Seismic Performance of Base Isolated Buildings

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Abstract

EC8 considers the reference seismic action with a return period of 475 years for which the structures must resist. Important structures, such as hospitals, have an importance coefficient, which increases the return period. However, for the damage limitation requirement, the EC8 imposes a lower action than the reference one.

This dissertation aims to evaluate the seismic performance of a class III structure subject to the reference seismic action. A model with a fixed base and a model with an isolated base were developed. Furthermore, its performance was evaluated through the relative displacement between floors and the shear basal forces.

For the fixed base solution, the seismic resistant capacity was analyzed by performing a Pushover analysis. The N2 method was used to assess the structure's seismic performance, concluding that it does not meet the minimum limits for 475 years return period seismic action, although it complies with the regulatory minimums. For the isolated base solution, a linear analysis by response spectrum was performed, and it was concluded that it meets the regulatory requirements for the reference action (T_R = 475 years).

Various noteworthy conclusions can be made. First, structures with seismic isolation present better seismic performance. Second, it is possible to be more rigorous and not just accept the regulatory minimums, which could be improved through new systems of seismic protection. This is particularly relevant given that these structures are vital for the community. For that to be done, entities need to be more involved and understand all the costs associated with poor seismic performance.

Key words: Seismic isolation, Pushover analysis, seismic performance, return period, damage limitation requirement,

1 Introduction

Seismic risk in a region is the product of hazard, exposure and vulnerability. Since it is not possible to prevent an earthquake from occurring (hazard), nor to prevent the exposure of people and infrastructures to the effects of an earthquake, engineering can only act on the capacity of buildings and infrastructures to resist the seismic action with the minimum of damage (vulnerability). In fact, vulnerability itself results from human action, and it is on this factor that man can act easily in many cases.

According to Lopes (2008), what causes the vast majority of victims and economic losses during the occurrence of violent earthquakes is the damage and collapse in buildings. Therefore, it is crucial to act at the construction level in order to mitigate the effects of seismic action.

Regarding the exposure, although the current knowledge of seismicity makes it possible to prevent construction in any seismic zone, it would not prevent the destruction of existing cities. Similarly, even if it would be possible to accurately predict the occurrence of earthquakes, and considering the evacuation of cities saves lives, it would not be enough to prevent the destruction of the city and the economy.

It is undoubtedly important to predict when and where an earthquake would occur. However, more important than that is to acknowledge what type of earthquake may occur in a given place, so that structures are adequately prepared to resist.

Following this, it is relevant to study what happened in the past so that one can understand what may happen in the future. It is through the study of the history of earthquakes (historical seismicity), recorded earthquakes (instrumental seismicity) and potentially

earthquake-generating faults (geological information) that seismologists can calculate the likelihood of earthquakes occurring in certain zones during certain periods of time (Oliveira, 2008).

These data provide a rich source to define design earthquakes, that are afterwards included in structural codes worldwide, such as the Eurocode for seismic regions, EC8.

Hence, seismicity can be represented as a function of the peak acceleration of the soil and the return period - Figure 1.1. This figure shows the seismicity in Portugal (Mainland and Azores). As can be seen, in the mainland there is a marked increase in peak acceleration (PGA - Peak Ground Acceleration) for high return periods, while the ground acceleration is very low for low periods. This territory is characterized by earthquakes with a high return period and high intensity, i.e. rare but intense earthquakes. The Azorean territory is characterized by periods of low return and low intensity, that is, an area with many earthquakes, but less intense.

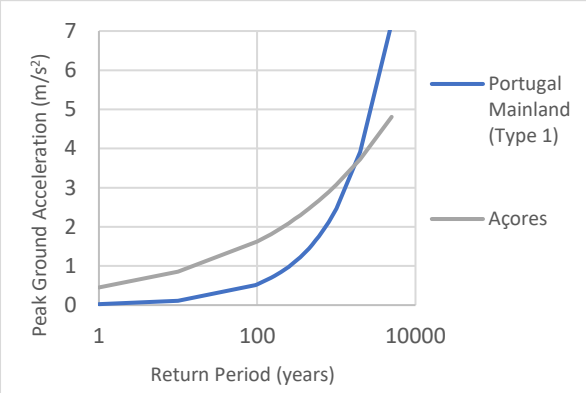

Figure 1.1- Seismic Action in Portugal – National Annex (CEN,2010)

According to Eurocode 8 (CEN, 2010), in the case of occurring an earthquake, the design of the structure must ensure that human lives are protected and important civil protection facilities are kept operational. The EC8 has two different performance requirements. Firstly, non-collapse requirement, that aims to protect human lives. It considers that the likelihood of an occurring seismic action is 10% in 50 years, equivalent to a 475-year return period. Secondly, damage limitation requirement, which aims to avoid structural damage and constrain the damage to non-structural elements. This “frequent” seismic action has a 10% probability of occurring in 10 years - a 95-year return period - and in design practice is equivalent to using a reduction coefficient compared to the design seismic action. These limits are shown in Figure 1.2.

The choice of the probability of collapse depends on the particular building. It is less a technical choice, but rather a political one, as it depends on the risk society is willing to take in case of seismic structural malfunction. It depends on cultural, historical and economic factors. It is commonly agreed that there are structures considered to be more important than others, such as hospitals. Consequently, for these structures, different probabilities of collapse are required.

