October 2021

# Structure design of a service building


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## Abstract

This dissertation aims to develop a structural project of a service building. The different stages inherent to its elaboration will be covered, where all design decisions and calculations developed will be demonstrated. Starting with the design in the face of architectural constraints, followed by the definition of the structural solution, the pre-design of structural elements and the analysis and design of the structure for the combinations that involve the gravitational and the seismic actions.

The project has some non-current conditioning areas, involving, namely a garden area located on floor 2, subject to a high live load, spans of 15.35 m on floors 3 to 6 and finally, the cantilevers slabs of 8 m of span with variable section on these same floors. These areas were the object of a design, structural analysis and design to obtain appropriate structural solutions. Another important aspect of the work was the design and dimensioning of the structure for seismic action. In view of the constraints of the architecture, the structure of the building presents relevant irregularities which led to the design of a structure resistant to seismic action with adequate robustness to be able to control its behavior. To perform the analysis and design of the structure, structural models were elaborated in the different phases of the study to pre-design the elements and finally check the safety to the ultimate and service limits states.



## Keywords

Project; Structure; Design; Prestress; Seismic design; Eurocodes.

## 1. INTRODUCTION

This master's thesis fits into the domain of a structural project of a service building located in the city of Lisbon. It aims to carry out a study of structural solutions to verify the safety and to ensure the principles of an appropriate structural design. The building in question consists of prestressed reinforced concrete.

This work manifests and emphasizes all the knowledge acquired throughout the master's degree in Civil Engineering, namely in the field of Structures, which is the main motivation for the execution of the project, in other words, to the application of the theoretical and practical concepts obtained, with the objective of creating autonomy at the level of design and structural detail. Aiming at a better understanding of the stages developed in this dissertation, it is structured in 9 chapters:

1) Introduction.
2) Description and characterization of the building.
3) Structural materials based on EC2 [1].
4) Project actions and combinations of actions according to EC1 [2].
5) Structural solution and pre-design, based on EC2 [3] and [4].
6) Three-dimensional modeling.
7) Design of the prestress of floors 3 to 5, according to EC2 [3] and [5].
8) Design and verification of safety, emphasizing the seismic analysis of the structure, following EC8 [6], EC2 [3] and [4].
9) Conclusion, with the objective of supporting the analyses and calculation procedures presented.

## 2. BUILDING DESCRIPTION

• **Characterization of the building:** it is a large building, whose structure will be in prestressed reinforced concrete. The vertical access is based on 2 cores of stairs ranging from the basement to the 6th floor.

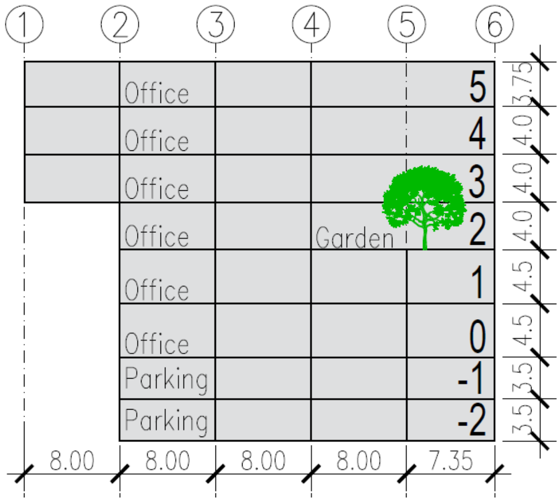


Figure 1 - Building schematic cut

• **Architectonic conditioning factors:** there are 3 principals, these being the 2nd floor that contains a significant load due to the garden area; the insulated blocks ranging from the 3rd floor to the roof which present a span of 15.35 m and finally, on these same floors, the cantilever with a span of 8 m. These zones will require special attention in the determination of the structural solution, to resist the gravitational actions and also to the effects of the seismic action.

