Seismic Analysis and Strengthening Proposals of a Masonry Building

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October 2021

Abstract

The Building stock in Portugal consists largely of old masonry buildings considered susceptible to seismic actions. Among the various construction typologies, the "Gaioleiro" typology in Lisbon is the most vulnerable; therefore, it will be the subject of this study.

Portugal has suffered numerous earthquakes throughout its history, and since new earthquakes are expected to occur, it became mandatory in 2019 to prepare a seismic vulnerability assessment report for buildings that undergo structural rehabilitation. It is therefore essential to know the procedures to be followed for the seismic assessment, and possible strengthening, of existing masonry buildings.

This dissertation evaluates the global and local seismic vulnerability of a structure representative of a "Gaioleiro" building, located at Avenida Duque de Loulé nº 70. The geometric and dynamic characterization, the structural elements, and the properties of the materials are defined based on studies carried out by M. Branco in 2006 and 2007.

Based on the information collected, the software 3MURI is used to develop an equivalent frame model. The calibration of the developed model considers the results of the ambient vibration tests developed by Branco in the aforementioned studies and the influence of the adjacent building on the behavior of the structure. Thus, for the seismic evaluation of the building, a non-linear static (pushover) analysis and a non-linear kinematic analysis of the local out-of-plane collapse mechanisms are performed.

Keywords: "Gaioleiro" Buildings; Seismic Assessment; Nonlinear Static Analysis (pushover); N2 Method; Nonlinear Kinematic Analysis; Seismic Retrofit.

1. Introduction

The need to assess the seismic performance of buildings built with structural masonry walls is of high importance, since masonry walls are responsible for supporting both vertical loads and ensuring stability in the event of lateral actions, (Lourenço et al., 2015). Masonry buildings represent around 34% of the housing stock in Lisbon and reach 50% nationally, of which it is estimated that around 50% require some type of intervention. Decree-Law No. 95/2019 (Decreto-Lei, 2019) made it mandatory to prepare a seismic vulnerability assessment report for buildings undergoing structural rehabilitation. Thus, it is essential to define analysis methods applicable to masonry buildings for the seismic assessment. The main objective of this dissertation is the seismic evaluation of a masonry building representative of the "gaioleiro" typology, to identify possible weaknesses and analyze the applicability and performance of seismic retrofit.

2. Old Buildings

Portugal throughout its history has suffered numerous earthquakes. The 1755 earthquake, which mainly affected the city of Lisbon, was the event that marked the urgent need to create measures to mitigate the material and human damage caused by earthquakes, which led to the creation of regulations and new construction techniques (Simões and Bento, 2012).

In 1755 the “Pombalino” era began and spearheaded a revolution in terms of the seismic resistance of buildings, quality of materials and land use planning, which lasted until the mid-19th century. Then came the “gaioleiro” period between the end of the 19th century and the beginning of the 20th century. The 40’s marked a revolutionary era in terms of construction, the era of buildings with a mixed structure of wooden and/or reinforced concrete floors. Soon afterwards, in the 40’s, 50’s and 60’s, the construction of buildings with a reinforced concrete structure started and continues until today.

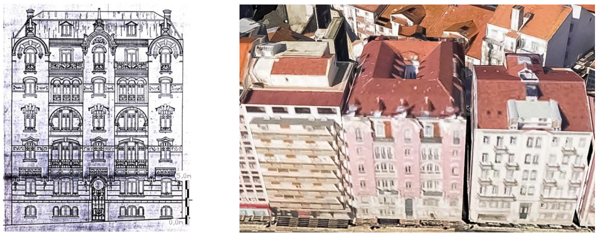
3. Case Study

In this work, it was decided to analyze the constructive typology “gaioleiro” in Lisbon as it presents significant seismic vulnerabilities. The building located on Avenida Duque de Loulé nº 70 was selected as the target for study, since there are studies carried out by Branco (2006 and 2007) on geometric and dynamic characterization in addition to the quantification of structural masses and loads.

