Previous Study of a Concrete Building in Tavira

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Abstract

The present manuscript aims to design and pre-dimension the structure of a residential building to be built in Tavira, Algarve. For this purpose, the seismic factor was preponderant for the structural modeling and verification.

The structural design germinated from the architectural project provided but presented singularities that made it difficult to create a structural model that simultaneously respected the architectural constraints and offered the necessary lateral stiffness to the proper behavior of the building. These difficulties were mainly the impossibility of placing beams in two parallel facades, generating a structure that was too flexible in relation to lateral displacements.

In order to ascertain an ideal solution for the structural design, the dynamic behavior of the building was studied using two different structural models: a structural model composed of a "monolithic" block to cover the entire building and a structural model composed of two separate blocks through a seismic joint, to cover each half of the building.

The safety check was carried out in accordance with Eurocode 8 and a comparative analysis was made to the dynamic behavior, to the construction process and to the cost of each solution analyzed to support the decision to be made. It was verified that the solutions did not meet the seismic performance criteria, being necessary to introduce resistant walls and to increase the cross sections of some structural elements to increase the structural stiffness of the building.

Finally, a proposal was made for structural improvement to the model without seismic joint and its analysis and verification of safety, in order to validate the adopted model.

Key-words: Structural project, Eurocode, Seismic analysis, Modeling, Seismic Joint.

1. Introduction

The scope of the present manuscript is based on the previous study of an armed concrete building to be built in the city of Tavira, Algarve. The influence of the geographic location of the structure on the factors to be considered in the modeling and what the best model to consider are matters of high importance. Certainly, the seismic aspect is a predominant factor for structural modeling and verification.

In order to study the influence of the choice of the structural solution, the dynamic behavior of the building was analysed using two distinct models: a structural model composed of a "monolithic" block incorporating the whole building and a second model composed of two blocks, dividing the structure in two, separated by a seismic joint.

The present study was designed on the basis of the project provided by the architecture. In the first instance the building plans were analysed and the main restrictions of the project were identified, namely the location of the columns and the beams. After this, the materials and the concrete cover were defined, the actions to be applied in the models were quantified and the structural elements were pre-dimensioned. After this initial phase, the structural modeling was carried out using the Finite Element Calculation Software: Robot Structural Analys 2018. Once the structural modeling was completed, the model was verified and the structural models analysed were analysed to the seismic analysis according to Eurocode 8 (EC8). Subsequently a comparative analysis was made of the dynamic behavior, the constructive process, the cost of the solutions to be built and the decision to be made. Also, in the chapter on seismic design, a proposal for structural improvement and its seismic analysis is presented. Finally, in order to validate the adopted model, the safety was verified.

For the preparation of this preliminary study, a number of structural regulations were consulted, in which the regulations applicable to the member states of the European Committee for Standardization (CEN), in particular Eurocodes 0, 1, 2 and 8. In addition, was also consulted a different bibliography to support the development of structural models and their analysis and verification of safety.

2. Structural conception and restrictions

The structural solution to be studied are a "monolithic" block covering the entire building and a solution composed by two blocks, separated by a seismic joint, to cover each half of the building. The plan dimensions of the model composed by only one block are 36.00 m x 11.20 m, while the dimensions of the model composed of two blocks are 18.00 m x 11.20 m each. The Figure 2.1shows the plan of the project under study, where the seismic joint can be identified to separate the two blocks. Since the building is located in a zone of high seismicity, the design of the structure will be based on the medium ductility class, DCM, in accordance with EC8 [1].

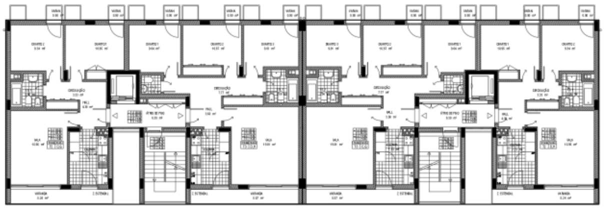


Figure 2.1 – Plan type of the architectural project.

The building consists of a ground floor, three upper floors and a non accessible roof, represented in Figure 2.2 through the elevation of the main facade. The architectural project has a distance between floors of 2.80m.

Figure 2.2 – Elevation of the main facade.

The architecture project did not contemplate beams in the x-direction, so the beams marked in red in Figure 2.3, were added in order to increase the stiffness to the lateral actions. The locations of the columns are presented in the layout of the next image and the beams are identified by the letters "V". In order to increase resistance to the lateral actions, the core of elevators was defined as element of structural walls. A massive flat slab with contour beams was adopted for the floors.

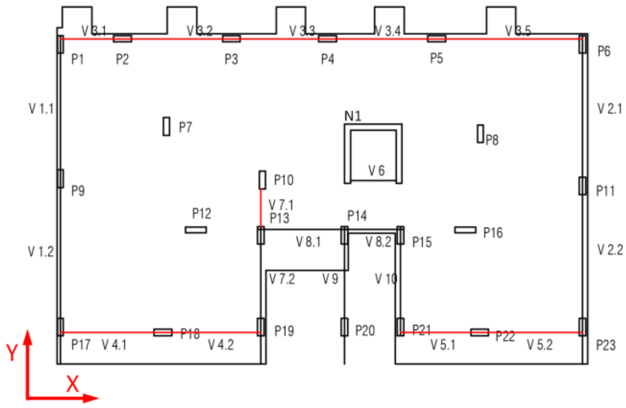


Figure 2.3- Identification of all the structural elements used in the initial structural design.

