phase is higher in this direction. As in direction X the walls present mostly a damage pattern caused by shear, the ductility value is reduced.

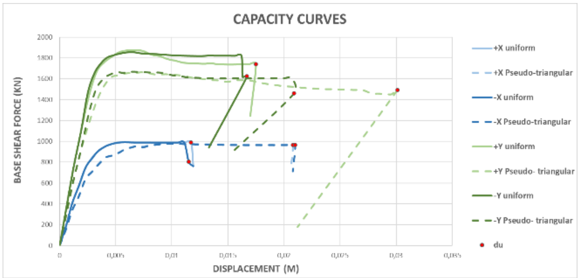


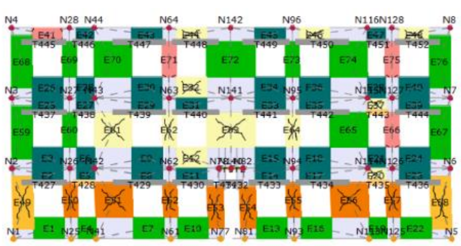
Figure 5 - Bearing capacity curves for the X direction (Node 4) and Y direction (Node 12), in both directions and for the different types of loading. Representation of the ultimate displacement.

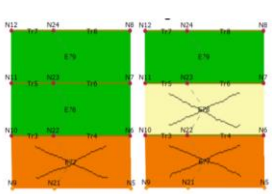
Table 7 shows the most conditioning values of the last displacement.

Table 7 - Ultimate displacements (in m) of the building under study.

Analysis	X uniform		Y triangular	
Sentido	Posit.	Negat.	Posit.	Negat.
d_u	0,0117	0,0115	0,0301	0,0209
$d_m = d_u \times \frac{3}{4}$	0,0088	0,0086	0,0226	0,0157

Figure 6 presents the damage maps for the back facade wall and for the gable walls for the ultimate displacement when respectively the uniform and pseudo-triangular load are applied. In the facade wall, the applied load causes a soft storey mechanism at floor 1, caused by shear failure and in the remaining floors some damage by bending has been identified. In the gable walls there is also the collapse by shear of the first floor.





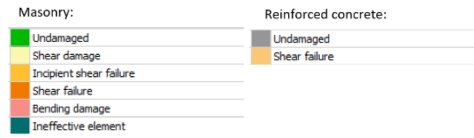


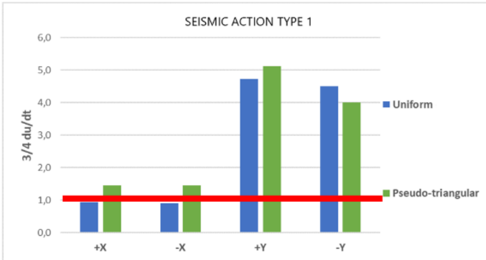
Figure 6 - Damage pattern of the back facade wall (top) and gable walls (bottom) for the conditioning loads.

3.3 N2 Method - Safety Verification

The N2 method recommended in Annex B of Part 1 of Eurocode 8 (NP 1998-1, 2010) was developed by (Fajfar, 2000) and is the recommended method for the evaluation of the structure performance.

In the N2 method, it is necessary to transform the capacity curves of a system of multiple degrees of freedom (MDOF) to a capacity curve of an equivalent single degree of freedom (SDOF) system, in order to obtain the bilinear curve equivalent to the capacity curve. Thus, it is possible to intersect the bilinear capacity curves with the elastic response spectra of the seismic action, in the acceleration-displacement response spectrum (ADRS) format, to obtain the target displacement (d_t). According to Part 3 of Eurocode 8 (NP EN1998-3, 2017) the value of the ultimate displacement, calculated by criteria 1 and 2, referred above, needs to be multiplied by ¾ to obtain the verification of the severe damage limit state (SD).

The safety is only verified when the ratio dm/dt value is greater than 1, which means that the displacement of the structure for the severe damage limit state (SD) must exceed the displacement imposed by the seismic action on the building. Figure 7 shows the safety verification of the heritage building under study, according to the d_m/d_t ratio, for seismic action type 1 and type 2.



