Seismic Design of Precast Concrete Industrial Buildings

Nuno Filipe Barata Feliciano

Department of Civil Engineering of Instituto Superior Técnico, August 2011

Abstract: The precast industry is growing in Portugal. However, in other countries, due to construction and design faults, the behaviour of these structures, when subjected to seismic events has been weak. To counter the negative association of precast structures to poor seismic behaviour, in France, CERIB produced a backup manual for the design of these structures under seismic actions, according to EN1998 (EC8). Nevertheless, in Portugal, there is no similar document. Therefore, this document intends to clarify some aspects and question others related to the design methodology of precast concrete industrial structures to seismic actions. In order to do that, a survey of common damages of precast structures due to seismic actions has been performed, some of the more relevant design clauses of CERIB's manual are reviewed and the design of a case study is presented. The case study structure analyzed in here represents a hypothetical typical precast concrete industrial building, in Lisbon area, Portugal, which serves as an exemplification of the design clauses, both for the overall structure, structural elements and connection between these.

Keywords: *Precast, Seismic Analysis, EN1998, CERIB*

1. Introduction

Precast concrete structures in Portugal are mostly used for commercial or industrial buildings, like warehouses, commercial spaces, logistic buildings, and so on. However, it would be interesting to expand the percentage of usage of these buildings in housing and business sectors. To do so, it will be necessary to correct some limitations in their seismic design and behaviour.

The existing framework for the design of precast structures to seismic actions is under developed and leaves room for errors and incorrect considerations.

Therefore, the main objective of this paper consists in the presentation of the design procedures adopted by EC8 and approach new methodologies and important considerations for this purpose.

To fulfill the objective, in a first phase, it was necessary to understand the most important problems that this kind of structure presents when subjected to seismic actions. These problems are mostly related to the damage of the critical zone in columns (column base), to the damage in the connections between structural elements, namely, the beam-to-column and column-to-footing connections, and to the damage in the connection of structural elements with non-structural elements: [1], [2] and [3].

The actual code for seismic design of structures was also studied – EN1998, or Eurocode 8 (EC8) [4]. In addition to this Eurocode, the manual of the *Centre d'Etudes et de Recherces de l'Industrie du Béton* (CERIB): *Guide de Vérification et de Dimensionnemment des Ossatures en Eléments Industrialisés en Béton pour leur Résistance au Séisme* [5] was also studied.

A case-study was used here in order to approach the most relevant and dubious issues on the design of precast structures to seismic actions. This case-study is a hypothetical precast structure in Portugal.

In the end, this document is completed with a summary of the final conclusions and an approach of future investigations in order to fulfill some aspects not developed in this study.

2. Seismic Analysis and Design of Precast Structures

This section presents the most important aspects of analysis and design approached by this thesis.

Beginning by the analysis, EC8 presents multiple methodologies of analysis: lateral force method of analysis, modal response spectrum analysis and non-linear methods, like non-linear static (pushover) analysis and non-linear time history (dynamic) analysis.

However, this work will be based in a modal response spectrum analysis and compared with the results obtained through a lateral force method, used in CERIB's manual.

To begin with, it will be necessary to define the seismic action to consider as acting in the structure. This action should be simulated through a design spectrum for elastic analysis. For horizontal seismic actions, the design spectrum is defined by the expressions 3.13 to 3.16 of EC8.

In order to define this spectrum, the location of the structure, the structural system, the conditions of regularity in plan and in elevation, the importance class of the structure, among others, should also be known.

With all these aspects defined, it is possible to compute another essential parameter for the definition of the design response spectrum: the behaviour factor, *q*. This parameter is used to reduce the forces obtained from a linear analysis, in order to take into account the non-linear behaviour of the structure, due to the formation of ductile mechanisms.

This factor should be further modified based on the type of connections between precast structural elements. EC8 states that this factor should be reduced by k_p factor, which is function of the energy dissipation capacity of the precast structure. The value of k_p depends on the design of the connections adopted in the structure. If these connections are not designed as stated in EC8's articles 5.11.2.1.1 to 5.11.2.1.3, k_p=0,5. Otherwise, k_p=1,0.

