Design methodology of flat-slab systems to the seismic actions

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Abstract: Over the last several decades, flat-slabs have been increasingly used in buildings, such as service buildings, schools, hospitals and residential buildings, due to a faster execution and easier installation of equipment on the ceilings. The NP EN 1998-1:2010 [1] does not consider flat-slabs as a primary seismic structural element, ignoring its resistance capacity to the seismic action and specifying that flat-slabs should be designed and detailed in order to have sufficient ductility to support gravity loads when subjected to the largest displacements, during the seismic design condition. However, this European Standard does not prohibit the consideration of flat-slabs resistance capacity in the seismic action. Thus, it would be economically advantageous to design flat-slabs as a primary seismic structural element, using design methods that consider their contribution to the building's seismic resistance.

For this purpose, a design methodology that considers the resistance of flat-slab systems in the seismic action, in accordance with the criteria set forth in the Portuguese specification for hospital buildings "Especificações Técnicas para o Comportamento Sismo-Resistente de Edifícios Hospitalares", E.T.05/2020 [2], will be evaluated. Three buildings with waffle flat-slab systems and different lateral stiffness will be analyzed. All buildings will be designed for the seismic zones 1.1 to 1.5, specified in the Portuguese National Annex - 3.2.1(2) of the NP EN 1998-1:2010 [1], considering the importance classes II and IV.

Keywords: Flat-Slab; Design; Seismic action; Ductility; Confinement.

1. INTRODUCTION

In recent decades, flat-slab systems have been increasingly used in service buildings, schools, hospitals and residential buildings, due to their many advantages. These structural systems reduce the construction time and cost, as a result of simplified formwork and materials; improve the floors height and overall architectural aesthetics. However, these slabs have some disadvantages regarding structural behavior, such as the high concentration of bending moments and shear forces in the slab-column connecting zone; the higher deformability and the lower resistance to horizontal loads, when compared to column-beam frames. In the case of gravity loads, the behavior of flat-slabs is well known and their design and reinforcement detailing do not raise much difficulties. Regarding the seismic action, there is still some uncertainty about their behavior, particularly in the slab-column connection zone. It is known that, in this area, shear forces are, in general, predominant compared to bending moments. This high concentration of stresses, associated with deformations induced by the seismic action, should be resisted without the occurrence of major structural damage or even punching shear failure, due to its brittle nature and potential total structural collapse. The contribution of the flat-slab system in seismic resistance, as a primary seismic structural element, is not fully covered by the NP EN 1998-1:2010 [1]. Thus, in seismic design situations, these systems can be designed and detailed for the effects of gravity loads when subjected to the maximum deformations induced by the seismic design condition, disregarding the contribution of its lateral stiffness. For these reasons, it is common to add frame systems and walls with adequate strength and lateral stiffness in order to not only provide an adequate behavior to the buildings, as far as lateral deformability is concerned, but also to reduce the demands on the flat-slab system, considering these systems as secondary structural systems to seismic resistance. In summary, the design of these systems usually considers an elastic behavior, which is associated with high stresses, induced by the seismic action. On the other hand, flat-slabs are classified as secondary structural system, which doesn't consider the lateral stiffness contribution to the resistance of seismic action. This is, therefore, a double restriction in the resistance design of flat-slab systems to the seismic actions.

2. CONFINEMENT OF REINFORCED CONCRETE SECTIONS

When subjected to horizontal cyclic actions, concrete suffers a progressive strength degradation that compromises its resistance capacity, stiffness and also ductility, since it has a low deformation capacity, inherent to its brittle behavior. Through Figure 1, it is possible to observe the successive loss of strength and accumulation of permanent deformations with the increase of the number of loading cycles.

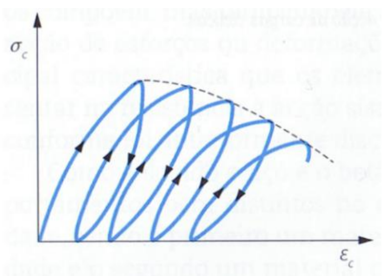

Figure 1 - Stress-strain diagram of concrete subjected to cyclic actions [6].

One of the most viable solutions to increase the ductility of concrete consists in using confining reinforcement, such as stirrups with relatively reduced spacing.

If the amount of longitudinal reinforcement is increased, the neutral axis decreases, resulting in a loss of ductility. On the other hand, the greater the reinforcement, the greater the resistance. It is, therefore, convenient to study the influence of confining reinforcement, since it increases the ultimate curvature, even for higher levels of longitudinal reinforcement.

