Seismic Vulnerability Assessment of Buildings Structures with "*Pilotis*"

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Abstract: This work addresses the seismic vulnerability assessment of structures of buildings with "*Pilotis*", characterised by having the ground floor with increased height, almost completely open, and, consequently, tend to concentrate the damage due to seismic action in the columns of this floor. This is an international architectural trend from the 1950s to the 1970s, originally launched by the architect *Le Corbusier* and which was very well received by Portuguese architects. In addition, and due to the time of construction, these buildings were dimensioned to seismic action using inadequate procedures, according to the current knowledge, representing an identified source of risk. Thus, the main objective of this work is to analyse the safety of an existing building representative of modern architecture in Lisbon - one of the buildings of the residential complex of Infante Santo Avenue - through different methods, from the simplest to the most complex, assessing the expected damage in the building.

Keywords: Reinforced concrete buildings, “*Pilotis”*, “soft-storey”, seismic vulnerability, masonry walls.

1 Introduction

In Portugal, more than half of the buildings have reinforced concrete structures and it is estimated that 50% to 70% of these were built before the introduction and dissemination of the seismic design regulation of 1983 (RSA, 1983) [1], the first standard that contemplates seismic design in line with the current philosophy of seismic analysis [2]. It is recognized by many authors that these buildings, not seismically designed or designed based on early seismic codes, have associated structural deficiencies, such as insufficient transverse reinforcement and low concrete strength, that are reflected in high seismic vulnerability since it is expected shear failure to occur in members. This subject is of special relevance in buildings supported by “*Pilotis*”, as history shows that in seismic events around the world, these buildings have shown a deficient behaviour. In fact, the architectural configuration of this type of buildings, characterised by a main volume of great mass and stiffness that transits to flexible columns of high slenderness, generates seismic vulnerability by the discontinuity of stiffness and concentration of deformation demands in the transition zone, namely in columns with low ductility, which can enhance the failure mechanism called "soft-storey". Therefore, it seems clear that in these buildings, located in seismic zones with high exposure, as Lisbon, and designed at a time when seismic action was not taken with great concern, the problem of seismic risk is considerable. Thus, it becomes crucial to evaluate and retrofit the most vulnerable buildings to minimise losses in a future seismic event.

However, the fact that large magnitude earthquakes in Portugal have long return periods has led to a reduction in the perception of seismic risk and its consequences by the general population, thus discouraging its prevention. In fact, urban rehabilitation has been on the agenda because of the deep crisis in construction a few years ago and the growing awareness of the regulatory authorities for the preservation of the built heritage, however, without the due concern for the seismic safety of structures, focusing only on improving living conditions [3]. Only in 2019, given the legal framework constituted by Decree-Law No. 95/2019 [4] and Order No. 302/2019, was it defined in building extension, alteration or reconstruction works, when the seismic vulnerability assessment should, or should not, be carried out, as well as the conditions under which the seismic retrofitting of the building should be dimensioned. These studies must be developed in accordance with the European standards for seismic safety assessment of existing structures.

In the scope of seismic analysis and retrofitting of existing buildings, the normative reference framework is the Eurocode 8-Part 3 (EC8-3) [5], being the non-linear analyses considered the most adequate analysis method, allowing a more realistic evaluation of the structures behaviour and identifying with more accuracy its critical components, comparatively to the linear analyses. Within the non-linear analyses, the static analyses, called "Pushover" analyses, stand out due to their greater simplicity and intuitiveness. On the other hand, the proposal of expedite methods for evaluating the seismic safety of reinforced concrete buildings becomes quite opportune, in the sense of simplifying and complementing the more complex methods of seismic analysis.

In the present study, the seismic performance of a representative building of the modern architecture in Lisbon is evaluated by the following methodologies, with varying degrees of complexity: (i) Methodology LNEC/SPES [6]; (ii) Methodology ICIST/ACSS [7]; and (iii) Methodologies of non-linear analysis, to verify the performance requirements of EC8-3, using "Pushover" analysis. For the non-linear analyses, the results of two computer programmes, SAP2000 and Seismostruct, which use different modelling approaches associated to different levels of computational demand, are compared, assuming as reference the results of Seismostruct. The effects of infill masonry walls are also evaluated.

2 Buildings with “*Pilotis*”

The origin of this architectural solution derives mainly from the first three points of the "Five points for a new architecture" published by the Swiss French architect *Le Corbusier* in 1926 [8], which define the principles of modern architecture: (i) “*Pilotis”* (open ground floor); (ii) Free floor plans; (iii) Free facades; (iv) Free windows; and (v) Terrace-garden. These fundamentals were possible due to the emergence of new construction materials, namely reinforced concrete, which allow greater design freedom, being able to enjoy larger areas unobstructed from structural elements [9].

Despite the aesthetic and functional advantages offered by this concept of modern architecture, as it allows good use and distribution of the ground floor space, this type of building is prone to the formation of the localized plastic mechanism of flexible floor, "soft-storey", when submitted to horizontal actions. In fact, the problem is that this architectural concept, initially implemented in regions without seismicity, working well from a static point of view, was imported to seismic zones around the world, as is the case of Lisbon. The drastic change in the amount of masonry walls between the ground floor and the upper floors generates a variation in stiffness and resistance at the transition floor level, which causes that, during the earthquake, the deformation of the structure is concentrated in that floor, being more flexible than the others (which behave as a rigid body), being stiffened by the masonry walls [9] – see Figure 1 (b).

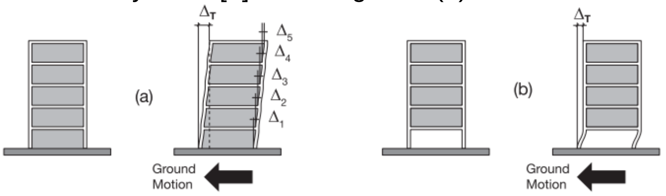


Figure 1: Distribution of the total displacement generated by an earthquake in a: (a) regular building; and (b) building with a soft-story irregularity. (Source: [9])

In fact, masonry walls are a non-structural element that can significantly influence the seismic behaviour of buildings, so disregarding their contribution to the seismic response of structures may be against safety, namely if these walls present an irregular distribution, since they can not only drastically modify the structural response, favouring flexible floor mechanisms, but can also substantially increase the global stiffness of the structure, which changes the seismic forces to which it will be subjected [10].

