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Capacity curves for reinforced concrete buildings designed in accordance with Portuguese regulation

M.J. Falcão Silva[1]

[1] *Pos-Doc Researcher and Auxiliary Professor, LNEC and ULHT, mjoaofalcao@lnec.pt*

Abstract

In recent decades, more specifically in the last 50 years, has been observed a growing concern related to seismic risks in large urban areas. Thus, as a result directly related to this awareness, new methods of analysis and design have been created and developed allowing the structures to become more resistant and seismically safe. Currently the civil engineering structures are designed to be able to survive the worst actions expected to occur during its lifetime. As such, the structures must be designed and analyzed to be endowed with the capacity to withstand the demands faced, being subject to a complex process of design covering various aspects, not just the structural and seismic safety but also other complementary aspects related to occupant safety and comfort.

This paper aims to disseminate the results obtained from the assessment of the resisting capacity of regular buildings designed in accordance with the application of the 1983 Portuguese regulation regarding the seismic action in current buildings, following the requirements of Regulamento de Estruturas de Betão Armado e Pré-Esforçado (REBAP) e o Regulamento de Segurança e Ações para Edifícios e Pontes(RSA). In these, buildings seismic analysis was based on RSA, which allowed obtaining earthquake actions equivalent forces, to be applied in each of the floors, and the resisting capacity, estimated through nonlinear static analysis (pushover). Some final remarks are presented and discussed, as well as envisaged future developments of the work in what concerns the definition of damage state thresholds and the development of analysis based on specific performance levels.

Keywords: Reinforced Concrete Buildings, Portuguese seismic regulation, Pushover analysis, Capacity curves

1. Introduction

In recent decades, more specifically in the last 50 years, has been observed a growing concern related to seismic risks in large urban areas. Thus, as a result directly related to this awareness, new methods of analysis and design have been created and developed allowing the structures to become more resistant and seismically safe. The architectural solutions adopted up to 70s associated with lack of role in the structural design of the seismic component when compared with other actions, give to buildings high vulnerability when requested by a seismic action being responsible for major materials and especially human losses, raising awareness in this way the whole civil society as well as the scientific community [1]. Currently the civil engineering structures are designed to be able to survive the worst actions expected to occur during its lifetime. As such, the structures must be designed and analyzed to be endowed with the capacity to withstand the demands faced, being subject to a complex process of design covering various aspects, not just the structural and seismic safety but also other complementary aspects related to occupant safety and comfort.

The aim of the present study is to assess the resisting capacity of reinforced concrete buildings designed after publication of RSA and REBAP and representative of Portuguese housing stock in order to obtain the corresponding capacity curves for further development of analyses based on specific performance levels. In a first phase of work were selected buildings with a frame structure, variable number of floors and different levels of ductility, which were considered representative of some typologies of the Portuguese housing stock. The design of the building structure was made in previous studies in accordance with Portuguese regulations. The chosen structures were defined based on a generic plan with a framed system constituted by columns, beams and slabs and having double symmetry in plan with its elements arranged according to an orthogonal grid.

Afterwards numerical modeling, based on geometric and material characteristics of each structural element, was conducted to reproduce the expected behavior when subject to seismic actions. Modal response analyzes and spectra analyzes on the different buildings considered were made allowing the dynamic characterization of the relevant structural system and the study of the response of the structures. Finally, capacity curves, representing the relationship between top displacement and base shear, as well as displacement profiles and interstorey drifts were obtained for the considered forces (proportional to modal shape) in both directions. In the end some final remarks will be presented and discussed looking ahead to future analysis performed over similar structures with variable floors, different levels of ductility, located in other seismic zones and based on other types of soil.

2. Regulatory framework

There are several sources that refer the existence of regulations created after the 1755 earthquake in order to equip new buildings with earthquake resisting capacity. Apparently it seems that Portugal was the first country to have a regulation devoted to the protection of buildings against earthquakes. The provisions of the Regulation, published shortly after the earthquake of 1755, were translated into original constructive solutions that can be identified in the “pombalino” buildings, which still exist in Portugal [2]. The recognition of the importance of the seismic action in the buildings behavior, and the need for their consideration in structural design is firstly introduced in the Regulamento Geral de Edificações Urbanas (RGEU) [3].

The first regulation to comprise seismic action was Regulamento de Segurança das Construções contra os Sismos (RSCCS) [4], being succeeded by Regulamento de Solicitações em Edifícios e Pontes (RSEP) [5]. Despite it was partially repealed by the following regulations, the RSCCS, complementing the generally stated in RGEU, has still in force some items applicable to small masonry structures. In Portugal, the Regulamento de Estruturas de Betão Armado e Pré-Esforçado (REBAP) [6] e o Regulamento de Segurança e Ações para Edifícios e Pontes (RSA) [7] realized, for the first time, an overview of structural reliability and modulating actions for different types of materials structures.