The NP EN 1992-1-1:2010 [3] defines that, in the absence of precise data, it is possible to adopt a stress-strain relation for confined concrete, as shown in Figure 2:

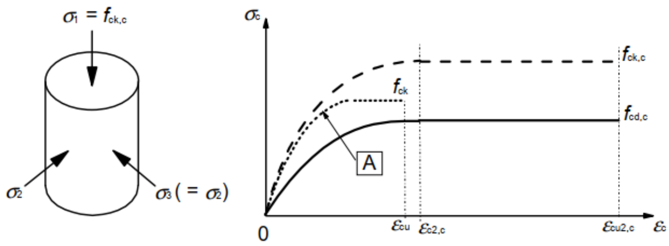

Figure 2 - Stress-strain relationship for confined concrete [3].

$$f_{ck,c} = f_{ck}(1{,}000 + 5{,}0\frac{\sigma_2}{f_{ck}})\ for\ \sigma_2 \leq 0{,}05 f_{ck} \quad (1)$$

$$f_{ck,c} = f_{ck}(1{,}125 + 2{,}50\frac{\sigma_2}{f_{ck}})\ for\ \sigma_2 > 0{,}05 f_{ck} \quad (2)$$

$$\varepsilon_{c2,c} = \varepsilon_{c2}(f_{ck,c}/f_{ck})^2 \quad (3)$$

$$\varepsilon_{cu2,c} = \varepsilon_{cu2} + 0{,}2\,\sigma_2/f_{ck} \quad (4)$$

Where f_{ck} is the characteristic compressive cylinder strength of concrete at 28 days; $f_{ck,c}$ is the characteristic compressive strength of confined concrete; σ_2 is the effective lateral compressive stress at the ULS due to confinement; ε_{c2} is the strain at reaching the maximum strength according to Table 3.1 of the NP EN 1992-1-1:2010 [3]; ε_{cu2} is the ultimate strain according to the Table 3.1 of the NP EN 1992-1-1:2010 [3].

Since the stirrups are spaced longitudinally and transversally, there is an area of concrete in between that is not effectively confined. Thus, the smaller the spacing, the greater the confinement efficiency factor. The figure 3 illustrates the distribution of effective confining stresses in slabs and the geometry of the confined concrete core area, in the longitudinal and transverse directions.

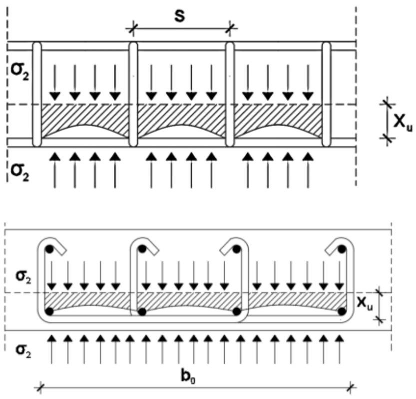

Figure 3 - Distribution of the confining effective stresses, in slabs, top: longitudinal; bottom: transversal.

The effective lateral compressive stress to due confinement is given by the following expression:

$$\sigma_2 = \alpha \frac{A_{sw}}{sb_0} f_{yd} = \alpha \rho_w f_{yd} \quad (5)$$

Where α is the confinement effectiveness factor; A_{sw} is the cross-sectional area of shear reinforcement; s is the spacing between shear reinforcement; b_0 is the width of the confined core of reinforced concrete; ρ_w is the reinforcement ratio of shear reinforcement, given by $\rho_w = A_{sw}/sb_0$.

The confinement effectiveness factor is given by the following expressions:

$$\alpha = \alpha_s * \alpha_n \quad (6)$$

$$\alpha_s = 1 - \frac{s}{x_u} \quad (7)$$

$$\alpha_n = 1 - \frac{\sum b_i^2}{6x_u b_0} \quad (8)$$

Where α_s is the confinement effectiveness factor for the longitudinal direction and α_n is the confinement effectiveness factor for the transversal direction.

3. EXPERIMENTAL TESTS

In 2014, Almeida A. and Inácio M. [4], under the FLAT project, developed a series of experimental tests to study the behavior of flat-slabs subjected to cyclic horizontal loads.

In these tests, 7 flat-slabs, subjected to gravity and cyclic horizontal loads, were analyzed. The models used are composed of reinforced concrete flat-slab panels 4.15m long, 1.85m wide with a thickness of 125mm.

The researchers concluded that the stiffness and deformation capacity are inversely proportional to the percentage of the applied gravity load, i.e., with the increase of the gravity loads, the ductility of the slab-column critical zone decreases. The same happens with the drift values. It was also concluded that there is an increase in the horizontal deformation capacity, when shear reinforcement is used, especially when the shear reinforcement is properly tied to the longitudinal reinforcement. Figure 4 illustrates the hysteretic diagrams of all experimental models. A clear increase in energy dissipation capacity and drift can be noted with the use of shear reinforcement.

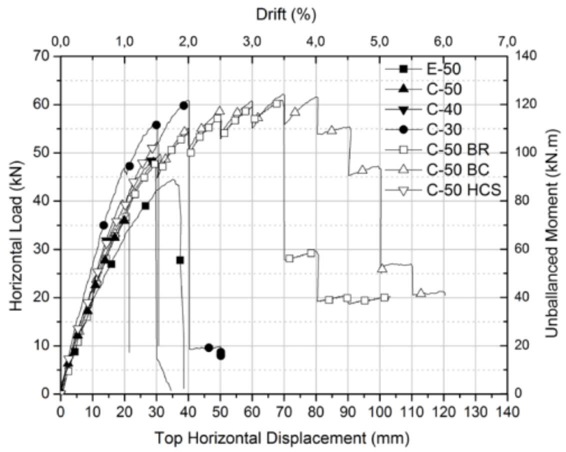

Figure 4 - Hysteretic diagrams of the models [4].