The current code of seismic design of new structures, Eurocode 8 – Part 1 (EC8-1) [11], recognizes the significant influence that structural irregularities in height have in the behaviour of buildings under seismic action, and, therefore, to avoid local plastic mechanisms of flexible floor, recommends basic principles of design and adopts the philosophy of "capacity design", associated to the principle "strong column-weak beam". In this sense, in EC8-1, it is not only recommended the introduction in the structure of structural reinforced concrete walls, with adequate stiffness and strength to uniform the displacements between floors, but it is also intended, with the "strong column-weak beam" principle, to maximize the number of plastic hinges in the frames, forcing them to form in the beams and that the columns remain in the elastic phase during an earthquake [12]. Nevertheless, these aspects were not considered in old buildings.

3 Case Study Building

The case study of this work - block 3 of Infante Santo Avenue Housing Complex (Figure 2) - is part of a set of 5 similar buildings, designed between 1949 and 1955, located in Infante Santo Avenue, in Lisbon.

Figure 2: General views of the building block under analyses

They are collective housing buildings with 9 storeys high, characterised by a visually open ground floor and infill masonry walls on the upper storeys. Conceptually, each block of Infante Santo Avenue Housing Complex is formed by two distinct volumes, to solve the relationship with the topography of the land: the block of houses (building based on "*Pilotis*") which is arranged transversally to the Infante Santo Avenue, and the body of shops that is parallel to the road, semi-buried, which serves as a support wall for the soil impulse.

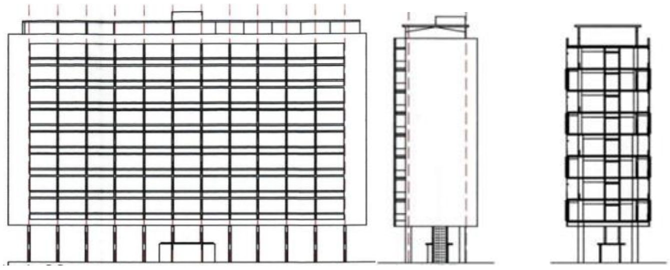


Figure 3: Case study building: elevations and cuts. (Source: [13])

As far as the resistant structure is concerned, this is relatively simple, made up of 12 transversal frames spaced 3,70 metres apart and with two cantilevered extensions of 2,70 metres at their ends. Therefore, the structure is symmetric in both directions and the transversal direction (Y) is much more rigid and resistant than the longitudinal one (X). It is important to highlight the fact that, on the roof of the ground floor, there are two robust longitudinal beams (V9 beams), in the alignments of the columns, intersecting only 2 transverse frames at each top of the building, which will have repercussions on the seismic analysis of the building in the longitudinal direction, since these beams give a much higher stiffness to these columns than to the other ones, only restricted to rotation by the slab. Other distinctive features of these residential building blocks are: (i) access cores to the upper floors in brick masonry; (ii) reinforced concrete elements using plain steel bars; (iii) solid reinforced concrete slabs with 0,11 metres thickness; (iv) columns with variable section in height; (v) beams unloading into beams; and (vi) ground floor with a higher height than the upper floors.

The building was designed according to the old Portuguese codes for reinforced concrete “Regulation of Reinforced Concrete” (RBA) [14], introduced in 1935, and the seismic actions in structural element´s design were taken according to an article of Maria Amélia Chaves and Bragão Farinha published in “Técnica” Magazine, as there were no national seismic regulations at the time [15]. The design methodology to consider seismic action consisted of equivalent horizontal forces, applied at the nodes of the structure, whose value depended on the mass and a correction factor that takes into account the structure's own frequency. Through Figure 4, it is possible to make a comparison, in terms of the intensity of the seismic action defined by the elastic response spectrum of the action values proposed in EC8-1 with those in the Regulation of 1958 (RSCCS) [16], it should be noted that the case study of this dissertation is prior to the latter regulation.

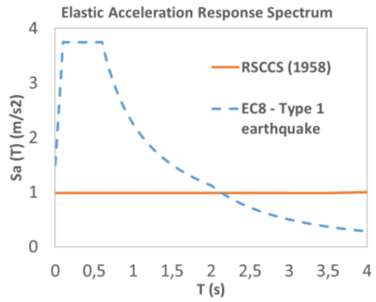


Figure 4: Comparison between spectral seismic accelerations of RSCCS and EC8-1, for a type A soil, and with an equivalent viscous damping (ξ) of 5%.

In fact, Figure 3 shows clearly that comparing the 1958 Regulation with EC8-1, for type A, the value of seismic action increases to about the double for structures with an eigenfrequency of 1 Hz (as is the case of the building in question), which indicates the problems that can exist in verification of the seismic safety of the structure in the light of the current normative demands.

4 Seismic Safety Assessment

4.1 Expedite Methods

The seismic performance of the building is firstly evaluated with the expedite methods: (i) Methodology LNEC/SPES and (ii) Methodology ICIST/ACSS. For a better understanding of the parameters and requirements involved in these methods, it is suggested reading [17].

The methodology LNEC/SPES proposes 2 expedite methods, method I and II, which allow the strength of reinforced concrete buildings to be evaluated only based on the geometric (method I) and mechanical (method II) properties of the vertical elements. According to method I, structural safety in relation to seismic action is verified if, at the level of each floor *j*, the percentage of the area of existing columns in relation to the floor area, $A_{PC,j}$, is equal to or greater than the percentage of column area required, $A_{PE,j}$. As can be seen in Figure 5 it is concluded that the building does not comply with safety, since the demand is higher than capacity on all floors.

