Evaluation of the seismic resistance of a reinforced concrete building

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Abstract

Earthquakes are natural catastrophes that cause severe damage both in material terms and in loss of human life. One of the main concerns is its unpredictability, which makes it important to minimize its effects. Building should be designed to withstand earthquakes, as human lives depend on the building performance during such events.

The main purpose of this dissertation is to analyze the performance of a reinforced concrete building during a seismic event. The building was analyzed taking into account a modal analysis by response spectrum, a linear analysis where the non-linear answer is considered through a behavior coefficient. The main limitation of these methods is their inability to provide information about the structure's non-linear behavior.

Seismic action is an action that imposes displacements on the structure. As these displacements increase, the structure becomes deformed and plastic hinges start to form, which lead to the loss of stiffness and, consequently, to its collapse. The non-linear analysis, better known as pushover, is intended to simulate this effect.

As nonlinear methods, the N2 method and the capacity spectrum method were used. The capacity spectra method is based on the overlap of the capacity curve with the reduced response spectrum, which is corrected based on the damping of the response spectrum. The N2 does not take into account the damping of the response spectrum, but the periods obtained.

Finally, the security check of the building was done and the results of the two methods were compared.

KEYWORDS: seismic analysis, non-linear analysis, pushover, capacity curve.

1 Introduction

Integrated in the Military Engineering Course, the theme of this master dissertation is the "Evaluation of the seismic resistance of a reinforced concrete building". It is important to analyse this issue since earthquakes are one of the most unpredictable natural disasters which cause the greatest damage, both in material and human.

Nowadays, seismic action is being considered in building projects by using a linear analyse, through a coefficient that takes into account the non-linearity of the building. Currently, the higher availability of information has made it possible to analyse buildings by using non-linear analyses, also known as pushover methods. This analysis is important when studying earthquakes, due to them being a displacement imposed in the structure and not a force. In reality, as the structure is being "pushed", it starts to suffer a deformation before eventually collapsing. However, before it collapses, plastics hinges are formed in the structure which lead to it losing stiffness. This phenomenon is explained in Figure 1.

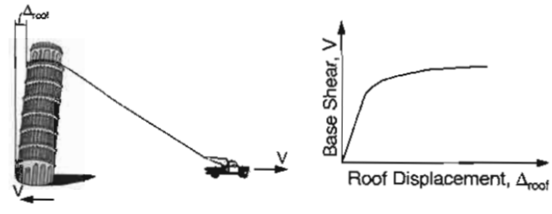


Figure 1 – Pushover method (ATC40, p. 8-4, 1996)

The curve illustrated above is known as the capacity curve, and it represents the behaviour of the building as it is being "pushed".

2 Objectives and methodology

This master dissertation aims to evaluate the seismic resistance of a military building belonging to the Portuguese Army. To reach this main objective, the building was modelled in SAP2000®, which allowed to obtain the capacity curve by pushover. Additionally, it was necessary to use a method to obtain the target displacement, with the decision being to use the N2 method proposed by EC8 and the capacity spectrum method by ATC40. After the target displacement is obtained, it is possible to verify the security of the building for that displacement.

The capacity spectrum method consists in a graphical approach where the capacity curve is superimposed on a reduced response spectrum to find the performance point. The reduced response spectrum is obtained by reducing the elastic response spectrum with the reduction factors until the performance point is found. In this method, the capacity curve is converted to a one degree of freedom, as well as to the acceleration-displacement format by the modal participation factor and modal mass coefficient.

If a building had infinite linear elastic capacity, then the capacity curve would be a straight line with a slope equal to the stiffness of the structure. Since real buildings do not have infinite linear elastic capacities, the capacity curve consists of a series of straight-line segments with decreasing slope, representing the progressive degradation of the structural stiffness as the lateral displacement increases, yields and finally collapses.

According to Bento and Lopes (1999), to maximize the building's dissipation capacity, the objective is to maximize the formation of hinges without the building becoming a mechanism.

After its conversion, its necessary to represent a bilinear curve of the capacity curve in order to estimate the effective damping so as to obtain an adequate reduction of the response spectrum.

Finally, if the displacement at the intersection of the response spectrum and the capacity curve is within 5% of the test performance point, then that is the performance point of the structure.