Regard all of this, it is important to notice the performance limits in Figure 1.2. The damage limitation requirement must be verified for a seismic action well below the design seismic action - T_R = 95 years for T_R = 475 years. Bearing in mind that a region like Portugal (mainland), which can be characterized by having rare and intense earthquakes, a question could be raised. Does it make sense that the seismic action of damage limitation is so inferior to the seismic action of the project, especially in buildings where a more demanding damage limitation is imposed? Although for buildings considered important, the seismic action is higher, the requirement to limit damage is reduced in the same proportion as for current buildings. In other words, the structure is sized for an earthquake higher than the reference earthquake (T_R = 475 years), but the damage check, which should be a condition in these cases, is carried out for an earthquake below the reference earthquake - Figure 1.3. It is important to note, however, that these established requirement limits are adequate for the Azores region (island), as there is no such sharp difference between the project earthquake and the “frequent” earthquake. However, it is always possible to be more demanding and not accept regulatory minimums.

Yet, the resistance to seismic action is always based on the ductile capacity of the structure, which consequently imposes damage to the structure. According to Guerreiro (2008), the exploitation of the ductile capacity of a structure necessarily requires the admission of more or less important structural damage, as it requires the formation of a mechanism based on plastic hinges, which only develop for important levels of deformation. Having that said, there are structures, such as hospitals and emergency centers, which, due to their importance, do not recommend a design based on the exploration of their non-linear behavior. Thus, new seismic protection techniques emerge in order to improve the structure's seismic behavior without resorting to its deformation capacity, as is the case of base isolation, as will be further studied in this dissertation.

In short, this dissertation aims to assess the seismic performance of a hospital building for the reference seismic action. A model with a fixed base and a model with an isolated base will be elaborated and its performance will be evaluated through the results in terms of displacement between floors and base shear forces.

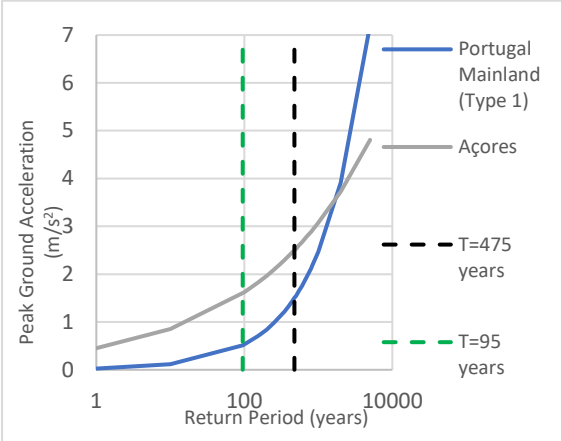

Figure 1.2 – Performance Requirements

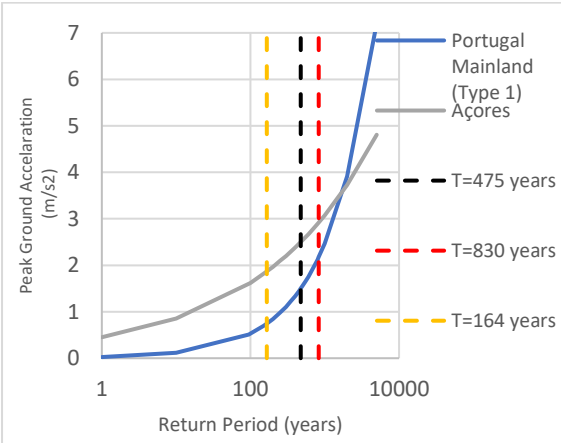

Figure 1.3 – Performance Requirements for this Dissertation

2 Seismic Base Isolation

Seismic base isolation is a seismic protection technique that aims to separate the movement of the structures from the horizontal movements of the soil: the seismic stress is reduced by creating a discontinuity plane between the soil and the structure (Mokha et. al., 1996).

The introduction of an appropriate basic isolation system in a building has the following effects:

- It increases the fundamental period of the structural set and modifies the corresponding mode of vibration;
- Increases the overall damping of the structure.

Concerning the first point, the interposition of a deformable layer between the structure and the soil is the key to isolation: the structure's low frequency decreases the accelerations and also the seismic forces in the structure (Kelly, 1990). This fact can be confirmed by looking at the acceleration response spectrum - Figure 2.1, left.

However, the reduction in the stiffness of the horizontal connection to the ground, essential for the reduction of the efforts in the structure, has as a negative effect, which is the increase of the horizontal displacements related to the ground (Guerreiro, 2008). Observe the displacement response spectrum - Figure 2.1, right.

In short, the isolation system makes the horizontal structure more flexible, thus achieving a reduction in efforts. The main disadvantage can be seen on the increase in displacements in terms of isolation, keeping the structure practically non-deformable.

To minimize these displacements, the isolation system must include some form of energy dissipation that guarantees an increase in the overall damping of the unit (Guerreiro, 2008).

Viscous damping absorbs the energy from the earthquake - it is the physical phenomenon that guarantees the energy dissipation capacity of the isolation system. Most structures have their own critical damping between 2-5%, while the isolation system can have critical damping between 10 and 20% (DIS, 2007).

