Project design of a public space roof

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Abstract

The project and design of a structure have as a main goal to ensure a suitable behaviour according to the purpose of which the structure was conceived. It is mandatory that a structure provides not only stability and resistance, but also durability and a proper structural performance in all of its lifetime.

Slabs have large dimensions in blueprints representing a determinant factor for the total cost of the project. Directly by consuming significant quantities of materials, and also indirectly, because its load affects the dimensions and materials of the remaining structural elements. Slabs are the structural element which most affects the allocation of resources to be used in the project and are the main focus of this study.

The main goal of this project consists in the development of a pre-stress solution for a roof slab of a public space and the design of the structure by applying regulations of eurocodes. A pre-design analysis of three solutions for the roof slab was developed to obtain the more appropriate solution. The criteria adopted for the choice of the solution were based on the comparison of the structural behaviour and economic reasons.

The solution chosen was the slab with beams grid because it uses a slab with a considerable smaller thickness when compared with the other case studies. In order to obtain a suitable performance of the structure towards seismic activity, an analysis was developed by using an automatic calculus software, SAP2000®. This software provides a static and dynamic structure's behaviour study.

Key-words: Project, structure, design, seismic analysis, Eurocodes, pre-stress.

1. Introduction

The project and design of a structure have as a main goal to ensure a proper behaviour according to the purpose of which the structure was conceived. It is mandatory to guarantee that the structure provides not only stability and resistance, but also durability and an appropriate structural behaviour during its lifetime.

Slabs have large dimensions in blueprints, representing a determinant factor for the total cost of the project. Directly by consuming significant material quantities, but also indirectly, because its load affects the dimensions and material quantities of the remaining structural elements. Slabs are the structural element which most affects the allocation of resources to be used in the project and are the main focus of this study.

The main goal of this project consists in the development of a pre-stress solution for a roof slab of a public space and the design of the structure by applying regulations of eurocodes.

First of all a pre-design analysis of three solutions for the project of the slab roof was performed. Afterwards it was decided which solution would fit better to the slab being studied by comparing the structural behaviour and taking in consideration material quantities and their economic costs.

The solution chosen was the slab with beams grid because it uses a slab with a considerable smaller thickness when compared with the other case studies.

Finally, in order to obtain a suitable performance of the structure towards seismic activity, an analysis was developed by using an automatic calculus software, SAP2000®. This software provides a static and dynamic structure's behaviour study.

2. Descriptive document

The roof slab under analysis in this project is located in Faro and could have lots of utilities. There is also an area that could be used as a bar or exhibitions space and other helping services.

In this stage it is necessary to understand that the architecture blueprints are the major factor that influences the position of columns and beams and its dimensions. Considering the spams between columns, of 20 and 40 meters, it is easy to understand that this project will need a pre-stress solution. This assumption assures some advantages like controlling the deformation and cracking of the concrete sections.

The foundation soil considered is a deep deposit of compacted sand and classified by EC8 [4] as a type C soil, considering a rupture tension of 400 kPa.

In order to guarantee correct levels of resistance and durability of the project it is necessary to choose the proper materials. In the following tables it is possible to understand the chosen materials and their characteristics for the concrete, steel and pre-stress [8] respectively.

Table 1 – Resistance charateristics of the concrete used in the project.

Concrete C30/37	f_{ck} [MPa]	f_{cd} [MPa]	f_{ctm} [MPa]	$E_{c,28}$ [GPa]
	30	20	2,9	33

Table 2 - Mechanical charateristics of steel used in the project

A500 NR SD Steel	f_{yk} [MPa]	f_{yd} [MPa]	ε_{yd} [x10^{-3}]	E_s [GPa]
	500	435	2,175	200

Table 3 - Mechanical charateristics of pre-stress used in the the project

Y 1860 (0,6''S)	f_{pk} [MPa]	$f_{p0,1k}$ [MPa]	f_{pyd} [MPa]	E_p [GPa]
	1860	1670	1452	195+/- 10

Consulting the EC1 [2], it is possible to obtain the corresponding dead loads and live loads used in this project. For seismic action it was applied the standards prescribed in EC8 [4].

