# Structural characterisation of reinforced concrete buildings built before 1983 in Benfica, seismic assessment and seismic retrofit solution of a representative building

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## Abstract

Portugal, as many European countries, is significantly prone to seismic hazard. Moreover, reinforced concrete (RC) buildings built between 1950 and 1983 are part of the residential building stock of most Portuguese cities. This period of construction marks a time when RC buildings were designed under rules without any strict seismic provisions, thus leading, most likely, to important seismic vulnerabilities.

A good knowledge about the building stock of a certain city can be used to estimate its seismic vulnerability. At the same time, the seismic assessment of a defined building stock is useful to delineate a seismic vulnerability map, to improve the control of damage to seismic actions and to assess possible retrofitting solutions to the most conditioning cases.

In this context, a database was developed through a data survey of the structural properties information of RC buildings located in Benfica. Following the database statistical analysis, the most representative building was selected, modelled, and calibrated from *in situ* experimental results (obtained from an ambient vibration test). Then, the seismic behaviour of this building was evaluated using a non-linear static analysis proposed by Eurocode 8 (EC8) and the structural safety was verified as provided in Part 3 of EC8. Finally, from the structural weakness identified in the previous assessment, a retrofit solution was developed and evaluated.

**Key Words:**

Vulnerability and Seismic risk; Structural Characterization; Reinforced Concrete Buildings; Seismic Assessment; Non-linear Static Analysis; Seismic Retrofit

## 1. Introduction

Portugal is vulnerable to earthquakes because it lies on the boundary of two tectonic plates. Its seismic hazard has negatively marked the country due to the high loss of human lives and material assets throughout its history (Borges, J. et al., 2001).

In the last four decades of the 20th century some important demographic factors were observed in the Lisbon Metropolitan Area, which can be observed generally across the country, such as, the demographic shifts with the ageing population and the young generations moving to the outskirts and the construction of RC buildings has increased to match the demand for residential houses. These reasons led to the construction of buildings without respecting the rules regarding seismic actions. Therefore, these buildings were built with low seismic behaviour and using simple and fast constructive techniques. Since there is a large percentage of these buildings built in Portugal and which have the same historical context, it is important to study and assess their resistance (Gago, A., 2011).

This paper corresponds to an extended abstract of the MSc Dissertation, which develops the study of the seismic assessment of old RC buildings in Lisboa, a work within the scope of the FCT Project "MitRisk Platform to support the seismic risk reduction using economically feasible strengthening solutions".

## 2. Benfica RC buildings characterization

First, all structural properties of RC buildings built before 1983 in a area of Lisbon (northern area of Benfica) were gathered from the blueprints available at “Arquivo Municipal de Lisboa” and stored in a database. Figure 1 represents the database map of Benfica.

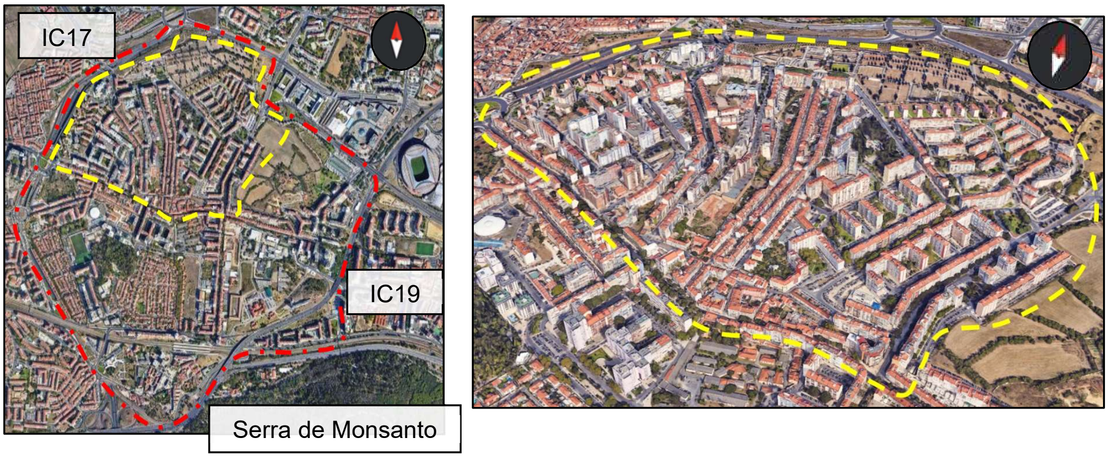



Figure 1 - Area of Benfica where RC building information was collected

The attributes collected and added to the database are:

- General data: project number, construction year, type of occupation (residential, commercial or mixed), number of floors, number of underground floors, material used, type of structure (reinforced concrete, masonry, and others), the class of concrete and steel used in the structure, type of configuration (frame or wall-frame), the presence of a soft-floor, the regularity in plan and height, the height of the first and remaining floors, the floor and height of irregularity and the type of slab (and its thickness);
- Structural data: the cross-section dimensions of the structural elements, longitudinal and transversal reinforcement, and the floor of the cross-section chances (columns, walls and beams).