In 2017, the building under analysis was demolished to make way for a new building. The gables, main and back façades were preserved.

3.1. Building characterization

The building had a rectangular floor plan and was deployed in an area of 537.6 m2, with the main facade (Figure 3.1a) and rear façade measuring about 19.2m, and the gables measuring 28m, which form a perimeter of approximately 94.4m. It was distributed over seven floors arranged by basement, ground floor, four elevated floors and a mansard. The left gable leans towards a more recent reinforced concrete building and on the right gable side there was a pedestrian access to the street. The configuration can be seen in Figure 3.1b.



(a) Main facade (Serra, 2004)

(b) View of the main facades (Google Earth Pro)

Figure 3.1: Building view

In this building there were two side airshafts, in the middle of the gables and a central airshaft, characteristic of “gaioleiro” buildings as they are essential for the provision of natural lighting and ventilation to the interior compartments. There were 3 staircases in the building: the staircase at the entrance door that gave access to the ground floor; the main staircase that connected the different floors; and the service staircase located on the back.

As the central part of the construction of masonry buildings, masonry walls are responsible to resist vertical loads, specifically gravitational loads, and horizontal forces such as wind and earthquakes. To give it strength, the walls are made up of rigid and heavy elements, in which tensile strength is neglected (Appleton, 2003).

There were two types of resistant walls in the building under study: irregular stone masonry walls that corresponded to the exterior walls of the facades, gables and airshafts; hollow brick masonry walls that matched the subdivision walls of the basement, as well as those that were present in the service staircase. The partition walls from the ground floor and upper floors were “tabique” walls made with a light timber structure.

The roof was made up of wooden trusses and a wooden slat, on which Marseille-type ceramic tiles were laid.

4. Numerical Modelling and Model Calibration

4.1. Nonlinear Modeling: Equivalent Frame

The numerical model was defined in the software 3MURI 12.6.2.8 (S.T.A.DATA, 2018), developed by S.T.A. Data and the University of Genoa, to obtain the response of the structure considering the nonlinear behavior.

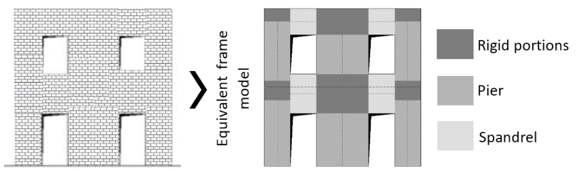


Figure 4.1: Wall idealization according to equivalent frame models, adapted from (Lagomarsino et al., 2013)

It uses the equivalent frame model (Figure 4.1) by using macro-elements, which are divided between the main (piers) and secondary (spandrels) elements. In these elements, the deformation is concentrated and the non-linear response is considered. Rigid portions are non-deformable elements which are generally undamaged. They connect deformable elements and are also responsible for transferring static and kinematic variables between elements, (Lagomarsino et al., 2013).

According to Lagomarsino et al. (2013), there are two main typical behavior patterns of masonry panels subjected to in-plane loading: the bending-conditioned behavior, which is the rocking/crushing mechanism, and the shear-conditioned behavior divided between bed joint sliding and diagonal cracking collapse mechanisms. The mixed failure mode that combines the behavior of bending and cutting is also frequent.

4.2. Material Characterization

Eurocode 8 Part 3 (EC8-3) (IPQ, 2017) specifies that to carry out the seismic assessment of an existing building it is necessary to obtain a certain level of quality of information about the geometry, construction arrangements and materials, which are the fundamental factors to determine the level of knowledge.

In this case, *in situ* visits and dynamic characterization tests were carried out by Branco (2006 and 2007) to determine the structure’s frequencies and modes of vibrations. Together they also provide information about the geometry and constituent materials, essential for the calibration of the numerical model.