## 3. STRUCTURAL MATERIALS

Structural materials must ensure the durability and strength of their elements. To determine the exposure class, structural class, service life and concrete covers, were followed the assumptions of EC0 [1] and EC2 [3], the adopted materials are indicated below.

Table 1 - Mechanical characteristics of structural materials

| Concrete C30/37 | | | |
|---|---|---|---|
| $f_{ck}$ (MPa) | $f_{cd}$ (MPa) | $f_{ctm}$ (MPa) | $E_c$ (GPa) |
| 30 | 20 | 2.9 | 33 |
| Steel A500 NR SD | | | |
| $f_{yk}$ (MPa) | $f_{yd}$ (MPa) | $\varepsilon_{yd}$ (‰) | $E_s$ (GPa) |
| 500 | 435 | 2.175 | 200 |
| Pre-stress steel Y1860 | | | |
| $f_{puk}$ (MPa) | $f_{pyk}$ (MPa) | $f_{pyd}$ (MPa) | $E_s$ (GPa) |
| 1860 | 1670 | 1452 | 195 |

## 4. PROJECT ACTIONS

They are divided according to their performance throughout the lifetime of the building, being permanent or variable. To integrate these actions into the structural analysis of the building, it was necessary to proceed to the combinations of actions following what is prescribed in EC0 [1]. To determine seismic effects, the EC8 [6] reference method will be adopted, which is based on an elastic-linear model of the structure and modal analysis by response spectrum.

## 5. STRUCTURAL SOLUTION AND PRE-DESIGN

• **Slabs:** waffle slabs with strips (0.80x0.80m), a height of 0.35 m for all floors, for the garden area of 0.45 m and for the cantilevers, a composite slab solution, with a solid reinforced concrete slab of 0.10 m high and steel beams on the lower face, ensuring the support of this structure by the prestressed strips. These strips ensure greater rigidity and resistance in areas where there are large concentrations of stresses, also provide a better behavior in the face of horizontal actions and better distribute the stresses due to vertical actions.

• **Columns:** for uniformity reasons, 3 standard dimensions were selected, being 0.60 m, 0.90 m and 1.55 m and fixed to another dimension, which is 0.30 m.

• **Walls:** in order to have a good structural design in the face of seismic action, these elements must ensure the proximity between the center of stiffness and mass of the structure, so as to have less torsional effects, it is also important to have rigid vertical elements in opposite areas in plan and that these are continuous from foundation to upper level. That said, 8 reinforced concrete walls were adopted, coupled in pairs, at one side of the building and 2 cores of stairs at the other.

• **Prestress spans of 10.5 m and 15.35 m:** prestress have the advantages to use greater spans, control of deformation and improvement of the behavior in service. For floors -1 to 2, cables with monostrand system with 150 kN of prestress force, and for floors 3 to 6, cables with 160 kN, both with $\emptyset = 0{,}6''$, these monostrands can be grouped in pairs. To determine the value of prestressing force, expression (1) was used.

$$P_\infty = \frac{q_{eq} \times L^2}{8 \times f} \qquad (1)$$

• **Prestress in the cantilever area:** the requirement for cantilever deformation was to limit the vertical displacement at the end to a limit value L/400. This limit corresponds to a long-term deformation equal to 20 mm and an elastic deformation of 6.7 mm. Through the diagram of Figure 2, the forces in the necessary prestress cables were determined and were adopted 29 strands in the central strips and 12 in the sides.

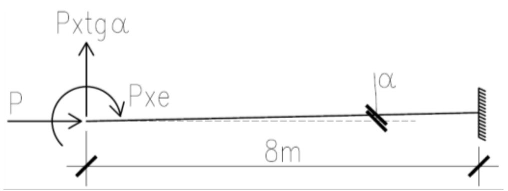


Figure 2 - Loads at the end of the cantilever

## 6. MODELING

Four different models were elaborated in the *SAP2000* automatic calculation program, which were intended for the verification of deformations in service and the ultimate limit states, to validate the dimensions previously adopted, both in terms of safety and functionality.