Regarding the current legislative panorama, in Portugal will become effective the new Eurocodes. The new design regulation of structures results from an harmonization effort in the European area of the existing rules and regulations in each country. Of these, Eurocode 8 (EC8) [8] and Eurocode 2 (EC2) [9], devoted to the seismic design of structures and to the definition of general design rules, present some modifications in what concerns the national RSA [6] and REBAP [7] regulations still in force. The changes introduced by the Eurocodes when compared with the Portuguese regulations will influence the earthquake-resistant design and the behavior of new structures, in particular the reinforced concrete buildings.

3. Case studies

3.1 Structural description

The study that served as basis for the present paper analyzes 22 buildings (frame and frame-wall) with reinforced concrete structure [10]. The structural solutions adopted in these buildings comprise slabs supported on reticulated mesh beams based on variable section over the height rectangular columns. The slabs have 0.15 m thickness constant on all floors and unload in beams also with 0.20x0.60 m^2 constant section. Depending on the structure number of floors, variable section columns in height, between 0.60x0.30 and 0.40x0.30 m2, were used. In a first phase 3 PT buildings type (frame structures) with variable number of floors (2, 4 and 8 floors) and normal ductility (η = 2.5), to be considered representative typologies with some expression in Portuguese housing stock (regulatory structures of reinforced concrete), were selected. The chosen PT structures were defined based on double symmetry 20x15m^2 generic plants constituted by frames, beams and slabs, with this elements arranged according with an orthogonal grid. The buildings composed by four (4) longitudinal frames spaced 5 to 5 meters and six (6) transverse frames spaced 4 to 4 meters with 3 meters height.

The design of PT8 building structure was made in previous studies [10], considering in addition to the permanent loads and overloads the floors the seismic action in accordance with the RSA [7] recommendations. The design of PT4 and PT2 buildings was obtained by extrapolation of the previous. It was considered that the PT structures were recessed at the ground floor, being this a very simplified solution for the buildings connection to the foundation soil. It is recalled that the simplification of the buildings connection to the ground is important since it can affect in a very significant way the stresses distribution in the structure [11]. It was not included wind action for the design of structural elements because the conditioning action was the seismic action. This procedure is due to the fact that was admitted, at this preliminary stage of the work, that the buildings structures in study are located in zone A, founded in middle soil (Art. 29.2 RSA), and a little exposed to wind. The parameters chosen to illustrate the seismic response were base shear force (V), directly representative of the force absorbed by the structures and top displacement (d) in each building and in each direction.

In Table 1 are presented the base shear and top displacement obtained in previous studies [8], for middle frames (transverse direction) and interior (longitudinal direction) of the structures object of the present study.

Table 1 – Maximum base shear and top displacement for the different studied structures, adapted from the original design [8]

	Transverse		**Longitudinal**	
Structure	V[kN]	d[m]	V[kN]	d[m]
PT2	130	0,040	170	0,017
PT4	190	0,050	260	0,047
PT8	320	0,095	410	0,110

3.2 Calculation tool

The calculation tool used was Seismostruct [11] which allow automatic generation models coupled with the visualization of all data needed for the same analysis. SeismoStruct, developed by University of Pavia, allows static (force or displacement) as well as dynamics (acceleration) action simulations and has the ability to perform several types of analyzes: i) modal, ii) non-linear static analysis (among which the analysis or conventional adaptive pushover); iii) analysis.

In the present study beyond modal analysis, that allowed obtaining the dynamic properties of the different structural systems, nonlinear static analyzes (conventional pushovers) have been developed. The pushover analysis allowed estimating both resistant and deformation capacity of buildings by nonlinear static analysis, enabling comparison with specific performance levels. The use of pushover analysis takes into account geometric and materials non-linearities, as well as the redistribution of the internal forces, thereby defining the structural capacity curve which represents the evolution of top displacement in function of base shear. With this type of analysis it is possible to determine the deformation of the element, the internal forces developed as well as the floors drifts or global drifts [11, 12].

3.3 Numerical modeling

Numerical modeling was performed with the goal of reproducing the expected behavior of structures against seismic actions and was based on geometric characteristics and material of each structural element of the different buildings studied. The structures were modeled using the average dimensions. To simulate more accurately internal stresses transmission between structural elements caused by the discontinuity between elements were used, where necessary, rigid sections. The slab contribution to the beams strength and stiffness was taken into account considering the effective width of T-beams, calculated using the proposed in REBAP Art.ª 88 [4], compatible with EC8 design provisions. The analyzes performed in numerical models were developed for the normal ductility structures, designed in accordance with RSA recommendations [5] and proposed in previous studies[8].