In this work, the limited knowledge level, KL1, was admitted, due to the uncertainties in the mechanical characteristics of the materials; in this case the confidence factor takes the value 1.35 (IPQ, 2017).

To define the building wall materials in 3MURI software, it is necessary to introduce the values of: modulus of elasticity (E), modulus of distortion (G), which according to (MIT, 2019) corresponds to $G = E/3$, volumetric weights (w), compression strength (f_m), shear strength (τ), which according to (MIT, 2019) corresponds to $\tau = f_t/1.5$ where f_t is the tensile stress, confidence factor (FC), safety factor (γ_m), shear drift and bending drift.

The wooden floors were modeled in the 3MURI software under the designation One-way timber floor with overlapped wood planks, the terrace floor, was modeled as Steel-beam and vault. The 3MURI software converts the floor geometry into an orthotropic membrane with equivalent properties presented in Table 4.1.

Table 4.1: Parameters calculated by the 3MURI software for floors.

floor	Equivalent thickness [cm]	Modulus of distortion $G_{equivalente}$ [N/mm^2]	Modulus of elasticity X E_X [N/mm^2]	Modulus of elasticity Y E_Y [N/mm^2]
wooden	2.2	12.00	33 818.18	12 000.00
terrace	4.0	18 790.00	57 833.33	0

4.3. **3MURI - Identification of difficulties**

In the construction of this structure, irregular stone masonry, hollow brick masonry and "tabique" walls were used. Due to the evolution regarding the knowledge of material properties over the years and the need for parameters to model the non-linear behavior, it was decided to change the values adopted by Branco (2006 e 2007), by new references made available in the Italian Regulation (MIT, 2019), in Simões (2018) and in the Techniques Tables (dos Reis et al., 2012).

The most relevant difficulties found during the modeling in the 3MURI software were: i) in the roof, problems were identified in terms of definition of the alignments of the trusses and the routing of the loads. Therefore, only its effect was simply modeled through linear loads, evenly distributed around the walls; ii) problems of modeling the mansard floor, due to the difficulty of modeling the sloping roof, it was decided to consider the structural walls of the mansard, through linear loads, uniformly distributed on the walls of the level below; iii) since the wall alignment has no continuity to the base the routing of vertical loads between the elevated floors and the ground floor was only possible with the use of beam elements at the basement level aligned with the plan of the elevated levels; iv) avoid openings close to perpendicular walls to ensure correct formation of the mesh of elements.

4.4. **Model Calibration**

To guarantee that the proposed model represents the real behavior of the building, the ambient vibration tests carried out *in situ* by Branco (2006) were considered. Calibration is completed when it is possible to ensure that the eigenfrequencies of the model obtained through modal analysis resemble the fundamental frequencies and modes of vibration of the structure obtained experimentally.

Through the dynamic modal analysis performed in the 3MURI software, the structure presents a first vibration mode with pure translation in the X direction. The vibration mode with the highest mass participation in the Y direction is coupled to a translation in X, therefore it corresponds to a torsional mode.

Table 4.2 shows the comparison between the numerical eigenfrequencies and the experimental frequencies. Due to the result of the modal analysis that presents errors greater than 10%, it is necessary to make changes to the model the model.

Table 4.2: Modal analysis results for the isolated model

Vibration mode	Frequency experimental [Hz]	numerical [Hz]	Mass participation X [%]	Y [%]	Error [%]
x translation	2.34	0.74	27.13	0	214.87
z twist	2.83	2.19	1.11	50.91	29.29

4.4.1 Model Validation

Initially, it was decided to reduce the volumetric weights of the masonry because it is a "gaioleiro" building, known for its constructive flaws and inferior materials. Therefore, the existence of possible voids in the walls was considered. For the irregular stone masonry it was considered the volumetric weights of 16 kN/m^3 and for the hollow brick masonry 12 kN/m^3.

