Project of a residential building in seismic zone

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1. INTRODUCTION

This thesis aims to design, develop and analyze the construction of a residential building in seismic zone. For the purpose, was performed a preliminary design of the structure , created a model by three-dimensional modeling program (SAP2000) and the analysis of the results. Subsequently, we performed dimensioning and proceeded to security verification necessary for a good constructive solution.

The preparation of a structure project is established on rules and decisions that determine their final quality, therefore, regulations were created for the project elaboration applied in member states of the European Committee for Standardization (CEN). The main regulations for the project of structures are: EC0, EC1, EC2 and EC8.

2. STRUCTURAL CONCEPTION

The concrete used in this project was class XC3 C30/37 because the moderate humidity existent in Azores, place where's located the building, with 35mm of nominal cover. The stell chosen for this project was A500 NR SD.

Were determinated the values of the acions that occur during the design working life, both permanent and variable actions. Were defined the combinations of actions recommended in EC0 such as quasi-permanent combination, fundamental combination, seismic design combination and rare combination.

None of the plans provided have a discretization of columns, beams, walls / cores, or foundations, so, it was necessary to define the most favorable location of these elements in order to ensure strength and ductility of the structure to the various actions.

As a structural solution, given the structural modeling, we chose a massive flat slab, supported on columns with a border beam.

It was taken special attention in the distribution of columns to avoid large spans of beams and slab.

On the stairs and elevators were provided cores as well as four walls at each end of the central zone of the building to prevent it from torsion.

Regarding to foundations, the weak resistance capacity of the ground, advise chosing indirect foundations through piles.

3. Pre-Design

For the pre-design of the flat slab we selected, in the structural plan, the longest span to defeat (6,5m).and applied the rule of pre-design given by the expression: $e_{slab} = \frac{L_{max}}{25\ a\ 30}$, so, $0{,}22 \leq e_{slab} \leq 0{,}26$. We adopted a thickness of 0.25m, in order to control the deformation and the punching shear. For the calculation of the balconies slab we used the same criterion followed previously: $e_{slab} = \frac{L_{max}}{10}$, so, $0{,}2 \leq e_{slab}$. The same thicness, 0,25m, was opted for this slab.

For the beams we opted a width of 0.20m, in order to comply with the architecture of the project. The beams of reinforced concrete shall comply with the values of slenderness, in accordance with the following expression: $h_{beam} = \frac{L_{max}}{12}$, with $L_{max} \approx 6m \ \rightarrow \ h_{beam} = 0{,}5m$. It was considered the value of 0.45m.

In pre-design of the columns were considered the combination quasi-permanent and the combination fundamental. Instead of the classical technique of the areas of influence, we chose to model the slab in a finite element slab plane, where the vertical supports, columns and walls were simulated by a simple model supports. In order to facilitate the constructive process was opted that columns have constant section throughout their development. Based on the reaction of the model that calculated the axial force at the base, and with this, limiting the normal effort reduced to 85% for the combination key and 60% in combination almost permanent, we chose to divide all the pillars in five families listed in the table at the end of the chapter.

The method used for pre-design of walls and cores is similar to columns.

As regards to the foundations, because of the low resistent capacity of the ground, was chosen piles.

In the definition of axial load resistant characteristic of a pile, it is assumed that the "steady", which was assumed to be a depth such that the pile length is 15m, ensures a tip resistance of 4.0 MPa.

We opted for piles with 600mm diameter.

The capacity of resistance of the cuttings was performed through $R_{Ed} = f_{cd,e} \times A_{estaca} = 1131\ KN$ and the for the calculation of the transversal and rotation rigidity of a pile was used the expressions: $K_{trans} = \frac{3EI}{L_e^3} = 16328{,}5\ KN/m$; $K_{rot} = \frac{3EI}{L_e^3} = 48985{,}5\ KN/m$ with modulus of elasticity of the pile 23 MPa.

		Dimensions (m)
Beams	V1-V15	0,2 X 0,45
Columns	P1	0,2 X 0,45
	P2	0,2 X 0,6
	P3	0,2 X 0,75
	P4	0,25 X 0,9
	P5	0,2 X 0,2
Walls/Cores	PA1	4,1 X 0,2
	PA2	4,1 X 0,2
	NES1	4,9 X 0,2
	NES2	3,3 X 0,2
	NES3	6,5 X 0,2
	NEL1	4,9 X 0,2
	NEL2	5,5 X 0,2

Tabel 1 –Summary of dimensions of elements.

