Evaluation of the seismic resistance of a framed reinforced concrete building Application to Military Building PM 203 Lisbon

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Abstract

The society's use of history as a source and guideline for the future is frequent. History shows that a seismic event may have consequences, larger or smaller, depending on the vulnerability of buildings.

The seismic vulnerability of buildings in Portugal, and of the ones from Army's, is a concern since old buildings, including those built before the 80's, were not built on a normative basis with the adequate seismic design considerations. Consequently, the seismic evaluation of the Army's building stock is required urgently to determine the needs of interventions and / or reinforcement, to guarantee their operationality in case of earthquake.

The main objective of this work is to identify the possible seismic vulnerabilities of the building. Thus a seismic assessment is needed and it is performed through an 3D computer modelling of the structure (developed in the SAP2000 program). The model is initially calibrated based on the results obtained with the in-situ environmental vibration tests and then non-linear static (pushover) analyses are developed. Different models were evaluated to contemplate the differences between design and built as well to evaluate the effects of infilled walls. Finally, the verification of the performance requirements, imposed in part 3 Eurocode 8, was made comparing the seismic demand, obtained with method N2 (static non-linear analysis) with capacity.

It is concluded that the structure has an adequate behaviour in bending and fulfil the requirements, but, in of shear a great number of columns have a premature brittle collapse.

Key-words: Reinforced concrete building; Environmental vibration test; seismic analysis; nonlinear static analysis; Eurocode 8 Part 3; method N2

Introduction

This master's thesis is part of the course in Military Engineering, and its theme is the evaluation of the seismic resistance of a reinforced concrete building - Application to Military Building PM 203 Lisbon.

Recent events showed us the great harm produced natural disasters and also the difficulty in anticipating them. Earthquakes are among these natural disasters. The danger associated with this lies with the building's damage and consequently the dangers to which people are subjected.

The army has a lot of buildings and all of them have some sort of importance to the institution. We can highlight some buildings for their historical and patrimonial value, such as the Army General Staff building (EME), the Military Museum or even the Queen's Palace in the Military Academy Headquarters (AM-Sede).

There are also buildings from the 60's and 70's, such as the CANIFA facilities, which were constructed before the standards for reinforced concrete (RC) were defined (REBAP 1983). In fact, the army has 392 military facilities, of different types, natures and ages, spread throughout the national territory. This fact arises the concern to study the necessity to reinforce these existing structures. The chosen as a case study is an old RC building from 1972 located in Lisbon.

The main objectives of this thesis were:

(i) Identifying the main structural inadequacies of the studied building for seismic performance, and analysing them based on the principles defined according to capacity design;

(ii) Apply the methodology proposed in the Euro code 8 - Part 3 (EC8-3) (CEN 2005), for the seismic evaluation of the structure, which as similar characteristics of the constructed buildings in the 70s in Lisbon, Portugal.

The used methodology involves:

(a) the development of a 3D computational model based on the nonlinear behaviour of the structure, using the structural analysis program SAP2000 v18.2.0 (CSI 2016).

(b) then a calibration and validation of the linear model are done by comparing the fundamental modes and frequencies obtained numerically with those obtained from the dynamic characterization test, more specifically an ambient vibration test.

(c) A sensitivity study analysing the dynamic characteristics of the building considering different approaches in the modelling of the building;

(e) The evaluation of the seismic performance of the structure through a static non-linear analysis, using the N2 method defined in the first part of Eurocode 8, EC8-1 (CEN 2010b);

w. Analysis and interpretation of the progressive collapse of the different structural elements and identification of the target elements for the subsequent intervention.

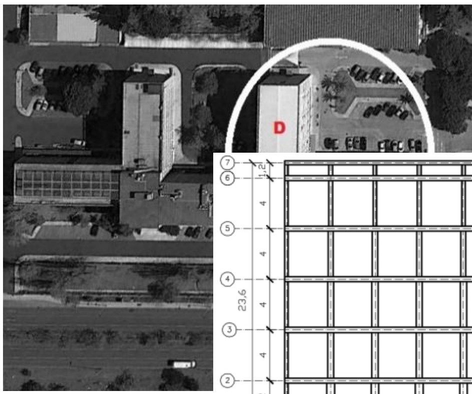
Case Study

The criteria used to choose the possible case studies were: that it was a framed RC building and located in the Lisbon area (in a seismic zone and accessible for visits for the characterization and performance of environmental vibration tests); that it was in current use and that it's construction date precedes the one of the REBAP´s entry in use; that it had design elements with sufficient information for a seismic evaluation and, finally, that it was a challenging to study.