CERIB's manual, however, presents different behaviour factors than the ones presented by EC8. On the manual, this factor is defined only according to the type of precast structure, neglecting regularity in plan and elevation, as well as the structural system. In here, the behaviour factor varies only with the existence, or not, of a rigid element in the roof, producing a behaviour of rigid diaphragm, and the existence, or not, of an intermediate floor in the structure (mezzanine).

EC8 also states that, in the case of inexistent rigid diaphragm behaviour in the floors and roof, deformability of these elements should be taken into account. This way, EC8 also considers, indirectly, the unfavorable effect of this situation in the seismic design.

Unlike EC8, CERIB's manual approaches a new element. In here, the behaviour factor is reduced or amplified according to elements' production and construction quality controls.

Once vertical seismic actions were considered in this study, it is necessary to define the vertical design response spectrum. This spectrum is defined according to EC8's 3.2.2.5.(5) article.

Once the seismic action was considered to be acting in all three directions, the directional combination should be performed according to EC8's 4.3.3.5.2.(4) article. This paper presents different directional combinations as the square root of sum of squares (SRSS) and the consideration of 30% of the seismic action in two directions and 100% in the other, among others.

In order to carry out a modal response spectrum analysis it is necessary to define which vibration modes should be considered. All the vibration modes that participate, significantly, in the general response of the structure should be considered. This aspect is guaranteed by the consideration of all vibration modes with which the sum of the mass participation is equal, or above, 90% of the total mass of the structure, and all the vibration modes with effective modal mass above 5% of the same total mass of the structure.

Seismic actions resulting of these vibration modes can be considered by combining the different modes of vibration with SRSS combination, if these modes are independent (EC8's 4.15 expression) or with complete quadratic combination (CQC) if not.

With this analysis methodology, it is possible to consider accidental torsional effects, due to asimmetry in the location of masses, through equivalent torsional moments, applied on each floor and roof. This moment depends on the accidental eccentricity and the horizontal force on each floor and roof.

With the seismic action defined, it is possible to calculate the effects on every structural element and proceed with the design of these.

The design of structural elements of precast structures is no different than the design of these elements on cast *in-situ* strucures. The main difference between these two kinds of structures is the design of the connections between structural elements. Once connections are discontinuity zones, it is indispensable to guarantee the good behaviour of these, even under seismic actions.

Once the columns are subjected to bi-axial bending, EC8 presents a simplified method of analysis that considers uni-axial bending but with the resistant bending moment reduced by 30% (EC8's 5.4.3.2.1.(2) article)

The design of the beams won't be addressed here, once these are simply supported elements.

The design of the footings is performed for the effects obtained from EC8's 4.4.2.6 article, which presents a overstrength factor, γ_{Rd}, and a coefficient Ω. This way, it is possible to magnify the actions' effects considered in the footings and include a *capacity design* consideration in these elements.

The design of the foundation beams can be performed in a similar to that of a normal beam, due to inexistent reference to the design of these elements on EC8. The design effects of these beams were obtained from EC8's 4.4.2.6 article. However, the considered axial force should be different from zero. Therefore, axial force was obtained from the article 5.4.1.2.(6) of EC8, part 5.

Once connections between structural elements in a precast structure are different from any other structure, the approach to these connections will be more careful.

The beam-to-column connection can be accomplished by a short corbel with bolts connecting the beam to the column. Between the beam and the surface of the column, a bearing pad should be placed in order to prevent a contact between these elements when a relative rotation between them should happen.

Therefore, for the design of the short corbel, it will be necessary to consider a strut-and-tie model, based on the effects from the beam to the short corbel and from this to the column. It will be necessary to place reinforcement in the ties and guarantee the resistance of the concrete in the struts.

Bolts should be design to resist to shear and tension forces. Shear forces are a result of the axial and shear forces of the beam. Tension forces are a result of the torsion and the moment of overturning of the beam, in addition to its other shear force. The existence of this overturning moment is due to the difference in elevation of the short corbel and the axis of the beam. When the beam is subject to seismic actions, a inertia force, eccentric to the short corbel, should occur, resulting in this overturning moment of the beam.

CERIB's manual presents expression (1) for the design of the bolt, where N refers to tension forces (positive if it is a tension) and G to shear forces.

$$\frac{3G+N}{A} < f_{yk} \quad (1)$$

However, there are two shear forces acting on the bolts. Therefore, G was considered equal to the SRSS of these two shear forces.