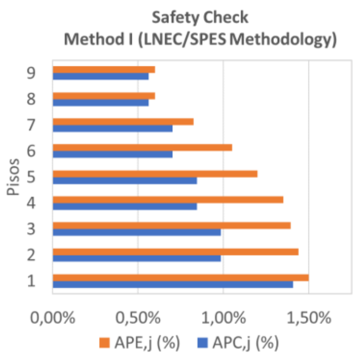


Figure 5: Safety assessment according to Method I of the LNEC/SPES Methodology

According to method II, the seismic safety of the building is verified if, at the level of each floor j, and in each main direction of the building, the resistant capacity of the building, measured in terms of the seismic coefficient, $CS_{C,j}$, is equal to or greater than the required seismic coefficient, $CS_{E,j}$.This method already considers the quantity and resistance of the longitudinal and transverse reinforcement, being necessary to calculate the resistance to horizontal forces of columns due to bending and shear mechanisms. Comparing the two coefficients at the level of each floor, it is noted that the safety of the structure is not verified at any floor, as shown in Figure 6.

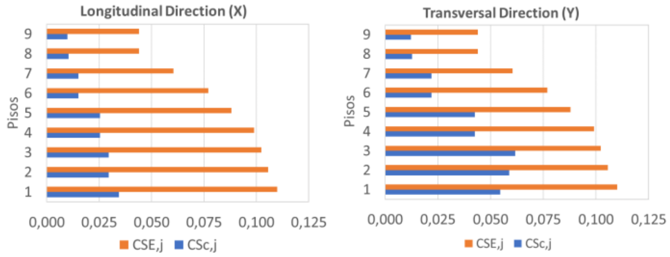


Figure 6: Safety assessment according to Method II of the LNEC/SPES Methodology

The last expedite methodology used, the ICIST/ACSS methodology, consists, in general, in determining and comparing two dimensionless indices, the seismic performance index, I_S, and the seismic load index, I_{S0}, corresponding to resistance and action, respectively, which is translated into a verification of the shear forces in all floors and according to the two main horizontal directions. In this method two cases of application were considered (Case 1 and 2), which differ in the average shear stress values adopted for the columns, in the column strength calculation, for the determination of the seismic performance index. In Case 1 the values suggested by the ICIST/ACSS methodology were adopted while in Case 2 more conservative values based on a calibration made in [16] were used, which intends to better represent the reality of the existing building at the time of construction of the building. The results are shown in the Figure 7. In Case 1, it is observed that the evaluation is satisfactory only on floors 8 and 9, in the longitudinal direction, and on floor 9, in the transverse direction ($I_S >$ 1,2xI_{SO}), and that in the remaining floors and directions the evaluation is either inconclusive (0,8x$I_{SO} < I_S <$ 1,2xI_{SO}) or unsatisfactory. On the other hand, applying Case 2, it is observed that the levels of resistance decrease significantly, so the building does not verify safety on any floor, in both directions.

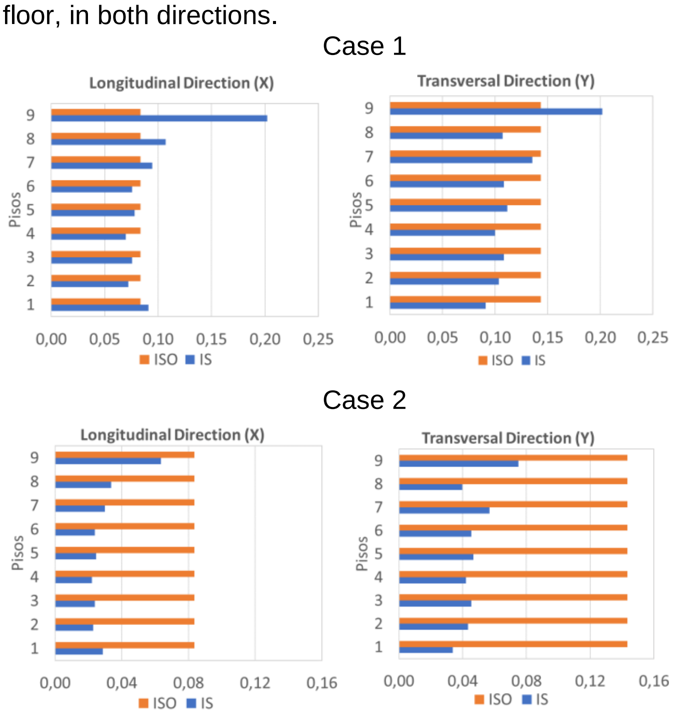


Figure 7: Safety assessment according to the ICIST/ACSS Methodology

4.2 Reference Method (EC8-3)

The main seismic performance evaluation of the building is conducted based on the assessment procedures prescribed in EC8-3.

The application of EC8-3 in Portugal is conditioned by the National Annex, which defines, for the case of current buildings (importance class II), the need to verify the Severe Damage Limit State (SD), one of the three foreseen in the European code and corresponding to the non-collapse design limit state for new structures, defined in EC8-1. It should be noted that the fact that existing structures may have already reached their useful life, leads to the acceptance of a lower demand in the verification of the limit states for existing structures compared to new structures, which translates into a less intense design seismic action, with lower return periods. Thus, for the Severe Damage Limit State, the National Annex defines a return period of 308 years for the definition of the seismic action, instead of 475 years for the reference action, as also designated in the seismic design of new structures. It should be added that with the new legislation, constituted by the Decree-Law nº95/2019 and by the Ordinance nº302/2019 (which imposes the need for the preparation of a seismic vulnerability assessment report in certain urban rehabilitation processes), it is mandatory to perform seismic retrofitting if the structure does not verify safety for 90% of the seismic action defined in the national annex of the EC8-3. In this work, the safety verification was performed for a seismic action with a return period of 308 years, corresponding to 100% of the regulatory action defined in the national annex of EC8-3.

It should also be noted that, contrary to what happens in projects for new structures, the usual methodology at the level of analysis of existing structures is based on the control of local and global displacements of the structure, a quantity which translates the real effect of earthquakes on structures. So, according to EC8-3, the structural performance is analysed by verifying the safety for local mechanisms, brittle and ductile, being the analysis based, respectively, on the control of the shear stress and of the chord rotation, a quantity which translates the deformation of the elements in the zone of formation of the plastic hinges.