Several probabilistic distributions were considered for each attribute collected in the database and their statistical parameters derived through the maximum likelihood estimation. This approximation was evaluated with the value of the mean and the coefficient of variation – COV (measure of sample dispersion) and the results were confirmed with Pearson's Chi-square Test for levels of 1%, 5%, and 10% significance.

From the statistical analysis results, it was possible to identify a representative building of the Benfica area. Then, statistical concepts are applied to the database of the area represented in Figure 1, and are compared to other similar studies such as Eurocode 2. It is important to mention that the analysis of this database obtained many results, however only the most relevant ones will be discussed in this chapter.

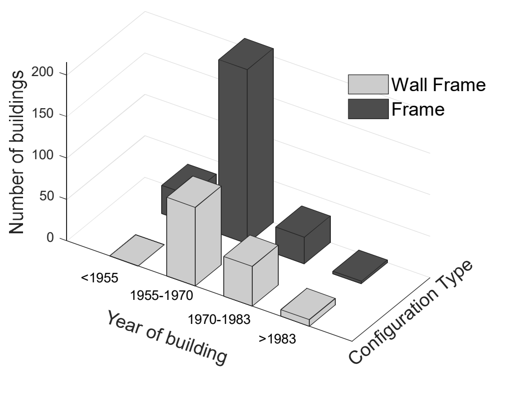
### i. Construction materials statistical analysis

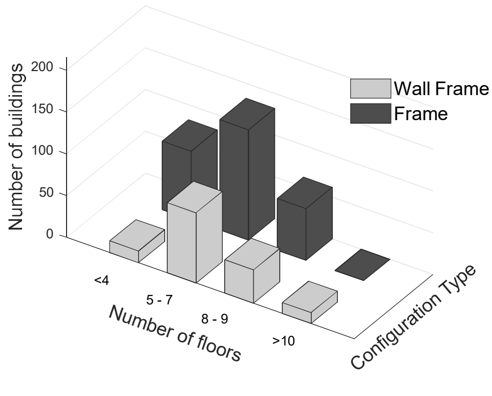
The characteristics of the construction materials were the only data analyzed without a probability distribution approximation since it is not a numerical class. This information is important because it allows to assess the characteristics of the materials (constitutive relationships) used for the structural analysis, because they are required to calculate the gravity loads, the concrete and steel resistance values. The materials most commonly observed in building structures data are, in general, concrete class B225 and steel class A40, usually smooth bars.

### ii. General characterization

Figure 2 (a) represents the results of the data collected from the buildings in Benfica, in function of the construction time and the type of configuration. As it can be observed, the data collected in Benfica area indicate that the construction period from 1955 to 1970 was the peak of building constructions, especially of the frame type.

From Figure 2 (b), it can be observed that most buildings with less than 7 storeys have frame structure while most buildings with more than 10 storeys have wall-frame type structure. As expected, for higher buildings there is a tendency to use the wall-frame configuration type, to take advantage of the shear walls and the response of wall-frame structures in height. For wall-frame type buildings the data follows a lognormal distribution, with a year of construction on average equal to 1970 and with a COV = 0.43% (and without satisfying Chi-square test). The average number of floors for wall-frame buildings is around 7 floors and a COV = 34%. For frame type buildings the data follows a normal distribution, with a year of construction on average equal to 1965 and with a COV = 0.34% (and satisfies the Chi-square test for level of 1% significance). The average number of floors for frame buildings is around 6 floors and a COV = 29%. In this context, it is globally accepted that the Benfica data survey area has a large number of frame type RC buildings with 5 to 7 storeys, built between the year 1955 and 1970.




(a) (b)

Figure 2 - Results of the data collected from the buildings in Benfica, in function of the (a) construction period vs type of configuration and (b) the number of storeys vs type of configuration.

iii. **Height floor statistical analysis**

Regarding the height between storeys, it was observed that the average height of the first storey for wall-frame buildings is 3.30m in a lognormal distribution with a COV=25% and for frame buildings is 3.60m in gamma distribution with a COV = 18% (both without satisfying the Chi-square test). Concerning the height of the remaining floors its average is equal to 2.85m in a lognormal distribution with a COV = 8% (without satisfying Chi-square test) and there was no difference between the structure type.

iv. **Slab thickness statistical analysis**

From the data collected, no significant difference in RC slab thickness was observed between the configuration building type. This data follows a lognormal distribution with a mean of 0.15m and a COV = 20%. According to Silva et al. (2014), the slabs of RC buildings built before the 1983 in Portugal have an average thickness of 0.17m and a COV = 19%, meeting the results obtained.

