ANALYSIS AND DESIGN FOR SEISMIC ACTION
APPLICATION TO A CASE STUDY

Author: Florentino Miguel Luz Coelho
Institution: Instituto Superior Técnico

Abstract

This article describes the design and particularities of the vertical structural elements of a specific offices building located in Lisbon and its behavior when affected by the seismic activity.
Throughout the whole design exercise, it is presented an analytical and explanatory look to the relevant parameters of the regulations that still require a special attention. Due to the fact that the regulation is recent there is a need to go deeper in it, in order to improve its comprehension and, at the same time, to make it more clear.
Aspects of the nature of the propagation of seismic action on the ground are evaluated as well as the reasons why Eurocode 8 considers half of the stiffness of the elements in the structural analysis.
Herein, the procedure to calculate the confinement reinforcement in composite wall sections (L-, T-, U-, I- or similar sections) is described and detailed. It is also presented the modeling of two cores, highlighting the differences and design procedures between them.

Keywords: Eurocode 8, Reinforced walls, confinement.

1. Introduction

The main goal of the dissertation is to analyze, design and detail the vertical structural elements of a building located in Alfragide - Lisbon (Figure 1) when submitted to seismic action according to EC 8. With a total of seven floors, three of them underground for parking and the upper floors for the offices, the building is composed by a structural system of wall and flat slab.

Figure 1 – Building's location (Source: Google)

In the undergrounded floors there is a rectangular configuration in plant limited by retaining walls, with an area 53x29 m^2 which is reduced on the upper floors to 43x17 m^2 (Figure 2).

For a better structural analysis of the building, it is relevant to approach some normative of the new standard EC 8 for which them are some interpretative doubts.

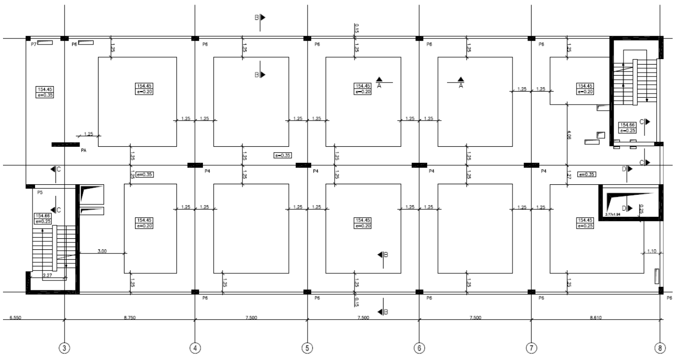

Figure 2 – Plant view of the upper floors of the building

2. Application of the Normative

2.1 Response Spectra

Comparing the Portuguese Standard (RSA) to the EC 8, it's easy to identify a clear diference about the influence of the soil in the Spectra response. Comparing both response spectra from RSA (Figure 3) one can conclude that they are more intense in a hard soil and that its intensity reduces in more deformable soils. On the other hand, EC 8 considers response spectra with higher values in more deformable soils (Figure 4). This is, due to the amplification of acceleration of earthquakes on structures with foundations in weaker soils.

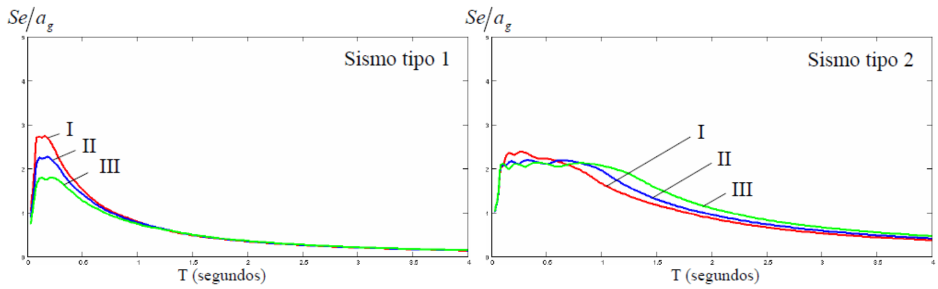

Figure 3 – Response spectra normalised according to the RSA as the type of soil (from the rigid soil I, to the most deformable one III). Left: for low magnitude ones at close distance; Right: for moderate to large magnitude earthquakes (ESTÊVÃO, SILVA, & SILVA, 2007)