The chosen building is the Army Geospatial Information Centre (CIGeoE), former Army Geographic Institute (IGeO), whose project dates to 1971. This building is in Avenue Dr. Alfredo Bensaúde between Encarnação and Moscavide (Figure 1) Lisbon, and its use is intended for offices. It has two distinct parts, with different functions and uses: on the left there is the military chemical and pharmaceutical laboratory (LMPQF) whose date of construction is 1968; and to the right the building under analysis CIGeoE built in 1976.

The CIGeoE is composed of a set of 3 buildings (B, C and D, identified in Figure 1, separated by expansion joints). It has 8 floors in blocks B and D, and 7 floors in block C, two of them (C and B) have an accessible terrace. They exhibit a height of approximately 27 and 23 meters, respectively. In the roof block C the is a pergola in RC and in block B there is an observatory. Block D's roof has a sandwich panel cover.

Figure 1 –CIGeoE's location (adapted from Google Earth®, 2016)



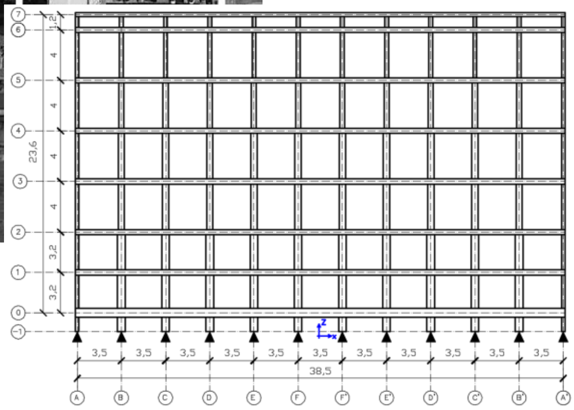
The building in study is block C, presented in Figure 1, which has the first two floors with a height of 3.20 meters and in the remaining floors has 4 meters. It also has a rectangular geometry with the dimensions of approximately 15 by 39 meters.

The structural system of the whole building is a framed structure, as shown in Figure 2 and Figure 3.

The structure does not have any type of resistant walls, being constituted solely by slabs, beams and columns. These elements are distributed as follows: slabs with a thickness of 0,12 m in both directions; Beams with different sections depending on the floor and alignment (see Table 1); Columns with a rectangular cross-section, decreasing in height, some of which are oriented according to Y axis, and others in accordance with X axis;

It also has two stairs, identified as E1 and E2 in figure 2, with E2 only going up to the third floor.

The foundations system is established with isolated footings. In the periphery of the building the footings are connected by foundation beams. In the project no information is given on the grounds on which the structure is founded, nor the depth of the footings. Given the necessity for this data in modelling, we assumed a depth of 2m for the footings.



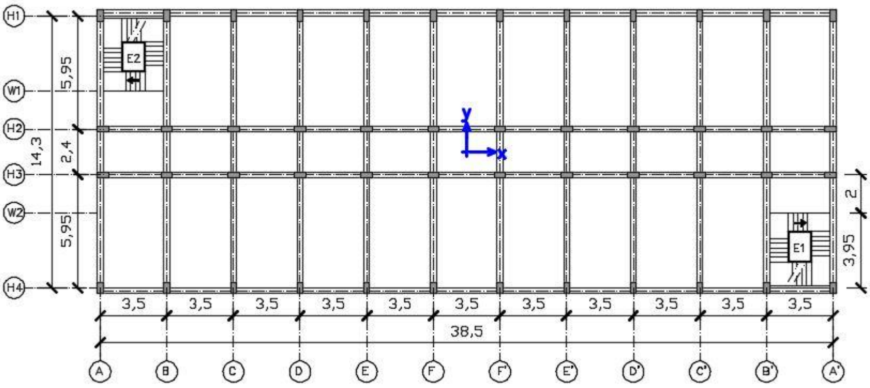


Figure 2 – Illustrative drawings of the structure of the building

Modelling

The software used to model the studied building was the SAP2000 v18.2 (CSI 2016) (Figure 3). This program allows a wide range of analyses. Nevertheless, in this thesis the following analyses were used: modal analysis, linear and nonlinear static analysis (pushover).