Where E-50 was subjected only to gravity loads, without punching reinforcement. Models C-50, C-40 and C-30 were subjected to horizontal cyclic loads, without punching reinforcement. The model C-50 HSC is composed of a high strength concrete where the applied gravity load corresponds to 50% of the strength. The models C-50 BR and C-50 BC were subjected to the same gravity load, but have different shear reinforcement solutions, using bolts with a radial and a cross pattern, respectively.

4. DESIGN RULES ACCORDING TO E.T.05/2020 [2]

E.T.05/2020 [2] Chapter 5 defines rules that allow, although in a somewhat limited way, the design of flat-slab systems in the resistance to the seismic actions. In general, E.T.05/2020 [2] advocates for structural simplicity. Factors such as uniformity, symmetry and simple path for flow of stresses caused by seismic action are basic design concepts that facilitate the analysis, design and detailing of these structures. With regard to some of the additional rules, it is defined that:

- In waffle flat-slabs, the solid strips between columns must have a minimum height of $h = 0{,}30m$ and a minimum width $b_{strip} = max(2h; b_c)$, where b_c is the column dimension that is perpendicular to the axis of the solid strip.
- Near the slab-column zone, it is necessary to consider a solid reinforced concrete zone with a minimum length from the column face of $3h_{slab}$ and a minimum width of $b_{eff} = b_c + 2h_{slab}$;
- Inside the solid reinforced concrete zone, within the effective width b_{eff}, an extension of the critical area of $l_{cr} = 1{,}5h_{slab}$ should be considered;
- The minimum longitudinal spacing of the punching shear reinforcement must be the minimum between ${}^{h_0}/_2$ and $8d_{bl}$, where h_0 corresponds to the distance between the top and bottom longitudinal reinforcement and d_{bl} is the diameter of the longitudinal reinforcement rebars;
- The minimum spacing between stirrup legs, placed in b_{eff}, in the perpendicular direction of the strip axis, must be less than $200mm$;
- The stirrups should be detailed to a distance of not less than $3h_{slab}$ from the column face.

Figures 5 and 6 illustrate the rules previously presented for a slab with a thickness of 0.35m:

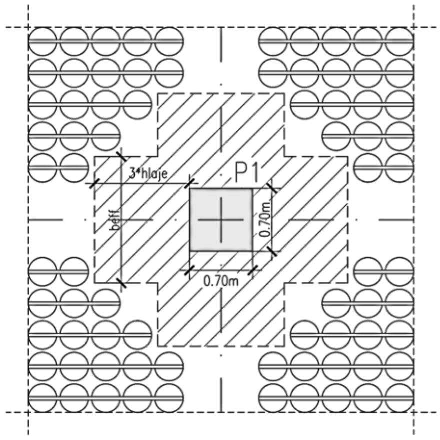

Figure 5 - Minimum dimensions of the solid slab-column zone.

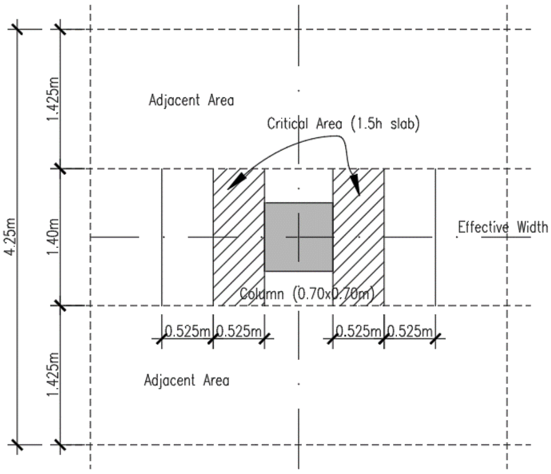

Figure 6 - Dimensions of the critical area, adjacent area and effective width.

- The longitudinal reinforcement designed for the seismic design combination must be placed, in its totality, in the central strip, where 50% of this reinforcement must be placed within the effective width.
- The longitudinal reinforcement designed to resist the bending moments transferred from the slabs to the columns must be placed within the effective width.
- Shear strength must be verified through the use of stirrups, calculated by the capacity design for medium ductility structures, as defined in subsection 5.1.2 of NP EN 1998-1:2010[1].

5. DUCTILITY EVALUATION

The ductility level of a section depends on the position of the neutral axis when the deformation capacity of the concrete exceeds its limit. In general, one way to control the position of the neutral axis consists in adding a certain amount of compression reinforcement. However, unlike beams, slabs do not have enough height that is favorable to the effective exploration of this reinforcement, because the strains at the fiber level which contains the compression reinforcement are relatively low, preventing it from reaching high levels of stress. This aspect has a relevant influence on ductility in situations before the spalling of the concrete cover, i.e., when the strains in the most compressed fiber of the section are less than 0.0035. When the strains exceed this value, the concrete cover spalls and the deformation diagram is affected by changing the maximum strain to the fibers where the compression reinforcement is located. It is in this situation that the influence of the compression reinforcement on ductility is relevant.