3.3.1 Materials

The PT buildings structures, object of the present research, are in reinforced concrete with B25 concrete and A400ER steel. In the concrete behavior's modeling a non-linear model with constant uniaxial confinement, initially proposed by Madas [13], was used. This model, based on the constitutive Mander et al. [14] relation, later modified by MartinezRueda and Elnashai [15] presents characteristics in accordance with Table 1. The concrete was modeled with different confinement factors ranging from kc = 1.0, for not confined concrete, and kc = 1.2, for confined concrete. In the steel's behavior modeling, the bilinear elastoplastic Menegotto-Pinto [16] with cinematic hardening model was used.

For this model the elastic range remains constant during the different loading stages and the kinematic hardening rule for assignment of the surface is assumed as a linear function of the increase of the plastic extension. This model is also characterized by easy calibration parameters and for its computational efficiency. It may be used in steel structures modeling as well as in models of reinforced concrete. The characteristics adopted for the steel of the PT buildings are indicated in Table 2

Table 2 – PT structures materials characteristics

B25 Concrete				
E_c [GPa]	f_{ck} [MPa]	f_{ctm} [MPa]	ε_{cm} [%$_o$]	γ_c [kN/m^3]
29,4	25	2,2	2,5	25
A400 ER Steel				
E_c [GPa]	ε_{sy} [%$_o$]	F_{sy} [MPa]	E_{sh} [MPa]	ε_{sm} [%$_o$]
210	1,9	400	0,58	100

3.3.2 Sections and elements

The nonlinear bending behavior of the transverse and longitudinal frame elements of the PT structures was modeled using distributed plasticity beam-column elements with type, with integration method based on displacement. These elements consider the geometric nonlinearity and the physically non linear behavior of the materials. The sections were subdivided into individual fibers, through which the integration of the material uniaxial nonlinear response was performed, obtaining the stress-strain state in the sections of the elements.

The propagation of nonlinearities through the elements length in beam-column elements is obtained from the cubic nonlinear formulation proposed by Izzuddin [17]. In numerical integration of the equations are used two Gauss points per element. The results obtained with the SeismoStruct program are always referred to the sections Gauss points and not to the ending sections of the elements. It should be noted that the Seismostruct considers the geometric nonlinearity automatically. In order to accurately estimate the propagation of plasticity along the elements length ware used discretizations ranging from 5 to 6 elements depending on the length of each structural element (beam or column).

The slabs rigid diaphragm effect was taken into account on each floor and in all PT type buildings models studied. This assumption, based on the use of the referred fiber elements, can lead to the artificial hardening / strengthening of beams, since these will be prevented from axial deformation, which will condition the moment-curvature relation. It is recalled that the unrestricted reinforced concrete elements subject to bending present axial deformation, as the neutral line is shifted from the center of gravity of the cross section. Aware that this effect is actually present in real buildings, in certain circumstances is still difficult to conceptualize how it can be simulated with a more realistic stiffness and without the use of shell elements [18]. However, there are experimental tests that have shown good results with the use of rigid diaphragms associated with distributed plasticity elements models [19].

The masses were linearly distributed along the beams and rotations at the base of the vertical elements were completely restricted.

3.3.3 Actions

Distributed vertical loads were assumed in the beams to simulate the static actions, including the reinforced concrete structural elements weight and the almost permanent value of the regulatory overload. The value of the permanent action comprising the slabs, beams and columns self weight, beams was 25kN/m^3. The slabs uniformly distributed vertical action (15cm thickness slab weight) was3.75kN/m^2 and the coatings weight was 2.50kN/m^2. The considered overload corresponds to a private use in which the people's concentration was the predominant element, taking the value 2kN/m^2 (ψ_2=0.2) in accordance with RSA [5].

4. Numerical analysis performed

4.1 Modal analysis

Linear plane modal analysis as well as response spectra analysis of the various PT type structures considered ware performed, which has allowed the dynamic characterization of the relevant structural system and to study the linear response of the building structures. With the purpose of calibrating the numerical models for each building type, modeling was complemented with a comparison with the results of previous studies carried out by LNEC [8].