To simulate the columns and walls, the torsional stiffness was reduced, so that the elements resist essentially by bending and shear. For the foundations, the building was fixed at the level of the 2nd floor, because the horizontal deformations are impeded on the basement floors.

The seismic action is simulated by introducing elastic response spectrum and calculation response spectrum into the global model. In addition, in a dynamic analysis, the stiffness of the resistant elements should be evaluated considering the effect of the cracking reducing the bending and shear stiffness of the structure by 50%.

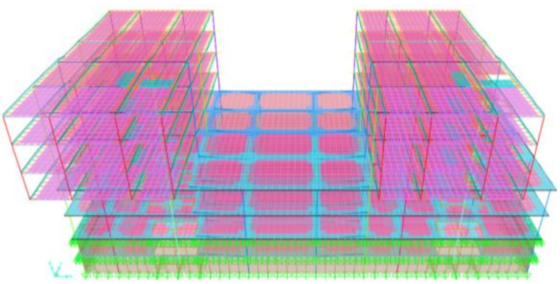

Figure 3 - Global model on *SAP2000*

## 7. PRESTRESS DESIGN OF FLOORS 3 to 5

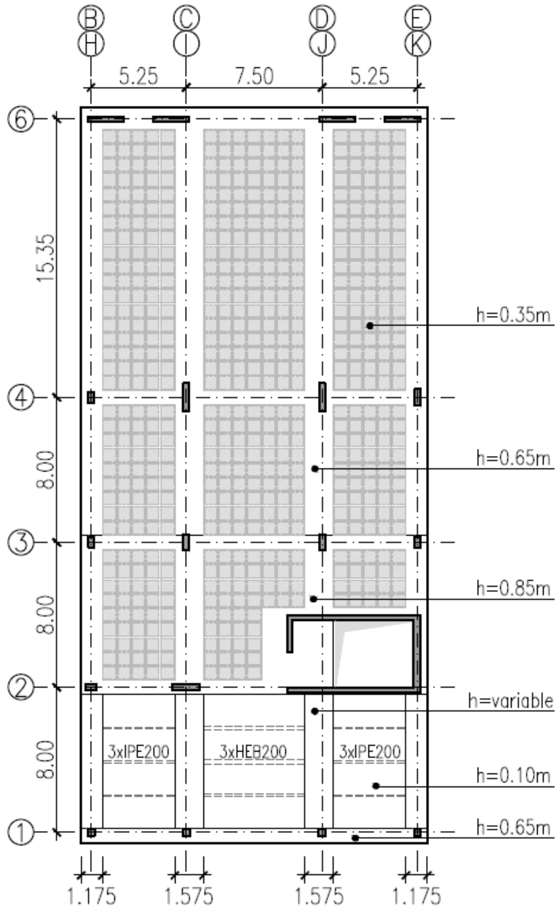


Figure 4 - Plan of a block of floors 3 to 5

- **Maximum eccentricities and determination of cable anchorage:** in the strips with $h = 0.65\,m$ there is $e_{max} = 0.205\,m$ and in strips with $h = 0.85\,m$, $e_{max} = 0.305\,m$. However, at the end of the cantilever, it was necessary to verify through the anchors adopted what would be the maximum eccentricity allowed, these with propellers of $\emptyset 295$, therefore, it was obtained, $e_{max} = 0.1345\,m$.

- **Cable layout:** accompanies the diagram of bending moments, thus opposing the deformations along the strips due to the external actions. Follows the moment diagram due to the quasi-permanent loads on the C/I strip on Figure 5.