The thickness of the bearing pad existing between the beam and the short corbel is obtained from the expression (2), proposed by CERIB's manual. This thickness depends on the relative rotation between the beam and the column and on the depth of the short corbel.

$$e_p \geq máx\left(\frac{a}{2}\theta; 5mm\right) \quad (2)$$

The relative rotation between the beam and the column, θ_d, is calculated according to expression (3), proposed by CERIB's manual.

$$\theta = \gamma_{Rd}(q-1)\frac{F_{Eh}}{LK_{\Delta}} + \frac{F_{Eh}L^2}{2R} \quad (3)$$

Another aspect that CERIB's manual addresses and there is no references in EC8 is the need for embrace the bolts in all the penetration height, between the corbel and the beam. This aspect is necessary once the bolts are subject to shear forces that could detach the surrounding concrete and jeopardize the beam-to-column connection.

The other type of connection addressed in this thesis is the column-to-foundation connection, generally guaranteed by a socket system. This system guarantees a fixed connection, through the injection of the space between the foundation and the column with mortar or other type of binding agent.

CERIB's manual presents a different design methodology for the footing and the socket system. As previously stated, the foundation, below the socket, should be designed for the effects calculated as described in article 4.4.2.6 of EC8. However, the socket should be designed according to the type of connection considered for the precast structure.

Lúcio, V. [6] propose a different approach to this connection. The same author indicates that the set of footing+socket can be designed as one through a strut-and-tie model. This system guarantees a link between the effects considered on the footing and on the socket.

Both connections studied in this thesis were design as being overdesigned connections (EC8's 5.11.2.1.2 article), in other words, they were design according to the capacity design.

Hereafter, the case-study, the results and some conclusion will be presented.

3. Case-study

In order to conclude this thesis, it was necessary to use a case-study, in order to analyze and study the current regulation to the design of precast structures to seismic actions.

3.1.Description of the structure

On a first stage, an hypothetical precast structure was defined, with usual dimensions and characteristics in Portugal.

This structure has the objective to shelter a commercial space, with an area, in plan, equal to 60x66m^2.

There are three types of columns. The ones from now on

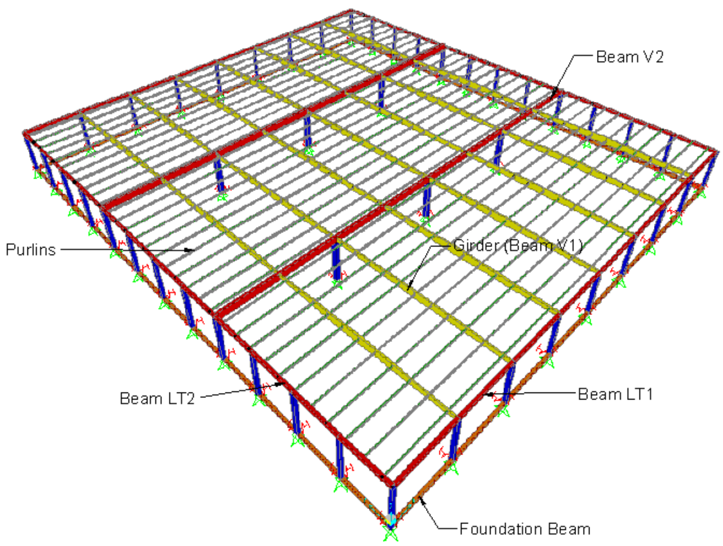

Figure 1: General aspect of the precast structure in study

identified as columns P1, with a cross section of 0,40x0,40m^2, are located on the façade of the structure. P2 columns are also located on the façade of the structure, but have a cross section of 0,45x0,45m^2 and are supporting V2 beams. Finally, columns P3 present a 0,60x0,60m^2 cross section and are located inside the structure, also supporting V2 beams.

The long façades of the structure have 13 columns, 5,5m away from each other. The other façades have 9 columns, 7,5m away from each other.

Figure 1 presents the general aspect of this structure.

Table 1 presents the dimensions of all the beams existing in the structure.