In the N2 method, the seismic demand is determined from inelastic spectrum and it depends on the idealized period of the system of 1 degree of freedom. In this method, the first step is to convert the capacity curve of multi-degrees of freedom to one degree of freedom. Then, we determine a bilinear curve representing the ideally elastic-perfectly plastic force, which is used to determine the displacement at the plasticity limit of the system with only one degree of freedom. Afterwards, the idealized period is calculated and then the target displacement is determined, depending if it the period is short or medium/long. If that value is not near the displacement point of formation of the plastic mechanism, then it is necessary to do a new iteration. However, if the value is near, then It needs to be converted into a target displacement of a one degree of freedom by multiplying it by the transformation factor.

3 Case Study

3.1 Location

The PM136 is located at the Regimento de Transportes, in Portela, Lisbon, which is responsible for logistical support, namely in the supply of vehicles. This building is a hangar for vehicles and it can be seen in Figure 2.

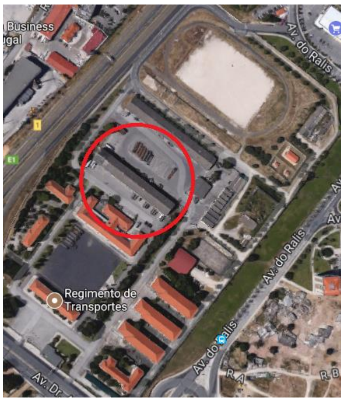


Figure 2 - Location of the Hangar (Satellite image taken from Google Maps, March 30, 2017)

3.2 Building Characterization

The hangar has a regular floor plan with 106,4mts of length and 16,2mts wide. It consists of twenty reinforced concrete frames filled with masonry walls and reinforced concrete slabs. The first floor has a height of 5mts and its function is to park heavy vehicles, whereas the second floor, at a height of 4,1mts, is used for lighter ones. Additionally, it has a gymnasium, a pavilion, an auditorium, a classroom and a warehouse. In Figure 3, it is possible to see a photograph of the building being studied.

Figure 3 – Case study PM136

This building has a specification that makes it more interesting and worthwhile as a case study: it has the ground at two different levels on each side. In Figura 4, it is possible to see the cross-section of the building and to notice this detail. This is of some interest because it will influence the dynamic analysis of the building due to the deformation in the structure.

Figura 4 – Cross-section of PM136

In order to carry out the seismic analysis, it is important to know the type of terrain we are dealing with since the seismic action is represented by the response spectrum. There was an excavation done at the regiment, the results of which can be seen in Figure 5. It is possible to conclude that the current soil type is type B, since the terrain allows for a considerable slope without collapsing.

Figure 5 – Excavation done at the Regiment

Knowing the type of soil, the seismic zone and using an important class of 2, both seismic actions were obtained: seismic action type 1 (S.A. 1) and seismic action type 2 (S.A. 2) elastic response spectrum, as it can be seen in Figure 6.

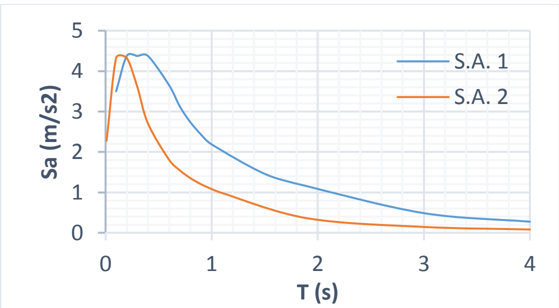


Figure 6 – Elastic Response Spectrum

3.3 Other actions

The PM136 has several compartments with their respective functions on the upper floor and, in the case of overloads, with their respective categories.

Based on EC1 (2009), as a permanent action only the concrete weight of the structure was considered (γ = 25kN/m^3). As for overloads, depending on the categories defined by the EC1, they were for the vehicles in the garage of 2.5kN/m^2, the gymnasium of 5kN/m^2, the pavilion of 5kN/m^2, the warehouse of 7.5kN/m^2, the auditorium of 4kN/m^2 and the classroom of 4kN/m^2.

4 Results

Two models were made for the results, one that took into consideration the soil adjacent to the building and another that did not. In addition to the beams and columns, the modelling of the building also took into consideration the expansion joints and the masonry walls. After modelling the building using the SAP2000®, a modal analysis was performed to obtain the vibration modes of the structure and its associated periods and frequencies. Through this modal analysis, it is possible to determine the mass participation factors in each direction and for each mode of vibration, thus allowing us to have a notion of the influence of each mode on the overall response of the structure.

4.1 Modal analysis

As for the seismic analysis of the structure, it is only of interest to analyse the horizontal directions, since there are a lot of vibration modes; as a result, the Ritz method was used. According to Penzien and Clough (1995), this is one of the most efficient methods for solving vibration problems. It optimizes the solution since it uses cinematic conditions to reduce the number of the degrees of freedom.