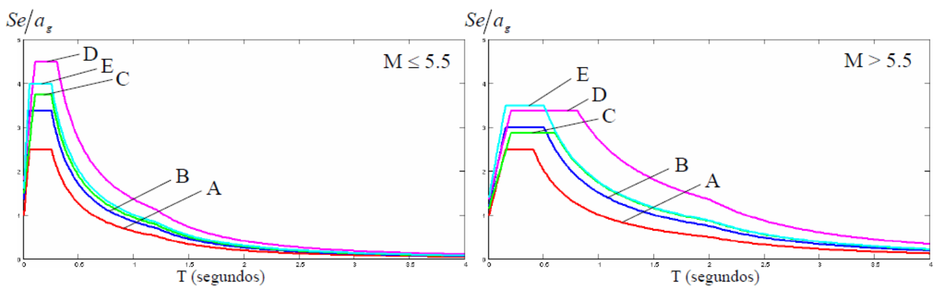

Figure 4 – Response Spectra according to the EC 8 as the type of soil (from the rigid soil A, to the most deformable one E). Left: for low magnitude ones at close distance; Right: for moderate to large magnitude earthquakes (ESTÊVÃO, SILVA, & SILVA, 2007)

2.2 Structural Seismic Elements

EN 1998-1 defines that a structure might have two types of elements: primary and secondary seismic elements. This division is of interest when the designer considers some structural elements not important for the seismic resistance or when, for architectural constraints, they can't follow the codes recommendations. Apart from this, it is recommended that all elements are considered as primary seismic elements for the following reasons (FARDIS, 2009):

- As the earthquake affects the structure globally, the primary elements offer a good performance due to the ductility provided in the design, but secondary elements may be seriously damaged.
- A structural system that cannot perform globally when earthquake resistance is concerned is a waste of resources, especially when the EC 8 is extremely conservative with these elements.

2.2.1 Primary Seismic Elements

Primary seismic elements are not considered as secondary seismic members and they are taken as being part to the structural system resistance to the seismic action. They are modeled on the analysis for the seismic design situation and fully designed and detailed for earthquake resistance in accordance with the rules of EN 1998-1.

2.2.2 Secondary Seismic Elements

These elements are not considered as part of the structural resistance system to the seismic action and its lateral strength and stiffness for seismic actions are neglected. These elements must be designed and detailed to ensure the support of gravity loading when affected by the displacements caused by the most unfavorable seismic design condition.

The contribution of these elements to the lateral stiffness shouldn't be over 15% of the lateral stiffness from the primary seismic elements. According to the EN 1998-1, they accomplish the requirements when the bending moments and shear forces, calculated due to the deformations imposed by seismic design situation, do not exceed their design flexural and shear resistance M_{Rd} and V_{Rd} , respectively.

2.3 Rigidity of the Elements for Seismic Action

The Portuguese Standard, Regulamento de Estruturas de Betão Armado e Pré-Esforçado (REBAP), establishes that the concrete stiffness can be estimated, in case of quick deformations, increasing by 25% its average value. In the other hand, EN 1998-1 considers that, in absence of more detailed data, the elastic flexural and shear stiffness properties of concrete should be considered as 50% of the corresponding stiffness of the non-cracked elements for the seismic combination.

REBAP justifies the increase of stiffness with the fact that seismic action is an instantaneous load. In the other hand, it is widely known that seismic action acts cyclically in the structures and the stiffness decreases after the first cycle due to cracking.

Considering an increased stiffness, the fundamental frequency becomes higher and there will be, in principle, bigger stress resultants in the structural elements. However, bigger stress resultants do not mean a good evaluation of the deformations that will be smaller due to the rigidity increased.

As shown in Figure 5, one can follow the curvature evolution as the bending moment increases. Before reaching the cracking moment (M_{cr}), the behavior is linear elastic and in a non-cracked state (state I); in this state, stiffness characteristics match with the ones in the section.

However, when the bending moment reaches the cracking moment occurs the opening of the first crack. This increases the curvature and modifies the stress distribution in that section with a consequent rise of the neutral line to a position corresponding to the cracked state (state II). By increasing the bending moment, the same will happen in the neighboring sections.