Table 1: Beam's dimensions

Beam	Dimensions (m)
Girder (Beam V1)	0,20x(0,45-1,00)
Beam V2	0,30x0,90
Beam LT1/LT2	0,30x0,45
Foundation Beam	0,30x0,45

Over the girders (V1 beam), purlins will be placed. These elements will support the roof, made of a metal sheet and connected to these purlins through metallic clamps. Due to the variation in girders height, the roof will have two 6,0% slopes.

Foundations will be made of a system of footing+socket. There will also be foundation beams, connecting all the façade's columns, as advised on EC8.

3.2. Modeling

In order to analyse the structure, a three-dimensional model was built in SAP2000, v.14. This software allows modal response spectrum analyses, which are relatively common in Portugal design procedures.

Beams and columns were modeled through *frame* type elements with the intended material cross section.

Foundations were simulated with a pinned support and two rotational springs, one for each horizontal direction. These springs are used to simulate the soil flexibility. Foundation beams were connected to this pinned supports and were modeled through a *frame* type element, with the intended material and cross section.

The spring's elasticity coefficient must be defined in the same time as the foundation's dimensions, once these two aspects are directly related. Therefore, these springs were calculated through an iterative process, once these springs will alter the effects on the foundations, which will, on the other hand, alter the dimensions of the foundations and the springs themselves.

To complete the structural model, it was necessary to consider some simplifications, including the consideration of the roof as being horizontal, the reduction of the inertia moments and shear areas on concrete elements to 50%, in order to take into account the effect of concrete cracking, as indicated in EC8, and the implementation of linear loads on the purlins, in order to simulate the effect of the weight of the roof.

3.3. Seismic action's definition

To establish the seismic action is necessary to define some aspects, like the structure's location and the regularity in plan and in elevation characteristics, as well as the structural system considered.

In order to do that, the location of the structure was considered to be in the area of Lisbon. This way, the seismic zones considered should be 1.3 and 2.3.

A terrain type C was adopted for this study: "Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of meters".

The other necessary aspect to define the seismic action is the importance class of the structure. The structure considered in this study will be used to support a commercial building and may be crowded. However, the importance class number III does not considered this type of structures. Therefore, this structure will be considered to belonging to importance class number II.

With all these characteristics defined, it is now possible to identify all the necessary elements to the definition of the horizontal elastic response spectrum. Table 2 presents these elements for 1.3 and 2.3 seismic actions (interpolate and intraplate earthquake scenarios, respectively).

Table 2: Fundamental elements to define the horizontal elastic response spectrum for seismic actions 1.3 and 2.3

Seisme	ag (m/s^2)	S	Tb (s)	Tc (s)	Td (s)	q	βag
1.3	1,5	1,5	0,1	0,6	2	1,5	0,3
2.3	1,7	1,46	0,1	0,25	2	1,5	0,34

The design spectrum is obtained from the elastic response spectrum, affected by the behaviour factor. This factor depends on the structural system and the conditions of regularity in plan and elevation of the structure.

This structure can be defined as a frame system or an inverted pendulum system. Considering that there is more than 50% of the total mass of the structure on the top third of its height, and there are some columns that are not connected along both horizontal directions, this structure can be considered to correspond to an inverted pendulum structural system. Therefore, the basic value of the behaviour factor, q_0, should be considered as equal to 1,5.

Once this structure does not have structural walls, the behaviour factor will be equal to its basic value, 1,5. Now, it is necessary to classify the conditions in regularity in plan and in elevation and find out if they will alter this value or not.

Despite the fact that the regularity in plan does not alter, directly, the behaviour factor, there is one condition necessary to verify the regularity in plan that, in case it is not fulfilled, will alter the structural system considered to a torsionally flexible structural system.

The regularity in elevation is straightforward, since there are neither setbacks nor discontinuities in all the structure's height.

The regularity in plan is a more complex process and implies the demonstration of a rigid diaphragm behaviour in the roof.

EC8 states that a floor or roof may be considered as possessing a rigid diaphragm behaviour if the displacement of different points of these elements, in a real situation, does not differ more than 10% from the displacements registered when a rigid diaphragm behaviour is imposed on those elements (article's 4.3.1.(4) Note).

In order to verify this condition, a rigid diaphragm behaviour was imposed in the roof and the displacements of various points of it were registered. These values will be used to compare with the values obtained from the real situation, properly braced.