v. **Structural elements cross-section statistical analysis**

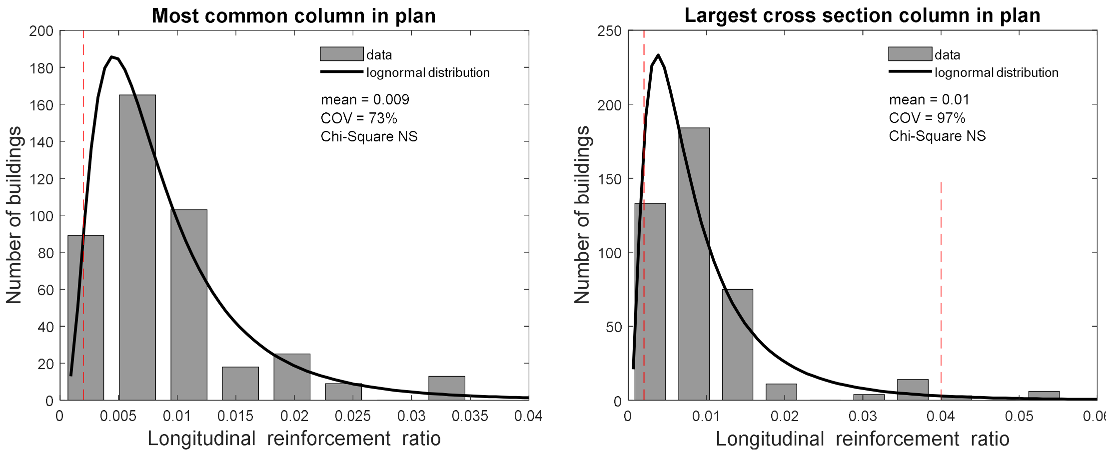
Table 1 provide the statistical analysis of the structural elements dimensions of the buildings in Benfica.

Table 1 – Statistical analysis of structural elements cross-section dimensions (* NS – Not Satisfied)

| Element | Dimensions [m] | Distribution | Average [m] | COV [%] | Max. [m] | Min. [m] | Chi-square [%] |
|---|---|---|---|---|---|---|---|
| Columns | Width | Lognormal | 0.29 | 28 | 0.55 | 0.12 | NS* |
| | Length | Lognormal | 0.55 | 28 | 1.10 | 0,20 | NS* |
| Walls | Width | Lognormal | 0.24 | 15 | 0.40 | 0.18 | NS* |
| | Length | Lognormal | 2.00 | 31 | 4.00 | 1.60 | NS* |
| Beams | Width | Lognormal | 0.24 | 28 | 0.50 | 0,10 | 10 |
| | Height | Lognormal | 0.48 | 25 | 1.00 | 0.25 | NS* |

vi. **Column reinforcement ratio statistical analysis**

Regarding the column's longitudinal reinforcement ratio and in order to compare with current modern regulations, the minimum and maximum reinforcement ratios were outlined in Figure 3, respectively, $A_{s,min}$ and $A_{s,max}$ according to the Part 1-9.5.2 of EC2 (NP EN1992-1-1, 2010). Note that these limits were only calculated for the section area component (since there was not enough data to consider all components for the $A_{s,min}$). As can be seen in Figure 3 (a), the average longitudinal reinforcement ratio for the most common columns in plan is 0.9% with a COV = 73% and in Figure 3 (b) for biggest cross-section column in plan is 1% with a COV = 97% (both follow the trend of a lognormal distribution). Regarding the limits of longitudinal reinforcement ratio imposed by EC2 (NP EN1992-1-1, 2010), there are columns with ratios lower than the minimum and with ratios higher than the maximum (for the largest cross-section columns in plan).




(a) (b)

Figure 3 - Longitudinal reinforcement ratio for the (a) most common column and (b) largest cross-section column in a plan

According to Furtado et al. (2015) the average longitudinal reinforcement ratio observed in 500 columns inspected in Lisbon is equal to 0.61% with a COV = 32%. This average ratio value is low due to the regulations in practice at the time, the REBA in 1967 and later the REBAP in 1983 (REBA, 1967; REBAP, 1983). According with another study of LNEC (2019) by the mentioned authors (with a survey of an additional 500 columns of buildings built between 1950 and 2000) it was possible to observe higher values of an average longitudinal reinforcement ratio equal to 1.27% with a COV = 40%. In summary, these studies are a reference for the statistical analysis results of this database because it is possible to check the coherence of the results with the existing buildings in identical environments. Note that the result match with what was expected, since the average longitudinal reinforcement ratio (0.9% and 1%), are between the reference values of other studies (0.6% and 1.3%). According to Part 1-9.5.2(1) of EC2 the minimum diameter for longitudinal reinforcement bars is 8mm (NP EN1992-1-1, 2010). The smallest diameter observed corresponds to a steel bar with Ø $3/16''$. In fact, it is possible to observe many buildings in the database with the diameter below or equal to the minimum limit, and consequently, in nonconformity with the current regulation. Regarding the transversal reinforcement ratio (for both types of columns) they tend to follow a lognormal distribution with same average equal to 0.1% with a COV = 65% (without satisfying Chi-square test). During data survey it was verified that there was not much concern in the dimensioning of the transversal reinforcement (opting for the same diameter and spacing for all the structural elements).