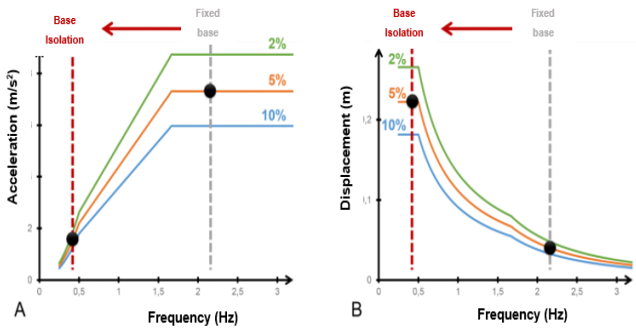

Figure 2.1 – Base Isolation Effects

2.1 Main Features

The main characteristics that a basic isolation system must present are:

- Support capacity for vertical loads:
- Low horizontal stiffness:
- Capacity to return to the initial position:
- Capacity to dissipate energy.

The isolation system also allows to correct the structural torsion effects. Torsion arises in the dynamic behaviour of structures when the centre of stiffness does not coincide with the centre of mass of the structural system. The correct dimensioning of the isolation systems must adjust the stiffness centre trying to reduce eccentricities (CEN, 2010; Skinner, Robinson, & McVerry, 1993).

2.2 Main Limitations

The effectiveness of basic isolation depends on the type of soil and the stiffness of the structure to be isolated. The higher stiffness a structure presents (i.e. the higher its own frequency) and the harder the soil is, the more

efficient
the use of base isolation systems (Symans, 2009). In soft soils, the proximity between the excitation frequency and the system's natural frequency in non-isolated structures provides an increase in the dynamic response.

Thus, basic seismic isolation is an appropriate anti-seismic protection method for small to medium-sized buildings, with a maximum of 10 to 12 floors, whose fixed-base structures have their own vibration frequencies within the usual range of seismic excitation frequencies (Komodromos, 2000).

The increase in displacement at the level of isolation implies free space around the structure, so that it can move freely without any kind of restriction. This limitation prevents the use of basic isolation in individual buildings integrated in bands or blocks.

3 Case Study

3.1 Building Description and Modeling

In order to make this case study the closest possible to an existing structure and not merely an academic case, the characteristics of the new Cuf Tejo hospital located in Alcântara, Lisbon, were adopted and further adapted to this dissertation, based on the article “Estruturas e Fundações do novo Hospital Cuf Tejo em Lisboa” (Appleton et. al., 2018).

Using the structural analysis software SAP2000, two buildings were modelled - one with a fixed base and the other with base isolation. The models in question consist of 6 floors. From the original block, basements and cantilever areas were not considered because they are not relevant for horizontal seismic analysis.

The slab was modelled with elements of the Shell type. To take into account the lightening in Cobiax (slab type used according to the article), it was considered a reduction in mass of the order of 30% and a reduction of inertia of 10%, according to the Technical Sheet of Cobiax da Ferca (FERCA, 2011).

The slab bands that connect the columns were modelled as frame elements.

The columns and walls were modelled as frame elements with a rigid section at floor level and the same section along the entire height of the building. In all frame elements, a reduction in torsional stiffness was considered, as they lose that ability with cracking.

C35 / 45 concrete and A500 steel were
used in all elements. In order to simulate the cracking effect, the stiffness of the concrete was reduced by half in all horizontal elements and reduced by 1.25 in all vertical elements.

3.2 Nonlinearity Modelling

Concrete

The Mander model (Mander et al., 1988) was adopted for confined and unconfined concrete – Figure 3.1

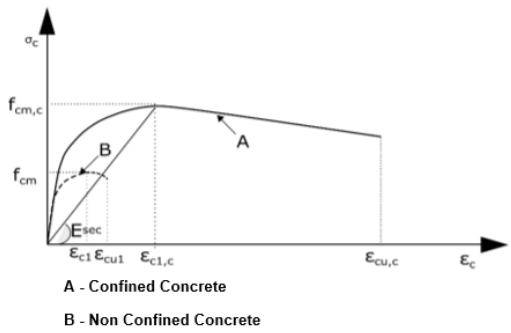

Figure 3.1 – Mander Model for Concrete

Steel

The model adopted for steel (Figure 3.2), presents the following values for A500 NR steel:

- Ultimate stress capacity, f_t = 540 MPa;
- Yield strain capacity, ε_{yk} = 2.1%
- Ultimate strain capacity, ε_{uk} = 11.6%.

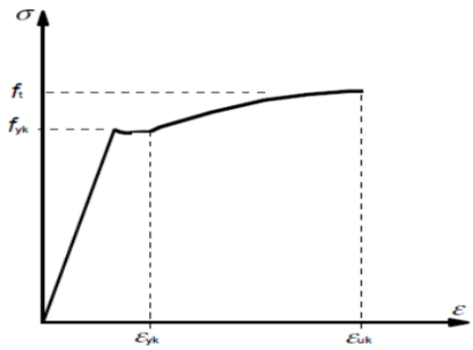

Figure 3.2 - Steel Model

Plastic Hinge

In the building under study, the Caltrans method was used to model plastic hinges. This method has a perfect bilinear elastoplastic idealized moment-curvature relationship. The plastic hinges are located at the ends of the elements, where inelastic deformations under seismic action are expected to occur.