4. Three-dimensional modeling of the building

Materials and their respective characteristics were defined. It was considered the siccative elasticity module, with half of its value, by the fact that the structure can crack during the occurrence of an earthquake, as recommended by EC8. In the modeling of the structure is used in a three-dimensional mesh structural where elements are represented. In this project, due to the geometry of the structure, was created a mesh through the intersection of points that represent the location of the columns, walls and cores. From the import of AutoCAD slab floor type - finite element shell (Shell) - for the program SAP2000, it was determined that the location of these points.

The beams and columns were considered bar elements (Frame), which corresponds to finite elements of 2 nodes. The elements were oriented according to the directions corresponding to the project.

In beams was not released the torsional rigidity due to the low value of these efforts.

The slabs and the cores were simulated as finite element shell (Shell) of 3 or 4 nodes. In these elements to torsional rigidity was despised, and so, efforts will be balanced only by bending. This option leads to a slab design on the side of safety.

In the three-dimensional model were used support, restricting the displacement and rotation of z axis, where we have applied the rigidities of translation and rotation calculated in the pre-design of the piles.

In the calculation of stresses on the piles, these were simulated as bar elements, restricting the vertical displacement, having been applied an impediment to translational displacements 5Ø top (reversal point displacement).

5. SEISMIC ANALYSIS

For seismic analysis is mandatory to take into account several factos like zone territory, soil type and class of Importance. As previously mentioned the building is located in the Azores, whence by Annex National of EC8, it is concluded that the earthquake is of type 2.1. For this type of earthquake the maximum accelerations at ground level is $a_{gr} = 2{,}5 m/s^2$. The foundations ground has weak capacity of resistance being a ground type D. The building is intended for the dwelling so it's classified as importance class II which advises the use of a importance factor: $\gamma_1 = 1{,}0$.

For a proper study of the building it must be analyzed and evaluated the number of vibration modes with a total value from mass participation in the order of 90% and acquires higher mode with a percentage higher than 5% in each direction. This value of mass participation was reached on the 5th vibration mode. Were determined values for the period and frequency.

MODES	Period (T) Seg.	Frequency (f) Hz	UX %	UY %	RZ %
1	1,67	0,60	0,1	0,0	94,2
2	1,39	0,72	90,2	0,3	94,3
3	1,18	0,85	90,5	96,4	94,3
4	0,50	2,00	90,6	96,4	99,0
5	0,43	2,34	99,0	96,4	99,1

Tabel 2 –Summary of. mass participation, period and frequency.

The EC8 determines that the concrete buildings are classified according to different types of structural systems in each direction, except for the torsionally flexible systems, therefore, it is necessary to determine the regularity in plan and elevation.

For regularity in plan is needed to calculate the slenderness of the building plant, it must be less than 4, and shall be in accordance with the two conditions: $r_i \geq l_s$; $e_{0i} \leq 0{,}3 \times r_i$ with : r_i (torsional radius); l_s (radius of gyration); e_{0i} (distance between the centre of stiffness and the centre of mass). This conditions must be fulfill for both directions. If does not fulfill the first condition, the structure is considered torsionally flexible.

Floor	e_{0x} (m)	e_{0y} (m)	r_x	r_y	l_s
1	-0,47	0,30	13,4	13,9	19,2
2	0,12	0,36	13,1	14,5	18,3
3	0,12	0,37	12,9	15,0	18,3
4	0,12	0,37	12,7	15,3	18,3
5	0,11	0,37	12,5	15,6	18,3
6	0,11	0,37	12,4	15,8	18,3
7	0,11	0,37	12,3	16,0	18,3
8	0,10	0,37	12,2	16,2	18,3

Tabel 3 – Values of eccentricity, torsional radius and radius of gyration.

It were analyzed the criteria for the determination of regularity in elevation. The building has a small indentation at the level of the 1ST floor but comply with the requirements of EC8, so it is classified as regular height.

It was considered that the structure belongs to medium ductility class.

The behavior factor was calculated for torsionally flexible buildings, therefore the basic behavior factor is $q_0 = 2$. The factor reflecting the prevailing failure mode in structural systems with walls (k_w) is 1,0 whence the behaviour factor is $q = q_0 \times k_w = 2{,}0$.