vii. **Shear wall reinforcement ratio statistical analysis**

In Figure 4, concerning the wall longitudinal reinforcement ratio statistical analysis, it is possible to limit the minimum ratio recommended in Part 1-9.6.2 of EC2 (NP EN1992-1-1, 2010). It is then verified that many wall-frame buildings are characterized by shear walls with lower longitudinal reinforcement ratio than recommended by the current regulations. Regarding the walls transversal reinforcement ratios, the results obtained are similar to those obtained for columns, where the average ratio is relatively smaller before 1970 increasing significantly after this year. This increase is due to the introduction of REBA with strict rules for the transverse reinforcement (to prevent buckling of the longitudinal reinforcement and concrete confine) (REBA, 1967).

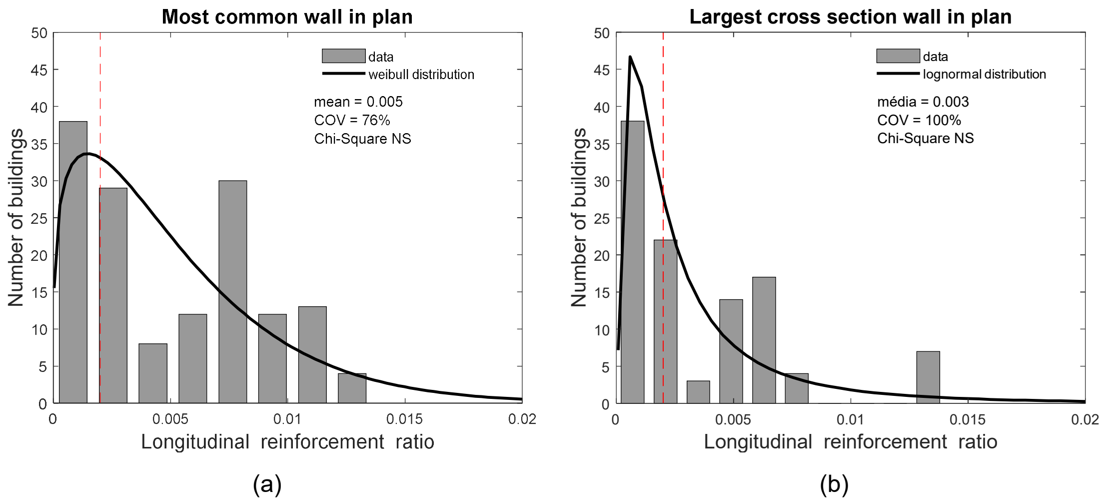



Figure 4 - Longitudinal reinforcement ratio for the (a) most common shear wall and (b) largest cross-section shear wall in plan

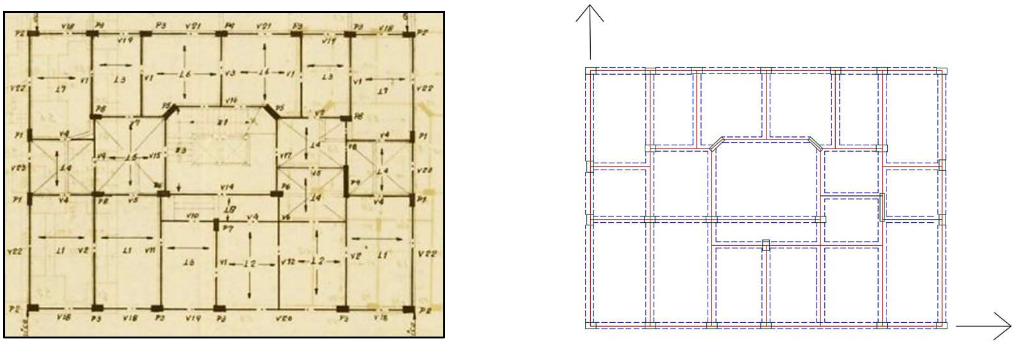
viii. **Beam reinforcement ratio statistical analysis**

Concerning the beam statistical analysis, it was necessary to compare the interior and border beams since the interior ones, usually, have restrictions in their design because they have to respect the clear headroom. The longitudinal reinforcement ratio observed in the border beams and interior beams tends to follow a gamma distribution, with an average reinforcement ratio equal to 0.6% and a COV = 84% and an average reinforcement ratio equal to 0.5% and a COV = 71%, respectively. The transversal reinforcement ratio does not vary with the beams type, and the average is equal to 0.2% with a COV = 116% (both tend to follow a lognormal distribution).