The vibration modes do not depend on the type of loads; instead, it depends on the mass and stiffness of the elements. This is why it is necessary to use 50% of elastic modulus of concrete to take into account the cracking of the concrete. Since the first mode is the one that mobilizes less energy to deform the structure, it is predictable that this mode is more mobilized in the y direction, since it has less mass, stiffness and inertia in that direction.

Comparing the vibration modes of the different models, in model A we have a greater participation of the y direction, which is to be expected. However, the same does not occur in model B, where it is more restricted by the support in the first floor where most of the mass of the structure will influence the dynamic response of the structure, hence it becomes more conditioned in the transverse deformation.

4.2 Non-linear analysis

To perform the non-linear analysis, it is necessary to first transform the acceleration response spectrum into a response spectrum in the acceleration-displacement format, more commonly known as ADRS.

The results can be seen in Figure 7.

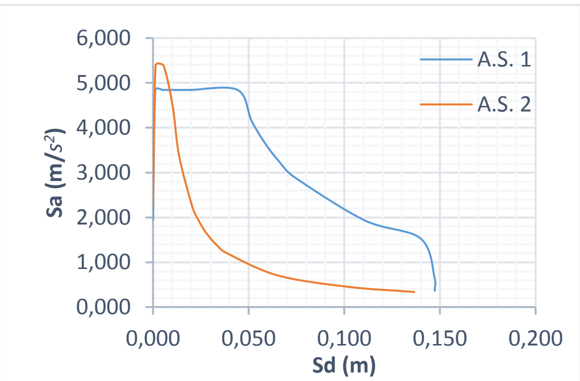


Figure 7 – Response spectrum at ADRS format

The concentrated plasticity model was used, which allows inelastic deformations to be concentrated at the end of the elements, represented as plastic hinges.

Since hinges are very important for the pushover analysis, since they are responsible for the yielding of the building, it was important to define them manually to have better control over the results.

To do that, it was necessary to obtain the Moment-curvature relation of each section and the length of the hinges.

According to Bento and Lopes (1999), the possibility of taking advantage of the non-linear behaviour of the materials and consequently of the structural system derives from the fact that the seismic action corresponds to displacements imposed on structures and not on forces applied.

Two types of loadings, one a uniform load and other modal, were applied for the pushover curve.

The control node used for the pushover was the rooftop node (see Figure 8), located at the middle of the building near the center of mass. Four capacity curves were obtained.

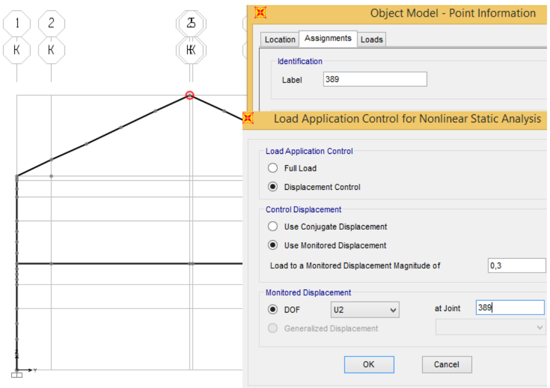


Figure 8 – Control node

First of all, by observing the two capacity curves represented on Figure 9, we can conclude that the modal load is more conditioning than the uniform load.

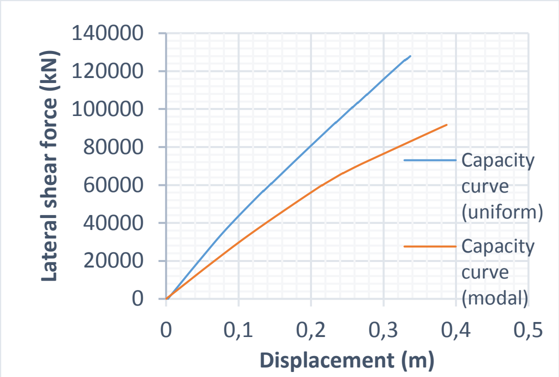


Figure 9 – Capacity curves for both loads

For the same value of total lateral shear force, we have more displacement in the modal load than the uniform one. This happens because the modal load takes into account the deformed and the modes of the structure when applying the forces. This will cause the stiffness to decrease, which is represented by a smaller slope/period.

In the model B, the first difference is in the shape of the capacity curve.