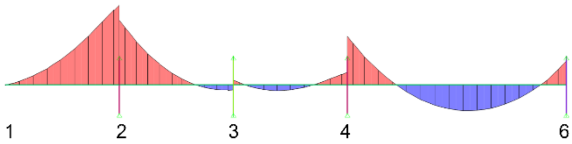


Figure 5 - Bending moments due to quasi-permanent loads on C/I axis

The following are the cable layout span to span:

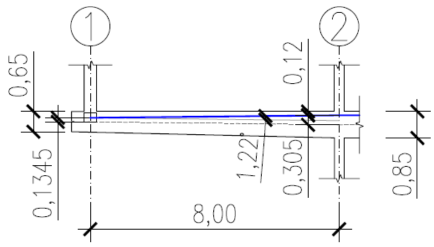


Figure 6 - Prestress layout between axes 1-2

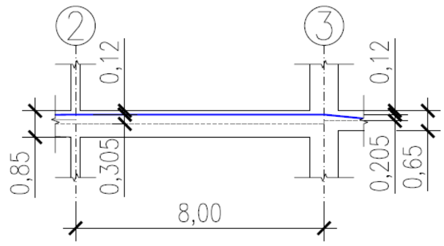


Figure 7 - Prestress layout between axes 2-3

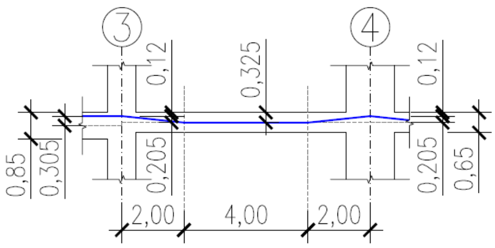


Figure 8 - Prestress layout between axes 3-4

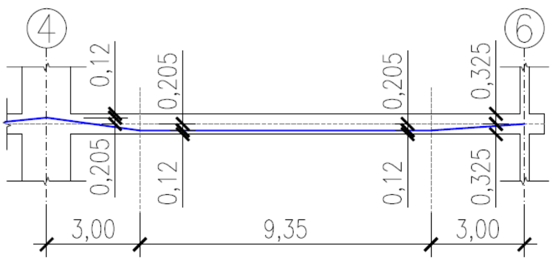


Figure 9 - Prestress layout between axes 4-6

• **Deformation control:** in order to compare the values of elastic deformations due to the quasi-permanent loads with those obtained after the insertion of the prestress, these are presented in the following figures and the values are in Table 2.

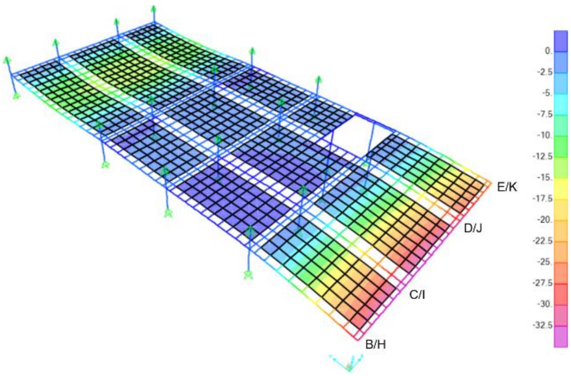


Figure 10 - Deformations due to quasi-permanent loads

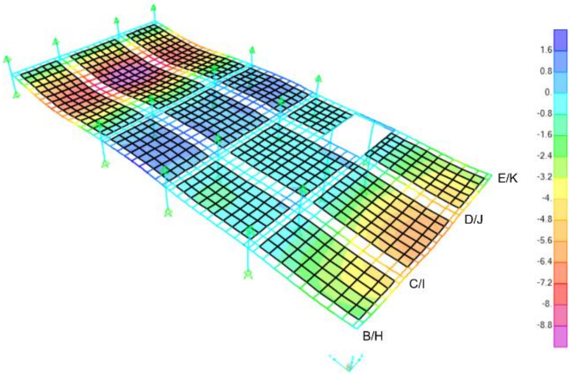


Figure 11 - Deformations due to quasi-permanent loads with prestress

Table 2 - Deformations on the slabs

| Deformation (mm) | | | | |
|---|---|---|---|---|
| Strip | Cantilever | | Span of 15,35 m | |
| | Pqp | Pqp+PE | Pqp | Pqp+PE |
| B/H | 31,5 | 0,4 | 10,7 | 6,5 |
| C/I | 34,3 | 4,7 | 13,4 | 8,3 |
| D/J | 31,0 | 5,1 | 13,4 | 8,1 |
| E/K | 24,5 | 1,2 | 10,3 | 6,0 |