## 3. Case study – Identification and modeling

The most representative building of Benfica (northern area) was sellected according to the results of the previous section. This building is located on Estrada dos Arneiros nº34, which belongs to a crowded location in Benfica. It is a residential frame type RC building with six floors built in 1965. This structure was designed following the portuguese regulations in practice at the time, the Regulamento Geral das Edificações Urbanas – RGEU and it is possible to identify some details (from the structure project) that are usually associated to poor seismic behaviour (RGEU, 1951). The columns have inadequate longitudinal and transversal reinforcement ratio (without spacing reduction close to beam-column joints). The beam sections are constant along the height of the building and have poor longitudinal and transversal reinforcement ratio. It is worth mentioning that both columns and beams have low transversal reinforcement area, and this confined concrete level has an adverse effect on the structural elements ductility. In fact, the poor seismic behaviour of RC buildings built before 1983 in Portugal, in general, is due to brittle collapse failures. The building was modelled and analysed with OpenSees software as represented in Figure 5, using fibre models with distributed plasticity, represented by a fibre section to which the defined materials are associated (for each structural element). Regarding the materials described above, the RC properties were modelled with a unconfined B225 concrete and the smooth A40 steel bars. The infill panels (mansonry walls) were modelled as two-diagonal compression struts.

Regarding the building model, the information that was not found in the original building project, was assumed from the statistical analysis results presented in the former chapter.



(a) (b)

Figure 5 – (a) Building plan available in Archive and (b) AutoCAD building layout.

## 4. Dynamic characterization of the building and model calibration

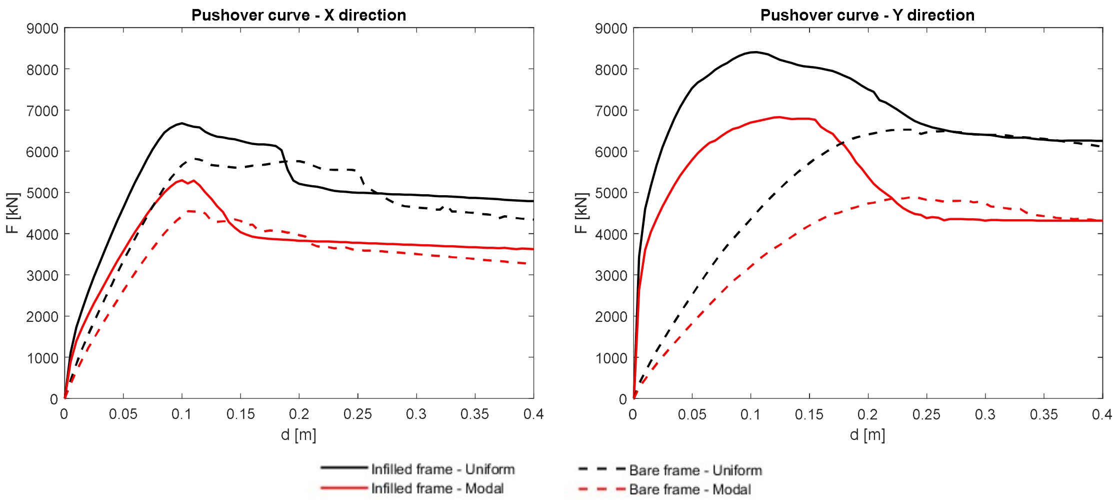
In order to validate the computational model, were performed an *in situ* ambient vibration tests to obtain thebuilding fundamental translation and torsion frequencies. The results of the *in situ* tests were recorded with an accelerometer as a wave graph and transformed using the Fast Fourier Transformation. This method provides the modulus spectrum of the experiment magnitude in each direction. Note that the eigenfrequencies of the vibration modes are identified by overlaying the spectrum line of all experiments for each direction on the same scale. Due to the structure being more rigid in one direction than the other, the direction of higher stiffness corresponds to the direction with higher frequency value, which results in the fundamental vibration mode. The numerical model developed in OpenSees was then subjected to several changes to approximate the actual model dynamic characteristics to the experimental dynamic characteristics (from the vibration test). According to EC8, unless a more accurate analysis of the cracked elements is performed, the elastic flexural and shear stiffness properties of concrete and masonry elements may be taken equal to 50% of the corresponding stiffness of the uncracked elements (EC8-1, 2004). The building was modelled as a bare and infilled frame, and then modelled alone and with adjacent buildings (as a block). The model with the dynamic characteristics closest to the ambient vibration test results was the model with masonry walls set into the block (which corresponds to the actual situation). Table 2 shows the frequencies obtained for each direction from the ambient test and for the numerical model, and also the error associated between both.