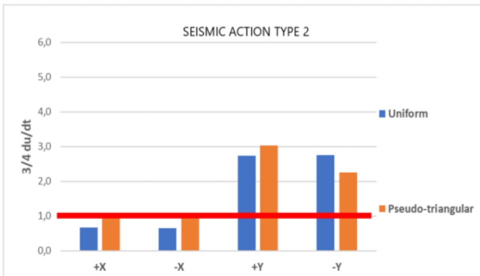


Figure 7 - Ratio between ¾ of the ultimate displacement and the target displacement for the different analyses for the type 1 earthquake (top) and type 2 earthquake (bottom).

4 Sensitivity Analysis

The sensitivity analysis allows for a more adequate choice of the confidence factor to be applied to the material properties and eventually to reduce the invasive tests, if these are dispensable (Lagomarsino & Cattari, 2014). The objective of the sensitivity analysis is to identify the parameters that most affect the seismic capacity of the building, and consequently its structural performance in the event of earthquakes.

In this case study, each type of masonry represents a study group, for which the values of some properties are varied. The following properties were considered: modulus of elasticity (E), shear modulus (G), compressive strength (fc) and shear strength (τ) for the masonry. The interval of values specified for ordinary stone and solid brick were defined according to the Italian Standard (NTC, 2018) and the hollow brick according to the reference work (Miloševic, 2019), in a total of 3 groups of variables. Another group associated with wooden floors was considered where only the shear modulus value was varied (G) according to a range considered acceptable to simulate floors with flexible and rigid behaviour. Table 8 and 9 presents the four groups of random variables and the respective minimum, mean and maximum values assigned to the mechanical properties of the material in question.

Table 8 - Mechanical properties of materials.

Material	Group	Variables	Min. value [N/mm]	Average value [N/mm]	Max. value [N/mm]
Ordinary Stone	G_1	E	690	870	1050
		G	230	290	350
		f_c	1	1,5	2
		τ	0,018	0,025	0,032
Solid brick	G_2	E	1200	1500	1800
		G	400	500	600
		f_c	2,6	3,45	4,3
		τ	0,05	0,09	0,13

E -young modulus; G - shear modulus; fc – average compressive strength; τ - shear strength

Table 9 - Mechanical properties of materials.

Material	Group	Variables	Min. value [N/mm]	Average value [N/mm]	Max. value [N/mm]
Hollow brick	G_3	E	2300	3015	3730
		G	770	1005	1240
		f_c	1,4	1,645	1,89
		τ	0,24	0,28	0,32
Wood flooring	G_4	G	12	56	100

E -young modulus; G - shear modulus; fc – average compressive strength; τ - shear strength

3Muri runs 2N+1 analyses, where N represents the number of parameter groups. In the first analysis, the average values of all random parameters are admitted as reference. While in the remaining 2N analyses all parameters maintain the mean value, except for one group, which will take the maximum or minimum value of the interval defined in the previous table. Thus, successively until all the parameters are run.

Figure 8 presents the results obtained from the sensitivity analysis performed in 3Muri for seismic action type 1.

It is notable that group G1 is the one that suffers the greatest variation with the alteration of the parameters. In other words, it is the one that presents the greatest sensitivity both in cognitive analysis and in improvement analysis.

Therefore, it is concluded that for the adequate definition of ordinary stone masonry, present in the facade and gable walls, it is important, a higher knowledge of the mechanical properties, to perform the seismic evaluation with higher accuracy. Hence, it is advisable to perform experimental tests applied directly to the building.

According to the improvement analysis, the ordinary stone masonry walls are also the ones that present the greatest need for structural reinforcement, especially the gable walls. As this direction verifies the seismic safety for all the analyses, their reinforcement is not considered necessary, however, the same does not happen with the facade walls. The same occurs for the properties of solid brick and hollow brick masonry when compared with those of ordinary stone masonry.

The group G4, referring to the shear modulus (G) of the wood floors, presents low cognitive sensitivity, therefore, it is not considered necessary an intensive study on this property, since it does not affect in almost anything the response of the structure. However, increasing the distortion modulus (G) as a reinforcement solution for wood floors may bring some benefit in the response of the structure in the Y direction, considering the obtained improved sensitivity index.