The braced system imposed in this structure consists on the implementation of circular steel cables, connecting the top of various columns. These cables tend to guarantee an equal displacement of the top of the columns connected by them and, therefore, a greater uniformity in all the columns' top displacements.

Figure 2 presents the cables' arrangement that leads to a braced situation complying with EC8's statements.

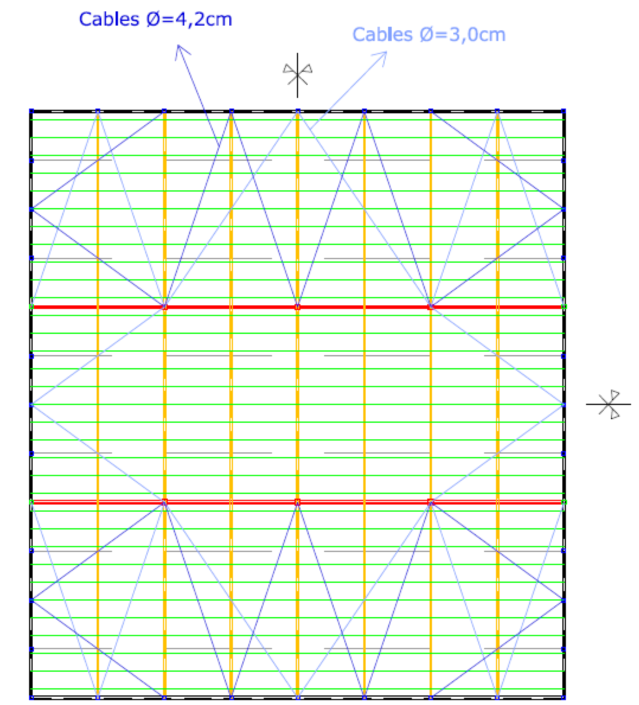

Figure 2: Cables' arrangement connecting some columns' tops

The greater difference between a point on the roof in the real situation and this same point in the imposed rigid diaphragm situation is below 7,05%, less than 10%, as intended. With the guarantee of a rigid diaphragm behaviour in the roof, the conditions in regularity in plan may now be evaluated.

All the regularity in plan conditions are easy to define, with the exception of EC8's expression 4.1b, which will dictate if this structure may be considered as possessing a torsionally flexible structural system or an inverted pendulum one.

Therefore, to calculate the torsional radius by x and y, r_x e r_y, it will be necessary to calculate the torsional stiffness, K_θ, and the lateral stiffness by x and y, K_x e K_y. In order to do so, a force, with an arbitrary value, was imposed through x and y, and a moment through the z axis. Based on the displacements and rotation obtained from this forces and moment, it is now possible to calculate the values of the torsional radius, being enough for the purpose apply a square root to the ratio of the torsional stiffness to the lateral stiffness in the direction perpendicular to the one pretended to calculate the torsional radius.

The radius of gyration of the floor mass in plan, Is, is obtained directly from the dimensions of the structure in plan.

The obtained values for these three quantities are presented in Table 3.

Table 3: Torsional radius r_x and r_y and radius of gyration of the floor mass in plan, l_S

rx(m)=	31,36
ry(m)=	31,14
ls(m)=	25,75

According to the values presented in Table 3, it is now possible to verify that EC8's condition 4.1b is checked and, with it, the regularity in plan. With this condition verified, the structural system previously identified as an inverted pendulum is correct.

Once this is a precast structure, it is necessary to guarantee that the connections between structural elements are designed in accordance to, in this case, EC8's 5.11.2.1.2 article, in order to use a behaviour factor equal to 1,5. Otherwise, a reduction coefficient, k_p=0,5 must be applied, in order to take into account a less energy dissipation capacity of the precast structure.

The seismic actions in x and y directions were combined with the help of the software SAP2000. The combination used was the SRSS, in order to simplify the number of analysis required.

3.4. Modal response spectrum analysis

This modal response spectrum analysis was executed by the software SAP2000, v.14. However, it is necessary to know the vibration modes' mass participations before defining, correctly, this analysis.