Table 2 – Dynamic characteristics *in situ* test results and the error related to the building in the block model

| Mode | Direction | Experimental frequency [Hz] | Model frequency [Hz] | Error [%] |
|---|---|---|---|---|
| 1 | X Translation | 3.60 | 3.78 | 5 |
| 2 | Y Translation | 7.20 | 6.15 | 15 |

## 5. Seismic assessment and retrofit solution

Following the numerical modelling of the case study, a seismic assessment was performed by means of a pushover analysis. The result is a pushover curve that plots the shear force at the base of the building as a function of the displacement at the centre of mass of the second-to-last floor (since the last floor is recessed and partial). The seismic action was defined according to Part 1 and Part 3 of EC8 (NP EN1998-1, 2010; NP EN1998-3, 2017). Two types of load distribution were selected to perform the pushover analysis: uniform and modal, for both main directions, X and Y (NP EN1998-1, 2010). Figure 6 shows the pushover curves obtained for the building inserted in the block as a bare and infilled frame.




(a) (b)

Figure 6 - Pushover curves for the bare and infilled frame models considering both lateral load types and for the (a) X and (b) Y direction

The collapse of the masonry walls is easily observed in the pushover curves with the drop of the structure resistance after reaching its maximum base shear force. After this point, the pushover curves of the bare and infilled frame models tend to overlap, meaning that at this stage, only the RC frame structure resists the lateral load. This is particularly evident in the curves corresponding to the transverse direction due to the presence of masonry wall model. Moreover, due to the higher structure's rigidity and strength in this direction, the structure's capacity is also higher when compared to the longitudinal direction (X direction).

Regarding the presence of the masonry walls, their presence increases the resistance and initial stiffness of the structure, which contributes to its seismic behaviour. As expected, the uniform lateral load leads to higher values of basal shear force, being the modal lateral load the most conditioning. Then, both models were subjected to a structural safety verification following Part 3 of EC8 (NP EN1998-3, 2017), where the presence of ductile and brittle mechanisms is evaluated by comparing the chord rotation and shear capacity with the demand, respectively. Since the case study is a residential building, only the significant damage (SD) limit state has be to verified in this seismic assessment. Even though the seismic safety of all structural elements was evaluated, only the results for the columns will be considered since was not detected any significant damage for the beams. Due to the reduced rate of transverse reinforcement, the columns have shown premature brittle mechanisms, demonstrating the inadequate behaviour of this buildings to shear forces. Hence, the displacement for which the brittle mechanism occurs is considered the ultimate displacement of the structure for the SD limit state.The displacements corresponding to attainement of brittle and ductile failure for both models and both lateral loads considered is shown in Table 3. Only the results concerning the X direction are shown because they are more stringent.

Table 3 – Displacement values corresponding to the attainment of each failure for the X direction

| | | **Uniform Load** | | **Modal Load** | |
|---|---|---|---|---|---|
| **Direction** | **Type of failure** | **Bare frame** | **Infilled frame** | **Bare frame** | **Infilled frame** |
| X direction | Brittle failure $d_b$ [m] | 0.015 | 0.015 | 0.020 | 0.015 |
| | Ductile failuire $d_d$ [m] | 0.120 | 0.110 | 0.115 | 0.110 |

The ultimate displacement was then compared to the target displacement obtained from the application of the N2 method (NP EN1998-1, 2010) for two types of seismic action in Lisbon: seismic action type 1.3 and type 2.3.

Once again, the results obtained have shown that the modal lateral load conducts to a weaked structural behaviour since it leads to higher period of vibration of the equivalent system (single degree of freedom), which is associated to a lower stiffness. Seismic action type 1.3 has given higher values of target displacement, being the most critical and according to law decree – *Portaria nº302/2019 (*DRE, 2019), if an existing building does not guarantee the safety verification considering 90% of the seismic action defined in Part 3 of EC8 (NP EN1998-3, 2017), a seismic reinforcement solution has to be developed.

Table 4 exhibits the target displacements obtained for the total seismic action and for 90% of it. Comparing the ultimate displacements displayed in Table 3 with the target displacement in Table 4, none of the models meet the safety requirements since the structure seismic capacity is exceeded before the target displacement (which means the structure will have to be reinforced).

Table 4 – Target displacement considering EC8-1 (NP EN1998-1, 2010) and law decree – *Portaria nº302/2019 (*DRE, 2019) for seismic action type 1.3

| **Seismic action – Type 1.3** | | | | |
|---|---|---|---|---|
| | **Bare frame** | | **Infilled frame** | |
| **Target displacement [m]** | **X direction** | **Y direction** | **X direction** | **Y direction** |
| $d_t$ (NP EN1998-1, 2010) | 0.052 | 0.068 | 0.042 | 0.017 |
| $d_t$ (DRE, 2019) | 0.047 | 0.062 | 0.038 | 0.016 |

Since the structure's safety is not verified due to a brittle failure, the reinforcement solution has to increase the column shear capacity, in particular for the longitudinal direction (X direction). In this context, it was decided to apply Carbon Fiber Reinforcement Polymers (CFRP) sheets around the columns to increase the concrete confinement. According to Part 3 of EC8 (NP EN1998-3, 2017), the confinement of columns with this material increace the shear resistance and also the ductility and the compression capacity.