In this work, the seismic performance of the structure is evaluated through static nonlinear methods of structural analysis, the so called “Pushover” analysis. These non-linear analysis methods imply extensive knowledge of the structure, in terms of geometry, constructional arrangements and materials. Thus, EC8-3 establishes rules and procedures to obtain the necessary knowledge of the structure and establishes a link between the knowledge obtained and the confidence with which the methodology can be used to assess seismic safety. This translates into the values of confidence coefficients, which will affect the values of response and capacities. The level of knowledge that is conservatively considered, through the information available from the drawings and written parts of the original project, is the limited knowledge level (KL1), which only allows linear analyses to be carried out. However, non-linear analyses will still be performed, reducing, when calculating the capacities of the elements, the average values of the material properties by the highest confidence factor ($CF_{KL1} = 1{,}35$), associated with the lowest level of knowledge.

4.2.1 Structural Modelling

The first step of the nonlinear methods of structural analysis consists of the development of a three-dimensional structural model. The case study was modelled using two automatic calculation programs, SAP2000 v22 [19], based on plastic hinges, and SeismoStruct 2021 [20], based on fibre models, essentially to validate the results obtained and to compare non-linear models using different types of numerical modelling, with distinct complexities and computational effort times. Although the objective of this work was to evaluate the existing building with masonry walls, numerical models were also developed in both programmes without consideration of the masonry walls, to study the influence that these non-structural elements have on the seismic behaviour of the structure. A complete and detailed description of the modelling of the building in both programmes can be found at [17].

It is noted that the main characteristic of Seismostruct consists in its stable capacity to consider the plasticity distribution along the length of the elements and their section, allowing a very accurate estimation of the damage distribution along the structure [20]. Despite its greater complexity in general and greater computational demand, the great advantage that this program presents in its latest versions consists in the processing of the data, allowing all the safety criteria of the structural elements to be automatically verified, as it has incorporated several international regulations, among them the EC8-3. In this way, since the aim is to evaluate the seismic vulnerability of an existing building, Seismostruct is considered as the reference tool in this work, since it allows simulating the effects of seismic action on the behaviour of the structure with greater approximation to reality, always emphasising the importance of carefully analysing the results.

A first validation of the structural numerical models can be achieved comparing the experimentally measured natural frequencies in [21] and the analytically estimated ones, as presented in Table 1.

Table 1: Structure Frequencies

Mode	Masonry Walls (PA)	Numerical programme	T (s)	F (Hz)	T “*in situ*” (Hz) [21]
1 (X)	**With PA**	**Seismostruct**	**1,03**	**0,97**	**1,08**
		SAP2000	1,20	0,84	
	Without PA	Seismostruct	2,77	0,36	-
		SAP2000	3,71	0,27	
2 (r)	**With PA**	**Seismostruct**	**0,62**	**1,60**	-
		SAP2000	0,77	1,30	
	Without PA	Seismostruct	1,08	0,93	
		SAP2000	1,23	0,81	
3 (Y)	**With PA**	**Seismostruct**	**0,59**	**1,69**	**1,75**
		SAP2000	0,73	1,37	
	Without PA	Seismostruct	1,02	0,98	-
		SAP2000	1,20	0,83	

As would be expected, the first vibration mode of the structure corresponds to the fundamental translation mode in the longitudinal direction (X), since its structure resistant to horizontal actions is all oriented in the transverse direction (Y) of the building, constituted by the transverse frames. The second mode is characterized by a torsional movement according to the vertical axis (r), being predictable also due to the reduced torsional stiffness of the building. On the other hand, the introduction of the masonry walls results in a significant increase in the stiffness of the structure, with greater expression in the longitudinal direction, which can be observed by the almost twofold increase in the frequencies of the fundamental modes.

4.2.2 "Pushover" Analysis

The non-linear static analyses ("Pushover" analyses), characterised by a representation of the non-linear behaviour of the structural elements, allow an accurate determination of the resistant capacity of the structure and its plastic mode. The objective of these analyses is to obtain the capacity curve or "Pushover" curve of the structure which, in practice, translates the relationship between the basal shear force and the displacement at the top of the structure (Centre of Mass of the last storey). It is emphasized that the capacity curve is an intrinsic characteristic of the structure, which describes its behaviour when subjected to an increasing lateral force distribution under constant gravity forces.

To evaluate the seismic performance of the structure using the "Pushover" analyses it is necessary to calculate the "target displacement", which corresponds to the maximum displacement that the structure will be subject to for the regulatory seismic action and for which the state of the structure and its components is evaluated. For this purpose, it is necessary to "intersect" the capacity curve with the most severe seismic demand defined by the national authorities to the building in study (elastic spectrum for soil type A and seismic action type 1, corresponding to a return period of 308 years, defined in EC8-1 through non-linear static methods, such as the N2 method proposed by EC8-1-Annex B.

Drifts

Firstly, to understand the structural response of the building to the earthquake and the influence that considering, or not, the behaviour of the masonry infill walls in the models has on this response it is important to analyse the structural displacements and drifts between floors (displacement between floors normalised by the height of the floor under analysis). Figures 8 and 9 show the structural displacement and inter-floor drifts, respectively, for the different load patterns of the Pushover analyses, whose significance can be consulted in [17]. Effectively, it is expected that the building, for the level of requirement prescribed in Lisbon for the limit state of Severe Damage (SD) of EC8-3, is conducive to the formation of the local plastic mechanism of flexible floor, "soft-storey", in the cast floor, which translates into a concentration of the deformation requirements on this floor, not allowing the structure to exploit the ductility available in all its structural elements, unlike what happens in the model without the walls that presents a more regular response in height.