The beams added to the initial solution, in their conception, led to the emergence of some particular situations due to architectural constraints. Since the architecture does not allow the design of usual beams, both in the main facade and in the late facade, the inverted beam solution is used for the V3, V4 and V5 beams, improving the lateral stiffness of the structure. The floors have a thickness of 10 cm for the layer of concrete cladding, which makes it possible to apply inverted beams on the facades considered for this purpose.

The building has superficial foundations linked by tie-beams joining the vertical elements in the two main directions.

3. General design criteria

As is a housing building, according to EC0 [2], a 50 year project life time corresponding to the S4 structural class was considered [3]. The strength class of the concrete and the minimum nominal concrete cover of the reinforcement depend on the classes of exposure to which the building is subject. For the foundation elements an exposure class XC2 was considered, for the inner elements an exposure class XC1 and for the external elements an exposure class XC3 or XC4 depending on whether or not they are protected from direct contact with rainwater [4]. With this, a concrete cover of 30 mm was adopted for the structure in general and 40 mm for the foundations.

A strength class C30 / 37 was considered for the concrete and for structural steel, it was assumed a high ductility steel A500NR SD.

The permanent loads are divided into the structure's own weight and remaining dead loads. For the specific weight of the structure, a weight of the reinforced concrete of 25 kN /m^3 [5], is considered. The remaining permanent loads represent the loads of non-structural materials, namely claddings and masonry walls, Table 3.1.

Table 3.1 - Remaining dead loads [6].

Designation	Quantification
Floor claddings 0 to 3	1,5 kN/m^2
Terrace roof not accessible	2,0 kN/m^2
Partition walls (floors 1, 2 and 3)	1,9 kN/m^2
Outer wall (linear load)	2,5 kN/m
Stairs claddings	1,5 kN/m^2

The live loads values are divided into categories according to their use [5]. On the basis of EC1 - Part - 1, the live loads presented in Table 3.2 were considered.

Table 3.2 – Live loads.

Category	Zone	Live loads (kN/m^2)
A	Floors	2,0
	Stairs	2,0
	Balconies	2,5
H	Not accessible roof	0,4

The effect of the seismic action on the structures is defined based on EN1998-1 for the quantification of the elastic response spectrum. The Portuguese national territory is divided into seismic zones, depending on the seismicity of the local. The building of this project is located in Tavira, corresponding to the seismic zone of type 1.3 and 2.3. The soil definition allows to determine the influence of local soil conditions on seismic actions and was defined as type C soil. The elastic response spectrum is affected by the behaviour factor (q) where the calculation spectrum is obtained in order to have considering the structure's ability to dissipate energy.

The values of the parameters defining the design response spectrum are shown in Table 3.3.

Table 3.3 – Definition of the seismic action according to the National Annex of EC8.

Seismic action	a_g (m/s^2)	S	T_B (s)	T_C (s)	T_D (s)
1.3	1,50	1,50	0,10	0,60	2,00
2.3	1,70	1,46	0,10	0,25	2,00

4. Pre-dimensioning

The structure definition followed the traditional methodology of structural pre-dimensioning, following the necessary bibliography simultaneously with the analysis of the plans of the architecture.

4.1 Slabs

Due to the columns distribution in plan and the absence of beams inside the building, the slab will be flat slab with contour beams. Through the ratio L_{larger}/h from 25 to 30, it was possible to obtain the range of values for the thickness of the slab [7]. Being the span of 6.00 m the most conditioning, the value of 0.20 m is adopted for the slab thickness. After this, the bending moments were calculated in order to verify the level of forces acting for the fundamental combination with a applied load of 14.3 kN/m^2.

4.2 Beams

Due to the architecture it was not possible to connect all the columns with beams, and the beams proposed by the architecture were adopted (yellow) and three inverted beams (red) were added, according to Figure 4.1.

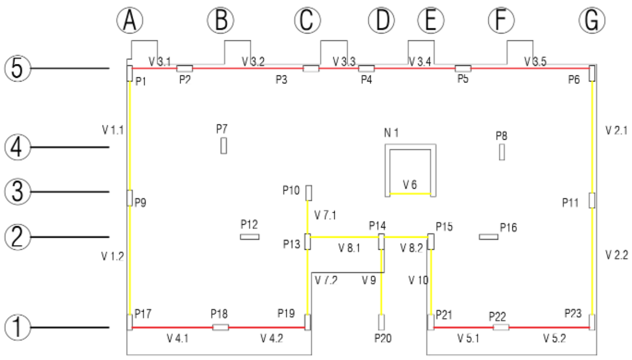


Figure 4.1 - Location of beams and columns in plan.

This option of adding three inverted beams is due to the fact that the building's architecture does not allow current beams and presents few beams in the x-direction (in red) and increase the lateral stiffness.

Assuming the L / h ratio from 10 to 12, a height of 0.45 m was adopted for the beams in the y direction and 0.30 m for the beams in the x-direction. After this, the level of stress in the beams was verified.

4.3 Vertical elements

The pre-dimensioning of the columns was done using the simplified method of the influence areas of the floors for each column, controlling the value of the normalised axial force to 0.50 and 0.65 in the case of primary and secondary columns respectively [7]. Simple models were adopted that allowed to estimate the load directed to each column, establishing the load paths in paving-beam-column, or, in the case of the zones of flat slab, in slab-column.