Executing a modal response spectrum analysis, it is possible to verify that, in the 200th vibration mode, with a period of, approximately, 0,038s, there's a sum of modal mass in z axis of only 73,3%, below the 90% stated by EC8. The sum of the modal mass in x and y directions are equal to the total mass of the structure so that, for these directions, the prior condition is verified.

EC8's also states that the last vibration mode must have a period below 0,20s. However, and once this is an automatic calculation, all the vibration modes with a period above 0,10s were considered. This way, 75 vibration modes were considered

Table 4 presents the most relevant values of this modal response spectrum analysis.

Summarizing, there are three types of vibration that is important to underline. The first ones, and most relevant, are the translations in the two horizontal directions, x and y. These vibration modes are the most relevant because they present the higher vibration periods (approximately 0,86s). The second most relevant vibration mode is the vibration of the structure in the z axis, in other words, the vibration of the roof's structure, due to girders and purlins connected with each other and supported by the columns. This vibration mode appears next to the translation vibration modes and has periods around 0,81s. The last relevant vibration mode is the torsion of the structure. This vibration mode appears in the 7^{th} position, which corroborates the conclusion that this structure does not have a torsionally flexible system. The period of this vibration mode is around 0,78s.

This 75 vibration modes considered were combined through CQC, once these modes can't be considered as independent of each other.

3.5. Design

In this section the most relevant aspects on the design of some structural elements and the connections between them will be presented. Thus, the section will be divided into two sub-sections with the designation *Design of structural elements (§3.5.1)* and *design of connections (§3.5.2).*

3.5.1.Design of structural elements

The structural elements chosen to present their design in this thesis were the columns and the foundation beams. Roof beams, once they are simply supported on their ends, won't have bending moments near the supports on the columns. Therefore, it won't be necessary to design these elements with EC8.

Since the objective is to present the design's methodology of structural elements, only some elements were design, as example.

In relation to the columns, P2 were chosen to illustrate these elements' design. These columns were chosen because they are located in the façade of the structure, therefore subjected to higher effects, and because they support V2 beams.

To begin with the design according to EC8, the bi-axial bending state that these columns are subjected to was considered through the simplified method presented by EC8. Based on the single bending moment effects, for each direction, separately, and with the resistant bending moment reduced by 30%, longitudinal reinforcement was calculated. The obtained value for longitudinal reinforcement must be displaced in all four faces of the section of the column, in order to guarantee an equal resistance in all four directions of bending. Summarizing, the adopted reinforcement was 3 - 25mm thickness bars, and 2 - 16mm thickness bars on each face of the column. The final resistant uni-axial bending moment will be M_{Rd}=334,79kNm, approximately.

In relation to the shear force, the column must be design in accordance to *capacity design*.

In order to do so, based on the resistant bending moment value, guarantee by the longitudinal reinforcement adopted, the design shear force must be obtained. Once the column is fixes on the base and simply supported on top, the design shear force must be obtained from expression (4):

$$V_{Ed} = \frac{\gamma_{Rd} M_{Rd}}{L_{Pilar}} \qquad (4)$$

Table 4: Modal masses and sums of 200 vibration modes of the structure

Mode	Period (seg.)	Modal Mass X	Modal Mass Y	Modal Mass Z	Sum X	Sum Y	Sum Z	Rotation Z
1	0,863	0,000	0,997	0,000	0,000	0,997	0,000	0,328
2	0,862	0,996	0,000	0,000	0,996	0,997	0,000	0,397
3	0,807	0,000	0,000	0,301	0,996	0,997	0,301	0,000
4	0,807	0,000	0,000	0,000	0,996	0,997	0,301	0,000
5	0,807	0,000	0,000	0,001	0,996	0,997	0,302	0,000
6	0,806	0,000	0,000	0,000	0,996	0,997	0,302	0,000
7	0,780	0,000	0,000	0,000	0,996	0,997	0,302	0,266
8	0,759	0,000	0,000	0,000	0,996	0,997	0,302	0,000
9	0,759	0,000	0,000	0,000	0,996	0,997	0,302	0,000
10	0,759	0,000	0,000	0,000	0,996	0,997	0,302	0,000
11	0,759	0,000	0,000	0,000	0,996	0,997	0,302	0,000
12	0,732	0,000	0,000	0,006	0,996	0,997	0,308	0,000
…								
15	0,732	0,000	0,000	0,000	0,996	0,997	0,514	0,000
…								
30	0,355	0,000	0,000	0,000	0,997	0,997	0,546	0,000
…								
50	0,218	0,000	0,000	0,000	1,000	1,000	0,582	0,000
…								
74	0,126	0,000	0,000	0,000	1,000	1,000	0,616	0,000
75	0,120	0,000	0,000	0,000	1,000	1,000	0,616	0,000
76	0,094	0,000	0,000	0,001	1,000	1,000	0,616	0,000
…								
100	0,088	0,000	0,000	0,000	1,000	1,000	0,638	0,000
…								
200	0,038	0,000	0,000	0,000	1,000	1,000	0,733	0,000