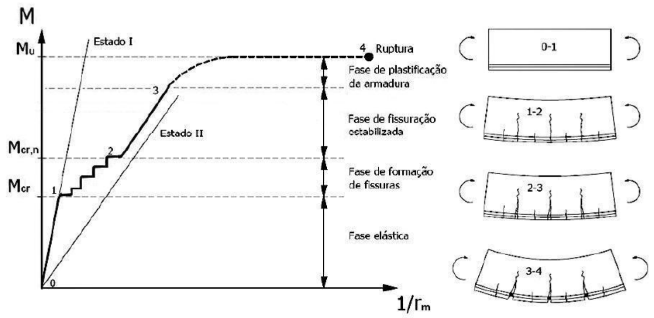

Figure 5 – Relationship between Moment-Curvature for a pure bending (TAVARES, 2010)

The flexural stiffness of a beam of concrete varies according to the amount of reinforcement in it (see Figure 6). This image shows that, for a percentage of reinforcement of 1%, the ratio of flexural stiffness of States I and II is approximately 30%.

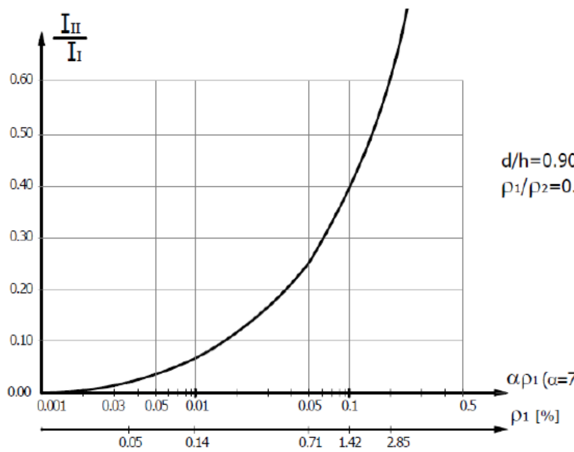

Figure 6 – Relationship between elastic and cracking stiffness for pure bending according different amounts of reinforcement (CAMARA, 1988)

In short, whether for elements subject to pure bending or to those subject to compound bending, one concludes that the stiffness of these elements will vary between the curvature of stage I and state II.

For pure bending elements, the flexural stiffness will be around 30% of non cracked stiffness while, for columns with bending and axial compression, the flexural stiffness will reach a value close to the non cracked stiffness. Therefore, considering the flexural stiffness of one half of the non cracked stiffness is a good approximation for a medium structural response.

2.4 Confinement of the core walls

Wall sections often consist of connected or intersecting rectangular segments (L-, T-, U-, I- or similar sections). In these cases, w_{wd} should be determined separately for each part of the rectangular section that may serve as the compression flange under any direction of the seismic action. (FARDIS, 2009)

Considering l_w the maximum dimension corresponding to the longitudinal section and b_c the width of the compressed flange adopted for confinement (as if the section was rectangular, with width b_c and depth l_w, see Figure 7), the compression area must be limited in compression flange width b_c. To verify this situation, the neutral axis depth of the ultimate curvature is obtained through:

$$\chi_u = (\nu_d + w_v) l_w \frac{b_c}{b_0} \quad (1)$$

The result obtained with the exercise described above is then compared with the compressed flange zone defined by b_c and its thickness. If the size exceeds the χ_u, the formula w_{wd} for the confinement reinforcement must be applied. If the value of χ_u exceeds the thickness of the compresses flange, two alternatives must be considered: (FARDIS, 2009)

1. Increasing the size of the area considered in the compression flange for confinement until χ_u is less than the flange thickness reduced by spalling cover concrete.
2. Providing confinement over the rectangular flange (web) instead of the compressed flange itself. The equation of confinement reinforcement should be applied with the width b_c equal to the thickness of the web (also in the normalisation of N_{Ed} and A_{sv} in v_d and w_{wd}). The value of w_{wd} from equation XXX should be implemented through stirrups in the web, sacrificing in terms of confinement the flange. However, it is more prudent to use the same confinement reinforcement in the flange as in the web.