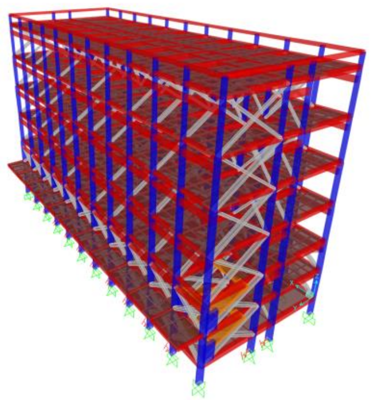


Figure 3 –3D model of the studied building in SAP2000® software (CSI 2016)

According to the information in the original project, the materials used in the studied building were a B225 concrete (equivalent to a resistance class of C20 / 25 defined in Eurocode 2 part 1, EC2-1, (CEN 2010a)) reinforced with A40N steel rods, whose mechanical properties are defined in the regulation of reinforced concrete structures (REBA 1967). Several concrete constitutive models were defined, varying with their degree of confinement, structural function (beams or columns) and dimensions. The constitutive relation that was adopted for the steel rods was defined according to the model proposed by (Park e Paulay 1975).

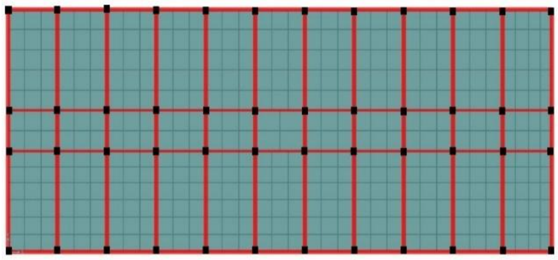
Table 1 shows the relevant properties of A40N steel considered in this thesis.

Table 1 – Properties of A40N steel rods

Proprieties	***Steel A40N***
Elasticity Module	$E_s = 210\ GPa$
Poisson Coefficient	$\upsilon = 0{,}30$
Tensile strength	$f_{sy} = 392{,}3\ MPa$
Tension break	$f_{su} = 470{,}7\ MPa$
Extending yield	$\varepsilon_{sy} =$
Hardening extension	ε_{sh} =0,010
Breaking extending	$\varepsilon_{su} = 0{,}14$

The beams and columns of the structure were modelled as linear bar elements (Figure 4). The nonlinear behaviour of these elements was done using the software SAP2000 Section Designer program (CSI 2016).

For the modelling of the slab ladders, there was a concern to properly model the axial stiffness of the inclined slabs, which have a negative effect on the columns in their supporting zones. To guarantee the right representation of this effect these elements had their torque released at one end and the bending moments at both ends. The remaining slabs of the structure were modelled with 0,12



m thick shell elements.

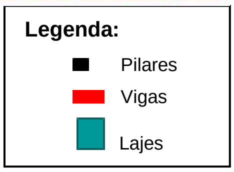


Figure 4 – Different elements of the modelling in SAP2000®

Given the uncertain nature and composition of the soils beneath the building, conservatively, the deformability module assumed was 400MPa.

Despite being complex the simulation of soil-structure interaction is possible using the Winkler model. This model allows this modelling by assigning a set of independent springs with linear and elastic behaviour to the footings. It were assigned springs of rotation in both directions of footing design (XX and YY).

The masonry infilled walls are not usually considered in the design of new buildings. However it may not be conservative in the analysis of existing buildings. Therefore, in this context the EC8 indicates that a consideration should be given to the filling walls.

In this work the method used was the one proposed by Mainstone (1971), where he proposes the modelling of the infilled masonry walls elastic behaviour with diagonal struts (figure 5) simulating the compression behaviour of the infilled walls (Fardis 2009).