The curvature ductility factor is defined by the ratio between the ultimate curvature and the yield curvature, given by de following expression:

$$\mu_\phi = \frac{\phi_u}{\phi_y} \tag{10}$$

The ultimate curvature is given by the following expression:

$$\phi_u = \frac{\varepsilon_{cu}}{x_u} \tag{11}$$

Where ε_{cu} corresponds to the ultimate compressive strain in the concrete and x_u is the position of the neutral axis.

The yield curvature, in slabs, is given by the following expression:

$$\phi_y = 2{,}1\frac{\varepsilon_{sy,d}}{d} \tag{12}$$

Where $\varepsilon_{sy,d} = 2.175‰$ for S500 steel and d is the effective depth of a cross-section.

The curvature ductility factor for situations in which no spalling of the cover concrete has yet occurred, i.e., the extension in the most compressed fiber of the concrete is limited to $\varepsilon_c = 0{,}0035$, is given by the following expression:

$$\mu_\phi = \frac{0{,}0035}{2{,}6(\rho - \rho')\varepsilon_{sy,d}}\frac{f_{cd}}{f_{yd}} \tag{13}$$

The ultimate compressive strain in the confined concrete is given by the following expression [5]:

$$\varepsilon_{cu2,c} = 0{,}0035 + 0{,}2\alpha\rho_w\frac{f_{yd}}{f_{cd}} \tag{14}$$

$$w_w = \rho_w\frac{f_{yd}}{f_{cd}} \tag{15}$$

Where w_w the mechanical volumetric ratio of confining reinforcement, α is the confinement effectiveness factor and ρ_w is shear reinforcement ratio.

Using the previous expressions 13, 14 and 15, the curvature ductility factor after the concrete cover spalling is given by:

$$\mu_\phi = \frac{0{,}0035 + 0{,}2\alpha w_w}{2{,}6(\rho - \rho')\frac{f_{yd}}{f_{cd}}\varepsilon_{sy,d}} \tag{16}$$

In cases where it is necessary to increase the deformation capacity of the concrete, i.e., situations where the available curvature ductility, after the spalling of the concrete, is lower than the required curvature ductility, it will be necessary to confine the concrete section. The confining reinforcement is obtained through expression 17, which is obtained from expression 16:

$$\alpha w_w = 13\mu_\phi(\rho - \rho')\frac{f_{yd}}{f_{cd}}\varepsilon_{sy,d} - 0{,}0175 \tag{17}$$

For the calculation of the behavior factor, it is used the expression 5.4, established in subsection 5.2.3.4(3) of the NP EN 1998-1:2010 [1]:

$$\mu_\emptyset = 2 * q_0 - 1 \tag{18}$$

$$q_0 = (\mu_\emptyset + 1)/2 \tag{19}$$

Where q_0 is the basic value of the behavior factor. This expression is used when $T_1 \geq T_C$.

6. STUDY CASES DESCRIPTION

In order to evaluate the design methodology for flat-slab systems indicated in the chapter 4, and to verify whether the solutions are feasible in practice, three structures were idealized in which each flat-slab system has increasing participation levels to the seismic resistance. The structures consist of an underground floor and 5 raised floors. In plan, it has a rectangular geometry with 30m x 38.50m. In height, the structures have 21.00m, with each floor having a ceiling height of 3.50m. The interior columns have a 0.70m x 0.70m cross-section. The flat-slabs have interior spans of 8.5m, end spans of 6.50m and have a thickness of 0.35m. It was used a waffle flat-slab solution with a solid area around the columns. Solid strips were also used between columns. The geometrical and mechanical characteristics of the molds are presented in the COBIAX CBCM-S-200 [6] technical sheets. The beams, in the boundary frame system, have a 0.30m x 0.70m cross-section. The dimensions assigned to the columns and concrete structural walls, of the three buildings, were conditioned by the percentage of contribution of the flat-slab systems in the resistance to the seismic action. The following table illustrates the resistance participation that each buildings flat-slab system has to the seismic action.

Table 1 - Participation of vertical structural elements in the resistance to seismic action.

Structure	Structural Concrete Walls	Boundary Frame System	Flat-Slab System
1	45%	25%	30%
2	-	50%	50%
3	-	40%	60%

The indicative minimum strength class of the concrete used is C30/37 and the characteristic yield strength of the reinforcement considered is A500 NR SD, with a ductility class C. All structures were designed for the seismic zones 1.1 to 1.5 and a type B foundation soil, specified in the Portuguese National Annex - 3.2.1(2) of the NP EN 1998-1:2010 [1], considering the importance classes II and IV. The adopted behavior coefficient is $q = 2.5$.

