Structural modeling spans of 10m versus spans of 7.5 / 8.0m in an office building in Lisbon

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Resume

The aim of this work is the elaboration of a structural project of an office building in Lisbon considering the change of the spans from 7.5m to 10m based on the available architectural elements. Taking into account the modification of the spans, the elaboration of all phases of the structural design was performed, from the early design stage to the final one, following the current European regulations (Eurocodes).

The building has three underground floors, a ground floor and five raised floors. The above floors have an open-space appearance used as office and the basements will function exclusively as parking zones for vehicles.

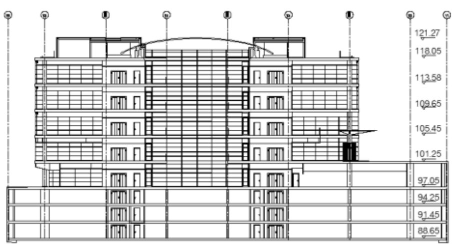

Figure 1 - Section of the building without the span modification.

The development of the new structural solution adapted to the new requirements, opted to obey the position of the vertical elements defined in the architectural design, only with small changes.

One of the main principles of structural behavior is the uniformity which ensures an adequate response to a seismic action. The uniformity guaranteed with a regular distribution of stiffness in plant. The existence of an elevated retaining wall on de 1st floor stiffness introduced an uneven distribution of stiffness which leads to an unpredictable torsional structural behavior. To obtain a more regular stiffness distribution it was assumed that the retaining wall is not structurally connect to the 1st floor slab, and were set up columns (shown in green in figure 2) which receives the vertical loads from the 1st floor slab. There was also the need to add four columns, shown in red in the next figure, in the underground floors once there was a span length of 17m.

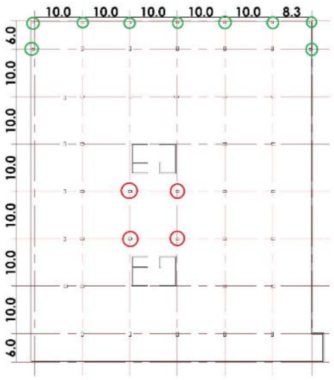

Figure 2 - Alterations made shown in the parking floor with 10m spans.

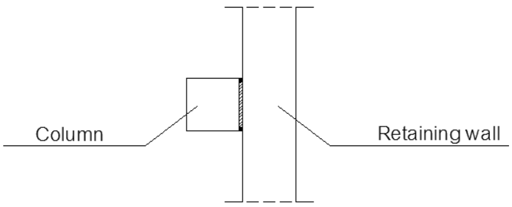

Figure 3 – Detail of the connection between the retaining walls and the introduced columns shown in green in Figure 2.

As an office building is considered to have a design lifespan of 50 years, corresponding to a structural class S4 according to the prescriptions of the EN1990. In terms of seismic action behavior the structure is classified as category II, ordinary building. The nominal cover is dependent on the exposure classes. The inner elements are considered as belonging to the class of exposure XC1, assuming that are in an environment with low air humidity and the elements in direct contact with the ground are classified as XC2. The cover for the retaining walls and foundations was considered of 50mm, for the slabs 30mm, for the columns 35mm and for the walls 45mm.

The choice of the structural materials was influenced by the strong seismic action on vertical elements. The resistance class of the concrete, for the main elements of the structure, was a C30/37 and the steel used for reinforcement was a high ductility steel A500NR SD. The self-height associated to the C30/37 concrete was 25kN/m^3.

The other permanent loads take into account nonstructural materials, which are required for the structure use are presented in the table 1.

Table 1 - Loads associated to nonstructural elements.

Roof	2,0 kN/m^2
Ground Floor to 4th Floor (Office)	1,5 kN/m^2
Stairs	1,5 kN/m^2
Floors -3 a -1 (garage)	1,0 kN/m^2

The values of the variable actions are defined on Section 6 of EN1991 and are dependent on type of use of the floor. In the table 2 are identified the actions on the structure, as well the combination coefficients, defined on the Portuguese Annex in the EN1990.

Table 2 - Loads associated to variable actions.

Action	kN/m^2	ψ_0	ψ_1	ψ_2
Office areas	3,0	0,7	0,5	0,3
Traffic areas	2,5	0,7	0,7	0,6
Acessible roof	3,0	0,7	0,5	0,3
Stairs	3,0	0,7	0,5	0,3
Vehicle on the land adjacent to the retaining wall	9,0	0,4	0,4	0,0

The effect of the seismic action on structures is defined based on the EN1998-1 from the quantification of an elastic response spectrum representing the ground acceleration. The Portuguese Nacional territory is divided into seismic zones depending on local hazard. The building in this project is localized in Lisbon area, corresponding to seismic zone type 1.3 and 2.3. The definition of the ground allows to determine the influence of the local ground conditions on the seismic actions, and was defined as ground type B. The elastic response spectrum is affected by the coefficient factor (q) and is obtained the design spectrum in order to take into account the capacity of the structure to dissipate energy.