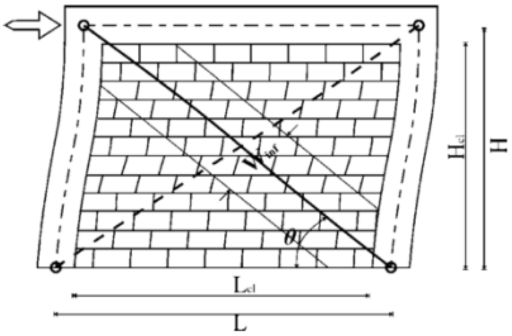


Figure 5 – Modelling the infilled masonry wall using a compression strut (Fardis 2009)

In order to characterize this phenomenon better it was adopted a double-struct model, taking into account that as stated in (Madan et al. 1997) the wall offers equal resistance to the lateral forces in both directions of the same plane. Nevertheless, the width of the struct (W_{inf}) was equally divided by each strut. This model does not take into account the form of collapse of the wall, which can be in the plane or outside of it. The proprieties of masonry bricks were considered those defined in the standards (NP834 1971). The modulus of elasticity of the walls can be estimated between 500 and 1000 times the compressive strength of the bricks (Fardis 2009), from which was adopted the midterm of 750 times.

The influence of the openings in the wall's stiffness was considered by applying a reducing factor to the compressed diagonals thickness (λ_0), which varies with the type, location and area of the opening.

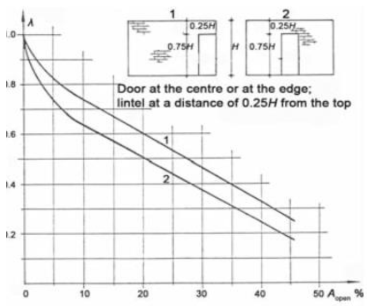


Figure 6 - Reducing factor to the compressed diagonals thickness (λ_0) (Fardis 2009)

For the noon-linear modelling the following options were considered: Beams and columns were modelled with plastic hinges ate at each end of the elements (model of concentrated plasticity); the masonry infilled walls were modelled using the "Axial P" hinges in the SAP2000® program.

The gravitational forces in the structure are a result of the self weight of the structural and non-structural elements, named permanent loads (CP) for the weight of slabs, columns, beams and other permanent loads (RCP) for the weight of coatings, partition walls and facade elements. Another type of load is the loading resulting of the building's use, named overload (Sc). These loads were applied as distributed loads to the floor slabs, except for the outer partition walls loads that were applied with a knife pattern on the periphery beams.

The performance of the studied building was also analysed for a reduced seismic action equivalent to a period of return of the seismic action of 308 years. It has a probability of exceedance of 15% in 50 years and foreseen in the national annex of EC8 -3 (CEN 2005).

For the seismic evaluation of the building by the N2 method, the response spectrum must be defined in the format acceleration-spectral displacement.

Seismic Analysis of the Structure

The existing building is significantly different from the original design. Also the stability and architecture projects do not match. Subsequently, in order to evaluate existing buildings it is necessary to make visual inspections and non-destructive tests on the building to better characterize it (Fib - Task Group 5.1 2003).

Thus comparing these projects with the *in-situ* observation of the building and with the existing photos from it's construction phases it was possible to conclude that the existing building is diferent from the in paper.

In order to understand the effect that these differences have on the seismic performance of the structure, there were performed different computer models, five in total. The different computational models were

used for comparison purposes, both in modal characterization and in pushover analysis.
In the static non-linear analysis the presence of masonry infills was considered, as well as the load eccentricity, predicted in EC8-3 (CEN 2005), in both directions.

For the analysis and of existing RC buildings there are requirements proposed by EC8-3.

Three boundary states (EL) being:

• Imminent Collapse Limit State (NC);
• Severe Damage Limit State (SD);
• Limit State of Limitation Damage (DL);

The national annex of EC8-3 (version not yet published) contemplates the definition of these requirements to be considered and verified in Portugal. So for buildings in Portugal and with an importance class II, it is necessary to verify the limit state of severe damages (SD) with a return period of 308 years. However, on conservatively it was performed for a seismic action with a return period of 475 years.

The EC8-3 also defines performance requirements complied by:

• Verification and detail criteria;
• Definition of seismic action;
•Analysis method.