In order to perform the design of a structure, the EC0 [1] defines the loads combinations and their respective coefficients to use. These combinations have as a main goal to consider all possible scenarios for loads to be acting simultaneous. There are two types of combinations corresponding to ultimate limit state and serviceability limit state.

The elastic response spectrum was defined here, showed in the figure 1, attending that the project is located in Faro, it can be used the zone 3 according to EC8 [4], The characteristics for definition of the seismic action are showed in the table 4.

Table 4 – Charateristics for the definition of seismic action

	Sismo 1	Sismo 2
Zona	1.2	2.2
Tipo de Solo	C	
$a_{g,r}$ (m/s^2)	2	2
γ_I	1	
a_g (m/s^2)	2	2
$S_{máx}$	1,6	1,6
S	1,4	1,4
T_B (s)	0,1	0,1
T_C (s)	0,6	0,25
T_D (s)	2,0	2,0

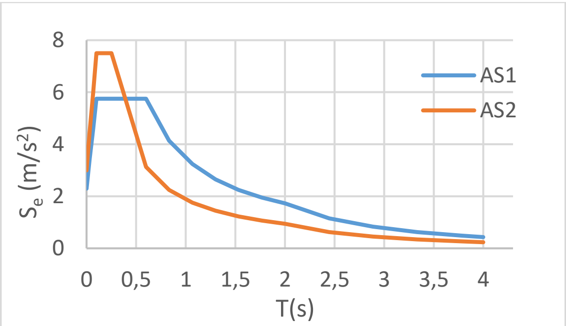

Figure 1 - Elastic response spectrum for zone 3

3. Study of the solutions for the project of the roof slab

In this chapter is discussed the pre-design analysis of the three solutions studied. Regarding that the self-height of the slab influences directly all the remaining structural elements, it is the first element to pre-design. The calculation of pre-design of the beams are also included in this chapter.

Attending that columns and foundations are normally constrained by seismic loads, the pre-design for these elements is made after the modulation of the structure in SAP2000.

The three solutions analysed were: waffle slab by FERCA, voided slab with loss moulds Cobiax and ribbon slab with beams grid.

The first solution studied was waffle slab by FERCA. This solution is conceived by using reusable formwork composed by plastic moulds called FG900 [8]. The main advantages of this type of solutions are the ability to reduce the resources used in the construction and to reuse the equipment that builds the slab. Through the minimization of concrete, by using this moulds, a T section is created allowing the slab to be calculated in the same way as a beam.

The second solution studied was a voided slab with loss moulds Cobaix. This solution is based on an economic and environment rationalisation, and enables the construction of a voided slab with the same behaviour as a bulk slab. Comparing this system with a bulk slab it is concluded that Cobiax [10] allows the reductions of the self-height in 30% and the inertia of the slab between 8 and 11%. These gains are shown in a low deformation of the slab, less reinforcing steel needed and low retraction effects.

The last solution is composed by a ribbon slab with beams grid trough the smaller span. The first step made was defining the distance between the beams grid. It was used for the grid distance 4 meters allowing the consideration of reinforce concrete slab instead of a pre-stress slab.

Slabs thickness were pre-design based on appropriate slenderness for each case. Considering the spans of the slab it is possible to obtain the thickness for this element and the exact loads on the slabs. The last pre-design step consisted on estimating bending moments for the service limit state, in order to obtain the minimum pre-stress force to guarantee the decompression of the concrete. The same steps were made to the beams.

Afterwards a comparison for the three case studies was performed, in order to choose the one presenting the best conditions for the project of the roof slab. This comparison was made by choosing the solution that uses the less resources as possible, meanwhile guaranteeing a suitable behaviour of the structure. The chosen solution was the ribbon slab with beams grid considering the quantities presented in table 5.