The pre-design of all structural elements was made with simple rules that allow to determine the optimal dimension of the cross-section of the elements, based on the internal forces and indirect control of deflection, in the case of slabs.

The slabs were pre-designed based on an adequate span/thickness ratio. The slab type selected was ribbed slab once that satisfies the span requirements. In order to determine if the level of internal forces is adequate was made an equivalent frame analysis considering the simplified apportionment of bending moment for a flat slab. With the obtained results it was considered a ribbed slab FERCA FG800 300-100. The areas in the slab above the columns were considered as massive reinforced concrete with 0,4m of thickness.

The pre-design of the columns was made by controlling the value of the normalized axial load in order to optimize the dimension of the cross-section, so that can be guaranteed an normalized axial load lower than 0,65 for the seismic action. The axial load in each column was obtained in a bi dimensional model of the floor modelled in SAP2000.

The structure was modelled in a three dimensional model in SAP2000 allowing to quantify the effects of the loads. The ribbed slabs were modeled as a flat slab with an equivalent inertia to the ribbed slab.

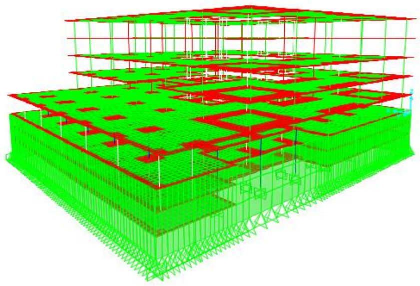

Figure 4 – Computacional model in SAP2000.

Due to concentration of stiffness in the center of the building and the inexistence rigid elements in the periphery is expectable that the building doesn't have the minimum torsional rigidity, and will show a torsional flexible behavior, which is responsible for introducing additional uneven forces, stresses and deformation in the peripheral structural elements. In order to avoid this type of behavior, a reduction of stiffness in the center of the building was done by progressively reducing the dimension of the structural walls, until the following equation present in the EN1998-1 was verified:

$$r_i \geq l_s$$

A spatial modal analysis was performed allowing to obtain dynamic characteristics of the structure.

Table 3 – Modal analysis of non-torsional model.

Mode	Period	Frequency	Translation along dir. x		Translation along dir. y		Rotation	
	(s)	(Hz)	Participation of mass (%)	Cumulative (%)	Participation of mass (%)	Cumulative (%)	Participation of mass (%)	Cumulative (%)
1	1,77	0,57	0	40	44	84	0	40
2	1,73	0,58	43	83	0	84	3	43
3	1,67	0,60	2	85	0	84	35	83

Based on the information of the modal analysis it was possible to achieve 95% of mass participation due to the mass of underground floors not being mobilized. In order to correct the values, the percentage of the mass of the underground floors, correspondent to approximately

40% of the total mass, is added to the obtained values of cumulative rate, allowing to achieve 95% of mass participation on the first 24 vibration modes.

Since the previous equation is verified proceeded to check the damage limitation imposed by the EN1998-1 on the column P28, located on the periphery of the building.

Table 4 - Verification of damage limitation on column P28.

Floor	d_{rx} (m)	d_{ry} (m)	d_{rx}.v (m)	d_{ry}.v (m)	h (m)	0,005h (m)	d_{rx}.v<0,005h	d_{ry}.v<0,005h
5	0,023	0,023	0,009	0,009	4,2	0,021	Verify	Verify
4	0,033	0,032	0,013	0,013	4,2	0,021	Verify	Verify
3	0,050	0,049	0,020	0,021	4,2	0,021	Verify	Verify
2	0,060	0,055	0,024	0,023	4,2	0,021	Fail	Fail
1	0,039	0,032	0,016	0,013	4,2	0,021	Verify	Verify

This model was not possible to validate, once it fails the damage verification of EN1998-1, and the consideration of flat slats frame as primary seismic elements is not a solution totally studied and fully covered by EN1998-1. Not being possible to guarantee the structural reliability, this option is set aside and the building will be designed considering the existence of walls in the center of the building as torsionally flexible with a behavior factor (q) of 2,0.