2.4.1 Expression χ_u

To understand the meaning of equation (1), defined by EN 1998-1 to calculate χ_u, there should be a demonstration for a variable section.

For an axial compression on the section, the equilibrium equation is given by:

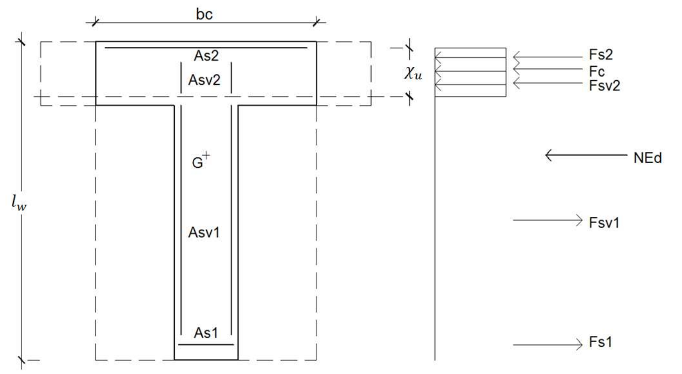

Figure 7 – T cross section for χ_u calculus

$$F_{s2} + F_c + F_{sv2} - F_{sv1} - F_{s1} = N_{Ed} \quad (2)$$

Taking into account that:

$$F_c = f_{cd}.b_c.\chi_u \qquad w_v = \frac{A_{sv} f_{yd,v}}{h_c b_c f_{cd}} <=> A_{sv} = \frac{w_v f_{cd} h_c b_c}{f_{yd,v}}$$

$$F_{svi} = A_{svi} f_{yd,v} \qquad A_{s2} = A_{s1}$$

The following is obtained:

$$\chi_u = (\nu_d - w_{v2} + w_{v1}) h_c$$

With ν_d as the normalised design axial force for the area defined by l_w and b_c and not the real area of the element. It is valid only when χ_u is smaller than the thickness of the flange.

As a simplification, $w_{v2} \cong 0$ e $w_{v1} \cong w_v$, by considering that the neutral line remains in the flange and all longitudinal bars from the web are in tension, we have the following expression similar to the one from EN 1998-1:

$$\chi_u = (\nu_d + w_v) l_w$$

3. Building analysis

Figure 9 shows an overview of the structural model of the building.

3.1 Modeling the core walls

The core walls were modeled in two different ways. The first one considered a frame element located at the geometric center of the cross section of the core wall and with the characteristics and properties of the real section of the structural element (model A). The second model consisted in subdividing the core in distinct walls. Then, in the geometric center of each one, a frame element is placed with the properties of the wall connected by an element that reconciles the displacements at the connections (model B).

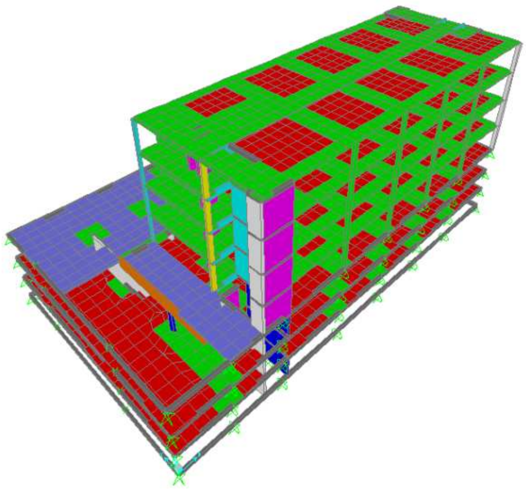

Figure 8 – Overview from the structural model (SAP2000)

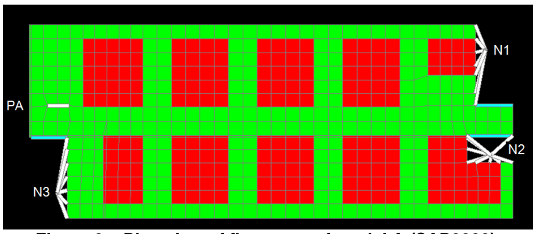

Figure 9 – Plan view of floor type of model A (SAP2000)