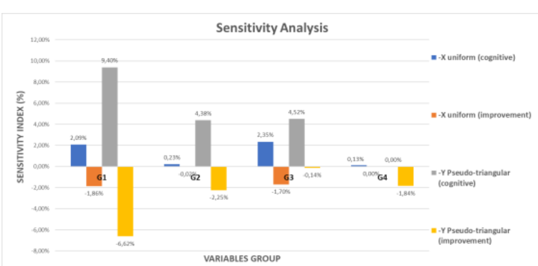


Figure 8 - Results of the cognitive and improvement sensitivity analysis, according to the most conditioning scenarios, in X and Y direction.

5 Local Mechanisms

The masonry buildings when subjected to seismic actions may also present out-of-plane collapse of the walls. For this reason, in this paper, local mechanisms analysis was performed. The studied is based on limit analysis and considers that the connections between floors and walls are not adequate, contrary to what was admitted in the analysis of the global behaviour of the building in the non-linear static analyses.

The 3Muri performs the safety verification according to the Italian Regulation (NTC, 2018), where it is stated that the verification it is assured if the seismic evaluation factor (α) is greater than 1.

This factor is calculated by dividing the spectral seismic acceleration of activation of the mechanism $(a\_0^\wedge *)$ and the minimum value of the same acceleration for the imminent collapse limit state (NC) amplified to the elevation of the mechanism $(a\_(0-min)^\wedge *)$.

Figure 9 shows the four mechanisms studied.

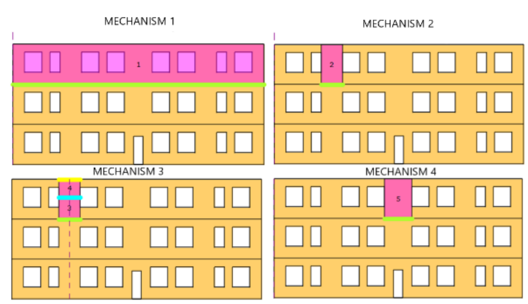


Figure 9 - Collapse mechanisms identified in the late façade wall.

From table 10 it can be concluded that only for mechanism 3 the safety of the structure is assured; this can be justified by the fact that, for this mechanism to happen it would require much more energy than for the others. For the block considered to rupture in the middle, it would be necessary to have a very strong connection between the masonry wall and the roof, which is not likely to happen in the structure. While the other mechanisms depend on weaker connections between the walls and the roof and are therefore more fragile, which compromises the safety of the building.

Table 10 - Safety verification of collapse mechanisms through non-linear kinematic analysis.

Mechanism	α	Security
1	0.16	Does not check
2	0.16	Does not check
3	7.44	Check
4	0.17	Does not check

6 Reinforcement

The reinforcement aims at minimising structural damage caused by seismic action, increasing the lateral resistance of the building and the deformation capacity and, in this way, satisfying the regulatory requirements for the verification of structural safety.

It was decided to choose reinforcement techniques of a less invasive character, so that they would not cause an excessive increase in the weight of the structure or the need to reinforce the foundation elements. Therefore, the techniques implemented were: lime injection, carbon mesh and introduction of tie rods in the walls. A recurrent intervention solution in masonry buildings was also studied, which consists in the addition of a concrete layer on the floors. This proved to be ineffective in relation to the seismic safety, presenting results similar to those of Model A, consequently, they are not presented.

After the implementation of the reinforcement, the seismic evaluation is carried out, again, through modal analysis and non-linear static analysis, regarding the behaviour of the walls in their own plane, and local mechanism analysis to predict the behaviour of the walls out of their plane.

The reinforcement made through the injection of lime in the facade walls presents a great efficiency to the seismic safety. All the analyses carried out verify the seismic safety, presenting a ratio d_m/d_t ratio of 1.15 and therefore higher than 1.

The reinforcement with carbon mesh was applied in 3Muri through the application of a FRCM mesh (inorganic matrix) only by the exterior of the facade walls at floor 1 of the building, with 44mm2/m and 0.044 mm thickness; these values are based on the standard (CNR-DT 200 R1/2013, 2014; CNR-DT 215-2018, 2020).

Figure 10 compares the capacity curves obtained for Model A and Model E, where this reinforcement technique was applied. It is concluded that the X direction presents a slight increase of the maximum basal shear force, which translates into an increase of strength and stiffness, and an increase of the top displacement values, which means an increase of ductility of the structure. All analyses verify seismic safety, presenting a minimum ratio d_m/d_t ratio of 1.35.