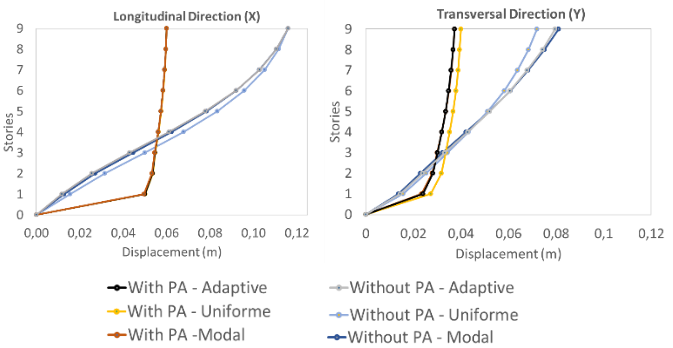


Figure 8: Structural displacements for the Target Displacement

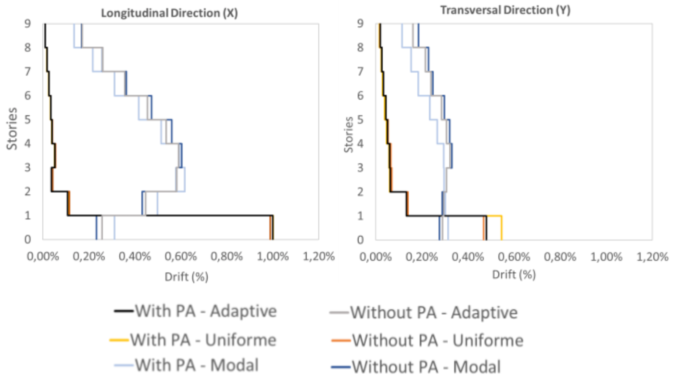


Figure 9: Drifts (Seismostruct) for the Target Displacement

Confronting the drifts results obtained from the building with certain international recommendations (Table 2), it is confirmed that at the ground floor level, in the longitudinal direction, the building is very close to reach, or has already reached, the limits recommended for the extensive damage limit state, which is associated with the risk of loss of human lives, while in the transverse direction, safety is apparently verified.

Table 2: Drift limits. (Source: [17])

Performance Levels	Drift limit				
	VISION 2000	FEMA-356	*Gobarah*	ATC-40	EC8-1
Moderate damage	0,50%	1%	0,50%	1-2%	
Extensive damage	1,50%	1-2%	0,80%	2%	1,25%
Imminent collapse	2,50%	4%	1%	2,5%	

Capacity Curves

The capacity curves, resulting from the pushover analysis for the different load patterns, which are described in detail in [17], are presented in Figure 10.

Firstly, from the analysis of this image, the masonry infill walls (PA), despite being fragile elements with limited strength, contribute significantly to the overall stiffness and strength of the structure, and can substantially influence the seismic performance of the building, so they should be considered in the analysis. As would be expected, it is clear from the Figure 10 that the direction with the greatest resistant capacity is the transverse direction since it is the direction of the frames of the structure.

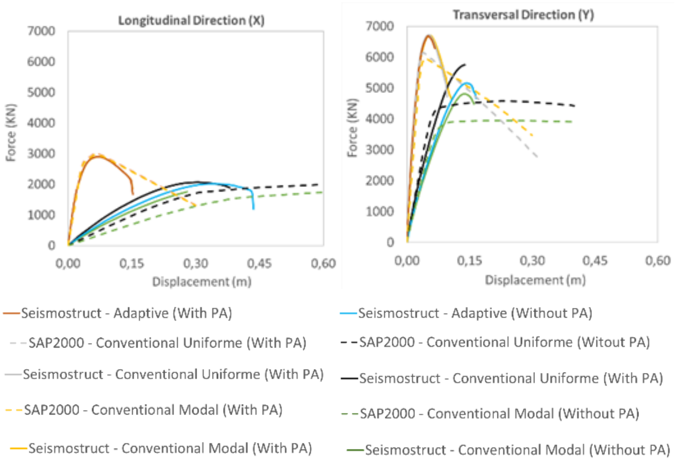


Figure 10: Comparison of Capacity Curves resulting from "Pushover" analyses performed in SAP2000 and Seismostruct (with and without consideration of PA) for the different load patterns

On the other hand, when comparing the Pushover curves between programmes, associated to the different load patterns and for the models with and without masonry walls, it is observed that the models developed in SAP2000 present a slightly lower horizontal resistance, in general. It is also found that, in the models without masonry walls, the concentrated plasticity approach used in SAP2000 does not reproduce the effect of strength degradation (well displayed in the Seismostruct curves), since these models reproduce a constant line with practically zero stiffness after the peak of maximum strength (like an elastic-perfectly plastic behaviour). Despite the differences between the results of the two programmes, which can be justified by the different modelling options adopted and the complexity of the programmes, the "Pushover" curves obtained are considered valid, assuming the Seismostruct results as the reference and will be those used in safety check.

Thus, safety verification will be assessed only for capacity curves based on adaptive Pushover analysis. The target displacements were automatically obtained by the program and were confirmed by applying the N2 method. The Figure 11 shows the capacity curves associated with the EC8-3 safety criteria and the target displacement. It is possible to verify that, in both directions, the failure mode of the structure is controlled by the shear collapse of the structural elements, namely the columns of the ground floor, since, for the displacement imposed on the structure by the earthquake, many of these columns have already exceeded their shear capacity according to the expressions of EC8-3.