4.2 Non-linear static analysis (pushover analysis)

For the corresponding transverse and longitudinal frames, as well as for all the non-linear models of the PT type structures, non-linear static analysis were performed . Normally, the more frequent forces pattern used in non-linear static analysis are the uniform, the inverted triangular or modal shaped pattern. Considering small-medium sized structures, with mass and stiffness uniform distribution in height, the configuration of the fundamental modes of vibration is similar to the inverted triangular forces pattern. In the present study were used load distributions proportional to the most significant vibration modes and corresponding to the lateral force applied in a certain node, proportional to the mass and to the normalized modal displacement relative to the control node. Following these nonlinear static analyses was possible to assess:

- Capacity curves (control node), through the progressive application of lateral load pattern until the required performance is achieved and associated with the maximum displacement. As the loading increases, the different building elements will enter yield, thus lowering the overall stiffness of the structure. From these curves, it becomes possible to identify various relevant parameters of the structures seismic response, such as stiffness variation with load increasing, the maximum base shear reached during the analysis and information about how the structure behaves under not linear conditions [12];
- Deformation, as the displacements on each floor in relation to the position at rest. In many circumstances the displacement profiles corresponding to the maximum displacement recorded on each floor are shown;
- Drift, as the ratio between the displacement difference among adjacent floors and the height between floors, is presented as a percentage

The results of non-linear static analysis, in particular with respect to resisting capacity curves and drift, constitute relevant data to conduct a seismic vulnerability assessment oriented for checking performance objectives. This type of evaluation allows, for example, for any type of PT structure studied verifying the actual need for a possible global or local strengthening, in particular, and the areas of greatest need for intervention. As such, this type of study may be important to preview, in future, less restrictive consequences of seismic actions, which result in a higher protection for buildings and surrounding areas users.

The more relevant results obtained during the study developed, both for dynamic analysis both for the non-linear static analysis (Pushovers) are resumed in section 5.

5. Numerical analysis results

5.1 Modal analysis

2D Modal linear response analyzes and spectra analyzes on standard PT different buildings considered were made allowing the dynamic characterization of the relevant structural system and the study of the linear response of the building structures. With the purpose of calibrating the numerical models for each building type, modeling was complemented by a comparison with the results of previous studies carried out by LNEC during the 1980’s.

The vibration modes for PT structures studied were obtained by means of simplified linear models (2D frames for longitudinal and transverse direction) on the selected and described calculation tool. This procedure was called AM_{2014}. The fundamental frequencies values obtained for frames oriented according to the longitudinal (L) and transverse (T) directions could be compared directly, and in situations where there was a registry, which allowed the validation of the performed numerical modeling.

Table 2 – Fundamental frequencies obtained for the different PT type structures

PT2		PT4		ΠT8	
2.47 (T)	2.62 (T)	1.47 (L)	1,46 (L)	0.91 (L)	0.87 (L)
2.58 (L)	2.69 (L)	1.54 (T)	1,50 (T)	1.08 (T)	1.07 (T)

5.2 Non-linear static analysis (pushover analysis)

Capacity curves, displacement profiles and interstorey drifts were obtained for PT type structures, considering the distributed plasticity models and the force pattern proportional to modal shape in both directions. The capacity curves drawn are characteristic and represent the relationship between base shear and top displacement (top floor center of mass nearest node).

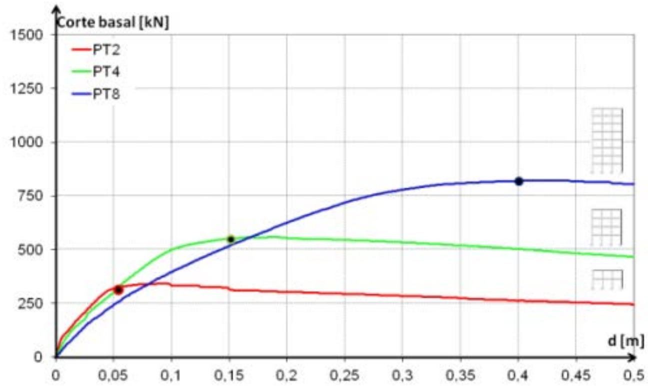

Fig. 1 – Capacity curves for PT structures: Transverse direction

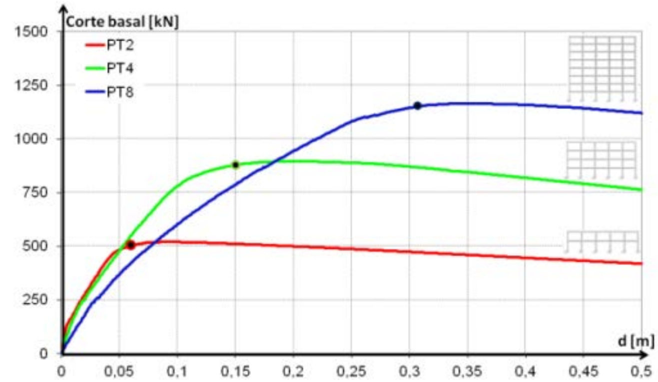

Fig. 2 – Capacity curves for PT structures: Longitudinal direction

It is possible to verify that both PT structures considered present more resisting capacity in the longitudinal direction than in the transverse direction, with approximate values of 1150kN and 760kN, respectively. Regarding the stiffness it can be observed, in both directions, a reduction from shorter PT buildings (PT2) to the higher ones (PT8). It was identified a slight degradation of stiffness or softening effect for all analyzed structures and both directions. The facts presented are compatible with geometrical characteristics and detailing of structural elements, and consequently with the results obtained from the dynamic characterization of the structures described in 4.1 and presented in 5.1.