For the "tabique" walls, $G = 0$ was admitted in order to model them as a secondary element that does not contribute for the lateral resistance. Furthermore, when analyzing the structural damage only for the vertical loads, failure by compression was observed in some elements, so it was necessary to admit a higher compressive strength than that established in the study (Simões, 2018) used to characterize the partitions. Through an iterative process it was verified that for 95 N/cm^2 the compressive strength is satisfied.

In addition, it was concluded that the consideration of the envelope is extremely important, so the adjacent reinforced concrete building was modeled in a simplified way until the numerical frequency values were close to the experimental values.

Table 4.3: Modal analysis results for the calibrated model

Vibration mode	Frequency experimental [Hz]	Frequency numerical [Hz]	Mass participation X [%]	Mass participation Y [%]	Error [%]
x translation	2.34	2.58	46.58	3.09	9.36
z twist	2.83	2.49	4.21	33.38	13.66

Table 4.4 shows the final mechanical properties used in modeling the materials of the walls and Table 4.3 presents the results of the modal analysis and percentage of error in relation to the experimental values for the calibrated model.

5. Seismic Assessment

The 3MURI software performs the global and local analysis independently, so in section 5.1 the global building response analysis will be performed by non-linear static (pushover) analysis according to the EC8-3 (IPQ, 2017), and later in section 5.2, the local analysis, non-linear kinematic analysis, will be presented according to the Italian Regulation (MIT, 2019).

According to Ordinance No. 302/2019 (Portaria, 2019), article 1, Point 3, the seismic performance assessment must be carried out for only 90% of the seismic action, and if safety is not verified, the definition of proposals for seismic strengthening is mandatory. For the evaluation of local performance, the reduction of seismic action was also considered.

5.1. Global Analysis

According to EC8-3 (IPQ, 2017) being an ordinary residential building, it is classified as class of importance II and therefore with regard to the damaged status of the structure it only needs to check the severe damage limit state (SD).

Nonlinear static analysis simulates through the imposition of inertial forces the effect of seismic action on the structure in order to define its overall resistant capacity. Inertia forces are applied asynchronously in the two main directions, X (longitudinal) and Y (transverse), and in both senses of direction, positive (+) and negative (-), with monotonic growth. Eurocode 8 Part 1 (IPQ, 2010) indicates the use of at least two vertical distributions of lateral loads, the uniform distribution, in which the lateral forces are proportional to the mass and independent of the height of the floors, and the modal distribution, proportional to the main vibration mode in the direction of the analysis. However, for masonry structures, due to the low mass participantion of the main modes of vibration, the pseudo-triangular (here static forces) distribution is recommended, which is proportional to the product of mass and height.

5.1.1 Capacity curves

The pushover curve is an intrinsic characteristic of the structure, regardless of the seismic ac-

Table 4.4: Mechanical properties of the constituent elements of the walls

	Modulus of elasticity E [N/mm^2]	Modulus of distortion G [N/mm^2]	volumetric weights w [kN/m^3]	compression strength f$_m$ [N/cm^2]	shear strength τ [N/cm^2]	source
Irregular stone masonry	1050	350	16*	200	3.2	(MIT,*2009 and 2019)
Hollow brick masonry	1800	600	12*	430	13	
Tabique wall	200	0	1.35**	95	1	(Simões, 2018) and **(dos Reis et al., 2012)

tion to which it is subjected, it is defined by the base shear force as a function of the displacement of the structure's nodes. The curves reflect the global response to the aforementioned force distributions and provides a series of information such as: the initial stiffness of the structure; the value of the maximum base shear force that the structure supports; the deformation capacity (ductility of the structure) and the ultimate displacement capacity (Lagomarsino et al., 2013).