4.4 Foundations

In the pre-dimensioning of the foundations, it was considered that the soil presents characteristics favorable to a solution of footings connected by tie-beams. Following the requirements of EC8 a section of 0.25 m x 0.50 m was adopted for the tie-beams.

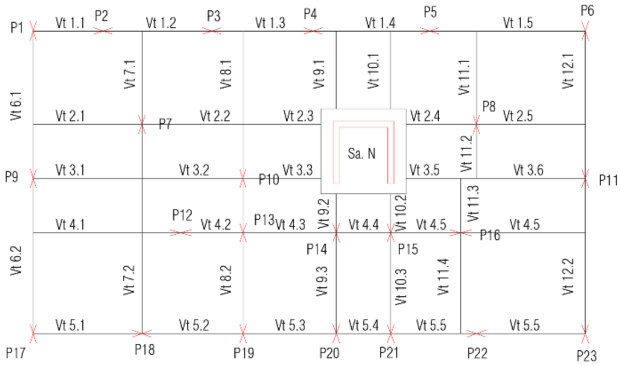


Figure 4.2 – Tie-beams disposition, Vt, in the case of the seismic joint model.

5. Structural modeling

The structural modeling of the two solutions was elaborated using *Robot Structural Analysis Professional 2018* software.

5.1 Structural elements

Following the recommendations of the EC8 the properties of the elastic stiffness of the concrete elements were reduced to half.

The columns and beams were modeled as bar elements and the walls modeled as shell elements.

The slabs were modeled using a rigid diaphragm for horizontal actions. For the evaluation of the forces in the slab, the thin slab option was used.

The foundations of the columns were modelled with hinges and connected with tie-beams and the foot of the resistant walls, it was modeled by elastic supports.

Given the difficulty in evaluating the participation of stairs and masonry to the stiffness of the models, these were not considered for the analysis of the primary structure [7] [8].

5.2 Actions

The permanent loads and live loads were evenly distributed across all pavements in accordance with their respective categories. Also, the exterior masonry loads were considered as linear load.

Following the EC8 prescriptions, the modal analysis was done through the Quadratic Combination Complete, for the combination of the modal maxima. Also, the seismic action was described by two orthogonal components.

6. Seismic design

After modeling the two structural solutions, the behavioral evaluation is carried out, analysing the contribution of secondary elements, plan regularity, frequencies and modes of vibration, structural system, behavior factor, response spectrum calculation, accidental torsion, 2nd order effects and limiting damage. The evaluation of the seismic behavior will be performed for the two models under study, so that a comparison will be made later.

6.1 Criteria

6.1.1 Model without seismic joint

The contribution of the secondary elements to the lateral stiffness of the structure is at most 3%, so that the model meets the requirement of section 4.2.2 (4) of EC8.

For a building to be regular in plan, it must meet all the requirements of section 4.2.3.2 of EC8. In this topic, it was verified that the model presents a reentrancy with respect to the convex polygonal line of 8.6% of its area, not respecting section 4.2.3.2 (3) of the EC8, that recommends a configuration in compact plan, reason why it is not considered regular in plan. However, the structure is not torsionally flexible, in accordance with section 5.2.2.1 (4) P of EC8, in which the structures must have a minimum torsional stiffness satisfying expression (6.1):

$$r_i \geq l_s \tag{6.1}$$

where r_i is the torsional radius of the structure and l_s is the radius of gyration of the mass of the floor in plan.

Evaluating the frequencies and modes of vibration in the results of the dynamic analysis, it is verified that the first two modes of vibration are of translation and the third mode is of rotation, as can be seen in the following table:

Table 6.1 - Periods and factors of mass participation by mode of vibration.

Mode	Period (s)	U_x (%)	U_y (%)	R_z (%)
1	1,15	71,31	0,00	5,11
2	0,99	0,00	75,09	0,00
3	0,94	5,47	0,00	70,00

In order to evaluate the type of structural system, ie the distinction between the various structural systems, it is necessary to calculate the level of forces or the percentage of compressive and shear forces in the various vertical structural elements. The model presents 52% of basal shear force on the walls in the x-direction and 57% in the y-direction. In relation to vertical loads, 70% of these are resisted by the columns, so the structural system is characterized by being a mixed system equivalent to walls in relation to vertical and horizontal loads in accordance with section 5.1.2 (1) of the EC8 [1].

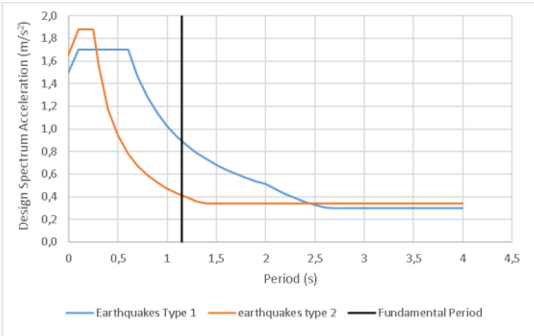
The behavior factor was calculated in accordance with section 5.2.2.2 of EC8. This section states that the behavior factor is defined by expression (6.2):

$$q = q_0 . k_w \geq 1{,}5 \tag{6.2}$$

where q_0 is the basic value of the behavior factor as a function of the type of structural system and k_w is a coefficient that reflects the predominant mode of rupture in the wall structural systems.

The basic value of the behaviour factor for the dual system equivalent to walls is q_0 = 3,0.α_u / α_1, where α_u / α_1 is a magnification factor being equal to α_u / α_1 = 1,1. The coefficient k_w takes the value 1.0 and the behavior factor is q = 3,3.