In the zone with the span of 15.35 m, the objective of limit elastic deformation is 8.56 mm, this value was determined by the expression (2).

$$\delta_{elástico} \leq \frac{L}{600 \times (1+\varphi)} \qquad (2)$$

• **Arrangement of cables in the strips:** were adopted cables with 12 and 7 strands, each with 160 kN of prestress force. The cantilever zone requires more prestress than the 15.35 m span, so some prestress cables end in alignment 3. The result is shown in Table 3, the arrangement of the anchors and the cable layout in the central strips are shown in Figure 12 and Figure 13.

Table 3 - Prestress forces

| Strip | Strands | P∞ (kN) | | |
|---|---|---|---|---|
| | | Total | Axis 1-3 | Axis 1-6 |
| C, D, I e J | 38 | 6080 | 2240 | 3840 |
| B, E, H e K | 24 | 3840 | 1920 | 1920 |

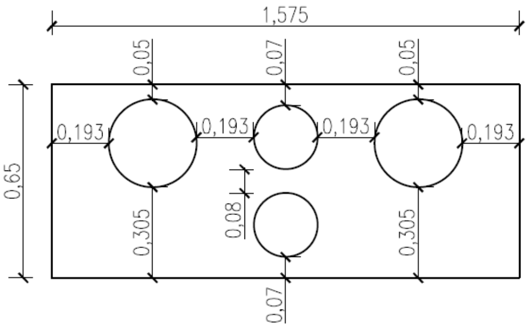


Figure 12 - Arrangement of the anchors of the prestress cables in the central strips

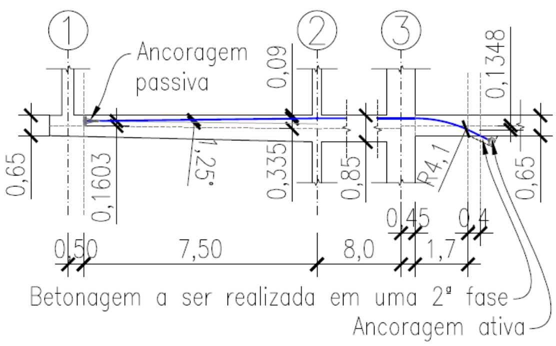


Figure 13 - Prestress layout, on central strips of axes 1-3

• **Verification of human-induced vertical vibrations:** the assumptions of [5] were followed, where the vibration modes of the structure corresponding to the vertical movement were first evaluated. It is also necessary to take into account the frequency value and the percentage of mass participation associated with each mode, in order to determine the corresponding mass, so:

- Cantilever zone: 1st vibration mode → $f = 3.8\ Hz$ e $Uz = 9\%$ → 118.08 ton
- Span of 15.35 m: 3rd vibration mode → $f = 6.3\ Hz$ e $Uz = 19.9\%$ → 261.23 ton

The critical damping of this floor is equal to 4% and the recommended performance class from A to D, so the evaluation of maintenance capacity for vibration performance is determined by analyzing the OS-RMS99 coefficient graph, thus verifying for both zones.

# 8. DESIGN AND SAFETY CHECK

## Combination of seismic actions

### Seismic analysis

• **Vibration modes:** through the mass participation factors, the type of movement associated with each vibration mode is determined, being the 1st mode corresponding to the translation movements in the Y direction with torsion coupled according to the Z axis; the 2nd twist around Z direction with Y-coupled translation; and the 3rd translation in X.

• **Characterization of the type of structure:** through displacements due to the elastic response spectrum, it was observed that there was no significant torsion. To define the structural system, the shear forces at the level of floor 0 were analyzed, and it was obtained that the walls resist more than 65% of the total shear force in both directions, so it is a system of uncoupled walls.