Moreover, as a direct consequence of the application of a force pattern proportional to the modal configuration, when compared to a uniform forces pattern distribution [20], was identified the presence of higher forces at higher levels, which leads to the increased of the total shear forces on each floor for the same level of base shear, leading to higher deformations.

Since the calculation tool (Seismostruct) used does not account for the resistance loss of the elements when the last deformations occur in the constituent materials, from a certain level (ten warnings in Seismostruct with respect to deformation in the structural elements corresponding to points marked in black in Fig. 1 and Fig 2) the capacity curves should not be taken as representative of the actual capacity of PT structures analyzed.

Em todas as estruturas analisadas, observou-se de uma forma sistemática que o deslocamento de topo máximo é mais elevado na direção transversal enquanto o corte basal máximo é mais elevado na direção transversal. Estas situações resultam diretamente da diferente rigidez efetiva em cada uma das direções sendo, conforme mencionado em parágrafos anteriores, consistentes com as características geométricas e pormenorização dos elementos estruturais das estruturas PT tipo estudadas.

In all analyzed structures, was observed in a systematic way that the maximum top displacement is higher in the transverse direction while the maximum base shear is higher in the transverse direction. These situations result directly from different effective stiffness in each direction and, as mentioned in earlier paragraphs, are consistent with the geometric characteristics and detailing of the PT type structures structural elements.

Based on the non-linear static analysis (Pushover) carried over the PT structures, were obtained, additionally to the capacity curves, displacement profiles (Fig. 3 and Fig. 4) and drifts (Fig. 5 and Fig. 6). The results shown refer to points indicated in the patent capacity curves in Fig. 1 and Fig. 2.