Once the behavior factor has been design, all conditions are met to define the calculation response spectrum. After entering the parameters of the seismic action, it was calculated the design response spectrum for earthquakes type 1 and type 2, where they are represented in Graphic 6.1 along with the mark of the fundamental period, where it is perceived that the earthquake type 1 is the most conditioning.



Graphic 6.1 - Design spectrum.

In order to take into account the accidental effects of torsion, section 4.3.3.3.3 (1) prescribes the application of a vertical axis torsor moment, applied in each floor, in accordance with section 4.3.2 (1)) of the EC8.

Regarding second order effects, EC8, in section 4.4.2.2 (2), provides for the possibility of dispensing the consideration if the coefficient of sensitivity to the relative displacement between floors is less than 0.1. After analysing the sensitivity coefficient, it was found that its value is mostly between 0.1 and 0.2, so the second order effects should be considered.

The verification of the damage limitation recommended in section 4.4.3.1 (1) of the EC8 aims to ensure that the values of the average relative displacements between floors are limited by a maximum value in the event of an earthquake with a higher probability of occurrence, so that the integrity of the structural and non-structural elements is ensured. In Table 6.2, is presented the verification of the damage limitation, where it can be observed that the structure does not verify the criteria of the limitation of damages.

Table 6.2 - Damage limitation check.

Floor	$d_{r,x}$ (cm)	$d_{r,y}$ (cm)	$d_{r,x}.v$ (cm)	0,005 h (x)	$d_{r,y}.v$ (cm)	0,005 h (y)
1	3,0	2,5	1,19	Ok	1,02	Ok
2	3,6	3,3	1,44	No	1,33	No
3	3,6	3,6	1,45	No	1,43	No
4	3,4	3,4	1,37	No	1,38	No

6.1.2 Model with seismic joint

As with the model without seismic joint, the contribution of the secondary elements to the lateral stiffness of the structure, is maximum 3%, fulfilling the requirement of section 4.2.2 (4) of EC8.

The model under study, by symmetry, presents the same relations between the total area and the reentrant area. In the same way as the model without a seismic joint, this one also does not fulfill the requirement of item (3) of section 4.2.3.2 of EC8, regarding the regularity in plan, reason why this model is not regular in plan.

After analysing the torsion radius and the radius of gyration, it is concluded that this model does not comply with section 4.2.3.2 (6) of the EC8, being torsionally flexible.

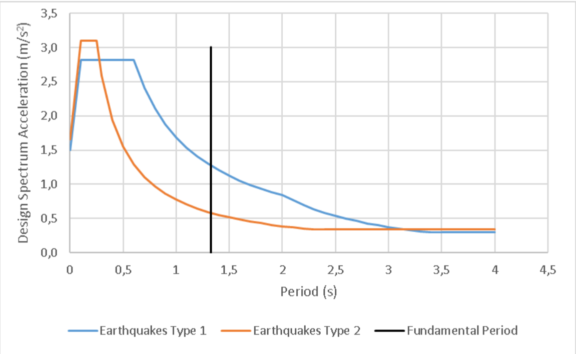
After analysing the results of the dynamic analysis, the first three modes show rotation associated with translation, as can be seen in Table 6.3.

Table 6.3 - Periods and factors of mass participation by mode of vibration.

Mode	Period (s)	U_x (%)	U_y (%)	R_z (%)
1	1,33	41,24	3,04	36,88
2	1,08	32,90	18,22	28,28
3	0,98	3,40	54,77	18,84

Since the building is a torsionally flexible system, the basic value of the behavior factor is equal to 2.0 and the value k_w, by symmetry, is equal to 1.0. In this way, the behavior factor takes the value equal to 2.0.

The design response spectrum for type 1 and type 2 earthquakes is shown in Graphic 6.2, along with the fundamental period mark.



Graphic 6.2 - Design spectrum.

In the same way as the previous model, the type 1 earthquake is the most conditioning.

After analysing the coefficient of sensitivity, it was verified that its value is mostly less than 0,1 for both directions. This resulted is lowering the behavior factor to 2.0, which implies an increase in the basal shear forces and consequently a decrease in the coefficient of sensitivity to the relative displacement.

In Table 6.2, is presented the verification of the damage limitation, where it can be seen that the structure does not verify the criteria of damage limitation.

Table 6.4 - Damage limitation check.

Floor	$d_{r,x}$ (cm)	$d_{r,y}$ (cm)	$d_{r,x}.v$ (cm)	*0,005 h (x)*	$d_{r,y}.v$ (cm)	*0,005 h (y)*
1	3,1	2,7	1,23	Ok	1,08	Ok
2	3,7	3,4	1,47	No	1,37	No
3	3,4	3,4	1,35	No	1,35	No
4	2,7	3,0	1,09	Ok	1,19	Ok

6.2 Comparative analysis

When comparing the two structural models, there are several factors to consider, such as the dynamic behavior, the constructive process, the maintenance and the cost of the model to be constructed. The seismic solution without joint, gains advantage by not being torsionally flexible and behaving like a dual system equivalent to walls. On the other hand, the seismic joint model behaves as a torsionally flexible system penalizing the behavior factor.

Making a seismic joint has higher costs than doing without a joint, since there will be a need to construct a double wall to separate the buildings, with all the structural elements that are part of the jointless structure to be duplicated in the wall in question. In addition, the construction of seismic joints requires not only greater attention in quality control processes, but also more specialized labour in the application of external protectors and specific materials. The maintenance costs for buildings with seismic joints are also higher because, even when well executed, the materials that make up the seismic joints have a life time shorter than the structure, thus requiring maintenance, replacement and repair costs.