The response spectrum is a graphical interpretation of the maximum response value of a set of oscillators of a degree of freedom, when incited by a seismic action. The values of the response spectrum depends on the period or frequency and the behaviour factor considered.

EC8 provides the method to determine the spectrum calculated by an elastic analysis for the horizontal components of seismic action, relating the acceleration (S_d) with the period (T). In accordance with the National Annex of EC8 (17), presents the following values.

Ground type	$S_{máx}$	T_B (s)	T_C (s)	T_D (s)
D	2,0	0,1	0,3	2,0

Tabel 4 – Values of S_{max}, T_B, T_C e T_D for type 2 earthquake and ground type D

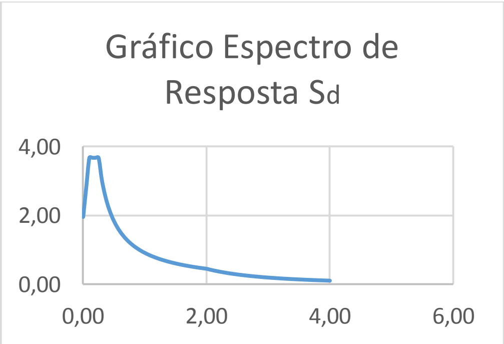

Figure 1- Response spectrum graphic.

According to the EC8 to calculate the torsional acidental effects it was determinated the accidental eccentricity and the on horizontal seismic forces of each floor. The horizontal seismic forces were calculated using seismic base shear, the displacements of the masses in vibration mode and the masses of the floors.

Given the seismic action, excessive relative displacements between floors may appear. This may cause an eccentricity of load on the vertical members, because of the axial forces.
The effects of 2nd order are considered in EC8 and take into account the interstorey drift sensitivity coefficient (θ). This value can be calculated by the expression: $\theta = \frac{P_{tot}.d_r}{V_{tot}.h} \leq 0{,}1$

It´s also necessary to control damage limitation, that means restriction of relative displacements between floors, according to the type of non-structural elements existing in the structure.

In this project the building have non-structural elements of brittle materials attached to the structure

, and according to EC8) the limit must be $d_r.\nu \leq 0{,}005h$

Floor	θx [rad]	θy [rad]	0,005h [m]	$d_{r.x}$ * v	$d_{r.y}$ * v
1	0,08	0,07	0,028	0,016	0,013
2	0,03	-0,01	0,015	0,004	-0,001
3	0,03	0,03	0,015	0,004	0,004
4	0,03	0,02	0,015	0,004	0,002
5	0,03	0,02	0,015	0,004	0,002
6	0,03	0,01	0,015	0,004	0,002
7	0,02	0,01	0,015	0,004	0,002
8	0,02	0,01	0,015	0,004	0,002

Tabel 5 – Values of interstorey drift sensitivity and limitation of interstorey drift.

The value of θ is always less than 0.1, which is despised the effects of 2º order in the design of the structure. The limitadion damage have been satisfied.

6. DESIGN

For the design and safety checks of slabs was considered a width of the unit pulled zone b_w = 1.0, so the distribution of reinforcements be made per meter.

The calculations performed for the Ultimate Limited States (ULS), was considered the fundamental combination cause it’s more limiting than the combination associated with seismic action. For the sizing of the slab is necessary to check the simple bending and the punching.

For a correct design, was calculated the minimum and maximum values of cross sectional area of reinforcement

For the calculation of longitudinal reinforcement bars were considered the values of bending constraints. We determinated the cross sectional area of reinforcement and defined the reinforcement bars necessary to resist the bending. From the result there is the need to put a mesh in the whole dimension of slab that will be reinforced with other reinforcement bars in areas where there are high bending.

In support areas of slab in columns it is important to check the need to strengthen the reinforcement bars in order to prevent the punching. It was calculated the maximum shear stress and the design value of the punching shear resistance of a slab without punching shear reinforcement along the control section considered. After comparing the results there was no need to punching shear reinforcement.

It was made a deflection control and a crack control. The deflection control was calculated using global coeficientes method and the crack control was determinated using control of cracking without direct calculation. In booth cases complies with the requirements.

For the beams only been studied the resistance to bending and shearing because the torsion is reduced. The efforts of calculation in the design of beams was taken into consideration a evelope the efforts of key combination and combination seismic, with the purpose to obtain the efforts for the situations more constraints. In the calculation of the efforts it was considered to be a simple bending and it was necessary to determinate the minimum and maximum values of cross sectional area of reinforcement.