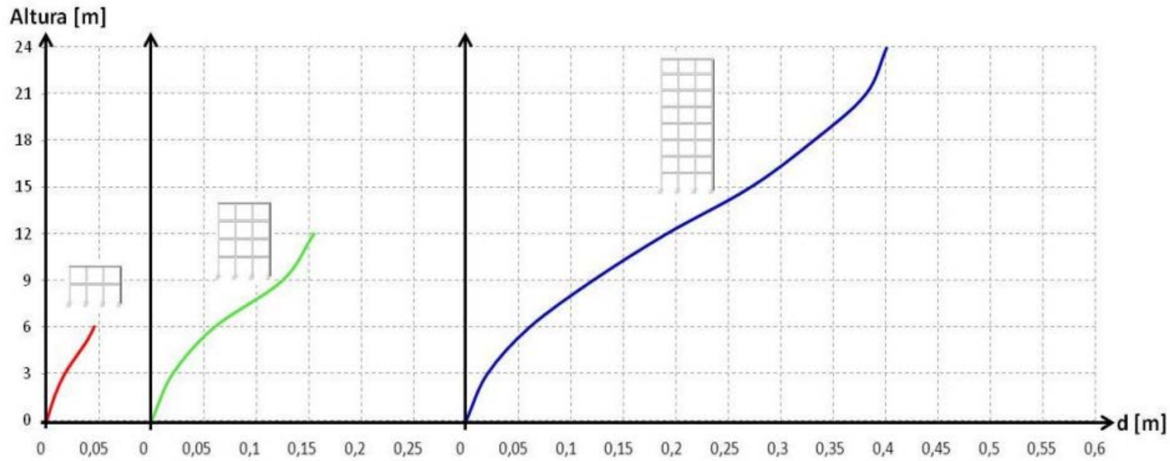

Fig. 3 – Displacement profiles for PT structures: Transverse direction

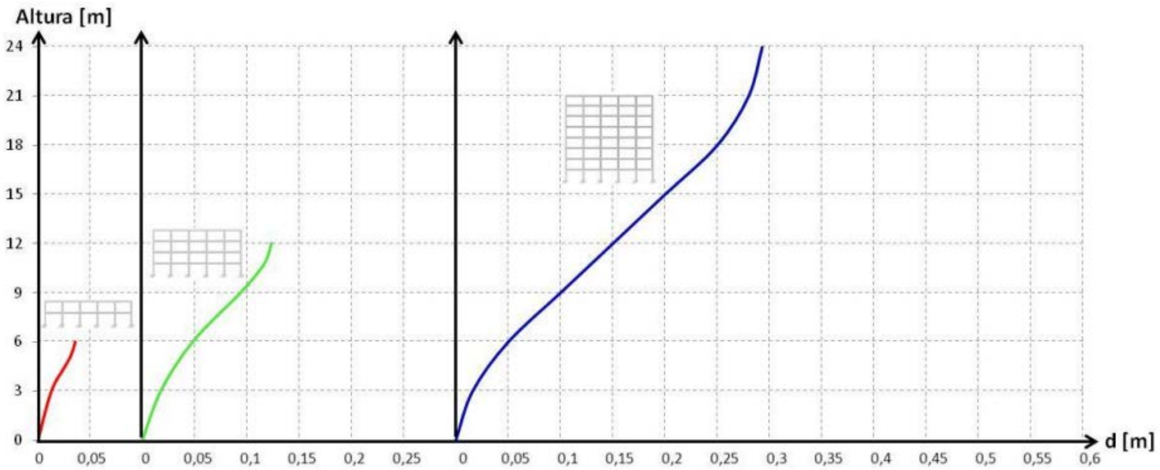

Fig. 4 – Displacement profiles for PT structures: Longitudinal direction

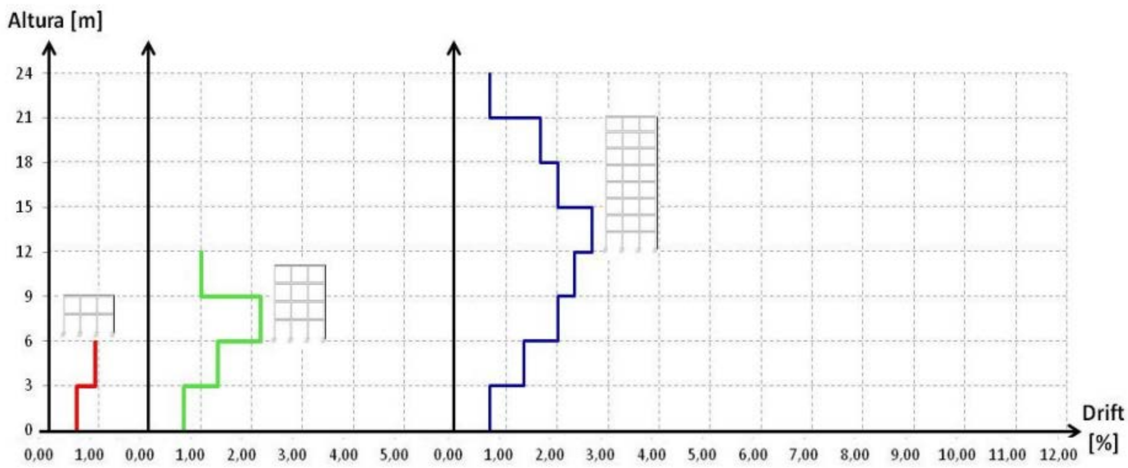

Fig. 5 – *Drift* profiles for PT structures: Transverse Direction

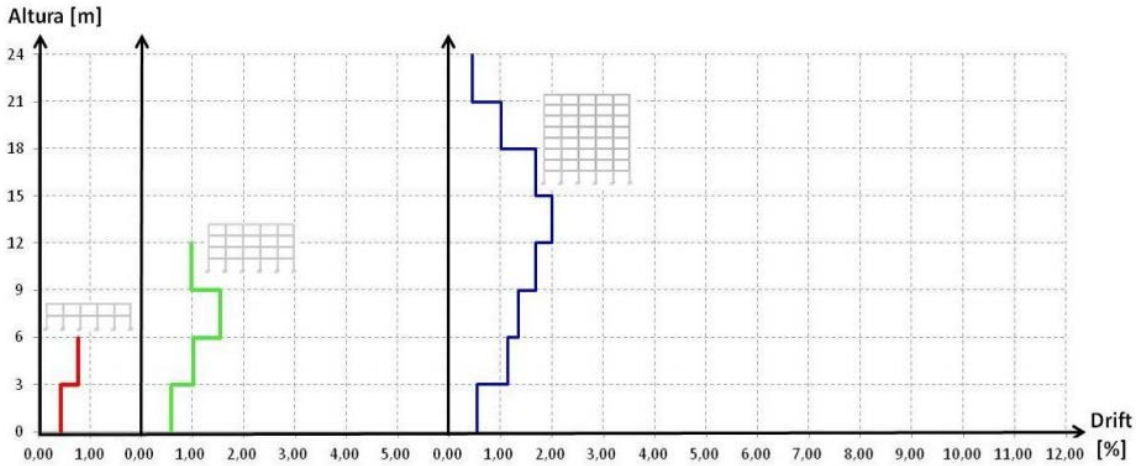

Fig. 6 – *Drift* profiles for PT structures: Longitudinal Direction

For all the the PT type structures, and in both analysed directions, it was observed a height distribution of both displacements of each floor, and the respective drift.