As we can see in Figure 10, this shape is different from the previous one; this is known as capacity curve with a "sawtooth" shape.

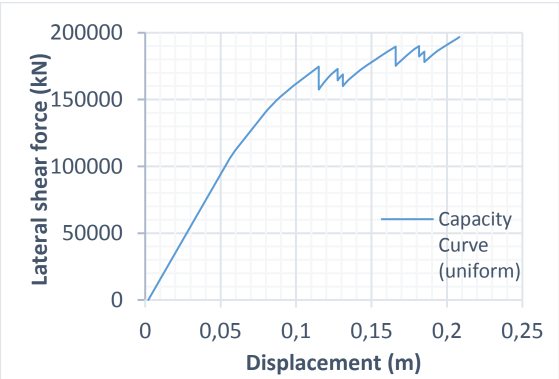


Figure 10 – Capacity curve for model B (uniform load)

According to Chen and Lui (2006), the lateral shear force applied on the structure increases until some elements reach 90% capacity. When this happens, the analysis is interrupted and the stiffness of that element is set for a smaller value or removed from the model. The loads are then reapplied to another element, which starts to initialize another capacity curve as it reaches its capacity, forming an effect similar to a tooth, known as "sawtooth".

This new capacity curve that takes into account the rigidity of another element will have its own displacement which may lead to values of shear force greater than the lateral shear force of the previous element.

It is possible to see in Figure 10 that, when a new capacity curve appears, its stiffness has decreased (note that the slope of each capacity curve is getting smaller).

The other difference is in the capacity curve of model B with the modal load, as visible in Figure 11.

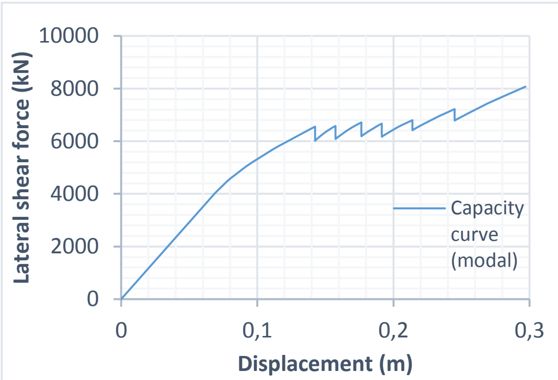


Figure 11 – Capacity curve for model B (modal load)

In the case of the capacity curve of model B with modal load, no method was applied since the structure has a much lower basal force compared to the uniform load, with a difference of about 160000kN. This is due to the fact that in model B only the top floor deforms due to the existence of the supports that simulate soil. As the building has some transverse walls on the second floor that separate the various rooms above, this ends up causing the building to not have a way in which the whole building moves to the same side, transversely, as it happens for the first vibration mode of model A, as it can be seen in Figure 12.

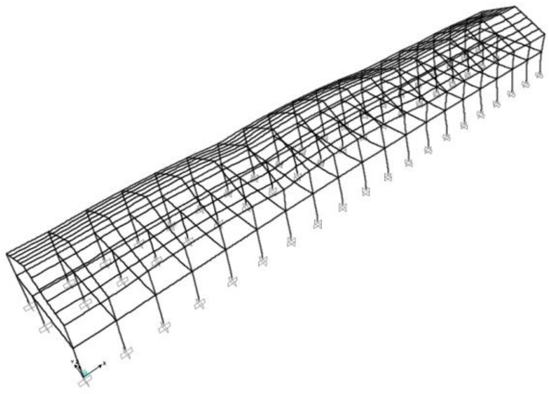


Figure 12 – First vibration mode of model A

In model B, the building moves in different directions which would make the values of the force cancel each other, which would result in a lower lateral shear force. The first vibration mode only has movement in the central zone (zone that has the pavilion). This means that we would have movements in different directions In order to take into account the transverse movement of the building, it would be necessary to combine the modes that cause the building movements transversally. In this model these would be the first, the second and the thirteen mode.

It can be concluded that applying the modal distribution in this model does not make sense since it takes into account the deformed ones in the different modes.

In spite of all this, I can say that the pushover method is a method that is thought for distribution in height and not in plan, which is the case with this model. Another way to look at this situation would be to evaluate the gantries transversely, rather than the building as a whole.

Obtained the capacity curve, we can apply the capacity spectrum method (CSM) and the N2 method.

To do the conversion of the capacity curve for both methods, it is necessary to know the mode of the structure to take out the amplitude modes.