Since the seismic joint will not bring about any significant improvement and considering all the drawbacks of its application, it is preferable to use the model without seismic joint. Thus, it is proposed to improve the structure in question (without joint), namely increasing the stiffness of the building to lateral actions.

6.3 Proposal for improvement

Considering that the previously analysed models did not verify the damage limitation criteria, an improvement is proposed for the model without seismic joint, since it is the most convenient one to be constructed. In this way, another structural option was verified that gives greater lateral stiffness to the structure, so that the parameters of the damage limitation are fulfilled and the sensitivity of the structure to the effects of 2^{nd} order is improved.

The initial model without seismic joint had the primary and secondary columns with section 0.22 m x 0.40 m. Therefore, with the proposed improvement, and after several attempts at accuracy, the primary columns become 0.22 m x 0.50 m. Four walls of 1.10 m x 0.22 m (Pa3, Pa5, Pa3' and Pa5') and four walls of 1.50 mx 0.22 m (Pa1, Pa4, Pa1' and Pa4') in the direction of *x*. Also, two walls with 1.50 m x 0.22 m (Pa 2 and Pa 2') were introduced in the *y* direction.

The representation of the location of the walls that were added to the structural model is shown in red in Figure 6.1.

It is emphasized that the image only represents half of the pavement, that by symmetry, is equal to the other half.

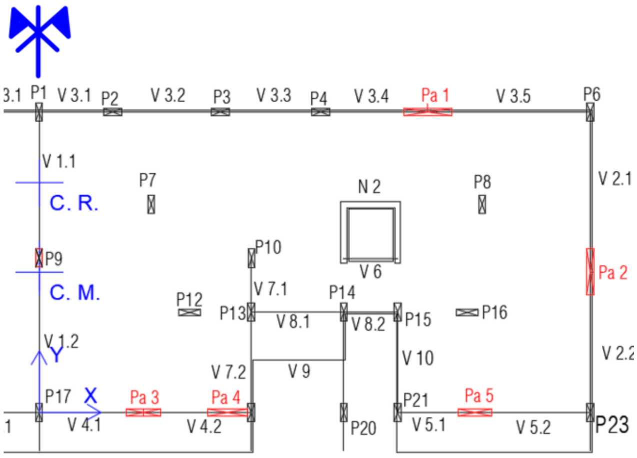


Figure 6.1 – Representation, in plan, of the location of the structural elements.

In relation to the beams, these structural elements have a fundamental role to increase the stiffness in the structure, choosing to increase in height the beams in the direction *y*, from 0,45 m to 0,60 m.

In the *x* direction, it was verified in the chapter referring to the structural design, that the beams were limited to 0,30 m in height by the architecture. Considering that the glazing and spans of the doors of the balconies are respectively 2.45 m and 2.20 m in height, it is assumed that the architecture allowed a reduction of 0.10 m height in these spans and a gain of the same value in the beams of the facades in question. Thus, it was decided to increase the beams in height from 0.30 m to 0.40 m and in thickness from 0.35 m to 0.40 m.

In order to give greater strength to the foundations and therefore to the structure in general, the tie-beams will be modeled with 0.70 m x 0.35 m. After analysis, it was verified that the tie-beam Vt10.2 was with very high moments, which required an reinforcement ratio close to ρ = 0,04 and it demands a high resistance of the transverse effort. This fact is justified by the free clearance of the beam (0.70 m) between the elevator core footing and the P15 column. In order to overcome the problem of crushing of the compression struts in the beam in question, and since the distance between of the column P14 and the column P15 are very close to the spread footing Sa.N (0.70 m), of the center of elevators, it is convenient only one spread footing (see Figure 6.2). Thus, the mentioned problems associated with the transverse stress are avoided and the overall dynamic behavior of the structure is improved.

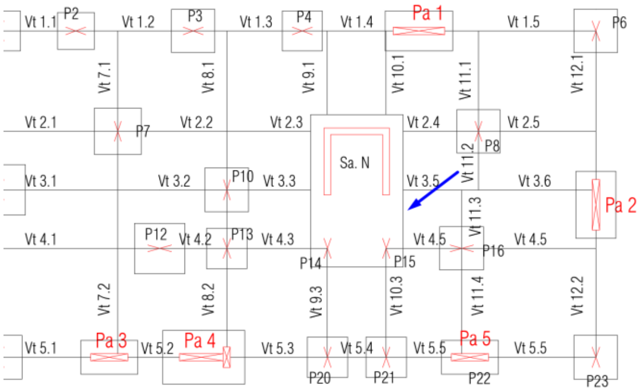


Figure 6.2 - Location of the footing that incorporates the spread footing of the two columns and the one of the nucleus.

After analysing the regularity in plan, it is concluded that the building is not torsionally flexible. However, due to the recess in the floor, the building continues to not be considered regular in plan.

In Table 6.5, the values of the frequencies, own periods and the mass participation factors of the modal analysis are presented.

Table 6.5 - Periods and factors of mass participation by mode of vibration.