- **Regularity of the structure:** the building is irregular in height, as the lateral stiffness and mass of each floor do not remain constant or show a gradual reduction, from the base to the top of the building. It is also irregular in plan, because of the two separated blocks above level 2.

- **Behavior coefficient:** the behavior coefficient is used to take into account the ability to dissipate energy. This depends on several factors, and the obtained value is equal to $q = 2.4$.

- **Calculation response spectrum:** it is defined in order to avoid a non-elastic structural analysis and is based on the elastic response spectrum affected by the behavior coefficient.

- **Accidental torsional moments:** it is necessary to take into account the effects related to constructive execution and the action of the ground on the structure, because of this, the accidental torsional moments need to be considered on the various floors being introduced as actions in the calculation model.

- **Limits of displacements between floors**: so that there is no damage to non-structural elements, the most critical situation was considered: non-structural elements consisting of fragile materials fixed to the structure.

- **Second order effects:** are related to the displacements at the ends of each column subject to a vertical loading, which leads to an increase of the bending moments. Was verified that the building is not subject to relevant 2nd order effects.

## Design

To design the vertical elements, stair cores, coupled walls and columns, the analysis of the forces in the structure was made first, for the most requested elements. From this, with the aid of the *Response 2000* automatic calculation program, the bending was dimensioned, obtaining the resistant bending moments for the adopted reinforcements. To verify the deflected bending, the simplified criterion of EC2 [3] was followed, according to equation (3).

$$\left(\frac{M_{sd,x}}{M_{rd,x}}\right)^{\alpha} + \left(\frac{M_{sd,y}}{M_{rd,y}}\right)^{\alpha} \leq 1 \qquad (3)$$

The design followed the calculation method of the capacity design, where it is expected the formation of a plastic hinge with high rotation capacity at the base of the resistant element, at level of floor 0, in order to control the mode of failure, preventing it from being fragile.

Regarding to shear force safety verification in the critical zone, EC8 [6] defines a coefficient of increase, $\varepsilon = 1.5$. In order to be able to verify the compression stress, it is necessary to first determine the maximum value of the admissible compression stress, using equation(4)and the maximum resistant shear force, through the equation (5).

$$\sigma_{c,max} = 0{,}6 \times \left(1 - \frac{f_{ck}}{250}\right) \times f_{cd} \qquad (4)$$

$$V_{rd,max} = \sigma_{c,max} \times b \times z \times sen\,\theta \times \cos\theta \qquad (5)$$

To design the reinforcements of the shear force, follow the equation (6).

$$\frac{A_{sw}}{s} = \frac{V_{sd}}{z \times \cot\theta \times f_{yd}} \quad (6)$$

The fixed support, located at the base at level 0, should be considered as a critical zone. To design it, is necessary to first determine the height of the critical zone, through the (7) and (8), and this height is defined for the case of the stair's core and coupled walls.

$$h_{cr} = max\left\{l_w; \frac{h_w}{6}\right\} \quad (7)$$

$$h_{cr} \leq \begin{cases} 2 \times l_w \\ \begin{cases} h_s, n \leq 6\, floors \\ 2 \times h_s, n \geq 7\, floors \end{cases} \end{cases} \quad (8)$$

Regarding the reinforcement, it should extend vertically along the height $h_{cr}$ and horizontally along the minimum length of the end elements, which is defined according to the expression (9) for the stair's cores and coupled walls, and (10) for the columns.

$$l_c = máx\{0{,}15 \times l_w; 1{,}5 \times b_w\} \quad (9)$$

$$l_{cr} = máx\left\{h_c; \frac{l_{cl}}{6}; 0{,}45\right\} \quad (10)$$

In these critical areas, it is necessary to guarantee a minimum value for the curvature ductility factor, according to the expression (11).