Table 4 - Comparison of choosing solutions in terms of concrete and number of pre-stress strands

	Beams Grid	Ferca	Cobiax
Concrete (m3)	526,9	667,1	700,3
Number of pre-stress strands	410	602	828

4. Analysis for vertical loads

After the pre-design of the beams and slabs the structure modulation was followed. Without the pre-design for the dimension of columns, the dimensions provided by architecture were used.

After the insertion of the structure in SAP, it was made a verification of the model by comparing the sum of the vertical loads in the model and the same sum for the pre-design loads. Regarding that the difference between those sums was like 1% the model was accepted.

Then it was made the deformation control of the slabs and beams. It is possible to see the elastic deformation of these elements in figure 2.

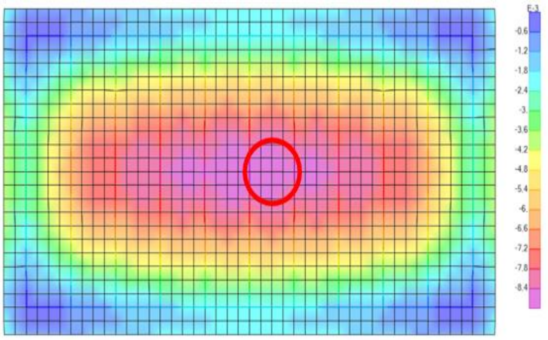

Figure 2 – Elastic deformation on the roof slab

The safety check was performed at the deformation limit for both the beams and the slab. The global coefficients method [6] was used to determine the long-term deformation of the slab. For this element, this check was made at the maximum arrow point, indicated in the figure 2.

Table 5 - The global coefficients method used in roof slab

δel (cqp+PE) [mm]	Kt	δlong (cqp+PE) [mm]	L/250 [mm]
0,3	2,88	0,8	16

The next table presents for each beam, considered as conditioning, the elastic deformation and the long term deformation estimated.

Table 6 –Elastic and long term deformation for beams, and the maximum admissible deformation.

					δmáx admissable	
	δe (cqp)	φ=2,5	δlong term	L	L/1000	L/500
	[mm]	(1+ φ)	[mm]	[m]	[mm]	[mm]
Viga 1	2	3,5	7	40	40	80
Viga A	2,8		9,8	20	20	40
Viga F	6,6		23,1	20	20	40

The pre-stress are pre-design applying the same calculations used for the slabs. In figures 3 and 4 is shown the two levels of cables disposition used in the beam 1 and 2. In figure 5 the layout of the cables used in the beams A and K, and in figure 6 the cable layout used in the beams of the grid.

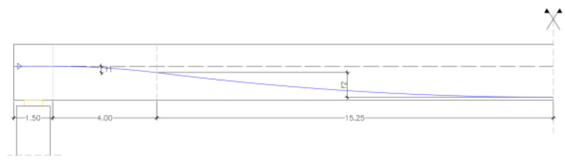

Figure 3 – Pre-stress layouts for the upper level of anchorage in beams 1 and 2

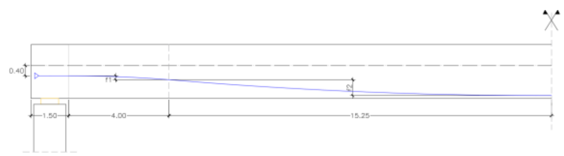

Figure 4 - Figure 5 – Pre-stress layouts for the bottom level of anchorage in beams 1 and 2

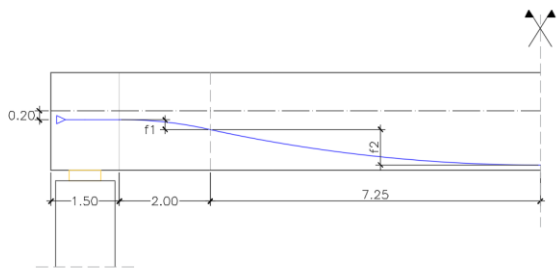

Figure 5 - Pre-stress layouts for beams A and K

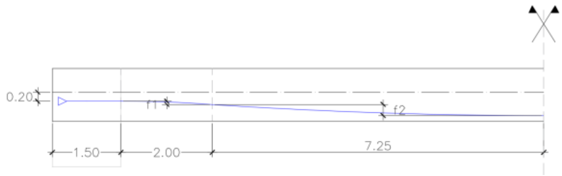

Figure 6 – Pre-stress layouts for beams B to J

5. Seismic analysis

In order to understand in which cases the structure was most effected by seismic activity, a modal analysis was performed.