It appears that, in what concerns the maximum registered top displacement in the transverse direction, the values range from 0.4m to 0,045m for PT2 and PT8, while with respect to the longitudinal direction values vary between 0,036m and 0,29m for the same PT structures. As can be seen the ratio between displacements, in the same direction, for frame structures with 2 floors and 8 floors is approximately 10 times higher. Whatever the structure analyzed, the maximum drift for the transverse direction is about once to one and a half to the corresponding value recorded for the longitudinal direction.

With regard to drift is observed that the maximum values vary between 0.93 and 2.67 for PT2 and PT8 in the transverse direction and between 0.77 and 2.0 for the same structures in the longitudinal direction. As can be observed, the drifts ratio for the same direction, considering frame structures between 2 and 8 floors, correspond to 2.5 to 3. For each of the analyzed structures, the maximum calculated drift for the transverse direction is, as indicated for displacements about once to one and a half time the corresponding value calculated for the longitudinal direction.

The direction that appears as having higher vulnerability is the transverse direction, due to be the direction where occur more significant maximum drift values . Whatever the structure examined a relationship between drifts in transverse and longitudinal direction in a range of 1.5 to 2 is observed.

The maximum drifts for both longitudinal and transverse directions, are near to the limit values set by structures performance requirements international guidelines against seismic actions [21, 22], considering the performance levels to life safeguard and collapse prevent, respectively. The areas where this condition may be identified are potential candidate areas to the introduction local reinforcement, thus correcting and eventually eliminating the formed mechanisms. The analysis of the results presented allows confirming a vulnerability increase in certain areas of the simulated structures. In fact, for both directions and analyzed PT type structures, it was observed the existence of more constraining floors, due to geometry and detail sections changes in height of the structural elements (columns and beams). The floors where this constraint is evident are 3rd and 5th floor for PT4 and PT8 buildings, respectively.

6. Final remarks and future developments

The presented work is part of a study in development in LNEC that focuses on the evaluation of the resisting capacity of reinforced concrete buildings designed in accordance with the actual Portuguese regulation. The results presented refer, as indicated in 3.1, the normal ductility structures located in Zone A, founded in middle ground (Art. 29.2 RSA). Until the present moment only the PT type structures described and presented in the paper have been studied, and the conclusions still have a preliminary nature:

- Good calibration of the numerical models representative of the PT type structures (fundamental frequencies error less than 5%)
- The non-linear static analysis (Pushovers) allowed to predict the response of PT type structures when facing seismic actions;
- The results obtained are in agreement with the results of linear dynamic analysis performed on previous works [19] to similar PT type structures. In fact, the non-linear static analysis provide good results for regular structures, such as the PT type structures studied;
- Higher resisting capacity and lower ductility in the longitudinal direction than in the transverse direction, with a corresponding ratio of approximately 2: 1 for any type of PT type structures;
- Stiffness reduction of the lower buildings (PT2) for the highest buildings (PT8). For all analyzed structures, after being achieved the maximum load capacity, was observed a slight rigidity degradation (softening effect) in both directions;
- Higher maximum top displacement and drifts in transverse direction than in longitudinal one. This observation is consistent with the stiffness and ductility characteristics in both directions of the PT type analyzed structures. Whatever the structure, it turns out that the maximum top displacement, in the transverse direction, is approximately one to two times the value recorded for the longitudinal direction;
- Higher vulnerability in the transverse direction and in certain areas of the structures, in the latter case due to the occurrence of geometry and detailing of structural elements changes

Presently, similar analyzes to those presented are in development for similar structures with normal and improved ductility (1 floor, 2 floors, 4 floors, 6 floors and 8 floors - with and without core - and 16 floors - with and without core) located in different seismic zones and based on different types of soil. The resisting capacity curves obtained contribute not only to the definition of damage stages and for the development of analysis based on specific performance levels, but also to support eventual needs of seismic reinforcement in structures.

The models presented do not include irregularities (plant or height), although it can be said that in some ways are representative of the building housing park under study. In what concerns structural irregularities (plant or height) is also provided the development of a complementary study to evaluate the effect in terms of resisting capacity and top displacement.

As was already mentioned, the obtained results are preliminary, being previewed to be subject to a more detailed analysis to subsequent publications. In these situations will be possible to frame the results of pushover analysis made with local seismicity, that is, with the objective displacement (target displacement) required to structures, eventually for different limits stages or performance levels.