3.3 Base Isolation Model

The behaviour model can be easily defined by a spring with a certain horizontal stiffness, associated with a damper with a certain value - Figure 3.3.

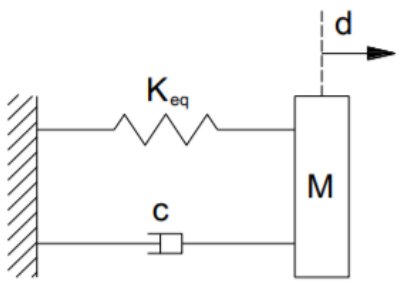

Figure 3.3 - Base Isolation System Model

Hence, using the SAP2000 software, linear elements (springs) with the desired stiffness value were considered, along with fixed supports (for the vertical direction).

The choice of horizontal stiffness implies choosing, as a starting point, the Natural Frequency value to be obtained for the isolated structure. This value is generally about one third of the frequency value of the fixed base structure (0.94Hertz), with a minimum of 0.3Hertz. Thus, the value of 0.33 Hertz was defined as the Intended Natural Frequency. The horizontal stiffness value of each support will be calculated using the following formula:

$$K_{isolator} = \frac{N_{isolator}}{N_{total}} \times K_{system} \quad (3.1)$$

Legend:

$K_{isolator}$ - Horizontal stiffness of each isolator;
K_{system} - Horizontal System Stiffness
$N_{isolator}$ - Normal Force in each isolator
N_{total} - Total Normal Force of the system.

4 Seismic Evaluation of the Building

4.1 Definition of The Elastic and Design Response Spectra

Table 4.1 defines the parameters necessary for defining the response spectra for a structure in Lisbon, a type 1 earthquake and a type B soil. The behaviour coefficient considered was that described in (Appleton et. Al, 2018), as well as the coefficient of importance.

Table 4.1 – Response Spectrum parameters

S_{max}	S	$a_{g,R}$ (m/s²)	γ_I	a_g (m/s²)	T_b (s)	T_c (s)	T_d (s)	q -
1,35	1,22	1,5	1,45	2,175	0,1	0,6	2	2,4

The elastic response and design spectra associated with a coefficient of importance III, as well as the reference elastic spectrum (T_R = 475 years) can be seen on Figure 4.1. Note that the elastic spectra will be used in the nonlinear analysis of the structure, while the calculation spectrum will be used in the design of the structure, which in turn will allow the execution of the non-linear Pushover analysis.

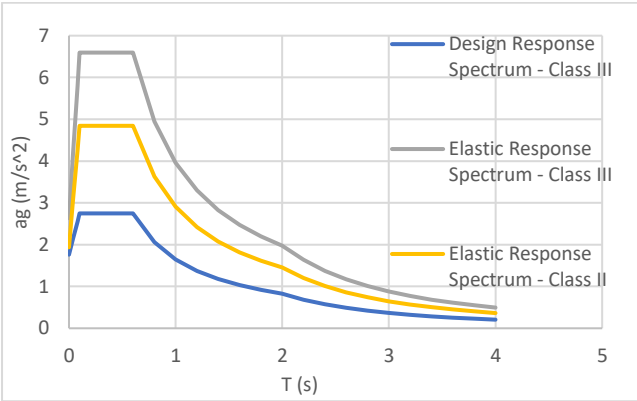

Figure 4.1 - Acceleration Response Spectrum

Structural Design

Regarding the design of the structure, it is important to note that the design of the building due to seismic actions is not the object of study in the present dissertation. However, due to the ductility it offers in the face of cyclical actions, the chosen reinforcement is vital for a good seismic performance. Therefore, it would not be possible to perform a nonlinear static analysis without knowing the structure's reinforcement. Thus, it is essential to design the building.

4.2 Pushover Analysis – N2 Method

A non-linear analysis characterizes the structure through a capacity curve (pushover) that relates the base shear and the displacement of the building at a control node, located on the top floor.

Eight analyses were performed: for each type of lateral loading (uniform and modal), the capacity curve was obtained in both directions (x and y) and in both ways (positive and negative).

In both directions, the capacity curves corresponding to the positive modal loading condition the seismic performance of the structure and will be analysed below using Method N2.