The retrofitting solution will be only designed and applies for the infilled building inserted in the block. First, the columns with brittle failures were identified (before target displacement was reached) and 19 out of 26 columns (total in plan) did not meet safety in first storey. For these columns, a retrofit layout was developed to make sure that the shear demand of the target displacement could be supported and for the most critical column, the following values were obtained: $V_{Demand}$ = 126 KN and $V_{Resistance}$= 43 KN. Since this is an iterative approach, the design of the retrofiting solution has started by selecting the CFRP's properties from a catalogue: SikaWrap-230C sheet from Sika AG (SIKA, 2021).

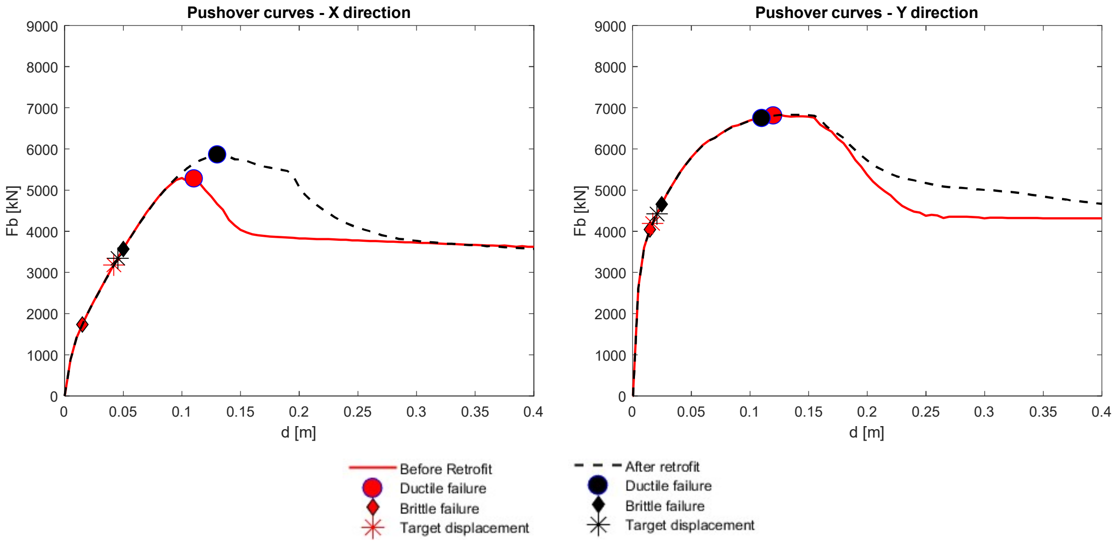
As mentioned before, this retrofitting solution improves the columns shear capacity and increases the compressive strength of the concrete. Hence, to account to this effect, it is necessary to increase the concrete parameters in the numerical model. Following Part 3-A.3.2.2 of EC8 (NP EN1998-3, 2017), the new values of the ultimate tensile stress of the confined concrete, $f_{cc}$, the respective extension, $\varepsilon_{cc}$, and the ultimate extension of the fiber in the compressed zone, $\varepsilon_{cu,c}$ were calculated.

In an initial phase, the retrofit solution was applied to all the previously identified columns and a new structural safety verification was perfomed. The results have shown that some columns that were previously verifying safety were no longer doing so. This is explained by the fact that the application of this retrofiting solution in several columns results in global stiffness increase and, therefore, has induced more lateral forces in the frame structure than expected. In summary, the retrofiting design in numerical models is usually a long and complex process to optimize due to the required iterations. Since this process began with the retrofit solution for the most constraining column, the final result of the reinforcement can be considered to be over-dimensioned for many columns.

Finally, Table 5 displays the target displacement for the retrofitted building and also the displacements corresponding to the attainment of brittle and ductile failures. Clearly, the retrofitting solution has improved the seismic behaviour of the structure.

Table 5 – Target displacement and ultimate displacement for the retrofiting infilled frame model

| Displacement | X direction | Y direction |
|---|---|---|
| Target $d_{t,ret}$ [m] | 0.045 | 0.021 |
| Brittle Failure $d_{f,ret}$ [m] | 0.060 | 0.025 |
| Ductile failuire $d_{d,ret}$ [m] | 0.130 | 0.110 |




(a) (b)

Figure 7 - Pushover curves with the indication of the target displacement and the failure displacements for the constraining action before and after retrofitting solution for (a) X and (b) Y direction

## 6. Concluding remarks

This study started with the development of a database, that contains all the available information of RC buildings located in the northern area of Benfica. Good knowledge about the building stock of a certain city can be used to estimate its seismic vulnerability. From a statistical analysis performed on this database it was possible to identify the main characteristics of RC buildings built between 1950 and 1983 located in Benfica.