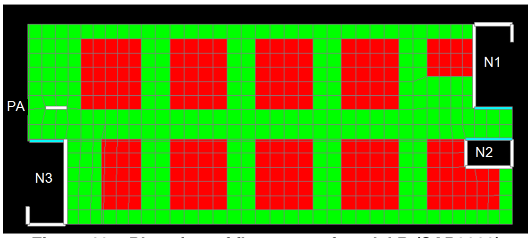

Figure 10 – Plan view of floor type of model B (SAP2000)

3.2 Distribution of basal shear forces by structural elements

Table 1 analyzes the shear forces on the first level of the floor for the core walls and its distribution in each axis. The shear from the xx axis is resisted from the blue sections and from the yy axis for the red sections, illustrated on Figure 11. One reaches to the conclusion that the use of different models didn't have significant influence in their distribution.

In the last line of Table 1 there is a quantification of the percentage that each core should resist according to the bending stiffness.

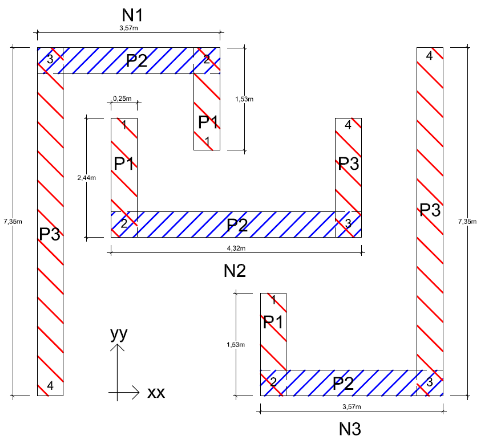

Figure 11 – Schematic representation of core N1, N2 and N3

Table 1 – Shear forces and respective percentage on core walls

Core walls Shear forces	N1		N2		N3		Total (xx)	Total (yy)
	xx	yy	xx	yy	xx	yy		
Model A [KN]	942,4	1300,1	1173,1	371,3	939,2	1926,5	3054,7	3597,9
% do total	31%	36%	38%	10%	31%	54%	100%	100%
Model B [KN]	852,1	1223,8	1257,6	389,0	849,5	1960,5	2959,2	3573,3
% do total	29%	34%	42%	11%	29%	55%	100%	100%
% from inertia section	27%	48%	47%	4%	27%	48%	100%	100%

3.3 Design the core walls

3.3.1 Calculation of longitudinal reinforcement according to model A

The calculation of the reinforcing bars that must be placed in the shaded sections on Figure 12 for the N3 core occurs as follows:
In the intermediate regions 2 and 3 (Figure 13), the calculation of the bending reinforcement is made in accordance with the following expression:

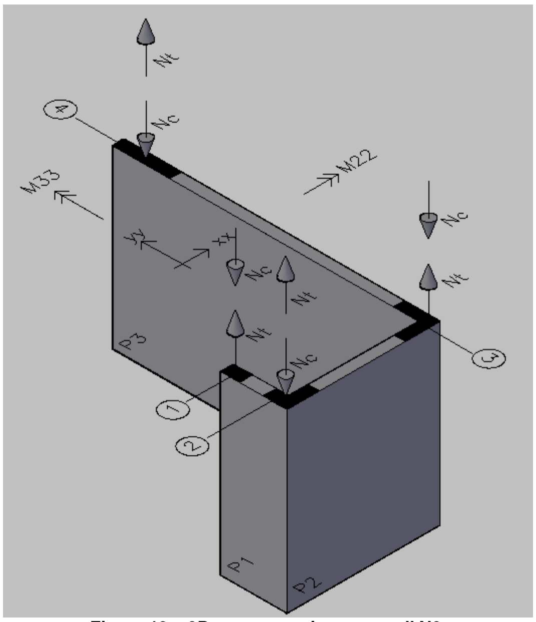

Figure 12 – 3D representative core wall N3

$$\mathbf{N_{equiv}} = \frac{\mathbf{M_{22}}}{\mathbf{z_{P3}}} + \frac{\mathbf{M_{33}}}{\mathbf{z_{P2}}} \quad (3)$$

One removes the effect of the compression load in this section, due to vertical loads, to calculate the axial tension, Nt:

$$N_t = N_{equiv} - N_{compr} \quad (4)$$

The area of longitudinal reinforcement required, A_s, is defined by the expression:

$$A_s = \frac{N_t}{f_{yd}} \quad (5)$$

At region 4 in the core, one uses the following expression:

$$N_{equiv} = \frac{M_{22}}{z_{P3}} \quad (6)$$

The remainder procedure is identic to the one used in regions 2 and 3 of the N3 core.