The EC8-3 depending on the quantity and reliability of the available information outlines performance requirements complied by the following levels:

• Limited level of knowledge
• Standard level of knowledge
• Level of total knowledge

Depending on the level of knowledge about the structure, an uncertainty factor is defined. In this work it were used as base information the architectural plans; the stability project; photos from it's construction period; environmental vibration test data, performed in-situ. Although, not enough to consider the level of total knowledge, given the academic nature of this dissertation, the level of total knowledge was considered, and therefore, static non-linear analyzes were performed without material properties being minimized.

Experimental Dynamic Characterization

In-situ Environmental Vibration Testing

This type of test allows to extract the dynamic characteristics of the structures, such as natural frequencies, vibration modes or damping ratios (Zhang et al. 2002). As mentioned in (Oliveira e Navarro 2010), it is usual to use environmental vibration tests to calibrate computer models in a linear regime, since they are easier to perform.

In order to perform this test, a Kinemetrics ETNATM triaxial accelerometer was used, with EpiSensorTM internal sensors suitable for picking up low frequency vibrations (0-10 Hz). The configuration of this equipment was done through a portable computer equipped with QuickTalkTM software (Kinemetrics 1995).

After this data is processed through a analytical method we can determinate the power spectral density (PSD) functions for each location and direction. Then it is possible to identify the frequencies of the vibration modes, by superimposing these graphics. The fundamental frequencies of vibration are identified by the simultaneity of peaks in the various functions. As shown in Figure 7, it can be concluded that the fundamental vibration mode along the X axis, occurs at a frequency of approximately 1.03 Hz (identified in Figure 7 with a dashed circle in black).

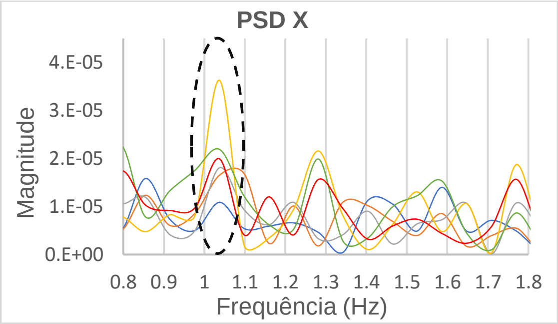


Figure 7 – PSD fuction in X axis

Then for the other direction it was possible to conclude that the fundamental vibration mode occurs at a frequency of 0.98 Hz (identified in Figure 8 with a dashed circle in black).

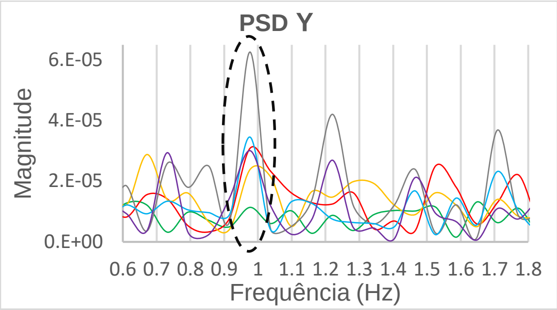


Figure 8 – PSD fuction in Y axis

For the dynamic modal analysis each modal configuration shows displacements and rotations in all directions (X, Y and Z). For each mode the following characteristics were obtained: the period T_n, and the corresponding frequency f_n; the vector of modal form Φ_n; the modal participation factors in the response to the seismic action in all directions: X, Y and Z - represented by Γ_{Xn}, Γ_{Yn} e Γ_{Zn} ;the effective modal masses in all directions (X, Y and Z), represented as M_{Xn}, M_{Yn} e M_{Zn};

The five models have their distinctions, however the most explicit are the central beams (being smaller in the stability and architecture model), the presence or not of infilled walls and the presence or not of some openings in the slabs. Taking this into account a sensitivity analyses was performed.

Through a summary analysis of the results presented in table 1, it is possible to notice the significant differences between models. The presence or not of infilled masonry walls is designed by NW (without walls) and W (with walls).