The values obtained are close to the results of similar studies, highlighting the interest in the development of additional databases in Lisbon. From the statistical analysis, the most representative RC building of Benfica was selected, numerically modelled, and analysed by nonlinear static analysis. Analysing a building believed to be representative of a certain area, allows to predict the damage distribution of similar typologies located in the same area. To calibrate the numerical model were performed an *in situ* ambient vibration test to obtain the fundamental dynamic characteristics of the building. As expected, the numerical model with masonry infill walls and located in a block provided the better approximation with the ambient vibration test results. Regarding the results of the nonlinear static analysis, it was found that the behaviour of the structure with the adjacent buildings is beneficially influenced by the presence of the masonry walls. Moreover, the results with the modal lateral load type are more critical, regardless of the direction. Then, following a seismic damage assessment as proposed by EC8-3, it was verified that the building does not meet the minimum safety requirements due to premature columns brittle collapse (poor shear resistance capacity). This is due to the inadequate transverse reinforcements of the columns, leading to the brittle collapse failure. The structure reaches its ultimate displacement (associated to a brittle failure) before reaching the target displacement (regardless of the direction).

Considering the structure's inadequate seismic behaviour, a strengthening solution employing CFRP was designed and numerically implemented in the structural model to confine the columns and increase the shear resistance capacity (since the beams do not have any safety problems). The results demonstrate the effectiveness of the retrofitting solution to eliminate the brittle mechanism. In conclusion, the study and knowledge of RC buildings built before 1983 allows us to adopt proactive measures regarding their vulnerability and seismic risk by means of retrofitting solutions.

## 7. References

(Borges, J. et al., 2001). J. Borges, A. Fitas, M. Bezzeghoud e P. Teves-Costa, "Seismotectonics of Portugal and its adjacent Atlantic area". Tectonophysics, p.373-387, 2001.

(DRE, 2019). *Diário da Républica Eletrónico*. "Diário da República n.º 175/2019, Série I de 2019-09-12 - Portaria n.º 302/2019".

(EC8-1, 2004). European Committee for Standardization. "EN 1998-1 – Eurocódigo 8: Design of structures for earthquake resistance " - Part 1: " General rules, seismic actions and rules for buildings ". Brussels, 2004.

(Furtado et al., 2015). A. Furtado, C. Costa, A. Arede e H. Rodrigues, "Geometric characterisation of Portuguese RC buildings with masonry infill walls." European Journal of Environmental and Civil Engineering, p. 396-411, 2015.

(Gago, A., 2011). A. Gago e J. Proença, "*Parque Escolar: Reforço Sísmica de Edifícios Escolares*", 2011.

(LNEC, 2019). Laboratório Nacional de Engenharia Civil "*Metodologia para a avaliação da segurança sísmica de edifícios existentes baseada em análises de fiabilidade estrutural - Edifícios de betão armado*". p. 32-134, 2019.

URL: http://www.lnec.pt/fotos/editor2/DE/rel081_2019_versao_revista.pdf, Accessed on July 2021.

(NP EN1992-1-1, 2010). Potuguese version of EN 1992-1-1:2004 + AC:2008 "*Norma Portuguesa – Eurocódigo 2: Projeto de estruturas de betão – Parte 1-1: Regras gerais e regras para edifícios*". Portugal, 2010.

(NP EN1998-1, 2010). Potuguese version of EN 1998-1:2004 + AC:2009 "*Norma Portuguesa – Eurocódigo 8: Projeto de estruturas para resistência aos sismos – Parte 1: Regras gerais, ações sísmicas e regras para edifícios*". Portugal, 2010.

(NP EN1998-3, 2017). Potuguese version of EN 1998-3:2005 + AC:2013 "*Norma Portuguesa – Eurocódigo 8: Projeto de estruturas para resistência aos sismos – Parte 3: Avaliação e reabilitação de edifícios*". Portugal, 2017.

(REBA, 1967). *Regulamento de Estruturas de Betão Armado* – REBA, "Decreto nº 47723, 20 de Maio de 1967" Portugal, 1967.

(REBAP, 1983). *Regulamento de Estruturas de Betão Armado e Pré-esforçado* – REBAP, "Decreto-Lei nº 349-C/83, 30 de Julho de 1983" Portugal, 1983.

(RGEU, 1951). *Regulamento Geral das Edificações Urbanas* – RGEU, "Decreto-Lei n.º 38-382, 7 de Agosto de 1951" Portugal, 1951.

(SIKA, 2021). Sika AG – Technical specification of SikaWrap-230C carbon fiber sheet.

URL:https://prt.sika.com/pt/solucoes-para-construcao/ref-colagem-estrutural/reforco-estrutural/sikawrap-230-c.html, Accessed on July 2021.

(Silva, V. et al., 2014). V. Silva, H. Crowley, H. Varum, R. Pinho e L. Sousa, "Investigation of the characteristics of Portuguese regular moment-frame RC buildings and development of a vulnerability model," *Bull. Earthq. Eng.*, p.1455-1490, 2015.