3.3.2 Calculation of longitudinal reinforcement on the model B

Knowing that the core is defined by three bar elements, one must consider the forces (as indicated on Figure 13) the relevant forces to the design of the longitudinal reinforcement.
To calculate the longitudinal reinforcement in the extremities (region 1 and 4 of Figure 13), the equivalent axial load for zone I was obtained as follows:

$$N_{equiv} = \frac{M_{Pi}}{z_{Pi}} + \frac{N_{Pi}}{2} \quad (7)$$

The remainder procedure is identical to the one used in chapter 3.3.1.
The calculation of the equivalent axial force in section 2 of Figure 13 was carried out as follows:

$$N_{equiv} = \frac{M_{P1}}{z_{P1}} + \frac{N_{P1}}{2} + \frac{M_{P2}}{z_{P2}} + \frac{N_{P2}}{2} \quad (8)$$

This way of considering the axial force of each element as being half applied to each end wall may not seem the correct way, by considering that the axial walls of the P1 and P3 correspond to a torque of forces that must be endured in regions 2 and 3. However the modeling and because there is an element of edge to it in section 4 to form a torque perpendicular as in the case, this binary eventually be built between the axial wall of P2 and P3 axial wall located in the center of gravity this wall, but that happens in the actual structure in Section 4, and therefore considered to be part of the axial wall of P3 should be included in region 4.

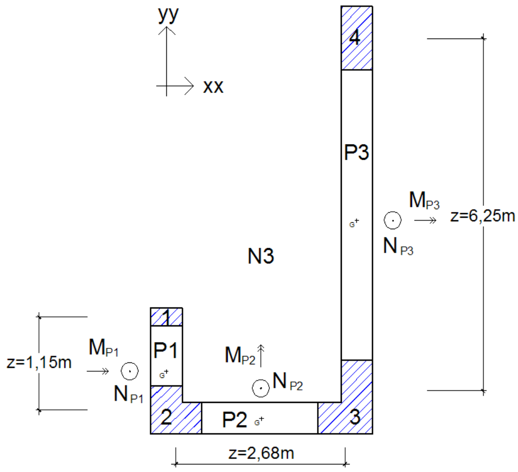

Figure 13 – Representation of the N3 core and relevant forces on the model B

3.3.3 Confinement of region 3

Considering M22 = 21336,9 KN.m and N_{Ed} = 3148,9 KN, the section of the compressed flange is defined by a confined rectangular section with width $b_c = 0{,}80m$ and it is verified if χ_u is lower than the flange thickness.

Using the following equations to verify the previous condition:

$$\mathbf{v_d} = \frac{\mathbf{N_{Ed}}}{\mathbf{f_{cd} l_w b_c}} = \frac{\mathbf{3148{,}9}}{\mathbf{20 \times 10^3 \times 7{,}35 \times 0{,}80}} = \mathbf{0{,}0268} \quad (9)$$

$$\mathbf{w_v} = \frac{\mathbf{A_{sv} f_{yd,v}}}{\mathbf{l_w b_c f_{cd}}} = \frac{\mathbf{26 \times 2 \times 1{,}13 \times 10^{-4} \times 435}}{\mathbf{7{,}35 \times 0{,}80 \times 20}} = \mathbf{0{,}0217} \quad (10)$$

$$\chi_u = (0{,}0268 + 0{,}0217) \times 7{,}35 \times \frac{0{,}25}{0{,}20} = 0{,}44\ m$$

As χ_u is higher than flange thickness, it is used a new lenght of b_c, until χ_u reaches the flange thickness.