Table 1 – Frequency, f_n, of the structure in the different computer models

Mode		*Stability*	*Architecture*		*Existing*		*Results*
			NW	W	NW	W	
1	Y	0,370	0,376	0,714	0,552	0,988	0,98
2	X	0,425	0,419	0,801	0,634	1,015	1,03
3	RZ	0,448	0,425	0,874	0,653	1,149	---

The diverse results obtained confirm the high importance of an adequate modelling as close as possible to the reality. The consideration of the masonry infilled walls behaviour in the computer modelling makes the structure more rigid and the fundamental vibration frequency increases. With these results it is possible to deduce that for the seismic evaluation of RC structures it is important to model these elements, especially because for the given level of seismic intensity, the structure behaves linearly.

So, we present the main modes of vibration of the existing structure, the model closest to reality and calibrated using the environmental vibration test. The modes of vibrations are:

- Mode 1: 1st mode of vibration in direction Y;
- Mode 2: 1st mode of vibration along the X direction;
- Mode 3: 1st mode of vibration of twist, on the central axis of the structure

Nonlinear Static Assessment

After the definition of the model and calibration based on the results of the dynamic identification test performed, there were performed non-linear static analysis, starting with pushover analyses to define the resistant capacity of the structure in the two main directions (X and Y). After that the method used to evaluate the seismic performance of the structure was the N2 method.

The pushover analysis is used mostly in the area of structural evaluation and / or reinforcement of existing buildings and consists of static non-linear analysis. This analysis is performed with the structure subjected to constant gravitational loads (g) and horizontal monotonic growth forces (p)

The characterization of the resistant capacity of the structure (Figure 9) is defined from the overall deformation of the structure (through the lateral displacement in a control node dtopo) as a function of the horizontal reaction at the base of the structure Vb (basal shear force) for increasing values of horizontal lateral seismic force applied. Using this curve, it is possible to roughly evaluate overall characteristics of the structure, namely its rigidity, resistance, or even ductility.

According to EC8, it is necessary to consider an accidental eccentricity. This eccentricity is meant to represent the uncertainty about the location of the masses on the floor. So, it was considered a eccentricity of the over load in both directions.

The N2 method, proposed by EC8-1 (CEN 2010b), consists in determining the target displacement of the structure for a given intensity of the seismic action. This target displacement was evaluated for the models with and without the consideration of masonry and for the most unsteady cases of load.

The pushover analysis was performed using the different types of lateral loading: uniform distribution and modal distribution of forces, using the capabilities of SAP2000® software (CSI, 2009), which automatically defines these loads. Then it was performed for the two translation directions, X and Y, as well as for the two loading directions: positive and negative. Then it was performed to consider the effects of accidental eccentricity in both directions of translation. Thus, the pushover analyses performed totalled 24 independent models of which the most conditioning cases were selected to apply the N2 method.

By analysing the capacity graphics, it is possible to conclude that the most constraining cases in the transverse direction will be the models in which the loads have eccentricities in x + (for a sense of loading according to y +), and vice versa. For the cases of analysis according to the longitudinal (direction X - See Annex B), due to the curves being approximately equal, only a typology of model was considered, being this the model that presents / displays eccentricity of load in y-.

These curves are then transformed into an equivalent one, representing a single degree of freedom structure. After tis numerical process an idealization of these curves is performed. Form this point on we can calculate the period of the structure, it’s ductility and also de displacement in rupture.

From the analysis of the periods of the SDOF systems, it is possible to verify that the presence of the infilling walls increases the rigidity of the structure (shorter periods). It is also possible to observe that the system in the X-direction is more rigid when compared to the system response in the Y + and Y- directions.

The values of the ductility that were calculated show that the structure has a more ductile response in the X direction. With regard to the consideration of the filling walls, a negative effect on ductility can be verified. If we compare the values obtained with the requirement levels defined by EC8-1 for a new structure with a mean energy dissipation capacity, it can be observed that the values of the prescribed coefficient behaviour are about 1.5 times higher than the ductility values verified for the Y direction. In comparison, the ductility values in the X direction are already approaching in one case and in others exceeding those defined by the EC8-1.

It is necessary to calculate the target displacement of the SDOF system. The determination of this displacement will depend on the dynamic characteristics of the system itself.

As previously mentioned, in this dissertation, the structure will be evaluated for the SD limit state of EC8-3, considering the reference seismic action for 475 years and for 308 years. Then the target displacement of the original structure MDOF is calculated by a numerical transformation on the value of the SDOF target displacement.