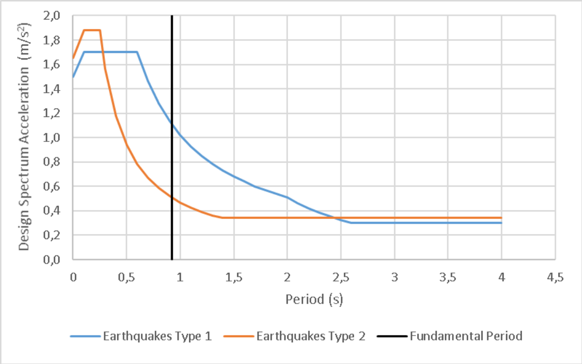
Mode	Period (s)	U_x (%)	U_y (%)	R_z (%)
1	0,92	74,97	0,00	2,33
2	0,76	0,00	75,89	0,00
3	0,73	1,83	0,02	16,82

By analysing the data presented above, the first two modes of vibration are composed by the translation component according to the two horizontal orthogonal axes, and the third mode present torsion. Also, there is a reduction of the fundamental period, evidencing an increase of stiffness in the improved model.

With the introduction of resistant walls, an increase in shear strength to be resisted by these vertical elements is observed in 21.3% in the *x* direction and 3.9% in the *y* direction. In relation to vertical loads, 61% of all loads are resisted by the columns, so the structural system, in the x direction and *y* direction, remains as a dual system equivalent to walls.

Since the structure in both directions is a dual system equivalent to walls, the basic value of the behavior factor takes the same value as the initial model without seismic joint. Thus, the value of q_0 is equal to $3{,}0.\alpha_u / \alpha_1$, keeping $\alpha_u / \alpha_1 = 1{,}1$. Also the value k_w remains equal to 1.0, so the behaviour factor is equal to 3.3.

The calculation response spectrum for the improved model is the same for the two orthogonal directions. Thus, by observation of the following graph, the type 1 earthquake is the most conditioning, as would be expected.



Graphic 6.1 - Design spectrum.

After improving the model without seismic joint, it was found that the coefficient of sensitivity θ, for the improved model, is less than 0.1 for both directions and on all floors.

The values for the limitation of damage are shown in Table 6.6, which is limited to 1.30 cm. As can be seen, the structure of the model under study verifies the criteria of limitation of damages and, therefore, it complies with the limits established in EC8.

Table 6.6 - Damage limitation check for the improved model.

Floor	$d_{r,x}$ (cm)	$d_{r,y}$ (cm)	$d_{r,x}.v$ (cm)	0,005 *h (x)*	$d_{r,y}.v$ (cm)	0,005 *h (y)*
1	1,9	2,1	0,75	Ok	0,82	Ok
2	2,6	2,8	1,02	Ok	1,11	Ok
3	2,7	2,9	1,10	Ok	1,17	Ok
4	2,6	2,7	1,02	Ok	1,08	Ok

7. Safety check

In order to check the safety of the ULS, only the reinforcement ratios that will be required for the dimensioning of the primary and secondary structural elements, i.e., to check the level of forces induced in the structure, will be compared. In this way, and in the case of a previous study, design details will not be considered, namely values of maximum spacing between reinforcements, diameters of reinforcing bars, critical zones, confinement, local effects of the masonry and local ductility.

7.1 Slab

The verification of the safety of the slabs, to the bending and to the punching, was made from the forces obtained in the finite element model, in relation to the fundamental combination and in relation to the seismic combination.

By analysing the slab panels, through the forces diagrams obtained by the finite element model (Figure 7.1 and Figure 7.2), it can be seen that the higher bending stresses are located on the column P7, for the negative bending moments of m_{xx} and for m_{yy}. In relation to the positive bending moments, the highest stresses are in the x-direction in the alignment "2" between the column P16 and the edge beam V2 and in the direction y in the "C" alignment between the column P10 and the edge beam V3 (Figure 4.1), these values being represented in Table 7.1.

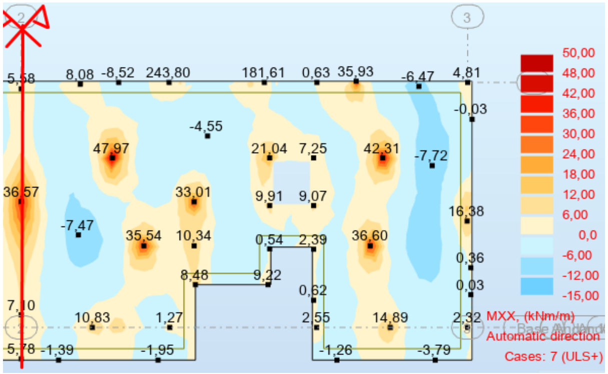


Figure 7.1 - Map of bending moments, in the *x* direction, fundamental combination.

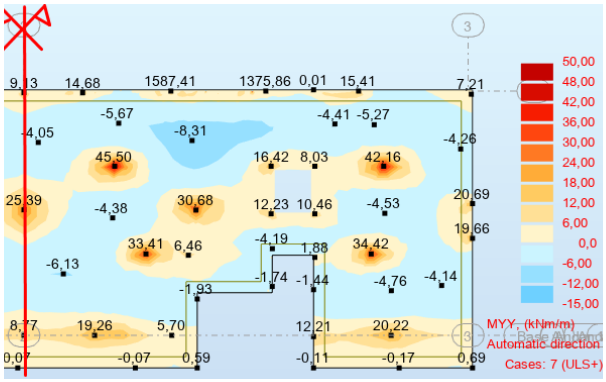


Figure 7.2 – Map of bending moments, *y* direction, fundamental combination.

The bending moments were calculated by the average of the forces (bending moments and torsors) in a measurement of width considered equal to 1,50 m.