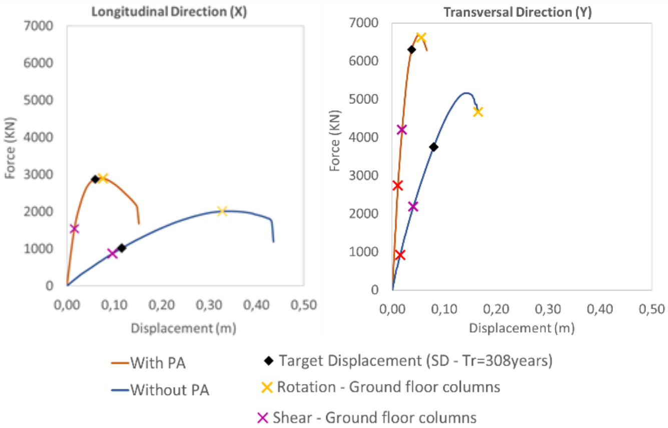
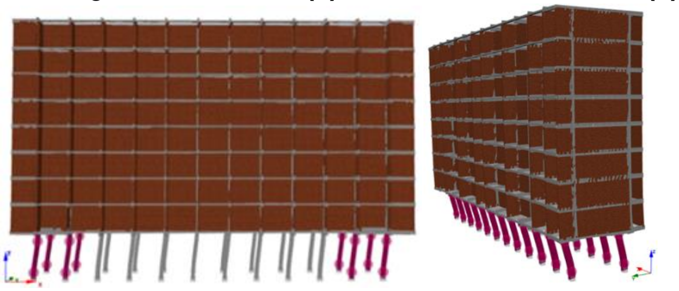


Figure 11: Capacity curves with indication of the Target Displacement, associated to the safety criteria of EC8-3

Seismostruct allows the identification of the elements that exceed a certain capacity according to EC8-3, as well as the location of the respective plastic hinges, through their colouring, and in the case study, as can be seen in Figure 12, for the Severe Damage limit state, only shear capacities were exceeded, identified by the colour purple.

Longitudinal Direction (X) **Transversal Direction (Y)**



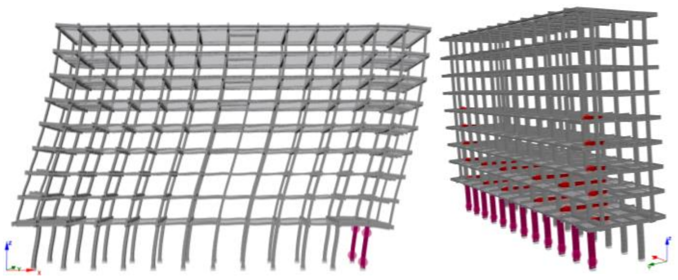


Figure 12: Deformations of the structure, with and without PA, for the Target Displacement (Tr=308years), with indication of the elements that exceeded their shear capacity according to EC8-3

In the longitudinal direction, it is visible that the 8 end columns of the ground floor are penalized by their greater stiffness, absorbing most of the shear stress of the floor and, consequently, reaching their shear capacity, while in the transverse direction, although the shear equilibrium is uniform across all the columns, given the greater forces involved in this direction, all the columns have already exceeded their shear capacity for the displacement imposed by the earthquake. It is noted that in the transverse direction, the beams of the first floor reach the shear capacity of EC8-3 before the columns of the ground floor.

Figure 13 demonstrates the shear performance of open floor columns (failure mode of these elements) throughout the Pushover analysis, comparing demands (dashed curves) and capacities (solid curves) until the maximum basal shear point is reached, according to EC8-3 safety criteria.

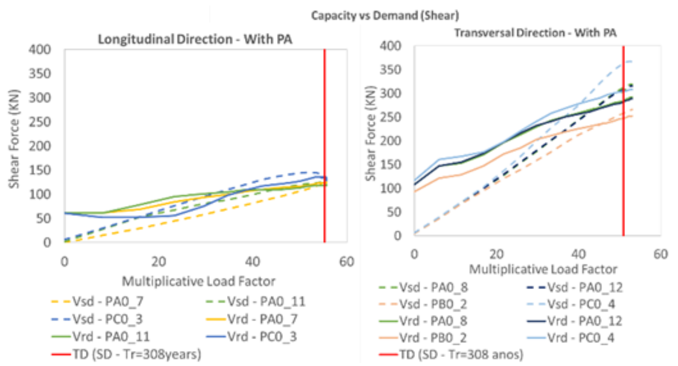


Figure 13: Analysis of the shear performance of ground floor columns according to EC8-3, throughout the analysis: comparison between Capacities (solid) and Demands (dashed)

As can be seen in Figure 13, the shear strength values of the elements may vary throughout the analysis, since they depend on factors such as the level of axial strain and the ductility requirement in displacement, while the requirements grow linearly until the peak strength is reached. In the longitudinal direction, it can be concluded that the end columns, for the target displacement (TD) imposed on the structure, slightly exceed their shear capacity (about 5%) and the intermediate columns, despite checking safety, are very close to reaching their capacity. In the transverse direction, no column guarantees safety for the target displacement (TD), with higher margins of non-compliance compared to the longitudinal direction, exceeding the shear capacity by about 25% in some columns.

Finally, to explore the potentialities of the Seismostruct program and to identify the progression of the damage in the structure, certain performance hypotheses were introduced in the adaptive Pushover analyses to the structural elements when they reach a certain safety criterion of EC8-3. In this case, the behaviour of the structure was analysed by assigning a residual strength of 30% ("Residual Strenght") of the total strength when a certain element reaches its capacity to shear or chord rotation, according to EC8-3.

Through the capacity curves of Figure 14, which include this "Residual Strenght" hypothesis, it is possible to observe that the analyses cease to converge before the target displacement is reached, since mechanisms are created that make the structure unstable when the columns of the ground floor reach their shear capacity according to EC8-3, which suggests a premature collapse of the structure. In the light of the above, it is possible to conclude that, for the displacement imposed on the structure by an earthquake with a 308 year return period, there are sufficient conditions to show that the safety of the structure is not assured, essentially by the brittle shear failure of the ground floor columns that tend to cause earlier collapse of the building, so that it will be necessary to support the design of a seismic retrofitting solution to improve the behaviour of the structure and limit its level of structural displacement.