Finally, considering b_c = 1,60 m

$$\chi_u = (0{,}0134 + 0{,}0109) \times 7{,}35 \times \frac{0{,}25}{0{,}20} = 0{,}22\ \mathrm{m}$$

$$\nu_d = \frac{N_{Ed}}{f_{cd} l_w b_c} = \frac{3148{,}9}{20 \times 10^3 \times 7{,}35 \times 1{,}60} = 0{,}0134$$

$$w_v = \frac{A_{sv} f_{yd,v}}{l_w b_c f_{cd}} = \frac{26 \times 2 \times 1{,}13 \times 10^{-4} \times 435}{7{,}35 \times 1{,}60 \times 20} = 0{,}0109$$

To calculate the required reinforcement for confinement one uses the exact same procedure used for calculating the single wall, with the parameters ν_d and w_v referring to the last iteration of the calculation χ_u (all parameters are defined in EN 1998-1 and in the appendix):

$$w_{wd} \geq \frac{30 \times 3{,}822 \times (0{,}0134 + 0{,}0109) \times 2{,}18 \times 10^{-3} \times \frac{1{,}60}{1{,}55} - 0{,}035}{0{,}425} = -0{,}068$$

If w_{wd} estimated above is less than the minimum determined on EC 8 (0,08 for DCM e 0,12 for DCH), we use the following expression with the highest value.

$$\max[-0{,}068; 0{,}08] = \frac{\text{vol. stirrups}}{1{,}55 \times 0{,}20 \times 1} \times \frac{435}{20} <=> \text{vol. stirrups} = 0{,}00114\ \text{m}^3$$

$$\text{area stirrups} = \frac{0{,}00114}{10 \times (1{,}55 \times 2 + 0{,}20 \times 9)} \times 10^4 = 0{,}23\ \text{cm}^2$$

$$\mu_\varphi = 1 + \frac{2(2{,}358 - 1)}{0{,}5775} \times 0{,}60 = 3{,}822$$

$$q_0 \text{ should be replaced by } 3{,}0 \times \frac{21336{,}9}{27150{,}6} = 2{,}358$$

$$M_{Rd} = 6{,}25 \times \left(63{,}67 \times 10^{-4} \times 435 \times 10^3 + \frac{3148{,}9}{2}\right) = 27150{,}6\ \text{KN.m}$$

$$\alpha = 0{,}726 \times 0{,}586 = 0{,}425$$

$$\alpha_s = \left(1 - \frac{0{,}10}{2 \times (1{,}50 - 0{,}05)}\right)\left(1 - \frac{0{,}10}{2 \times (0{,}25 - 0{,}05)}\right) = 0{,}726$$

$$\alpha_n = 1 - 18 \times \frac{0{,}20^2}{6 \times (1{,}60 - 0{,}05) \times (0{,}25 - 0{,}05)} = 0{,}586$$

It is needed to put stirrups Φ6mm//0,10m (0,28 cm^2) in the region defined by b_c and χ_u.

By using the same procedure for the perpendicular axis, one obtains the confinement reinforcement represented on Figure 14. Blue colour stands for the confinement reinforcement for bending moment M22. Red colour stands for the M33 moment. Despite the calculations suggesting stirrups Φ6 mm / 0,10 m, the use stirrups with a diameter of 8 mm is considered a good practice.

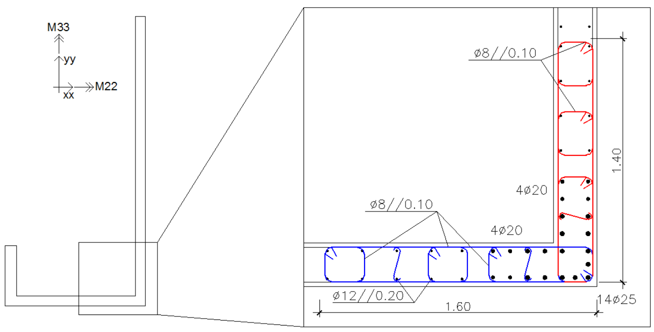

Figure 15 – Detail of confinement reinforcement in section 3 of the core N3

4. Conclusions

In this project, code aspects were presented, focusing on relevant aspects where some doubts on interpretation could rise on the design process.

The evolution of the spectral response from the Portuguese code RSA to EC 8 was referred. It was also explained why half of the rigidity of the non-cracked elements is proposed in the seismic analysis, and a definition was provided for the procedure of calculating the confinement reinforcement in a composite wall element (e.g. elevator core).