The original capacity curves the MDOF system were compared with the target displacements obtained.

The last displacements of the MDOF system will have to be lower than the target displacements.

From the analysis of the results, it was verified that the Y direction, where there is a lower relative non-linear deformation capacity, reaches the target displacement at the threshold of the linear zone with the non-linear zone of the curve capacity. This displacement does not reach the ultimate displacement, either for the reference seismic action or for the reduced seismic action as can be seen in Figures 10 to 12.

With regard to the effect of the infilling masonry, the reduction of the target displacement due to the increase in stiffness is not very high, so that there is no great reduction in the deformability of the structure. However, it can be seen that the ultimate displacement of the structure occurs at a lower seismic intensity.

In Figure 10 to 12, the target displacements are determined above the capacity curves of the pushover analysis of the building.

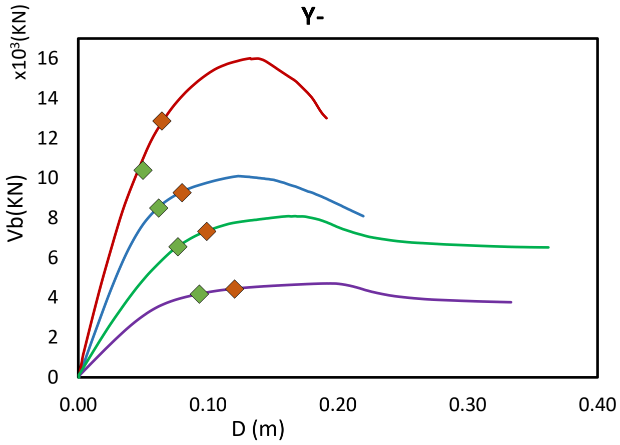


Figure 10 –Pushover curves with the target displacements in Y- direction

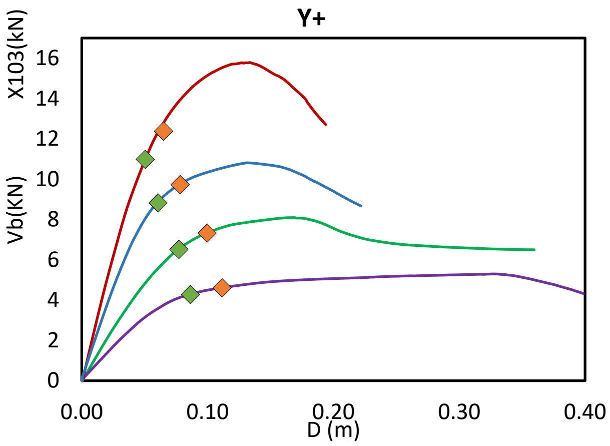


Figure 11 –Pushover curves with the target displacements in Y+ direction

Figure 12 –Pushover curves with the target displacements in X direction

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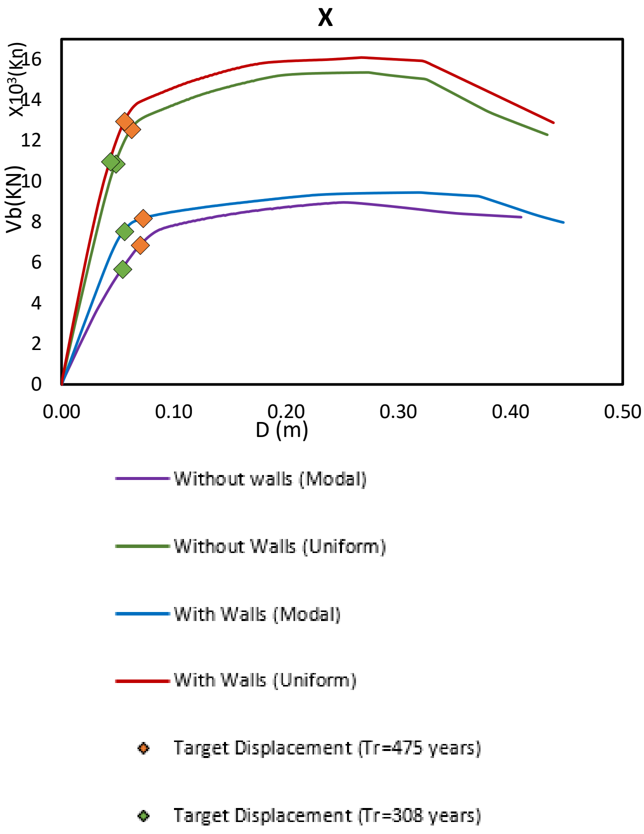