In the next tables the damage limitation verification by EN1998-1 and the principal modes of vibration are presented, being possible to verify that the first mode of vibration is rotation around the vertical axis.

Table 5 – Modal analysis of torsional model.

Mode	Period	Frequency	Translation along dir. x		Translation along dir. y		Rotation	
	(s)	(Hz)	Participation of mass (%)	Cumulative (%)	Participation of mass (%)	Cumulative (%)	Participation of mass (%)	Cumulative (%)
1	1,66	0,60	0	40	3	0,43	35	75
2	1,21	0,82	46	86	0	0,43	0	75
3	1,04	0,96	0	86	46	0,89	3	78

Table 6 - Verification of damage limitation on column P28.

Piso	d_{rx} (m)	d_{ry} (m)	$d_{rx}.v$ (m)	$d_{ry}.v$ (m)	h (m)	0,005h (m)	$d_{rx}.v<0,005h$	$d_{ry}.v<0,005h$
Terraço	0,022	0,017	0,009	0,007	4,2	0,021	Verify	Verify
4	0,026	0,019	0,010	0,008	4,2	0,021	Verify	Verify
3	0,028	0,020	0,011	0,008	4,2	0,021	Verify	Verify
2	0,028	0,020	0,011	0,008	4,2	0,021	Verify	Verify
1	0,027	0,022	0,011	0,009	4,2	0,021	Verify	Verify

Due to the high stiffness concentration in the center of the building the stresses introduced on the cores make impractical their design. So was assumed a 30% reduction in the stiffness of the elements that compose the cores according to the article 5.4.2.4(2) of EN1998-1, introduced in the computational model by reducing the width of each element.

Initially was evident that the limited number of pillars for the large development in plan of the building would influence the design of the core, verifying that largely resisted the seismic action. Due to this condition there were high demands of reinforcement, which made it necessary to increase the thickness of the walls to 0.4m.

The design of the walls began by performing the verification of the limitation of normalized axial load for seismic action imposed by EN1998-1 of 0,40.

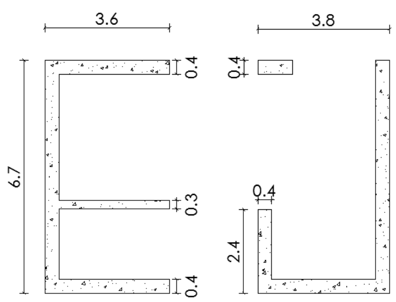

Figure 5 - Dimensions of the cores.

The calculation of longitudinal reinforcement is based on the method of fictitious columns which allows a simple way to perform design of walls by considering the reinforcement steel near the ends.

The internal forces considered for the design of the longitudinal and transversal reinforcement were determined by the EN1998-1 design envelope to ensure that plastic hinge develops only at the base. In order to ensure that the plastic hinge is formed at the base, on the ground floor, it was necessary to provide a superior ductility in this zone with the adoption of an adequate distribution of rebar stirrups.

The design of columns was divided into primary and secondary elements. The columns defined as primary elements are characterized by having the capacity to resist the seismic action. In contrast, the secondary elements are not designed to resist the seismic actions, but only to gravity loads and the displacements due to the seismic action. In the case of the building in study, all columns above the ground floor were defined as primary elements and all columns on the underground floors were classified as secondary elements. The design of the primary elements must follow EN1998-1 regulation allowing to ensure a design with ductile behavior. Since it is not necessary to endow the secondary elements with ductility, they were designed following the EN1992 1-1 regulation.

The determination of the longitudinal reinforcement was performed using the software xD-COSEC, which based on a defined distribution of reinforcement in the cross-section, the values of resistant capacity are provided and compared with design values associated to the seismic action allowing to check if the safety is verified. The longitudinal reinforcement must comply the EN1998-1 and EN1992 1-1 prescriptions related with the minimum reinforcement area and the disposition of the rebar.

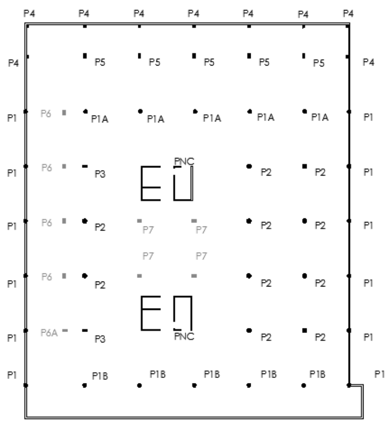

Figure 6 - Columns distribuition on the ground floor.