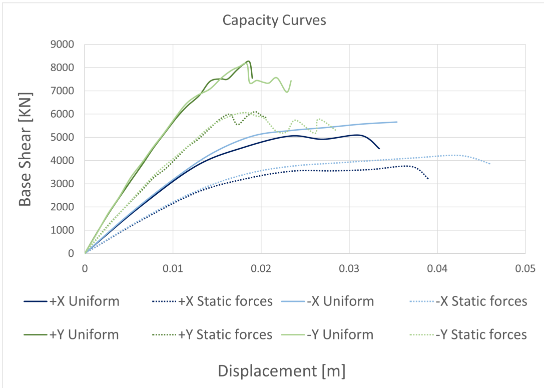


Figure 5.1: Capacity curves

Based on the results obtained in Figure 5.1, it is evident that the Y direction has greater stiffness and resistant capacity since the gables do not have any openings. The Y direction presents a more fragile behavior, except for the capacity curve in the negative direction with pseudo-triangular distribution, which presents a more ductile behavior. In the X direction, due to the openings in the main and rear façades, the structure presents greater ultimate displacement and ductility, a consequence of the various elements that allow exploring the distribution of the non-linear behavior.

When comparing the two distributions, it is possible to verify that the structure for the pseudo-triangular distribution has lower resistance capacity, since its curves are always developed under the curves with uniform distribution, however they reach higher values of ultimate displacement.

However, only through the analysis of the capacity curves it is not possible to determine which situation is the critical one. It will therefore be necessary to apply the N2 method, for the safety verification, which will be discussed in section 5.1.2.

5.1.2 N2 Method

The N2 method developed in the study carried out by Fajfar (2000) and proposed in Annex B of EC8-1 (IPQ, 2010) compares the capacity curve of a system with n degrees of freedom with a response spectrum (Fajfar, 2000).

The evaluation of structural performance is done through the control of displacements in which safety is verified if inequality $dt \leq dm(SD)$ is satisfied. This corresponds to the comparison of the displacement associated with the severe damage limit state ($dm(SD) = \frac{3}{4}dm(NC)$), with the displacement that the structure presents due to the application of seismic action for this severe damage limit state (dt), obtained through the N2 method. Here $dm(NC)$ is the ultimate displacement of the structure associated with the near collapse limit state.

According to the results presented in Figure 5.2, it is concluded that the structure does not meet the requirements of the EC8 criterion based on the N2 method for any of the situations for the type 1 earthquake. In the case of a type 2 earthquake, only the Y direction in the positive direction with static forces distribution is not satisfied. Therefore, the type 1 earthquake is considered critical, the result was expected since it is a tall and flexible structure.

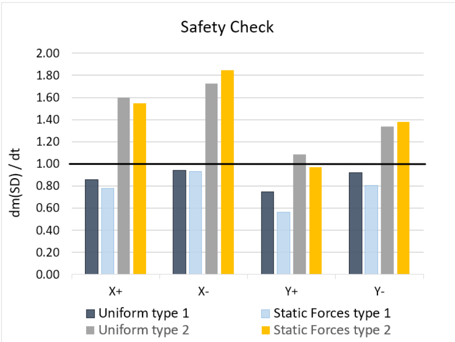


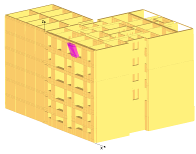
Figure 5.2: Safety Check

Since all cases must verify the safety requirements defined in EC8, it is necessary to proceed with the structural strengthening.

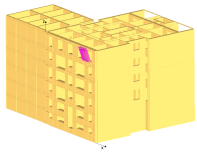
5.2. **Local Analysis**

In this section, the analysis of local collapse mechanisms related to the out-of-plane response of the façade walls caused by the occurrence of the seismic action will be addressed. Since EC8-3 (IPQ, 2017) does not address local analyses, this study will be carried out in light of the Italian Regulation (MIT, 2009). It proposes the verification of safety through geometrically non-linear kinematic analysis based on the definition of possible local mechanisms of failure associated with overturning collapse. The objective is the determination of the seismic acceleration that activates the mechanisms and compare it with the seismic action for the near collapse limit state (NC) as stated in the Italian Regulation (MIT, 2009). The analysis is performed through macroblock modeling in the 3MURI software.