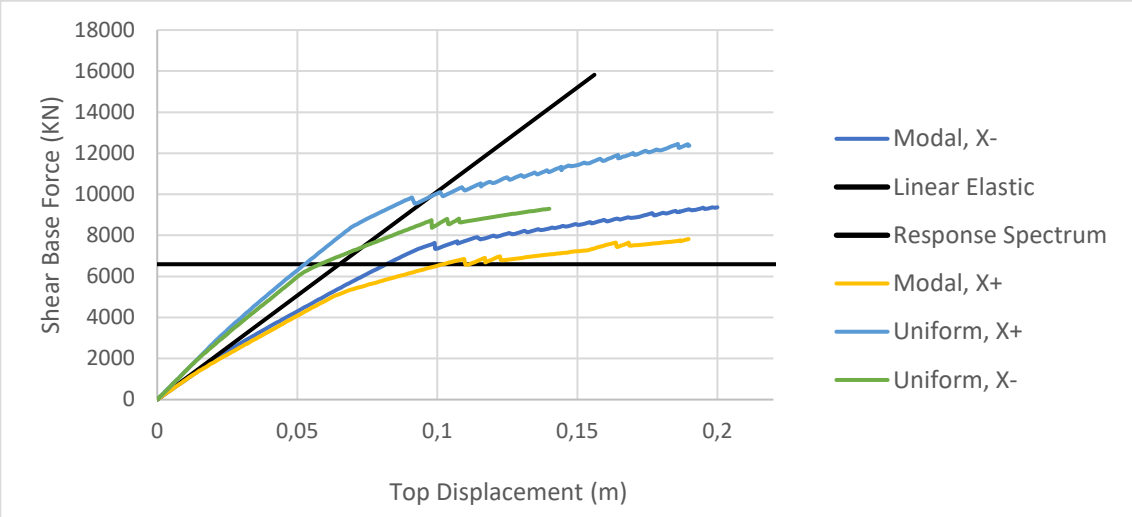

Figure 4.2 - Capacity Curve - X Direction

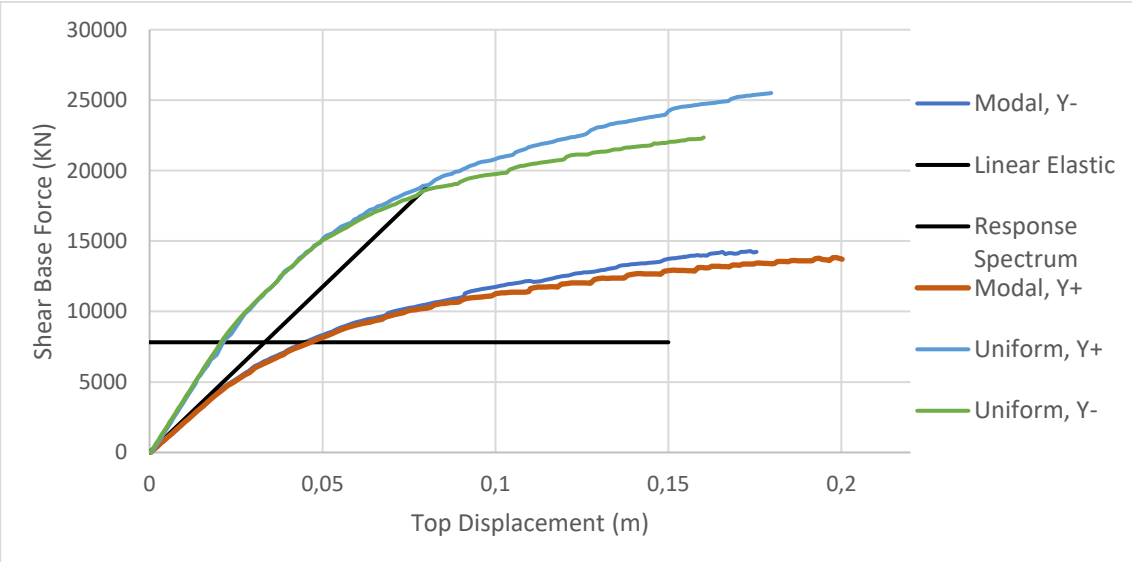

Figure 4.3 - Capacity Curve - Y Direction

N2 Method

The target displacement for the seismic assessment of the building was defined according to the N2 method developed by Fajfar (2000) and suggested in the Eurocode 1998-1 CEN (2010) - EC8-1.

The target displacement of interest to analyse corresponds to the point of intersection of the capacity curve with the elastic reference spectrum. This spectrum has a damping coefficient value of 5%, referring to the viscous damping for reinforced concrete structures. However, it is possible to estimate the equivalent damping value through its hysteresis cycle. Thus, for a given cycle it is possible to estimate the value of the equivalent damping coefficient from the quotient between the interior area of the cycle and the area of the outer rectangle. For the present structure, the cyclical

behaviour outlined in Figure 4.4 was considered.

Hence, the target displacement for the evaluation of the damages of the structure is obtained through the intersection of the capacity curve of the structure with the elastic response spectrum with an equivalent damping value of 12% - Figure 4.5.

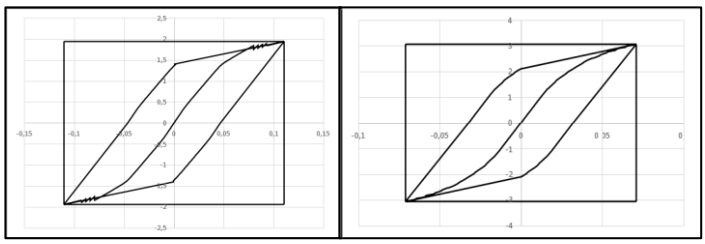

Figure 4.4 - Hysteresis Cycle

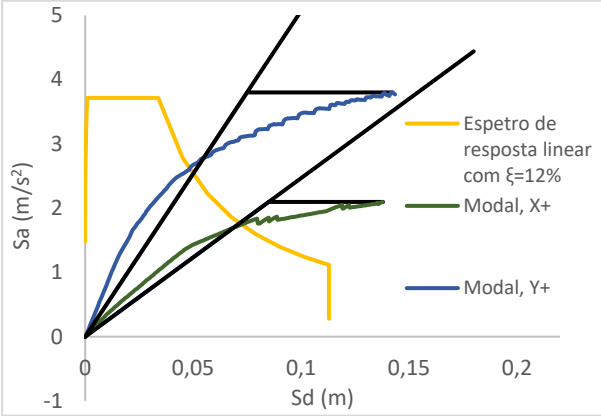

Figure 4.5 . N2 Method

Finally, the last step of the N2 method consists of multiplying the target displacement obtain with the system with 1 degree of freedom by the transformation coefficient, and thus obtaining the target displacement for the structure with multiple degrees of freedom. Table 4.2 presents the results.