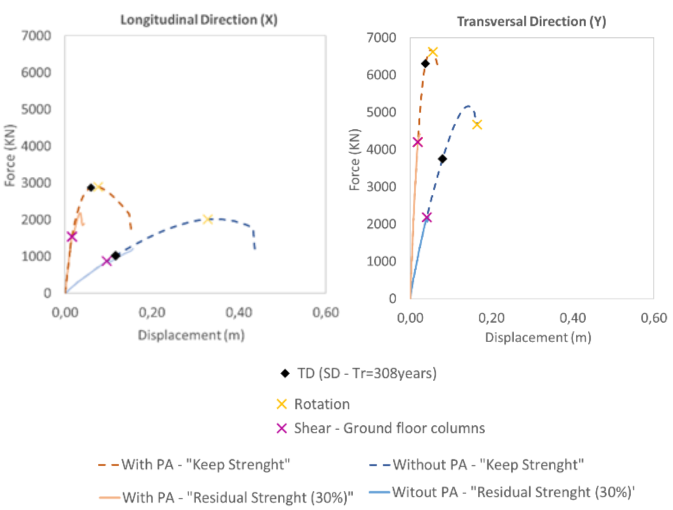


Figure 14: Capacity curves, with the addition of the "Residual Strenght" performance assumptions, associated with the EC8-3 safety criteria

4.2.3 Retrofitting Strategies

It is concluded that seismic retrofitting of the case study is necessary to install safety levels compatible with current requirements, which can be achieved by reinforcing the existing structural elements or by introducing new elements to the structure. The improvement of behaviour may be achieved by adopting one of the following approaches, or even combining them: (i) reducing the seismic demands; and (ii) improvement of original performance, with locally or globally interventions. In this work, only the seismic retrofitting solutions to improve the original performance of the structure will be addressed, namely through CFRP-wrapping of columns (Carbon Fibre Reinforced Polymers) and by adding steel braces.

The intervention of reinforcement by jacketing with CFRFP is necessary to avoid the premature brittle failure of the columns of the ground floor, providing an increase in shear strength, and to bending, of these individual elements, without altering, in a relevant way, the properties of the structure. According to Annex A of EC8-3, the total capacity, as controlled by the stirrups and the CFRP, is evaluated as the sum of the contribution from the existing concrete member and the contribution from the CFRP. The Seismostruct software allows carbon fibre fabrics to be selected for structural reinforcement from a list of the most commonly used products on the market, or alternatively, by entering user-defined values. In this way, the blanket "SikaWrap Hex 300C" of the producer company Sika was selected.

On the other side, by adding a new lateral load resisting system, such as steel braces, the lateral stiffness can be increased. Circular tubular profiles of 0,60 metres diameter and 0,02 metres thick of S355 steel were chosen to be added to the ground floor end spans, in both directions.

Figure 15 compares the capacity curves of the retrofitted structure with the two types of solutions and the original structure, associated to the safety criteria of EC8-3. About the CFRP jacketing of the ground floor columns, it can be seen that the original performance of the building has been improved, significantly increasing the global deformation capacity of the structure and slightly increasing the global resistant capacity, through local interventions. Regarding the structure retrofitted by the addition of steel bracing in the end spans of the ground floor, it is clear that this retrofitting technique conferred an increase in initial stiffness and overall horizontal resistance in both directions, as well as reduced the maximum deformation that the structure will be subject to for the seismic action of safety verification, thus improving the original performance.

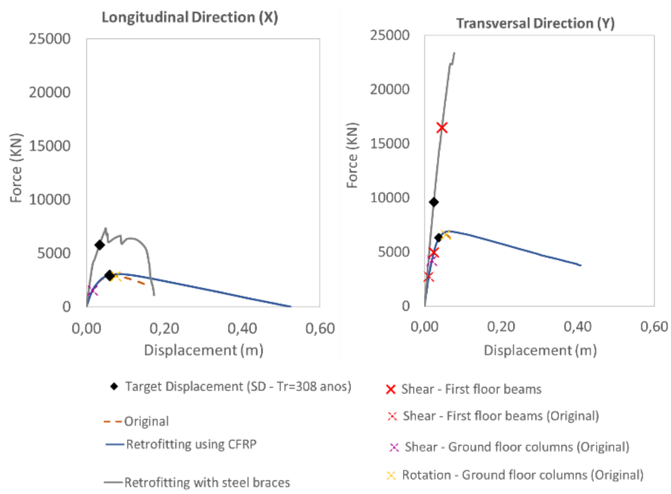


Figure 15: Comparison of the capacity curves for the different retrofitting solutions versus the original structure

Again, Figure 16 illustrates the evolution of the shear performance of the ground floor columns throughout the non-linear static analysis until the maximum basal shear is reached, comparing the capacities according to EC8-3 (solid curves) and the demands (dashed curves) of these elements, with the addition of the two reinforcement solutions to the structure. The application of CFRP blankets does not increase the stiffness of the columns (the requirements of the elements remain similar to those of the original structure, see figure 13), but increases their shear capacity, enabling them to reach their bending capacity without developing fragile mechanisms.

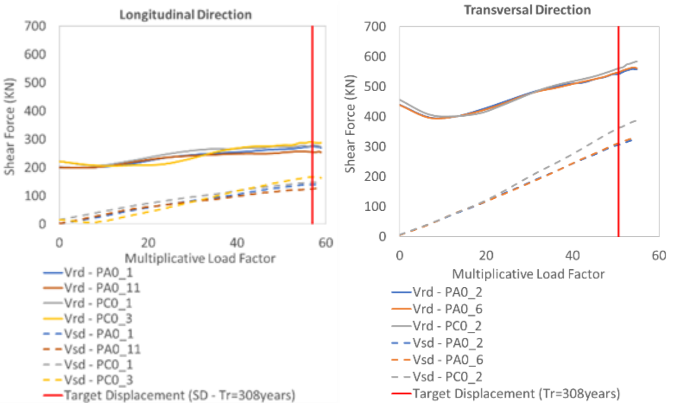


Figure 16: Comparison between Capacities (solid) and Demands (dashed) to the shear according to EC8-3 along the "Pushover" analysis, for the retrofitted structure with CFRP wrapping

In relation to the technique of adding metal bracing on the ground floor, as can be inferred from Figure 17, the shear requirements of these columns were reduced compared to the original structure (Figure 13), with the bracing elements being responsible for almost all the full seismic demand.