The beam / column connection is a critical area, so that, according to EC8 it is necessary to consider an critical extension (l_{cr}) with the same height dimension of the beam (h_w).

The Anchorage of longitudinal reinforcement is a matter of great importance, which is why the EC8 provides expressions that limit the longitudinal diameters of the beams. This longitudinal diameters were calculated and it was determinated the cross sectional area of reinforcement and defined the reinforcement bars necessary to resist the bending.

For shear is also necessary to calculate the minimum value of cross sectional area of reinforcement.

To be in compliance with the rules set forth in EC2, it is necessary to take into account the longitudinal spacing of shear reinforcement and the transverse spacing between branches as steps addressed in article 9.2.2 (6) and 9.2 .2 (8) of the same EC [15

To be in compliance with the rules laid down in EC2 (15), it is necessary to take into account the spacing between longitudinal reinforcement of shear force and the transverse spacing oh the legs. Both of those spacing were calculated.

7. CONCLUSION

A project in seismic areas requires studies and particular information, such as location, characteristics of the ground and type of earthquake, class of importance of the building, among others, and special attention in conception and design of the structural elements.

In conception of structural solution, pre-design and design of the structure, it is necessary to comply with certain rules, for the results be the most realistic possible, i.e. how the structure will respond to requests from efforts..

We followed the principles of EC2 and EC8 to the design of the structure, with particular care in control of ductility of elements. For an appropriate structural performance it is essential that the structure has good capacity resistant and a good ductility, in order to be able to suffer minor displacements without loss of resistance. However, an excess of ductility is harmful, why were fully complied with the rules in EC8.

The calculation of the design of the structure was a lengthy process and insightful, in order to obtain an appropriate result.

Special attention has been given to armour of confinement of the columns, walls and cores, since these are the main responsibles for the resistance of the structure to seismic actions.

After the verifications and calculations required to design the structure of the building, the values, without exception, meet the required safety levels by EC's.

8. Bibliography

APPLETON, Júlio - **Estruturas de Betão Vol. 1**. 1ª Ed. Lisboa: Orion, Julho de 2013.

APPLETON, Júlio - **Estruturas de Betão Vol. 2**. 1ª Ed. Lisboa: Orion, Julho de 2013.

APPLETON, Júlio; MARCHÃO, Carla - **Folhas de Apoio às Aulas de Estruturas de Betão I: Módulo 2 – Verificação da Segurança ao Estados Limites Últimos de Elementos Com Esforço Axial Desprezável.** Instituto Superior Técnico, 2009.

APPLETON, Júlio; MARCHÃO, Carla - **Folhas de Apoio às Aulas de Estruturas de Betão I: Módulo 3 – Verificação do Comportamento em Serviço (Estados Limites de Utilização – SLS)**. Instituto Superior Técnico, 2009.

APPLETON, Júlio; MARCHÃO, Carla - **Folhas de Apoio às Aulas de Estruturas de Betão I: Módulo 5 – Verificação da Segurança ao Estados Limites Últimos de Elementos Com Esforço Axial Não Desprezável**. Instituto Superior Técnico, 2009.

APPLETON, Júlio; MARCHÃO, Carla - **Folhas de Apoio às Aulas de Estruturas de Betão II: Módulo 2 – Lajes de Betão Armado**. Instituto Superior Técnico, 2012.

APPLETON, Júlio; MARCHÃO, Carla - **Folhas de Apoio às Aulas de Estruturas de Betão II: Módulo 3 – Fundações de Edifícios**. Instituto Superior Técnico, 2012.

NP EN 1990 – **Eurocódigo – Bases Para o Projeto de Estruturas.** 2009

NP EN 1991-1-1 – **Eurocódigo 1 – Ações em Estruturas – Parte 1-1: Ações Gerais.** 2009

NP EN 1992-1-1 – **Eurocódigo 2 – Projeto de Estruturas de Betão – Parte 1-1: Regras Gerais e Regras para Edifícios.** 2010

NP EN 1997-1 – **Eurocódigo 7 – Projeto Geotécnico – Parte 1: Regras Gerais.** 2010

NP EN 1998-1 – **Eurocódigo 8 – Projeto de Estruturas para Resistência aos Sismos – Parte 1: Regras Gerais, Ações Sísmicas e Regras para Edifícios.** 2010