The following picture illustrates the plan of the structure 3:

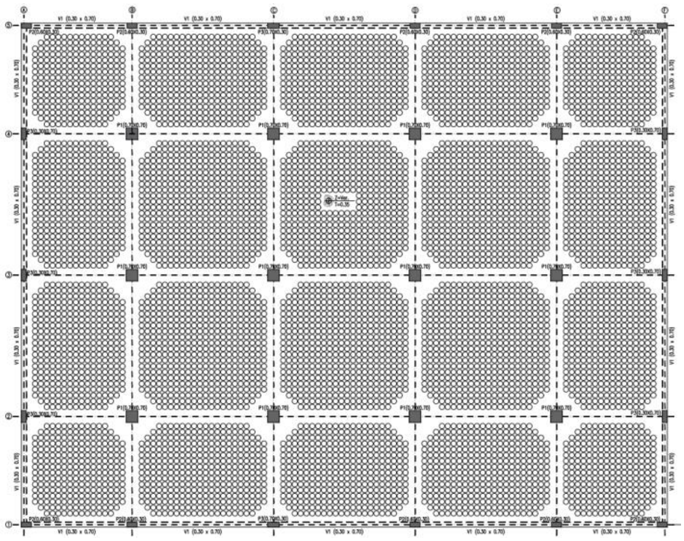

Figure 7 - Plan of the structure 3 floors.

7. DAMAGE LIMITATION

Due to the high number of study cases, it will only be shown, in this paper, the damage limitation requirement for the structure 3 considering an importance class IV.

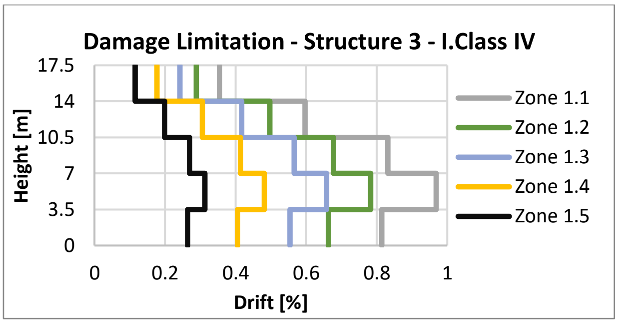

Figure 8 – Drift values as a function of height, for the seismic zones 1.1 to 1.5, for an importance class IV, for the structure 3.

Structure 3, classified as importance class IV, meets the NP EN 1998-1:2010 [1] criteria 4.33, in the seismic zones 1.1 and 1.2, which defines that when the relative displacements between floors are higher than 0.75% it should be ensured that non-structural fixed elements should not interfere with the structural deformations.

8. DESIGN AND RESULTS ANALYSIS

Using the equivalent frame analysis, the following table summarizes the bending moments in the central and lateral strips, in the alignments C3 and D3, for the gravity loads:

Table 2 - Bending moments in the strips and the respective longitudinal reinforcements.

			A_s (cm²/m)
M^- [kNm/m]	CS	175	14.8
	LS	58	5.65
M^+ [kNm/m]	CS	70	5.65
	LS	47	5.65

A minimum reinforcement of Ø12//0.20 (5.65cm²/m) was adopted. It is verified that, for the gravity loads combination, Ø20//0.20 (15.71cm²/m) would be necessary to resist the negative bending moments in the central strip.

Next, the tables 3 and 4 show the top and bottom longitudinal reinforcements needed to resist the seismic action combination and the bending moments transferred from the slabs to the columns. It is considered that $As^{-beff} = 0.4 * As^{beff}$.

Table 3 - Top longitudinal reinforcement.

Struc.	1 (30%)		2 (50%)		3 (60%)	
I.Class	II	IV	II	IV	II	IV
Zone	As^{beff} adopted					
1.1	Ø25//0.125	-	Ø25//0.125	-	Ø25//0.10	-
1.2	Ø20//0.10	-	Ø25//0.15	-	Ø25//0.125	-
1.3	Ø20//0.125	Ø25//0.10	Ø20//0.125	Ø25//0.10	Ø20//0.125	Ø25//0.10
1.4	Ø20//0.15	Ø20//0.10	Ø20//0.15	Ø25//0.15	Ø20//0.15	Ø25//0.15
1.5	Ø20//0.175	Ø20//0.125	Ø20//0.175	Ø20//0.125	Ø20//0.175	Ø25//0.20

Table 4 - Bottom longitudinal reinforcement.

Struc.	1 (30%)		2 (50%)		3 (60%)	
I.Class	II	IV	II	IV	II	IV
Zone	As^{-beff} adopted					
1.1	Ø16//0.125	-	Ø16//0.125	-	Ø16//0.10	-
1.2	Ø16//0.15	-	Ø16//0.125	-	Ø16//0.125	-
1.3	Ø16//0.20	Ø20//0.175	Ø16//0.15	Ø20//0.15	Ø16//0.20	Ø20//0.15
1.4	Ø12//0.125	Ø16//0.15	Ø16//0.20	Ø16//0.15	Ø12//0.125	Ø16//0.15
1.5	Ø12//0.15	Ø12//0.15	Ø12//0.125	Ø16//0.20	Ø12//0.15	Ø12//0.125

In general, the longitudinal reinforcement needed to resist the bending moments transferred from the slabs to the columns are the highest.