$$\mu_\theta = 2 \times q_0 - 1 \quad (11)$$

In order to determine the confinement reinforcement, an iterative method were followed, through the equation (12), in which the requirements of EC8 [6] for the confinement of the critical zone are considered satisfied if this equation is verified. For the columns the parameter $\omega_v$, does not exist.

$$\alpha \times \omega_{wd} \geq 30 \times \mu_\theta \times (\nu_d + \omega_v) \times \varepsilon_{sy,d} \times \frac{b_c}{b_o} - 0.035 \quad (12)$$

The maximum spacing of the links in the longitudinal direction in the confined areas, is determined according to the expression (13).

$$s_{max} = min\left\{\frac{b_o}{2}; 175; 8 \times d_{bl}\right\} \quad (13)$$

The volumetric rate of the confinement reinforcement is determined through the expression (14).

$$\rho_w = 2 \times min\left\{\frac{A_{sw,x}}{b_o \times s}; \frac{A_{sw,y}}{h_o \times s}\right\} \quad (14)$$

Finally, through the expression (15), the volumetric mechanical rate of links in the critical zones is determined, for the reinforcements adopted, and the minimum value imposed by EC8 [6] is $\omega_{wd,min} = 0.08$.

$$\omega_{wd} = \rho_w \times \frac{f_{yd}}{f_{cd}} \quad (15)$$

• **Stair's core:** the forces diagrams are represented in Figure 14 in sequence. It serves to qualitatively evaluate the progress of forces in the structure, where it can be observed, that are effectively equal in both stair's cores.

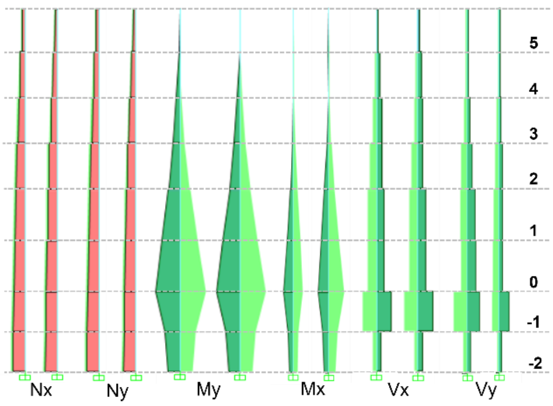


Figure 14 - Forces diagrams in the stair's cores for seismic combination

The core section geometry is shown in Figure 15, in which the zones indicated by A and B correspond to the free ends, in this case, to the critical zones.

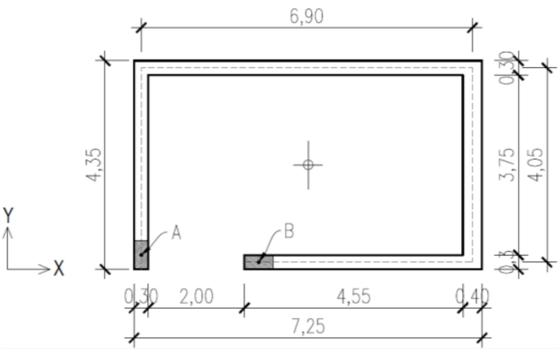


Figure 15 - Geometry of the core of stairs

The arrangement of the calculated reinforcements for the critical zone is shown in Figure 16.

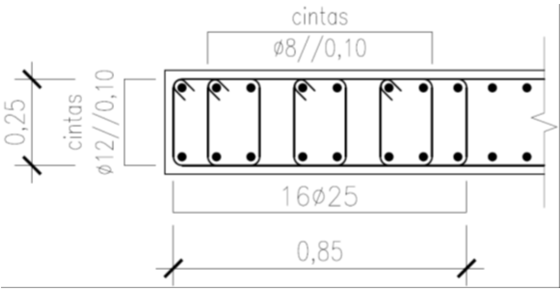


Figure 16 - Confinement of the core of stairs

• **Coupled walls:** have 1.95x0.30 m in the X and Y directions, respectively.