Table 4.2 - Target Displacement of the Structure

	d_t^* (m)	Γ	d_t (m)
X Direction	0,072	1,38	0,099
Y Direction	0,048	1,40	0,067

4.3 Damage Analysis in the Fixed Base Solution

According to EC8-1 (CEN, 2010), in order to minimize damage to non-structural elements, the relation between the displacement within floors and the height of each floor must not exceed 0.5%.

Table 4.3 shows the displacement values among floors and the respective relation between that value and the height of the building. As can be seen, the limit is exceeded in both directions. That is, the structure dimensioned for an earthquake higher than the reference earthquake does not meet the damage limitation requirements imposed by EC8 for the reference action seismic.

Table 4.3 - Relative Displacement for the Reference Seismic Action

X Direction		Y Direction	
d_r (m)	d_r/h	d_r (m)	d_r/h
0,016	0,41%	0,015	0,37%
0,024	0,60%	0,026	0,66%
0,026	0,66%	0,028	0,70%
0,026	0,66%	0,025	0,62%
0,025	0,63%	0,021	0,52%
0,023	0,57%	0,018	0,44%

Table 4.4 shows the displacement between floors for the "frequent" seismic action, lower than the reference earthquake. These values were obtained following the procedure proposed by EC8 for the requirement of damage limitation. As can be seen, for this level of seismic action, the displacement limit between floors meets the requirements. This is the current procedure in the design offices for the level of damage requirement.

Table 4.4 - Relative Displacement for the Damage Limitation Requirement

X Direction		Y Direction	
dr*v	**dr/h**	**dr*v**	**dr/h**
0,006	0,15%	0,005	0,12%
0,012	0,29%	0,009	0,24%
0,014	0,35%	0,012	0,29%
0,014	0,36%	0,013	0,32%
0,014	0,34%	0,012	0,31%
0,012	0,29%	0,012	0,30%

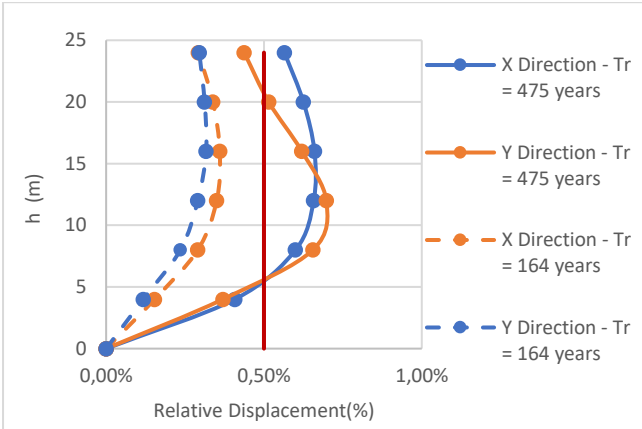

Figure 4.6 - Relative Displacement – Fixed Base

It is then possible to conclude that, following the indications of EC8, this solution meets the requirements for the Damage Limitation Requirement. However, when subjected to the seismic action of reference, the structure may not comply with the operational levels – Figure 4.6.

4.4 Seismic Performance of the Base Isolated Solution

Figure 4.7 shows the response spectra for the solution with base isolation. The support system of this type of solution has a damping coefficient between 10% and 15% for the fundamental periods. In this sense, a critical damping of 10% was considered for periods greater than 2 seconds (fundamental periods). For periods of less than 2 seconds, the structure vibrates with the frequencies proper to the superstructure, associated with a critical damping of 5%.

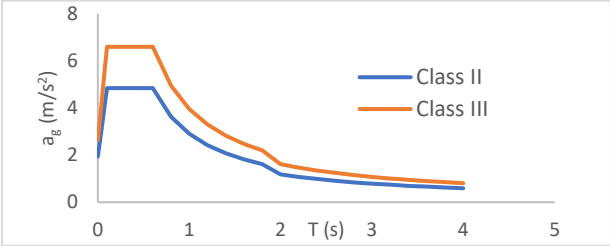

Figure 4.7 - Acceleration Response Spectrum for Base Isolation System

Table 4.5 shows the values of relative displacements between floors. In addition to the displacement values for the reference earthquake, the values for the class III earthquake were also obtained - Table 4.6. As can be seen, the values obtained are much lower than those obtained for the fixed base structure and comply with the regulatory requirement – Figure 4.8.