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8. References

[1] Varum H (2003): Avaliação, reparação e reforço sísmico de edifícios existentes. *PhD Thesis*, Secção Autónoma de Engenharia Civil, Universidade de Aveiro.

[2] RSCCS (1958) :Regulamento de Segurança das Construções Contra os Sismos, *Decreto n º 41658 de 31 de Maio*, Imprensa Nacional – Casa da Moeda, Lisboa.

[3] RSEP (11961): Regulamento de Solicitações em Edifícios e Pontes, *Decreto nº 44041 de 18 de Novembro*, Imprensa Nacional – Casa da Moeda, Lisboa.

[4] REBAP (1983): Regulamento de Estruturas de Betão Armado e Pré-esforçado, *Decreto-Lei 349-C/83 de 30 de Julho*, Imprensa Nacional – Casa da Moeda, Lisboa.

[5] RSA (1983): Regulamento de Segurança e Acções para Estruturas de Edifícios e Pontes, *DecretoLei nº 235/83 de 31 de Maio*, Imprensa Nacional – Casa da Moeda, Lisboa.

[6] EN 1998-1 (2005): Eurocode 8: Design of structures for earthquake resistance – Part I: General rules, seismic actions and rules for buildings, *Comite Européen de Normalisation*, Bruxelas, Bélgica.

[7] EN 1992-1-1 (2004): Eurocode 2: Design of concrete structures – Part I : General rules, *Comite Européen de Normalisation*, Bruxelas, Bélgica.

[8] Carvalho E, Coelho E (1984): Análise Sísmica de Estruturas de Edifícios Segundo a Nova Regulamentação - Análise Estrutural de um Conjunto de 22 Edificios", Vol. II e Vol. III, LNEC, Lisboa.

[9] Antoniou S, Pinho R. (2003): SeismoStruct – Seismic Analysis program by Seismosoft, *Technical manual and User manual.*

[10] Marques M (2011): Avaliação Probabilística da Segurança Sísmica de Edifícios", *PhD Thesis*, Faculdade de Engenharia Universidade do Porto.

[11] Falcão Silva M J (2014): Avaliação da capacidade resistente de edifícios em betão armado dimensionados de acordo com a regulamentação Portuguesa de 1983, *Jornadas Portuguesas de Engenharia de Estruturas*, Paper 289.

[12] Mota M (2010): Aplicação e Comparação de Métodos Pushover de Análise Sísmica de Estruturas de Edíficios, *Dissertação de Mestrado*, Faculdade de Engenharia da Universidade do Porto.

[13] Madas P (1993): Advanced Modelling of Composite Frames Subjected to Earthquake Loading, *PhD Thesis*, Imperial College, University of London, London, UK.

[14] Mander JB, Pristley M, Park R (1988): Theoretical Stress-Strain Model for Confined Concrete, Journal of Structural Engineering, **114** (8)8, pp.1804-1826.

[15] Martinez-Rueda J E, Elnashai A (1997): Confined Concrete Model Under Cyclic Load, Materials and Structures, **30** (197), pp.139-147.

[16] Menegotto M, Pinto P (1973): Method of analysis for cyclically loaded reinforced concrete plane frames including changes in geometry and non-elastic behavior of elements under combined normal force and bending. *Proceedings IABSE Symposium of Resistance and Ultimate Deformability of Structures Acted on by Well-Defined Repeated Loads*, International Association of Bridge and Structural Engineering, Lisbon, Portugal, **13**, pp. 15-22.

[17] Izzuddin B (1991): Non-linear Dynamic Analysis of Framed Structures, *PhD Thesis*, Imperial College, University of London.

[18] Carvalho G, Bhatt C, Bento R (2013): Nonlinear static and dynamic analysis of reinforced concrete buildings – Comparison of different modelling approaches. *Earthquake and Structures*, **4**(5), pp. 451-470.

[19] Bento R, Bhatt C, Pinho R (2010): Using nonlinear static procedures for seismic assessment of the 3D irregular SPEAR building, *Earthquake and Structures*, **1**(2), pp. 177-195.

[20] Furtado A (2013): Avaliação de soluções de reforço para edifícios com r/c vazado. *Tese de Mestrado*, Departamento de Engenharia Civil, Universidade de Aveiro.

[21] FEMA 356 (2000): Prestandard and Commentary for the Seismic Rehabilitation of Buildings, *Federal Emergency Management Agency*, U.S.A.

[22] SEAOC (2005): Performance based seismic engineering of buildings, Part 2: Conceptual framework, *Vision 2000 Committee*, Structural Engineers Association of California, Sacramento, California, U.S.