The recommended limits for reduced positive and negative bending moments are respectively $\mu \leq 0.18$ and $\mu \leq 0.30$. Thus, once calculated, it has been found that the reduced bending moments are in the order of 0.02 and 0.07 respectively for positive and negative moments.

7.1.1 Punching and resistance of the slab-column connection

The punching shear resistance check shall be made in accordance with section 6.4 of EC2, on the slab of floor 1, for the fundamental combination of actions and for the seismic combination, as this combination influences the eccentricity of the punching. Thus, after analysing the efforts on the columns, it was verified that the column P7 is the one that presents greater levels of efforts transmitted by the slab, reason why the punching shear has a heavier effect on the same one.

In order to verify the strength of the slab at the moment transmitted by the column due to the seismic action with $q = 1,5$, a distribution in a width of 2*h* is considered for each side of the column faces, where *h* is the thickness of the slab. The values of the obtained moments must be increased by the relation between the displacements of the primary structure, d_{e1}, and the displacements of the global model, d_{e2}, where the secondary elements participate.

At the perimeter of the column and at the control perimeter, the shear stress results from the existence of concentrated forces and the transmission of bending moments between the slab and the columns, whereby safety has been verified with the eccentric punching. The verification of punching shear resistance at the perimeter of the column for fundamental and seismic combinations is shown in Table 7.1, with safety being verified.

Table 7.1 - Verification of punching resistance at the column perimeter for fundamental and seismic combinations.

Comb.	Column	V_{Ed} (kN)	B (-)	v_{Ed} (kPa)	$v_{Rd,max}$ (kPa)
Fund.	P7	204,3	1,15	1151,7	5280
Seismic	P7	128,5	1,98	1248,3	5280

The Table 7.2, shows the verification of puncture safety in the control perimeter at *2d* of the column, for fundamental and seismic combinations safety is also verified.

Table 7.2 - Verification of puncture safety in the control perimeter at 2d of the column for fundamental and seismic combinations.

Comb.	Column	v_{sd} (kPa)	$v_{Rd,c}$ (kPa)	v_{min} (kPa)
Fund.	P7	414,4	609,5	542,2
Seismic	P7	449,1	609,5	542,2

By observing Table 7.1 and Table 7.2, the shear stresses at the perimeter of the column and at the basic control perimeter of the P7 column are below the limits, so it is not necessary to adopt an enlarged column head or specific shear reinforcement. Also, it can be verified that the seismic combination of actions is conditioning with regard to punching forces.

7.2 Beams

In this chapter, it is only of interest to investigate the ductility of the beams, to ensure compliance with the maximum reinforcement ratio for bending and transverse stress, and to verify the compressive stresses in the web.

The beams in the x-direction are 0,40 m x 0,40 m (width x height) and the most conditioning forces are located in beam V4.2, for the seismic combination. The value of the maximum negative moment of the studied beam, next to the resistant walls Pa.3 and Pa.4, and its required area of reinforcement are represented in Table 7.3. In turn, the value of ρ_{max} is calculated in accordance with section 5.4.3.1.2 (4), being equal to 0.014 for the beam in question.

Table 7.3 - Verification of the longitudinal reinforcement ratio for the seismic combination (Beam V4.2).

Beam	M_{Ed} (kNm)	μ (-)	ω (-)	A_s (cm^2)	ρ (-)	ρ_{max} (-)
V4.2	145	0,29	0,322	14,80	0,012	< 0,014

As can be seen in the previous table, the required reinforcement ratio is lower than the maximum limits recommended by EC2 and EC8. However, a verification must be made on the basis of equilibrium by the real capacity, in order to calculate the maximum transverse force V_{Ed}, and, later, verification of the compression struts next to the supports, $V_{Rd,\,max}$.

The design shear force and the design shear resistance are calculated by considering the same flexural resistance, for cyclic moments in the beam under study (lower and upper reinforcement).

Table 7.4 - Check compressed struts (Beam V4.2).

V_{QPR} (kN)	$M_{Rb,1}$ (kN.m)	$M_{Rb,2}$ (kN.m)	Υ_{Rd}	l_{cl} (m)	V_{Ed} (kN)	$V_{Rd,max}$ (kN)
15,2	155,0	155,0	1,0	1,5	221,9	380,5

As can be seen in Table 7.4, the beam verifies the safety of the maximum design shear resistance limited by crushing of the compression struts.

The beams in the y-direction are 0.22 m x 0.60 m (width x height) and the most conditioning stresses are located on beam V 2.2, for the seismic combination. The value of the maximum moment in the studied beam, next to the resistant wall Pa.2, and its required area of reinforcement are presented in Table 7.5.

Table 7.5 - Verification of the longitudinal reinforcement ratio for the seismic combination (Beam V2.2).

Beam	M_{Ed} (kNm)	μ (-)	ω (-)	As (cm^2)	ρ (-)	ρ_{max} (-)
V 2.2	275	0,21	0,231	12,90	0,010	< 0,013

As can be seen in Table 7.5, the required reinforcement ratio is lower than the maximum limits recommended by EC2 and EC8.

In the Table 7.6 shows the calculation of the shear acting force and the resistant shear force.

Table 7.6 - Check compressed struts (Beam V2.2).

V_{QPR} (kN)	$M_{Rd,1}$ (kN.m)	$M_{Rd,2}$ (kN.m)	Υ_{Rd}	l_{cl} (m)	V_{Ed} (kN)	$V_{Rd,max}$ (kN)
51,0	179,0	179,0	1,0	3,5	216,3	460,4

As can be seen in Table 7.6, the beam verifies the safety of the design value of the maximum shear force, limited by the crushing of the compression struts.