Table 4.5 - Relative Displacement for the reference seismic action

X Direction		Y Direction	
dr X	**dr/h**	**dr Y**	**dr/h**
0,0080	0,20%	0,0031	0,08%
0,0070	0,18%	0,0033	0,08%
0,0072	0,18%	0,0036	0,09%
0,0068	0,17%	0,0036	0,09%
0,0061	0,15%	0,0035	0,09%
0,0052	0,13%	0,0033	0,08%

Table 4.6 - Relative Displacement for the class III seismic action

X Direction		Y Direction	
dr X	**d/h**	**dr Y**	**d/h**
0,0110	0,28%	0,0043	0,11%
0,0096	0,24%	0,0044	0,11%
0,0097	0,24%	0,0049	0,12%
0,0092	0,23%	0,0049	0,12%
0,0083	0,21%	0,0048	0,12%
0,0071	0,18%	0,0045	0,11%

However, as previously explained in Chapter 2, this type of system has a considerable initial displacement value, about 17 cm in both directions for the reference earthquake (Figure 4.9), and that is why it is a challenging system to implement.

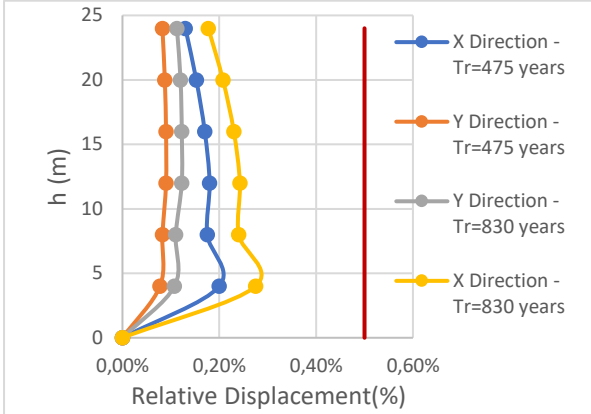

Figure 4.8 - Relative Displacement – Base Isolated

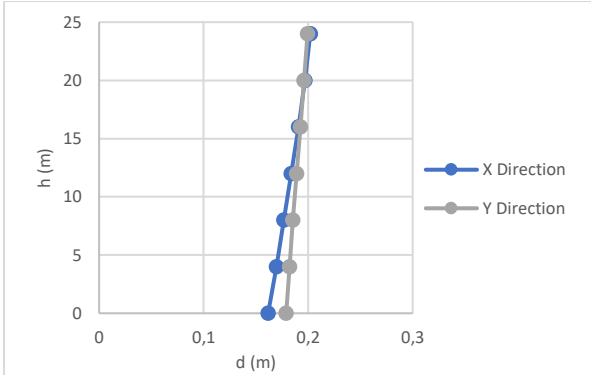

Figure 4.9 - Base Displacement

4.5 Analysis and Comparison of Results

Figure 4.10 shows the relative displacement for the reference seismic action of the two solutions analysed. As can be seen the base isolation system presents much lower results.

For the same seismic action, Figure 4.11 shows the displacements in height. Note that in order to better visualize the differences between the two systems, the origin of the displacements of the two systems coincided, thus ignoring the initial displacement of the system with base isolation, expressed in Figure 4.9. Thusly, we can see that the displacement at the top of the structure is about 6 times higher for fixed-base solution for the reference earthquake.

Figure 4.12 shows the height displacements of the fixed base solution for the reference seismic action and the height displacements of the isolated base solution for the design seismic action (class 3 structure). Note that the displacements of the fixed base solution are still about 3 times greater. These results show that, even considering a higher seismic action, the system with base isolation presents quite significant benefits in terms of reducing the deformation of the structure.

In Table 4.7 the basal shear forces of the two solutions are also compared. Comparing the results for the reference seismic action, the basal forces are about double for the fixed base structure. It should also be

noticed that, even considering a higher action for the isolated base structure (class III), the values of the fixed base structure are higher. Table 4.7 shows the values in terms of seismic coefficient - the relationship between horizontal and vertical forces - in which, as mentioned, the same mass was considered for both structures, so that the results are comparable.

These results reveal once again the benefit of the basic isolation system. For the same seismic action and for a structure with the same mass, the isolated base system develops baseline shear forces significantly lower than the fixed base system.