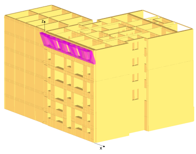
The choice of the out-of-plane collapse mechanisms shown in Figure 5.3, it is conditioned according to (Simões et al., 2020) and (MIT, 2009) by the geometry, state of preservation of the walls, poor quality or lack of adequate connections of the façade walls to perpendicular walls, roofs and intermediate floors and interaction with the adjacent buildings.



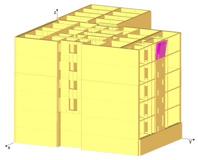
(a) Mechanism 1



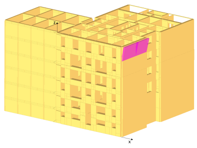
(b) Mechanism 2



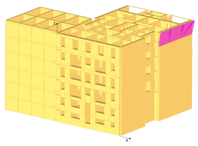
(c) Mechanism 3



(d) Mecanismo 4



(e) Mechanism 5



(f) Mechanism 6

Figure 5.3: Local collapse mechanisms

The performance evaluation of the local mechanisms can be obtained by comparing the seismic acceleration amplified to the height of the mechanism a^*_{0-min}, and the spectral seismic acceleration of activation of the mechanism a^*_0. Safety is checked for $a^*_0 \geq a^*_{0-min}$.

Through the analysis of Figure 5.4, where the performance evaluations of the local mechanisms are presented, it is concluded that the safety criteria are not met for any of the local mechanisms for both seismic actions, therefore the strengthening of the zones will be futher studied.

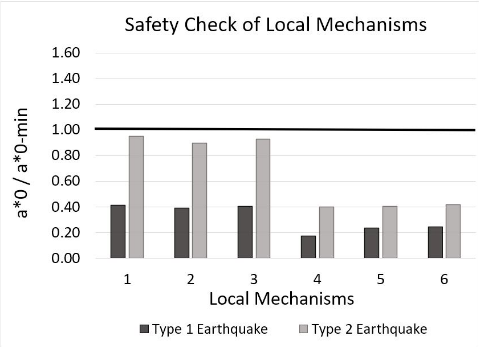


Figure 5.4: Safety Check of Local Mechanisms

6. Seismic Retrofit

6.1. Global Strengthening Solutions

As an intervention strategy to achieve the global seismic performance requirements established by EC8-3 (IPQ, 2017), it was decided to increase the deformation capacity of the structure. Two strengthening solutions are proposed: the injection of natural hydraulic lime; and the application of Fibre-Reinforced Cementitious Matrix/Mortar (FRCM).

6.1.1 Strengthening with natural hydraulic lime injection

The strengthening by injection of lime was applied according to the Italian Regulation (MIT, 2009) which suggests the adoption of a multiplicative coefficient to increase the mechanical characteristics of the masonry in order to take into account the effect of the strengthening on the walls. The ultimate displacement that leads to the collapse of the structure is conditioned by the deformation/ductility capacity which, according to Vanin et al. (2017), will be higher when the structural strengthening is applied. Therefore, the values of shear drift and bending drift must also be increased. Several irregular stone masonry walls that showed significant damage were iteratively reinforced. It was observed that the reinforced capacity curves show an increase in inclination in the elastic phase, developped over the unreinforced capacity curves and reach higher ultimate displacement values. Based on the analysis of the results (Figure 6.1), it is concluded that only the structural safety for the Y direction in the negative direction with pseudo-triangular distribution for the type 1 earthquake was not verified.