7.3 Columns

It was decided to choose only the representative column for axial load (P9 column), which will be exposed to the short column effect in the staircase area (P20 column), which will have the largest relative displacements between floors throughout the structure (column P23) and the secondary column that has the greatest influence area (P7 column). Thus, the analysis of the level of stress in these columns was initiated by the verification of the normalised axial force, v_d, being obtained by the maximum combined value between the forces of the quasi-permanent load combination and the seismic combination, according to section 5.4. 3.2.1 (3) P of EC8 shall be less than 0,65 in the primary columns. It was also carried out the verification to the bending with axial and to the shear force, namely the verification the crushing of the compressed struts.

Although the detailing is not part of this study, it is necessary to use the calculation of the reinforcement in the columns to verify the level of bending, compression and shear. The flat slab will be checked in an elastic regime, since the slab-column connecting zone should behaver in an elastic regime. In this way, it was verified that the reinforcement ratios are within the admissible limits for the seismic combination. The design value of the shear force is determined according to the capacity design rule. After calculating the shear force and the design shear resistance, was verified these elements respect the resistance requirements.

7.4 Walls

The safety check on all the walls that constitute the structure, considered the verification of the design value of axial force, bending moment and shear force. Regarding normalised axial force, EC8 recommends that this value be less than 0.4 on primary seismic walls. For this verification, it was considered an quasi-permanent combination of actions.

Following the recommendations of EC8, the longitudinal reinforcement should be concentrated near the ends of the wall sections. In the following image can be highlight the concept of fictional column method.

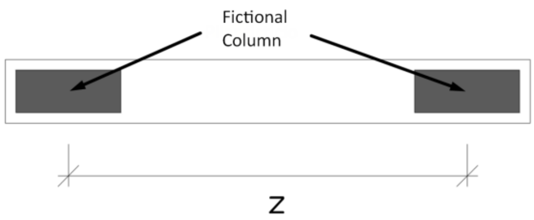


Figure 7.3 - Schematic representation of a shear wall, including the end elements and the bending arm, z.

In order to verify the shear force according to EC8, namely the crushing of the compression struts, the value of the transverse acting force must be increased by 50% in order to consider the possibility of an increase of the transverse forces after plastification at the base of a seismic wall. In relation to the bending moment, as if the forces are being checked only at the base of the walls, it does not undergo any changes for the purposes of the safety check.

After analysing all the walls, it was found that the longitudinal reinforcement ratios and the verification of the transverse stress through the crushing of the compression struts in the resistant walls, verified the safety.

7.5 Foundations

The safety check of the tie-beams was performed considering a behavior factor of q = 1,5, as defined in section 5.8.1 (4) of the EC8-1, considering elastic behavior. In this way, the safety check was made in accordance with EC2.

As the tie-beams have all the same geometry, 0.70 m x 0.35 m, to check the level of stress in the beams it was decided to identify the largest forces and to make the necessary checks. For the safety check, the beams Vt 10.1 and Vt 5.2, marked with blue arrows in Figure 7.4, are the ones that present the greatest bending forces and a relatively short length in comparison with other beams. Thus, the value of the maximum moment of the beams in question is next to the spread footing Sa.N of the elevator core and next to the wall Pa 3.

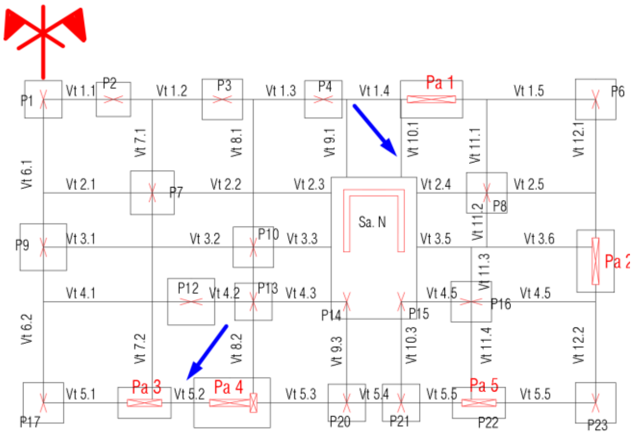


Figure 7.4 - Schematic indication of the location of maximum bending moments in the tie-beams.

After analysis of the beams in study, it was verified that respect the safety to the shear force through the crushing of the compressed struts and that the reinforcement ratios were below the recommended maximum limit.

8. Final notes

Throughout the present study it was possible to go through the main phases of a structural project where there was constant concern to apply the regulations concerning the structural Eurocodes, and it should be emphasized that the bibliography consulted for the implementation of the various chapters was fundamental in the execution of the whole job.

The main difficulties encountered were the elaboration of a model with the required stiffness in the x direction, with the beams in this direction having to be inverted. The model with seismic joint, presented torsion in the first vibration modes while the model without seismic joint presented translation.

After analysing the two structural solutions, it was verified that both solutions did not meet the criteria of the seismic design, so the chosen model was the one without joint and its seismic behavior has been improved. Then, in this way, ten resistant walls were introduced in order to increase the lateral stiffness and the dimensions of the columns and beams were increased. Therefore, the improved structural solution verified all seismic design criteria.

Finally, a safety check was made in order to validate the improved structural solution, and the effort levels were verified in all the structural elements.

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