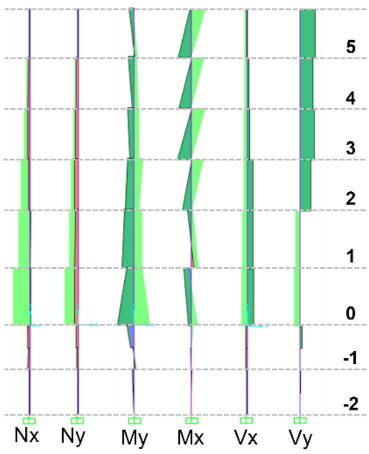


Figure 17 - Forces diagrams in the coupled walls for seismic combination

• **Columns:** two columns were analyzed, one with 1.55x0.30 m and the other with 0.30x0.60 m. A particularity in the sizing of the columns is that the acting shear forces are determined by balance of the column, considering the resistant bending moments in the extremities associated with the formation of plastic hinges, through the expression (16).

$$V_{sd} = \gamma_{Rd} \times \frac{(M_{Rc,1} + M_{Rc,2})}{l_{cl}} \qquad (16)$$

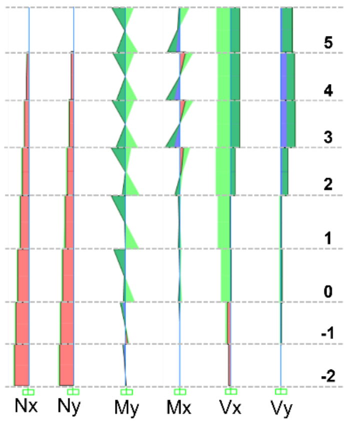


Figure 18 - Forces diagrams in the column 0.30x0.60m for seismic combination

• **Coupling beams:** the prescriptions of EC8 [6] were followed regarding the coupling beams. The resistance to seismic actions should be conferred by reinforcements arranged according to the two diagonals of the beam, according to the expression (17), with its geometry represented in 19.

$$V_{sd} \leq 2 \times A_{si} \times f_{yd} \times sen\ \alpha \qquad (17)$$

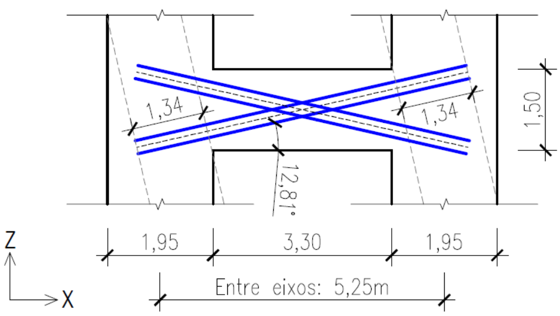


Figure 19 - Coupling beam geometry

## 9. CONCLUSION

The developed dissertation aimed to go through all the necessary steps for the elaboration of a structural project of a prestressed reinforced concrete service building. The choice of the structural solution was adapted to the existing architectural conditions.

From a structural point of view, the building presented 3 major challenges, considerable spans, 8 m long cantilevers and a garden area on a raised floor, which required the adoption of a structural system that provided adequate rigidity, counteracting the deformation of the solid slabs and strips, but also light in order to reduce its own weight. To this end, a structural system of waffle slabs was chosen, and to ensure adequate rigidity, stairwell cores were adopted with resistant walls on one side of the structure and on the other, walls coupled by coupling beams. This structural system leads to an adequate behavior against earthquakes, as it provides a good torsional rigidity to the building, controls the relative displacements between floors and the global deformation.

The building is located in a region with relevant seismicity, Lisbon, therefore, it was necessary to pay special attention to the seismic behavior of the structure, which means that the structural elements were mainly emphasized, which the combination due to seismic actions is a conditioning factor, namely, the stairwells core, the coupled walls, the coupling beams and the columns. These followed the requirements imposed in EC8 [6] and also in EC2 [3].

Finally, it was possible to obtain a valid, functional, safe and feasible solution from a structural point of view, respecting the regulations.

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