The 1st mode will be used for both models since it is in these modes that the building has to vibrate to the same side. In other words, it is similar to static charging. In the case of model A, this is the mode with the greatest participation in the y direction, and in the case of model B, this mode does not correspond to the mode with the highest participation.

To obtain the amplitudes, it was necessary to know the center of mass of the first floor of the building; as for the second one, the rooftop node was used. The center of mass obtained is in the coordinates (X_{CM}, Y_{CM})=(53,2m; 8,24m).

Therefore, as we are only checking the pushover for y direction, then for model A, we obtained in the 1st mode an amplitude of 0.0205m for the 1st floor and one of 0.0427m for the 2nd floor. In model B for the 1st mode, the amplitude of 0.0002m was obtained for the 1st floor and for the 2nd floor an amplitude of 0.1163m.

Now, it is possible to start converting the capacity curves for each method. This is where the differences start to appear.

About the conversions of the capacity curve, the CSM converts the capacity curve directly to capacity spectrum in the acceleration-displacement format, with the acceleration units given in g's. The N2 method converts the capacity curve to a capacity curve of one degree of freedom system. In the CSM, the intersection of the capacity curve with the response spectrum in the ADRS format is used.

As we can see in model B, with the uniform load we see a different result from the others loads and models.

We must take into account that the seismic action type 1 has higher values of spectral coordinates, which means that the acceleration and displacement caused by this type of seismic action will be greater than the seismic action type 2, and that its verified in model A, as we can see in Figure 13.

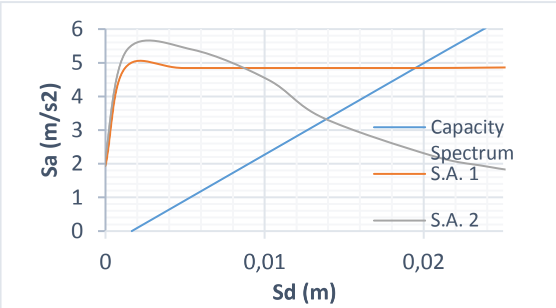


Figure 13 – Capacity spectrum (ATC40) model A uniform load

In model B, the uniform loading, since the building is very rigid, has a very low period that the target displacement of the structure is within the zone represented in Figure 14. In this case, the seismic action type 2 causes greater displacement and acceleration of the structure than the seismic action type 1.

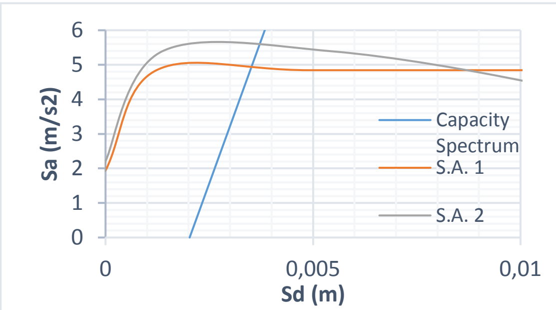
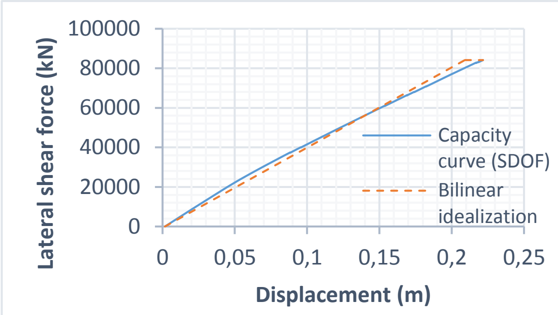


Figure 14 – Capacity spectrum (ATC40) model B uniform load

The N2 method uses a bilinear idealization based on the equal energy principle using an elastic-perfectly plastic force-displacement relationship for the equivalent Single Degree of Freedom (SDOF) system. It is possible to see an example in Figure 15 of model A with uniform load. In this case, the displacement and the yielding force at the formation of plastic mechanism are used.

Figure 15 – Bilinear idealization model A (uniform load)



In the CSM method, we use the bilinear idealization after obtaining the test performance point. If this point occurs in the linear phase, it will not be necessary to the bilinear idealization since it will give us a linear graphic.

Since all the results were obtained in the linear phase of the structure, it was not necessary to do iterations, except in the N2 method, where they were necessary to take into account the different type of seismic action since it has different acceleration values.

The results obtained from the CSM method and N2 method were converted from one single degree of freedom to multi degree of freedom and they are represented below, corresponding Table 1 for the CSM method results and Table 2 for N2 method results.