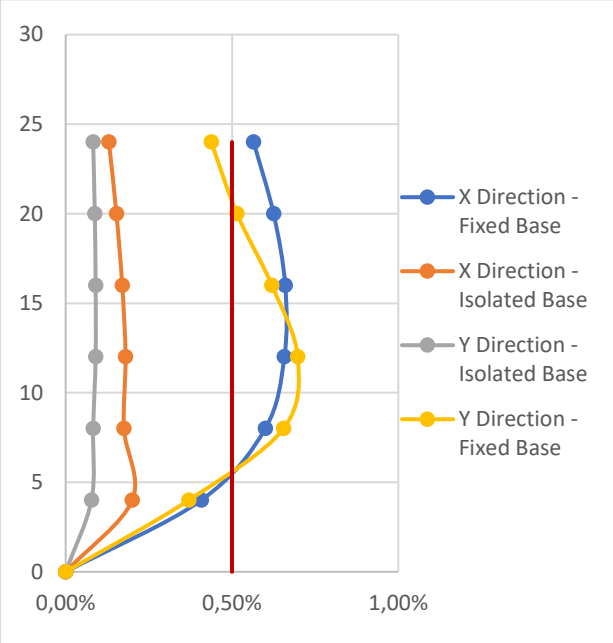

Figure 4.10 - Relative Displacement for the reference seismic action

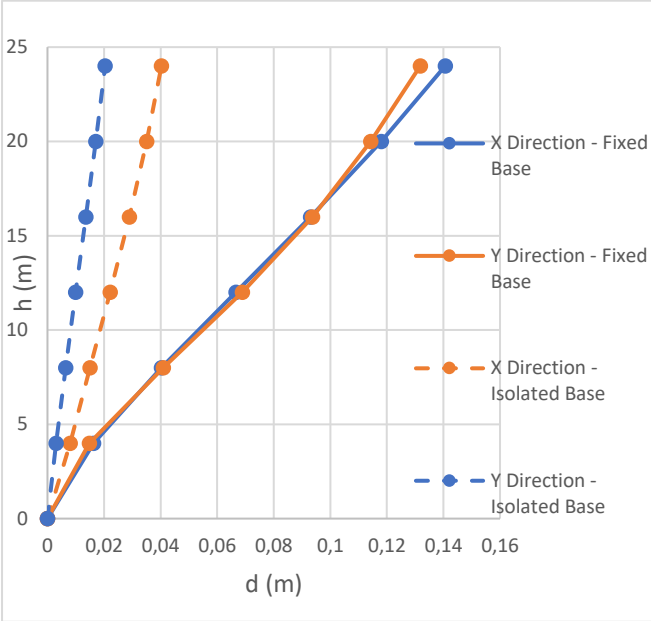

Figure 4.11 - Displacements - Isolated vs Fixed Base - Reference Action Seismic

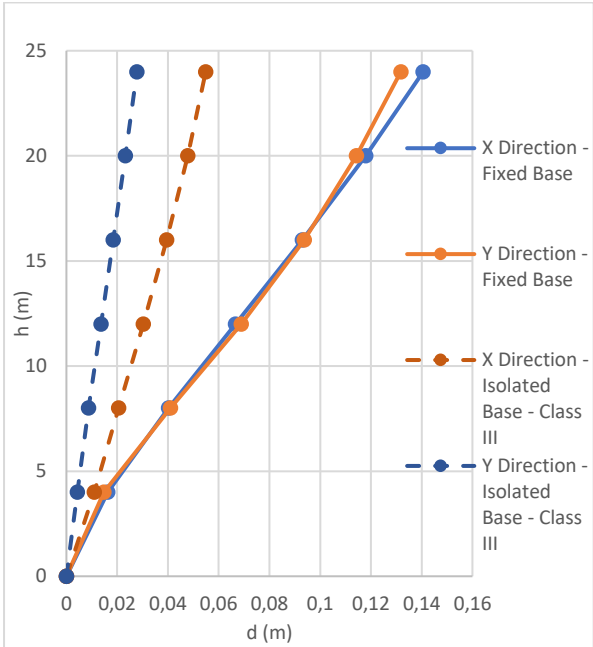

Figure 4.12 - - Displacements - Isolated Base (Class III Action Seismic) vs Fixed Base (Reference Seismic action)

Table 4.7 - Base Shear Force

	Global FX	Global FY	Global FZ	βx	βy
	KN	KN	KN		
Base Isolation - Reference action	3894	4347	49815	8%	9%
Base Isolation - Class III action	5301	5916	49815	11%	12%
Fixed base - Reference action	6880	10131	49815	14%	20%

5 Conclusion

The first conclusion that can be drawn from these results is that the structure with the base isolation system presents a better overall seismic performance compared to a usual structure with direct or indirect foundations. However, its use is often challenging due to the free space that is required to install this system. Adding to that, this system is not yet very popular (as there are only two buildings in the mainland of Portugal at the time of this dissertation), but this trend is expected to change in the coming years.

Despite the fact that using this system can be challenging in many cases, it is still possible to have a detailed analysis and design. Nonlinear analysis methods, such as Pushover analysis, allow designing to be made based on displacements and not forces. More than allowing the detection of the structure's resistance in seismic action, they also allow a greater control of the damage in the structure.
However, once again, nonlinear analyses are time consuming comparing to other methods. Firstly, it demands a structure design through the analysis by response spectrum. Secondly, it needs a later readjustment of the results obtained through the non-linear analysis. Because of all this, it is not a usual method of structural engineer offices.

The second conclusion that can be taken from of this dissertation is related to the results of the fixed base structure and the level of requirement of EC8. Previously on Chapter 1, the level of demand for the EC8 was questioned given the importance of hospitals and the costs associated with their inoperability/unfeasibility/impracticability. Despite the fact that the present results corroborate this issue, it is important to note that it is not mandatory to accept the regulatory minimums. The regulation allows to be stricter.

Along these lines, it is possible to be more rigorous regarding the damage to structures, either through seismic protection systems (such as base isolation) or through more rigorous analyses, such as the Pushover analysis. The involved entities need to clearly define the acceptable level of damage to buildings and the costs and consequences of such damage. For this to happen, it is crucial that knowledge is up to date and that there is information sharing. It is absolutely necessary that the entire framework is known from the seismic action (and the way it is characterized), to new methods of seismic protection and analysis methods. This is everyone who is involved' responsibility, namely the design engineers, the customers to the owners. This is the only way that the levels of damage to structures can be consciously defined.

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