As seen previously, it is established in E.T.05/2020 [2] that all longitudinal reinforcement that resists the bending moments transmitted from slabs to columns must be placed within the effective width $b_{eff} = b_c + 2h_{laje}$. It can be concluded that this requirement becomes too demanding for these structures located in seismic zones 1.1 and 1.2, when classified as an importance class IV.

For example, in the structure 1, it would be necessary Ø25//0.06 for seismic zone 1.1 and Ø25//0.085 for seismic zone 1.2, inside the effective width, which is an impractical solution.

Table 5 - Transverse reinforcement.

Struc.	1 (30%)		2 (50%)		3 (60%)	
I.Class	II	IV	II	IV	II	IV
Zone	$A_{sw}/s/r$ [8 legs]					
1.1	Ø6//0.10	-	Ø6//0.10	-	Ø8//0.125	-
1.2	Ø6//0.10	-	Ø6//0.10	-	Ø8//0.125	-
1.3	Ø6//0.10	Ø6//0.10	Ø6//0.10	Ø8//0.125	Ø6//0.10	Ø8//0.125
1.4	Ø6//0.125	Ø6//0.10	Ø6//0.125	Ø6//0.10	Ø6//0.10	Ø6//0.10
1.5	Ø6//0.125	Ø6//0.125	Ø6//0.125	Ø6//0.10	Ø6//0.125	Ø6//0.10

The results obtained show that the transverse reinforcement required to resist shear forces leads to solutions that do not raise difficulties regarding the detailing.

Next, it was verified if the available curvature ductility is higher than the required curvature ductility, considering that there is no concrete cover spalling, using the expression 13. For this, the concrete strain, in the extreme fiber of the cross-section, of 0.0035 was adopted. Since the adopted behavior coefficient is 2.5, the required curvature ductility is (expression 18): $\mu_{\phi required} = 2 * 2{,}5 - 1 = 4$.

As mentioned previously, it is impractical to detail the longitudinal reinforcement for structures classified as an importance class IV for the seismic zones 1.1 and 1.2.

Table 6 - Available curvature ductility before the concrete cover spalling.

Struc.	1 (30%)		2 (50%)		3 (60%)	
I.Class	II	IV	II	IV	II	IV
Zone	$\mu_{\phi available}$ Before spalling of the concrete cover					
1.1	2.45	-	2.45	-	1.96	-
1.2	3.06	-	2.93	-	2.45	-
1.3	3.82	2.15	3.82	1.96	3.82	1.96
1.4	4.59	3.06	4.58	2.93	4.59	2.93
1.5	5.35	4.28	5.35	3.82	5.35	3.91

Analyzing the results in the table 6, it can be verified that, for the seismic zones 1.1 to 1.3 and for the importance class II, the value of the available curvature ductility is lower than the required. In addition, the structures classified as an importance class IV, in general, did not verified this condition above the seismic zone 1.3. In these cases, it can be concluded that the concrete cover will spall.

Next, the curvature ductility was verified considering that there is spalling of the cover concrete. Thus, it was adopted strains, at the level of compression reinforcement, of 0.0035 (expression 16).

Table 7 - Available curvature ductility after the concrete cover spalling.

Struc.	1 (30%)		2 (50%)		3 (60%)	
I.Class	II	IV	II	IV	II	IV
Zone	$\mu_{\phi available}$ After spalling of the concrete cover					
1.1	4.14	-	4.14	-	3.31	-
1.2	5.33	-	4.97	-	4.14	-
1.3	6.37	3.60	6.37	3.26	6.37	3.41
1.4	8.08	5.33	8.07	4.89	8.08	4.97
1.5	9.22	6.46	9.22	6.37	9.22	6.20

Analyzing the results of the table 7, it can be verified that confining reinforcement is required for seismic zone 1.1 (structure 3), when the structures are classified as an importance class II, since the available curvature ductility is lower than the required. In addition, the same is verified for the seismic zone 1.3 when the buildings are classified as an importance class IV. However, transverse reinforcement will be required, in all cases, to prevent the buckling of the compressed reinforcements.

Table 8 - Confinement reinforcement.

Struc.	1 (30%)		2 (50%)		3 (60%)	
I.Class	II	IV	II	IV	II	IV
Zone	A_{sw}/s [cm^2/m]					
1.1	-	-	-	-	6.84	-
1.2	-	-	-	-	-	-
1.3	-	4.21	-	7.31	-	5.96
1.4	-	-	-	-	-	-
1.5	-	-	-	-	-	-

By analyzing the results of the previous tables, it can be seen that, in general, the transverse reinforcement calculated by the capacity design is sufficient to ensure good ductility of the slab-column zone, since the required curvature ductility is lower than the available curvature ductility.

After the confining reinforcement design, the curvature ductility coefficient and the behavior coefficient were evaluated.

The following tables summarize the values of the available curvature ductility coefficients and the behavior coefficients, considering the increase in the deformation capacity of the concrete, provided by the confinement and transverse reinforcements.