Table 1 – Target displacements obtained by CSM method

	Final target-displacement (m)	
	A.S. 1	A.S. 2
M. A - unif	0,027	0,021
M. A - mod	0,042	0,025
M. B - unif	0,0034	0,0036

Table 2 – Target displacements obtained by N2 method

	Final target-displacement (m)	
	A.S. 1	A.S. 2
M. A - unif	0,027	0,017
M. A - mod	0,036	0,020
M. B - unif	0,0034	0,0035

All of these displacements occur in the linear phase of the structure. This means that, if this type of seismic action occurs, the structure will respond with that displacement.

According to EC8 (2009), the damage limitation requirement is obtained by calculating the displacement between floors. For PM136, the following admissible displacement represented on Table 3 was obtained.

Table 3 – Displacement between floors

Displacement between floors (m)		
Piso	h	0,005h
1	5	0,025
2	4,1	0,0205

Since the second floor has more displacement than the first one, only the second will be verified.

Using the target-displacement obtained from the different methods and loads, it was possible to obtain the values for both walls, the right one (R.W.) and the left one (L.W.) and determine the displacement between floors. The verification was made by subtracting the displacement at the bottom and top of the walls and verifying if the result was lower than 0,0205m.

Table 4 – Displacement between floors

		h (m)		Verify (m)
M. A - unif	R.W.	0,0286	0,0155	0,013
	L.W.	0,0254	0,0154	0,01
M. A - mod	R.W.	0,0421	0,0204	**0,022**
	L.W.	0,0204	0,0204	**0,022**
M. B - unif	R.W.	0,0051	0	0,005
	L.W.	0,0021	0	0,002

As it can be seen in Table 4, the results obtained for model B with uniform load conclude that the structure is secured for the damage limitation requirement. The same does not happen for model A with modal distribution, which was obtained a higher value. In this case, the structure does not have an admissible displacement, so it is not secured for the limitation of damage requirement.

According to Fafjar (2005), the ductility factor (Table 5) is defined by the ratio between the maximum displacement (d_m^*) and the yield displacement (d_y^*).

Table 5 - Ductility factor for the various models and types of loading

	dm*/dy*
M.A. Unif	1,06
M.A. Mod	1,13
M.B. Unif	1,16

Therefore, we can know what type of ductility is required for this structure depending on the type of seismic action.

We can notice the required ductility level is low which would correspond to a lower coefficient of behaviour if we were to apply a linear analysis and we wanted to take into account the nonlinear effect. It makes sense, since this structure has linear behaviour.

Finally, since the structure, within the assumed parameters, is well dimensioned for this seismic action, it is not necessary to make a proposal for any seismic reinforcement. However, although it is well-suited for this type of seismic action, it does not indicate that the structure can withstand any type of seismic action, since they may have higher accelerations which may cause the intersection with the capacity curve in the non-linear phase and then it can no longer check for safety.

5 Conclusions and future developments

In this dissertation, we studied the seismic action, the methods for its analysis and the seismic behaviour of the building PM136 of the Portuguese Army.

As a method, non-linear static analysis, better known as the pushover method, was used. Although the linear analysis takes into account the non-linear effects with the coefficient of behaviour, through this method it is possible to determine the weaknesses of the structure, with the formation of plastic hinges.

As the structure in question showed a good resistance to earthquakes, the results were all in the linear phase of the capacity curve, which did not lead to the formation of plastic hinges nor the application of the methods as planned.

It is of extreme importance to highlight the fact that this result was influenced by the characteristics in the study, namely the amount of reinforcement in the sections of reinforced concrete. In order to make a more careful analysis, it is necessary to survey the existing reinforcements on the pillars and beams of the building.

Since our country is at risk of seismic action, it is of most importance to continue the research in this area. A catastrophic phenomenon of these can shake our country causing immense losses, both in terms of Portuguese heritage and human lives.

As for future development, I propose the verification of the efforts obtained, but for this it would be necessary to be able to make the survey of the steel bars in the reinforced concrete sections of the building to be able to do a more correct analysis. To do this, I propose the use of the "*proceq profoscope*" in order to obtain information about the reinforcements. Efforts could be made by comparing our regulation, EC 2 with ATC40.

Although a rather low ductility factor was obtained, the sections of the structure could be checked for the minimum ductility that these types of earthquakes requires the structure.

The best way, in my opinion, of designing a building nowadays is by using the capacity design. This method is also present in our regulation and it is an area that deserves more attention and should be studied and used more.

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