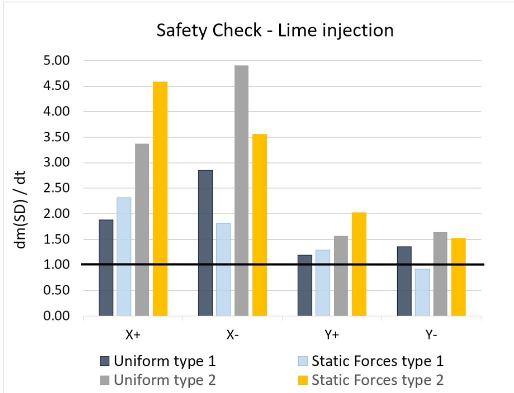


Figure 6.1: Safety Check after lime injection

It is verified that the Y direction is the most conditioning one also due to the location of the reinforced walls, which are mostly concentrated on the right side of the structure, and thus, it causes a change in the stiffness center and some irregularity in plan, which consequently aggravates the torsion effect.

6.1.2 Fibre-Reinforced Cementitious Matrix/Mortar (FRCM)

It was decided to model the FRCM in two ways: through the multiplicative coefficients (approach 1), proposed in the Italian Regulation (MIT, 2019), to increase the mechanical characteristics of the masonry; and the application of the FRCM system available in the 3MURI software (approach 2).

6.1.2.1 Modeling through multiplicative coefficients

The modeling through multiplicative coefficients (approach 1) is carried out in a similar way to the lime injection strengthening technique. For reinforced plaster, the Italian Regulation (MIT, 2019) proposes the adoption of a multiplicative coefficient. For irregular stone masonry, the results of study (Ponte et al., 2021), which addresses the application of a mesh of glass and carbon fibers in masonry walls, are used as a reference for the multiplicative coefficient that effects the bending drift. For the shear drift, Vanin et al. (2017) multiplicative coefficient is used.

After analyzing the results, it was decided to analyze the strengthening solution with glass/carbon fiber mesh since they present equivalent results.

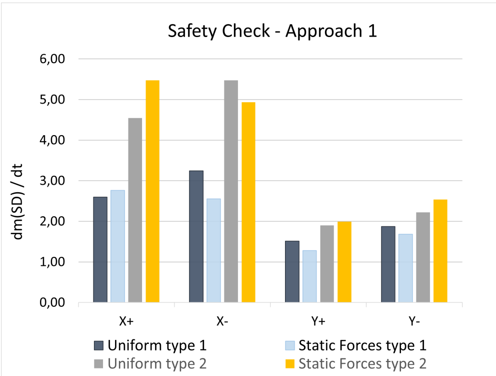


Figure 6.2: Safety Check after approach 1

After evaluating the performance by the N2 method (Figure 6.2), it is concluded that the structural safety is verified for all cases for the type 1 and type 2 earthquake. It was observed that the type 1 earthquake is the most conditioning one. For the X direction, the most conditioning situation occurs for the negative direction with static forces distribution. For the Y direction the safety is closer to not being verified, for the positive direction with static forces distribution.

6.1.2.2 Modeling through the function provided in 3MURI

For the strengthening modeling, through the function provided in the 3MURI software (approach 2), it was observed in Figure 6.3, that safety is not verified for the type 1 earthquake for three situations in the Y direction, for the two force distributions in the positive direction and for the pseudo-triangular distribution for the negative direction. Furthermore, it should be noted that the Y direction is 67% more conditioning than the X direction.

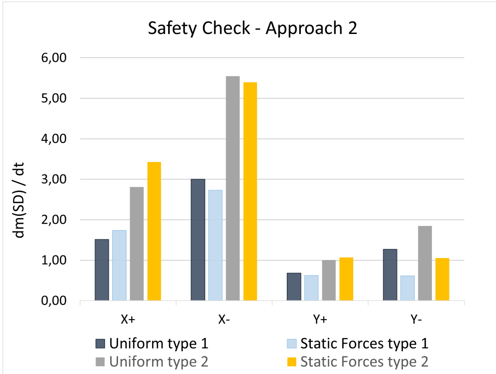


Figure 6.3: Safety Check after approach 2

6.1.2.3 Comparison between the results obtained by the two calculation approaches

It is observed that the capacity curves obtained with approach 1 present greater resistant capacity when compared with approach 2. Approach 1 also presents greater stiffness and ductility than approach 2, which has a direct impact on the verification of seismic performance. Approach 1 presents higher ratio values.