Table 9 - Final available curvature ductility coefficients.

Structure		1 (30%)		2 (50%)		3 (60%)	
Importance Class		II	IV	II	IV	II	IV
Seismic Zone	1.1	6.1	-	6.1	-	6.6	-
	1.2	6.6	-	6.5	-	6.9	-
	1.3	6.9	5.8	6.9	6.6	6.9	6.7
	1.4	8.1	6.6	8.1	6.4	8.1	6.5
	1.5	9.5	6.9	9.5	6.9	9.5	6.9

Table 10 - Behavior coefficients.

Structure		1 (30%)		2 (50%)		3 (60%)	
Importance Class		II	IV	II	IV	II	IV
Seismic Zone	1.1	3.6	-	3.6	-	3.8	-
	1.2	3.8	-	3.7	-	4.0	-
	1.3	4.0	3.4	4.0	3.8	4.0	3.8
	1.4	4.5	3.8	4.5	3.7	4.5	3.7
	1.5	5.3	3.9	5.3	4.0	5.3	3.9

Through the analysis of the results of the previous tables, it can be seen that, in general, the available curvature ductility granted by the transverse reinforcement is higher than 6.0, and it is possible to explore a behavior coefficient higher than 3.0. These behavior coefficients correspond to values that are close to those of structures of medium ductility.

It can be concluded that this design methodology allows, in a slightly demanding way in some of its requirements, the exploration of flat-slab systems in the resistance to seismic action through the adoption of transverse reinforcement that guarantees the resistance to shear, confines the concrete cross-section and, consequently, increases its deformation capacity.

9. DETAILING EXAMPLE

To illustrate the design methodology adopting the criteria set forth in E.T.05/2020 [2], the example of building 2, located in Lisbon, was taken, considering that the building in question is a hospital. Regarding seismic condition, the building is located in the seismic zone 1.3 and is classified as importance class IV.

The table 11 shows the load values for the fundamental and quasi-permanent combinations.

Table 11 - Load values used in the design.

Equivalent Self Weight [kN/m^2]	6.9
Remaining Perm.Loads [kN/m^2]	3.0
Variable Actions [kN/m^2]	4.0
γ_G	1.35
γ_Q	1.5
ψ_2	0.4
P_{sd} [kN/m^2]	19.4
P_{qp} [kN/m^2]	11.5

The maximum bending moment, due to the quasi-permanent combination in the central strip, is M_{qp}^{CS}=442kNm. The bending moment in the central strip due to the seismic combination, is M_{sd}=695+442=1137 kNm. Since the center strip has a width of 4.25m, the bending moment in the central strip is m_{sd}=268kNm/m, resulting in 23 cm^2/m of reinforcement. These reinforcements are then distributed over the effective width and adjacent zones: $As^{beff} = 34.9\ cm^2/m$ and $As^{adj.} = 17.1\ cm^2/m.$

As seen previously, all the reinforcement that resists the bending moments transmitted by the slabs to the columns must be designed for the effective width. Thus, m_{sis}^{beff}=695/1.40=497kNm/m, resulting in 49.00 cm^2/m of longitudinal reinforcement. It is adopted, in the effective width, a top longitudinal reinforcement of Ø25//0.10 (49.09cm^2/m) and a compression longitudinal reinforcement of Ø16//0.10 (20.11cm^2/m). In the adjacent zones, a top reinforcement of Ø20//0.175 (17.95cm2/m) is adopted.

Figure 5.9 illustrates the longitudinal reinforcement in the central strip, near the central columns.

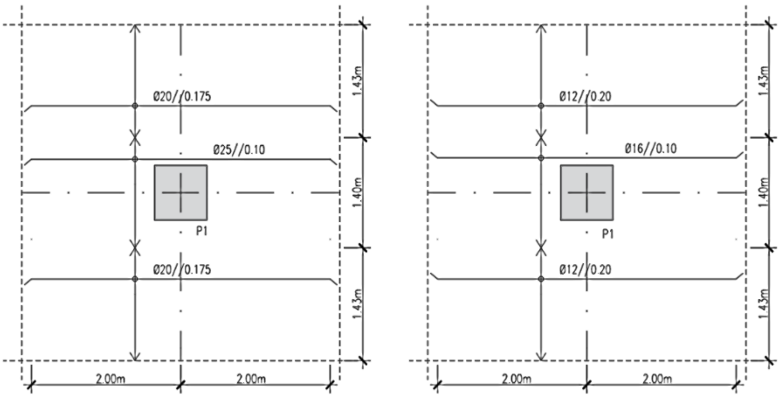

Figure 9 - Longitudinal reinforcements. Left:Top reinforcement; Right: Bottom reinforcement.

The design shear force is determined by the capacity design, $V_{sd} = 249 + 208 = 457\ kN$, being the first value related to the resistant bending moments and the second value related to the quasi-permanent load. The adopted inclination of the compression struts is θ=30°, and results in a transversal reinforcement of $A_{sw}/s = 24.3\ cm^2/m$. Since the reinforcement must not be higher than 200mm apart and must be arranged within the effective width, this results in 8 legs of $A_{sw}/s/r = 3.03\ cm^2/m$. Thus, stirrups of Ø8//0.125 (4.02cm2/m) are adopted.