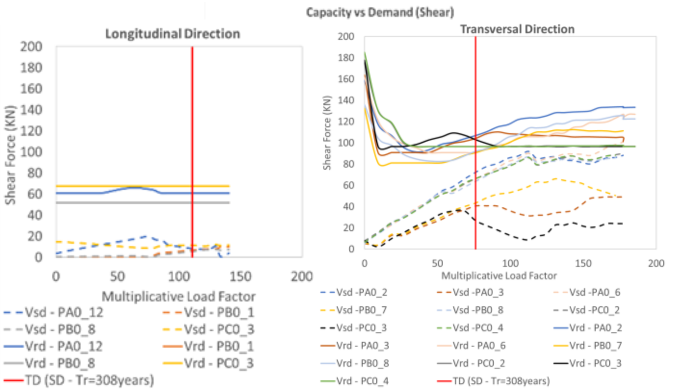


Figure 17: Comparison between Capacities (solid) and Demands (dashed) to the shear according to EC8-3 along the "Pushover" analysis, for the reinforced structure with additions of steel bracing

Drifts

It is also important to evaluate how the introduction of the two defined retrofitting techniques influence the evolution of drifts and structural displacements compared to the original structure without strengthening.

Figures 18 and 19 clearly show that the most efficient reinforcement technique in reducing maximum drift is the addition of steel bracing in the casting floor, since it allows the local plastic soft-story mechanism to be minimised/eliminated, reducing the maximum drift by more than half and the levels of structural displacements by about half, in both directions.

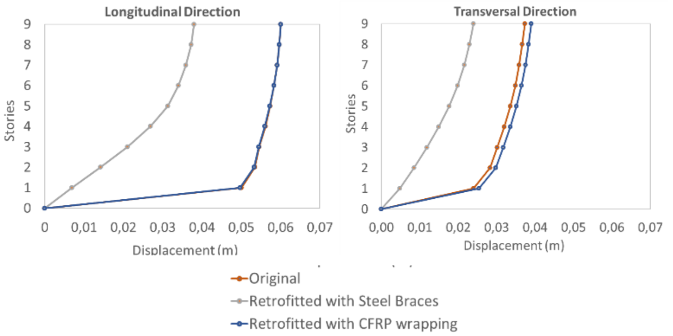


Figure 18: Comparison of drifts for the different reinforcement solutions against the original structure, for the Target Displacement (Tr=308years)

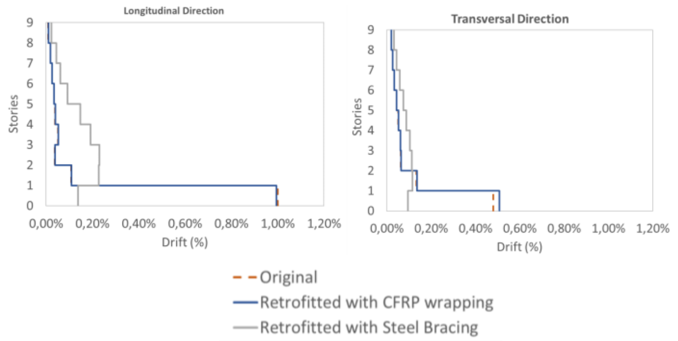


Figure 19: Comparison of drifts for the different reinforcement solutions against the original structure, for the Target Displacement (Tr=308years)

5 Conclusions

This paper addresses the issue of seismic vulnerability of one of the most vulnerable class of existing reinforced concrete buildings in Lisbon, namely buildings with an open ground storey ("*Pilotis*"), designed to seismic action using inadequate procedures, according to current knowledge. The safety of an existing building representative of modern architecture - one of the buildings of the residential complex of Avenida Infante Santo, in Lisbon - was analysed by different methods, from the simplest to the most complex, in order to assess the expected damage to the building.

The expedite methods reveal themselves to be coherent with their ease and rapidity of application consisting of a first balance of the structure's safety. It is concluded that the building does not verify safety in any method, however the most conservative results were derived from the least expedient methods, the ones that considers the mechanical properties of the elements. On the other hand, it is emphasized that these expedite methods are not able to identify the main vulnerabilities of irregular buildings (as is the case of the study), namely, the concentration of seismic demands on the ground floor. This is due to the fact that these methods do not take into account the effect of the masonry walls which, in practice, are the elements that introduce this singularity.

The main analysis and reference to the case study follows the procedures of EC8-3, in which the evaluation of the seismic performance of the structure is obtained using "Pushover analysis. At the modelling level, in order to increase the accuracy and validity of the final results, the structure was numerically modelled using two different programmes, SAP2000 (the most widely available programme) and Seismostruct (a programme with more complex analyses that require greater computational effort, which is not very compatible with the current practices of project offices). The results obtained with Seismostruct are considered as a reference, being the ones that best reproduce the real behaviour of the structure, however it is noted that the concentrated plasticity models (based on plastic hinges) used in SAP2000 constitute an interesting alternative to the more complex fibre models, since they require significantly less computational effort time and reproduce reliable results. It is concluded that the building is prone to the formation of a soft-storey mechanism on the ground floor, in both directions, and the columns of this storey are vulnerable to brittle shear failure, thus not verifying the safety for the target displacement imposed on the structure by the regulatory earthquake defined in the national annex of EC8-3 (associated to a return period of 308 years). Given the mechanism observed, two alternatives of seismic retrofit to be implemented at the ground floor are studied: (i) CFRP column wrapping; and (ii) addition of steel bracings. Both prove to improve the seismic performance of the building, on the one hand increasing its deformation capacity, as is the case of CFRP wrapping of elements and, on the other hand, increasing its stiffness and lateral resistance, as is the case of the addition of steel bracing. It was found, however, that the only retrofitting technique capable of eliminating the plastic flexible floor mechanism and limiting the level of structural displacements is the technique with metal bracing.

Finally, in terms of comparison of the results obtained with the expedite methods and the reference method, it is clear that certain particular aspects of the structure, such as the discontinuity and concentration of local deformation demands, or even, the penalisation of the end columns of the ground floor by the longitudinal beams V9, are not correctly evaluated by the expedite methods, so the design of strengthening solutions deserves further studies.

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