Considering the approximate way in which the multiplicative factors of approach 1 affect this problem, and since it does not take into account the specificities of the mesh, it is interesting to observe that this approach provides less conservative results. This outcome puts in evidence the importance of reassessing and refining the multiplicative proposed by the Italian Regulation (MIT, 2019).

6.1.3 Combination of strengthening solutions

As it was possible to observe in the seismic performance evaluation that lime injection strengthening techniques and Fibre-Reinforced Cementitious Matrix/Mortar (FRCM) modeled with approach 2, applied individually, do not guarantee structural safety verification. It was decided to combine the strengthening solutions that do not verify safety, that is, lime injection and application of the FRCM system by approach 2.

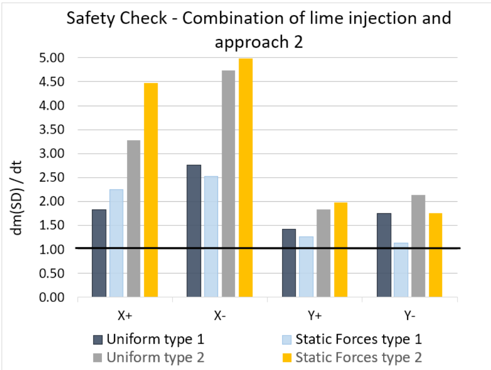


Figure 6.4: Safety Check after combination

After the seismic performance evaluation (Figure 6.4), it was concluded that the combination of strengthening results in the verification of structural safety for both types of earthquake. It should be noted that the combination of strengthening techniques makes the reinforced elements more rigid, and thus alter the structure's distribution of efforts, which in this case causes a reduction in seismic performance in some situations and an increase in others.

6.2. **Local Strengthening Solutions**

Regarding local strengthening solutions, it was decided to introduce prestressing cables in specific places, to avoid the formation of out-of-plane collapse mechanisms, with as little intervention as possible. The minimum force required to achieve the seismic performance was defined iteratively. By verifying the safety of local mechanisms through geometrically non-linear kinematic analyses (Figure 6.5), it was concluded that safety is verified for all mechanisms.

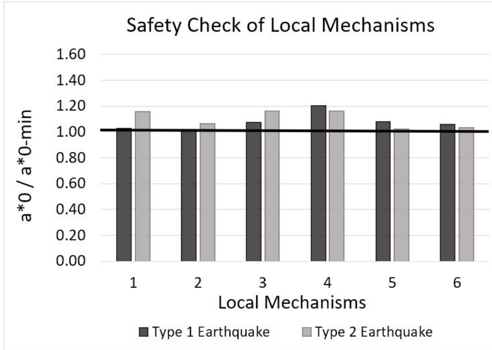


Figure 6.5: Safety Check of Local Mechanisms

7. **Conclusions**

The work presents the seismic assessment of a masonry building according to non-linear analysis procedures. After the global evaluation, it was observed that the structural safety is not verified for earthquakes of types 1 and 2. Different strengthening solutions were studied individually and combined, including the injection of natural hydraulic lime, which does not verify the seismic performance, and the application of reinforced plaster by two approaches: modeling through multiplicative coefficients, which ensure structural safety; and through the function available in the 3MURI software, which only checks safety when applied in conjunction with the injection of natural hydraulic lime.

For the local analysis, the safety of the collapse mechanisms considered was not verified. As a strengthening solution, the introdution of prestressing cables in specific places, to avoid the formation of the out-of-plane colapse mechanisms was studied.

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