Afterwards the behaviour coefficient was defined. In order to do so, it was necessary to classify the structure regarding its regularity in blueprint and in height. Given the geometry conditions of the roof and for only existing one floor, the structure can be classified as regular in blueprint and in height.

Considering that the structure is classified, structurally, as a portico system and is design as DCM class, the basic value of the behaviour coefficient (q_0) is 3,3. This coefficient can be calculated by the expression 1.

$$q = q_0 \times k_w = 3{,}3 \qquad (1)$$

It is important to point out that a behaviour coefficient of 3 was considered. This assumption does not represent significant changes in the seismic definition, because the difference is not substantial and by decreasing this value it is only considered that the structure sustains less damages.

The seismic coefficient was obtained by following expression 2.

$$\beta = \frac{F_b}{W} \qquad (2)$$

Table 7 – Seismic coeficient obtain for each type of seismic action.

SA1		SA2	
βx	βy	βx	βy
0,25	0,25	0,11	0,10

As shown is table 7, the value of the seismic coefficient is higher for both directions for the seismic action type 1. Therefore, only the seismic action type 1 was considered for further analysis.

Then it was verified the need of considering the P-Δ effects [9] of the structure, by considering a beam-column continuity. These effects are not necessary to verify, because the value of sensibility coefficient for the displacements between floors is less than 10%.

However the design stress, regarding seismic activity, for the columns and foundations are extremely high, leading to an excessively reinforcement needed for columns and with difficult detailing, and also to higher dimensions for the foundations. Consequently, this possibility was excluded and the possibility of inserting an elastomeric isolator was analysed.

The elastomeric isolator [11] has a main goal to accommodate the displacements that arise in the structure upon seismic activity and to increase the damping on the structure.

Therefore, it was necessary to correct the response spectrum in order to consider this increase of damping, through the expression 3.

$$\eta = \sqrt{\frac{10}{5 + \xi}} \geq 0{,}55 \qquad (3)$$

A second modal analysis was performed considering the elastomeric isolator between the main mass of the structure (the roof) and columns.

A considerable increase of the vibration periods was verified, which can translate in lower seismic accelerations in the structure.

For this model the P-Δ effects were considered, according to the increase of displacements by the utilization of the elastomeric isolator.

It was verified that even though these displacements increase, when compared with the ones from the structure with beam-column continuity, the stress for this model with the isolation decreased significantly. Allowing the decrease of the dimension of the columns, comparing with the dimensions suggested by the architecture, to one column section less armed and yet a reduction of the dimensions of the foundations.

6. Design

Concluding all the safety verifications presented previously, the design of the structural elements that compose the chosen solution for the roof slab was developed. The procedures adopted in order to obtain the reinforcement steel needed for a proper structural behaviour are described in the present chapter.

It was necessary to define which of the structural elements were conditioning, meaning that these elements presented higher stress. The elements selected for this purpose can be observed in figure 7.

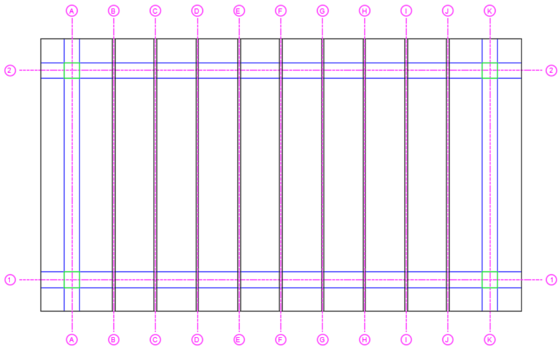

Figure 3 – Grid considered for the roof slab.