Conclusions and future developments

From the nonlinear analysis made to the structure under study it was possible to identify the main structural inadequacies, with influence on the seismic performance. In flexural behaviour of the building it was possible to conclude that the structure presents a more ductile behaviour in the longitudinal X direction, showing a greater last displacement than in the Y direction.

The inadequate detailing of the elements, namely at the level of the transversal reinforcements, allied to a smallest dimension in geometry, can justify the reduced ductility coefficients in the Y direction.

It can be observed that the structural components of the building, in both directions, that are more conditioning for the horizontal actions are the columns which for the target displacement present a shear strength collapse. By analysing the fragile collapse of the columns by shear strength it was possible to observe the high impact of considering the requirements imposed by EC8-3. It was also possible to conclude that many vertical elements are prematurely conditioned by a fragile rupture from shear strength.

Finally, is concluded that the structure has an adequate behaviour in bending and fulfil the requirements but in terms of shear strength a great number of columns have a premature brittle collapse.

In order to obtain a more complete and accurate seismic analysis of the structure, the following future developments are suggested:

• Consideration of the adjacent building, separated by expansion joints in the seismic evaluation of the building under study;

• Non-destructive or semi-destructive tests and in-situ inspections with the purpose of assessing the real properties of the materials, for later calibration of the computational model and to identify possible anomalies in the structural elements;

• Evaluation of the economic feasibility and possible dimensioning of the seismic reinforcement to be applied to the structural elements conditioning.

References

CEN. 2005. «European Standard EN 1998-3 - Eurocode 8: Design of structures for earthquake resistance - Part 3: Assessment and retrofitting of buildings». 1–97.

CEN. 2010a. «Norma Portuguesa EN 1992-1-1 - Eurocódigo 2 – Projecto de estruturas de betão - Parte 1-1: Regras gerais e regras para edifícios». 1–259.

CEN. 2010b. «Norma Portuguesa EN 1998-1 - Eurocódigo 8 - Projecto de Estruturas para resistência aos sismos - Parte 1: Regras gerais, ações sísmicas e regras para edifícios». 1–230.

CSI. 2016. «SAP2000® Integrated Software for Structural Analysis and Design».

Fardis, Michael N. 2009. *Seismic design, assessment and retrofitting of concrete buildings: based on EN-Eurocode 8*.

Fib - Task Group 5.1. 2003. *Monitoring and Safety Evaluation of Existing Concrete Structures*. fédération internationale du béton.

Kinemetrics. 1995. «Altus Quick Talk® - Altus Software and Firmware for Windows Support Software».

Madan, A., A. M. Reinhorn, J. B. Mander, e R. E. Valles. 1997. «Modeling of Mansory Infill Panels for Structural Analysis». *Journal of Structural Engineering* (October):1295–1302.

NP834. 1971. «Tijolos de barro vermelho para alvenaria - Formatos».

Oliveira, C. S. e M. Navarro. 2010. «Fundamental periods of vibration of RC buildings in Portugal from in-situ experimental and numerical techniques». *Bulletin of Earthquake Engineering* 8(3):609–42.

Park, Robert e Thomas Paulay. 1975. *Reinforced Concrete Strutures*. New York: John Wiley & Sons.

REBA. 1967. *Decreto Lei no 47/723 de 20 de Maio do Ministério das Obras Públicas - Regulamento de Estruturas de Betão Armado*. Portugal.

REBAP. 1983. *Decreto Lei no 349-C/83 de 30 de Julho do Ministério das Obras Públicas e Transportes - Regulamento de Estruturas de Betão Armado e Pré-Esforçado*. Portugal.

Zhang, Lingmi et al. 2002. «Ambient Vibration Testing & Modal Identification of an Office Building». 1–6.