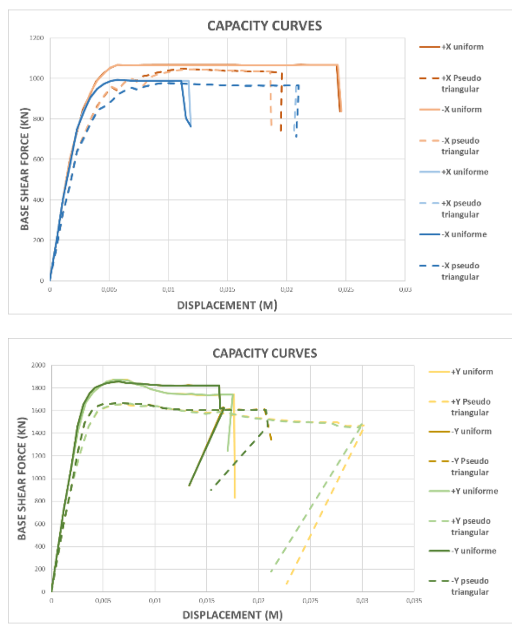


Figure 10 - Resistance capacity curves obtained for Model E (in orange and yellow) and Model A (in blue and green) in the X direction (top) and Y direction (bottom).

As for the local analyses, the quantity of tie rods and its respective force were applied by an iterative process, resulting in an optimized value of 55kN. It was applied one tie rod in the mechanism 2 and 4 and seven tie rods in the mechanism 2. For the mechanisms where the seismic safety was not verified before, the results are presented in table 11.

Table 11 - Safety verification of collapse mechanisms after strengthening through non-linear kinematic analysis.

Mechanism	α	Security
1	1.01	Check
2	2.53	Check
4	1.29	Check

7 Conclusions

This work focused mainly on the analysis of the seismic vulnerability of a typical "placa" building from the 40's built in the city of Lisbon. The building studied consists of a structure with masonry walls, floors and roof with wooden beams. There are also some reinforced concrete elements, namely slabs and beams.

Two three-dimensional models (Model A and Model B) were developed in the 3Muri calculation program. In Model A the roof is not structurally modelled and in Model B it is modelled using the Roof tool of the programme. It was concluded that the structural modelling of the roof does not add significant value to the results obtained (Model B), so as simplification, the roof effect may be considered only through its load.

It was verified that the Y direction (direction of the gable walls) presents higher values of stiffness and resistance. This behaviour is justified by the fact that the walls in Y direction do not present openings. It is also in this direction that the building presents greater ductility, since the damage pattern in the X direction is mostly due to shear, which does not allow such high ductility values to be achieved. The safety verification performed by 3Muri is not satisfactoryfor the two analyses performed for the X direction. Thus, it can be concluded that the Y direction exhibits a better behaviour to horizontal forces.

Through Part 1 and Part 3 of Eurocode 8 and the correspondent National Annexes, it is concluded that seismic safety is not verified for the X direction, for two analyses performed for seismic action type 1 and three analyses for seismic action type 2. It was also verified that seismic action type 2 is the most conditioning.

The sensitivity analyses performed show that the mechanical properties of ordinary stone masonry walls are the ones that present the highest sensitivity in the cognitive analysis and, therefore, there is a need to improve the knowledge of their mechanical properties, through experimental tests to be performed on the stone masonry walls, to obtain more accurate results in the seismic analysis. On the other hand, the properties of irregular stone masonry walls are also those that present greater sensitivity in the improvement analysis, thus identifying the need to study their structural reinforcement.

In addition, the seismic assessment of the masonry building was also analysed using expedited methods published technical notes applicable, respectively, to buildings with rigid floors and buildings with flexible floors. It was concluded that the seismic safety is not verified for any of the methods applied, thus the results are consistent with the application of more refined analysis methods (nonlinear static analyses).

The study of the local behaviour indicated the safety is not verified, with only one mechanism satisfying the safety requirements; nevertheless, this mechanism will only occur if strong connections between the walls and floors exist, which is not expected to happen.

Finally, several strengthening solutions were applied to Model A, where it was concluded that the addition of a concrete layer on the floor does not present an effective improvement of the seismic performance of the building. On the other hand, both the injection of lime in the façade walls and the introduction of carbon mesh cause a significant increase in the resistance and capacity of deformation of these walls, thus allowing the building to satisfy the code safety requirements.

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