- Roof slab
- Beam 1
- Beam A
- Beam F
- Column A-1
- Foundation

Roof slab

In order to perform the design of the slab it was verified the safety regarding the ultimate limit state of bending, as a mean to obtain the necessary reinforcement for this element. After the verification of the design stress, it is concluded the need to have superior reinforcement with #Φ10//0,10, corresponding to 7,85 cm^2 and a inferior reinforcement steel of #Φ8//0,10 (5,03cm^2).

Beams

The design of these horizontal elements was affected by the serviceability limit states, regarding the use of pre-stress beams that was expected. It is important to point out that it was considered a strength by pre-stress string of 160 kN, representing approximately 20% of losses.

Regarding the ultimate limit state, it has already been mentioned that it is not a constraint, but the values presented by the reduced bending moment (μ) of these structural elements were evaluated. As expected these values were relatively low.

However, a bending safety check of the beams was carried out in order to obtain the quantities of ordinary reinforcement needed to verify the safety in relation to this limit state.

In relation to the transverse reinforcement [6], this was calculated through the expression 4, considering a θ = 2,0.

$$\left(\frac{A_{sw}}{s}\right) = \frac{V_{sd}(z.cotg\theta)}{z.cotg\theta.f_{cd}} \quad (4)$$

Columns

Considering that the design of these elements is being done in elastic regime, by using a behavior factor equals to one, the minimum and maximum amounts of armatures were obtained considering EC2 [3].

After that it was analysed the design stress presented in these elements, considering the seismic action as constraint. The resistant stress were also verified, taking into account the reinforcement used in the pillars, according to the expression 5.

$$\frac{M_{sd,x}}{M_{rd,x}} + \frac{M_{sd,y}}{M_{rd,y}} < 1{,}0 \quad (5)$$

Regarding shear reinforcement, it was calculated according to the expression 4, but using an angle ϴ equal to 45 °. For the nodes beam-column and beam-foundation the spacing was reduced to half in order to prevent rupture, because the concentration of stress in these nodes.

Foundation

The foundations were design, according to the model of strut-and-tie presente [5] in figure 4, and as a rigid element. It is important to point out that it was considered a soil rupture tension of 400 kPa.

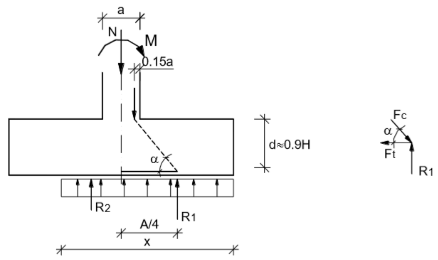

Figure 4 – Model of strut-and-tie considered in the foundations

7. Conclusions

The main goal of this project consisted in the development of a structural solution for a roof slab. Considering the architectural constraints, it implied the usage of a rectangular slab with 20 and 40 meters of spams. Therefore the solution could only be design by using pre-stress. Assuming a pre-stress solution, it was possible to acknowledge countless options not only regarding the drawing of tendons, but also the type of pre-stress to consider.

Since the beginning it was understood that the design of the horizontal elements, beams and slab, would be restricted by its behavior in service because of the spans considered. However as pre-stress is characterized by imposing to the structure a deformation in the opposite direction of gravity, it becomes more accessible the control of the deformation.

It was relevant to evaluate the node beam-column, being a sensitive zone because of the high concentration of stress and the level of displacements showed on the top of the columns for the seismic activity. By using elastomeric isolators, it was possible to verify that these problems were minimized, decreasing the stress and consequently the dimensions of columns and foundations in blueprints.

For future work considerations, in order to improve the project developed the usage of columns with a hollow section could be considered. This consideration could improve a better seismic behavior by decreasing the stiffness of these elements. Therefore it would imply an increase of the fundamental period of the structure, resulting in a minimization of seismic accelerations. However this consideration must take into account the limitation of the reduced axial effort to 0,4 and also the non-increasing P-Δ effects.

It is possible to affirm that the objectives of this study were achieved. The structural project developed was compatible with the architect proposed and ensured the safety and functional requirements and also considered a minimization of resources.

8. References

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