Figure 5.10 illustrates the detailing of the transverse reinforcement within the effective width.

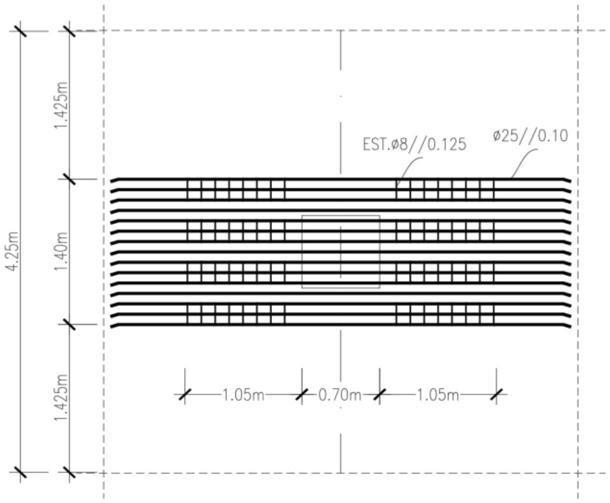

Figure 11 – Plan of the transverse reinforcement detailing.

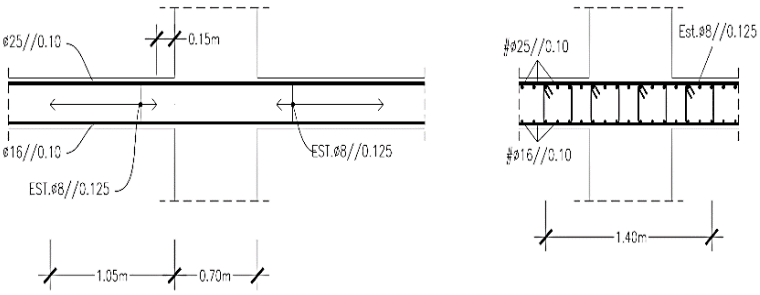

Figure 10 - Cross-section of the transverse reinforcement detailing. Left: Longitudinal; Right: Transversal.

10. CONCLUSIONS

It was verified that the seismic action combination is the most significant, comparatively to the gravity loads condition. Additionally, it was found that, in some seismic zones and for buildings classified as importance class IV, the longitudinal reinforcement ratios, in the effective width, would be so high that its detailing would be unfeasible. This high longitudinal reinforcement ratio results from the requirement established in E.T.05/2020 [2] that imposes the placement of all the reinforcement that resists the bending moments transmitted from slabs to columns within the effective width, in order to guarantee a frame effect. It is concluded, therefore, that this requirement is a very demanding factor for structures of class of importance IV, located in areas of high seismicity. Regarding transverse reinforcement and confining reinforcement, it was found that, in general, is sufficient to ensure a good ductility of the slab-column connecting zone.

The table 12 summarizes the feasibility of the detailing for each of the structures analyzed, for each importance class and seismic zone.

Table 12 - Feasibility of the detailing solutions.

<table>
<tr><th colspan="2">Structure</th><th colspan="2">1 (30%)</th><th colspan="2">2 (50%)</th><th colspan="2">3 (60%)</th></tr>
<tr><th colspan="2">Importance Class</th><th>II</th><th>IV</th><th>II</th><th>IV</th><th>II</th><th>IV</th></tr>
<tr><td rowspan="5">Seismic Zone</td><td>1.1</td><td rowspan="5">Feas.</td><td rowspan="2">Unfeas.</td><td rowspan="5">Feas.</td><td rowspan="2">Unfeas.</td><td>Avoid</td><td rowspan="2">Unfeas.</td></tr>
<tr><td>1.2</td><td rowspan="4">Feas.</td></tr>
<tr><td>1.3</td><td>Avoid</td><td>Avoid</td><td>Avoid</td></tr>
<tr><td>1.4</td><td rowspan="2">Feas.</td><td rowspan="2">Feas.</td><td rowspan="2">Feas.</td></tr>
<tr><td>1.5</td></tr>
</table>

Where green is feasible, yellow is advisable to avoid and red is unfeasible detailing.

It can be concluded that this design methodology, which allows the exploration of flat-slab systems in the resistance to the seismic action, establishes some highly demanding requirements for the design of these structural solutions in zones of high seismicity.

Although research has been carried out regarding the behavior of flat-slab systems to the seismic actions, it can be seen that there is not yet an adequate knowledge of the behavior of this type of structures. It is suggested, for future researches, the study of the following topics:

- the evaluation of the shear capacity of the column-slab connection under the effect of cyclic actions that develop large plastic deformations;
- the ductility of the column-slab connection with and without transverse reinforcement;
- the behavior evaluation of the system with medium and thick slabs, since a relevant part of published studies refer to thin slabs that hardly reflect the behavior of slabs used in service buildings;
- the development of design methodologies based on the results obtained from previous studies.

11. REFERENCES

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