In the case study, we showed two ways of shaping the cores and corresponding reinforcement calculation procedures. It was concluded that shaping the core with one single element (A model) is the most direct way. In the B model, problems arise when quantifying the axial stress resultants in each element, in choosing which part should be considered as a force binary for an axis and which part should be left for the perpendicular. However, it was concluded that both models are valid.

We also showed the details of a core section according to EC 8.

5. References

CAMARA, J. (1988). "Comportamento em Serviço de Estruturas de Betão Armado e Pré-Esforçado". Instituto Superior Técnico.

ESTÊVÃO, J. M., SILVA, E., & SILVA, J. (2007). "Avaliação da Resposta Sísmica de Edifícios Sitos em Solos Brandos Estratificados". 7º Congresso de Sismologia e Engenharia Sísmica.

EUROCÓDIGO 8. (2010). “Projecto de estruturas para resistência aos sismos Parte 1: Regras gerais, acções sísmicas e regras para edifícios".

FARDIS, M. N. (2009). "Seismic Design, Assessment and Retrofitting of Concrete Buildings Based on EN-Eurocode8" (Vol. 8). Department of Civil Engineering, University of Patras, Greece: Springer.

REBAPE. "Regulamento de Estruturas de Betão Armado e Pré-Esforçado". Decreto-Lei n.º 349-C/83, de 30 de Julho.

RSA. "Regulamento de Segurança e Acções para Estruturas de Edifícios e de Pontes". DL nº 235/83 de 31 de Maio, Porto Editora.

TAVARES, R. M. (2010). “State-of-Art sobre o Controlo da Fendilhação devido a Deformações Impostas”. Instituto Superior Técnico: Dissertação para obtenção do Grau de Mestre em Engenharia Civil.

Appendix

$$\alpha w_{wd} \geq 30\mu_{\varphi}(\nu_d + w_v)\varepsilon_{sy,d}\frac{b_c}{b_0} - 0{,}035; \qquad w_{wd} = \frac{\text{volume of confining hoops}}{\text{volume of concrete core}} \times \frac{f_{yd}}{f_{cd}};$$

$$\mu_{\varphi} = \begin{cases} 2q_0 - 1;\ \text{if } T_1 > T_c \\ 1 + \frac{2(q_0 - 1)}{T_1}T_c;\ \text{if } T_1 < T_c \end{cases}; \qquad q_0 \text{ should be replaced by } q_0 \times \frac{M_{Ed}}{M_{Rd}};$$

$$w_v = \frac{\rho_v f_{yd,v}}{f_{cd}}; \qquad \rho_v = \frac{A_{sv}}{A_c}; \qquad M_{Rd} = z\left(A_s f_{syd} + \frac{N_{comp}}{2}\right); \qquad \alpha = \alpha_s \alpha_n;$$

$$\alpha_s = \left(1 - \frac{s}{2b_0}\right)\left(1 - \frac{s}{2h_0}\right); \qquad \alpha_n = 1 - \sum_n \frac{b_i^2}{6b_0h_0}$$

w_{wd} – mechanical volumetric ratio of confining reinforcement

χ_u – The neutralaxis depth at ultimate curvature
μ_{φ} - curvature ductility factor

w_v - mechanical ratio of vertical web reinforcement

ρ_v - reinforcement ratio of vertical web bars in a wall

A_{sv} - total area of the vertical reinforcement in the web of the wall

M_{ED} – design bending moment from the analysis for the seismic design situation

M_{RD} – design flexural resistance

v_d – normalised design axial force ($v_d = N_{Ed}/A_c.f_{cd}$);

$\varepsilon_{sy,d}$ – design value of steel strain at yield

h_c – cross-sectional depth of column in the direction of interest

h_0 – is the depth of confined core (to the centreline of the hoops)

b_c – cross-sectional dimension of column

b_0 – width of confined core in a column or in the boundary element of a wall (to centreline of hoops)

α – confinement effectiveness factor,
n – total number of longitudinal bars laterally engaged by hoops or cross ties on perimeter of column section

b_i – distance between consecutive bars engaged by a corner of a tie or by a cross-tie in a column