The rebar stirrups in the primary columns were determined using the maximum acting shear determined using the capacity design method corresponding to plastic hinge formation, defined by the EN1998-1. Due to the low shear forces verified in the underground floors for the seismic action and no ductility requirements, the rebar stirrup in the secondary columns were determined using the EN1992 1-1.

All the primary columns must have an adequate confinement defined by the EN1998 1-1 to guarantee enough ductility in order to provide capacity for deformation without rupture.

The slab was only designed for the roof floor due to the high dimension in plant of the building. Using the bending moments provided by the structural model in SAP2000 from the combination envelope of ultimate limits state it was possible to determine the design values. Considering the imposed minimum and maximum reinforcement areas by the EN1992 1-1 and the obtained design values it was possible to admit an adequate rebar placing.

The damage limitation verification was done using the global coefficients method, allowing to determine if the deflection of the slab verifies the limitation L/250 of the defined in the EN1992 1-1.

Due to the presence of vibration modes at the slabs in the sixth mode, it was considered the analysis of vertical response spectrum in order to obtain the forces associated to the participation of the mass in the slabs. It was possible to verify a very little increment of the stresses in the slabs however was considered negligible.

The punching shear is caused by concentrated forces and the transmission of bending moments in the slab. The reinforcement was determined according with EN1992 1-1.

The connection between the slab and column is the critical zone structures with flat slabs, as such is needed to ensure that when the structure is subjected to seismic action this zone has sufficient ductilit to allow deformation capacity without the occurrence of rupture. In order to ensure a more efficient confinement and allowing to increase the ductility in the connection zone was considered a minimum shear reinforcement of 3x3ESTØ10//15.

Considering the requirements of EN1998-1 to the structures resist seismic action without partial or total collapse was adopted a progressive collapse reinforcement placed in the lower face of the slab upon the pillars. This reinforcement allows to ensure the suspension of the slab, avoiding its collapse in case of failure by puncture.

The reinforcement in the base of the retaining walls was designed with the combination of bending and axial load, due to, the bending moment introduced by the eccentricity between the center of gravity of the retaining wall and reaction on the foundation caused by the tension on the soil or the micropile. In the remaining height of the retaining wall the design bending moments were caused by the horizontal pulses of the soil and the external bending moment applied at the base off the wall introduced by the eccentricity with the reaction in the foundation (see next figures).

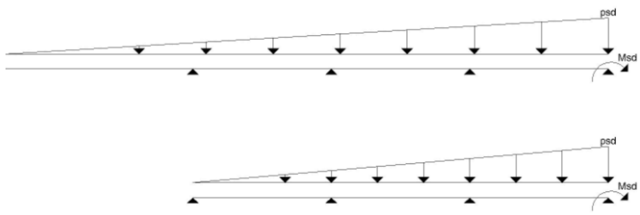

Figure 7 - Calculation method in the remaining height of the retaining walls.

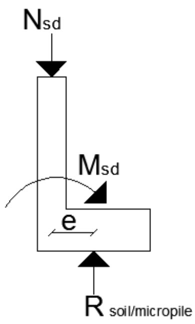

Figure 8 - Calculation method in the base of the retaining walls.

In the lateral retaining walls were verified higher values of axial load due to the existence of columns that directly discharge over the wall. The higher values of axial load forced the increment of the foundation size, in order to check the safety of the soil, resulting in a greater eccentricity among the center of gravity of the wall with the reaction of the soil. The design values obtained showed a very high magnitude, making impossible to guarantee a safety design of the wall. As a solution, micropiles were adopted, allowing to control the eccentricity between the center of gravity of the wall and the reaction, in this case, at the micropile.

It was possible to verify that the bending moments on the footings for seismic action were negligible due to the existence of basements. Thus, the design of the footings are performed assuming a model of concentrated load with the fundamental combination of actions following a strut and tie model.

Some of columns have a shared footing due to their proximity. The design of the footing rebar was determined considering the bending moment introduced by the eccentricity between the two columns and the center of gravity of the footing and the sum of the axial load of the two columns.

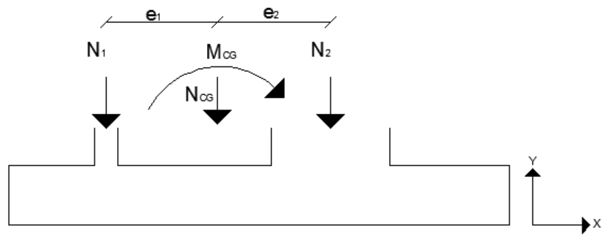

Figure 8 - Calculation method for footing with two columns.

The rebar on the foundation of the retaining walls is guaranteed by the continuity of the reinforcement determined at the base of the retaining walls.

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