The value of γ_{Rd} must be equal to 1,1, in accordance to EC8's 5.4.2.3.(2) article. Based on these values, and once the height of the column, L_{pilar}, is equal to the height of the structure, h=5,8m, the design value of the shear force will be V_{Ed}=63,5kN.

To define the transverse reinforcement to place in the critical zone of the column (near its base) it is necessary to first calculate the length of the critical zone through EC8's 5.14 expression. The critical height of the column will be, from the top of the foundation, approximately, equal to 1m. Therefore, in this height, it is necessary to adopt confining hoops so that the ductility needed for a plastic hinge to resist is guaranteed.

In accordance to EC8's 5.4.3.2.2.(11) b) article, two hoops must be placed in order to guarantee that the maximum distance between two consecutive braced longitudinal bars is less than 200mm.

The maximum separation of hoops in this critical zone is calculated through EC8's 5.4.3.2.2.(11) a) article, and is equal to 100mm.

According to EC8, if the column has a reduced axial force below 0,2, and the behaviour factor is less than 2,0, as it is in this case, the transverse reinforcement in this critical zone could be calculated with EC2, in other words, it won't be necessary to guarantee the ductility, according to EC8, for this critical zone. Nevertheless, the mechanical volumetric ratio of confining hoops within the critical regions, ω_{wd}, will be calculated.

The value for ω_{wd}, obtained according to EC8, is negative, what corroborates the no need to adopt confining hoops in all the critical height of the column.

The minimum value of transverse confining reinforcement, according to EC8, is less than the value of this reinforcement calculated according to EC2. So, the hoops in this critical region have an 8mm diameter.

Concluding, two hoops of 8mm in diameter, with 100mm away from each other, have been placed in all the critical height of the column P2.

On the rest of column P2's height, the transverse reinforcement adopted was calculated according to EC2, resulting in 2 bars with 300mm away from each other and 8mm in diameter.

It would be interesting to incorporate the effect of the eccentricity between the axis of the column and the support of V2 beam in the design of the columns, what would cause an additional bending moment in the column. However, and once the seismic effects on the column are, by itself, relatively weighty, this effect was neglected in the design of these columns.

With all of these, the design of P2 columns is concluded.

Foundation beams were designed according to the principles for primary beams, in general, as indicated in EC8's 5.8.1.(3)P article. Despite these are not roof or floor beams, these foundation beams present a similar behaviour as the first ones.

Once there are two foundation beams, ones in the direction of the larger horizontal dimension of the structure and other in the perpendicular direction, the design presented here applies to the beams with the highest effects. Therefore, the foundation beams arranged in the smaller horizontal dimension of the structure are the ones whose design is presented here.

The effects used in the design of these beams were obtained from EC8's 4.4.2.6.(4) article:

$$E_{Fd} = E_{F,G} + \gamma_{Rd} \Omega E_{F,E} \quad (5)$$

where $E_{F,G}$ represents the effects due to permanent loads, γ_{Rd} is the overstrength factor, Ω is the value of $R_{di}/E_{di} \leq q$, where R_{di} is the design resistance of the element *i* and E_{di} is the design value of the action effect on the element *i* in the seismic design situation and $E_{F,E}$ represents the action effect from the analysis of the design seismic action

Once the foundation beams are connecting various vertical elements (columns), Ω is obtained from EC8's 4.4.2.6.(8) article and is equal to $\Omega \approx 1,438$.

This way, the design value of the bending moment will be $M_{Ed} \approx 38,3$kNm. According to this value, the longitudinal reinforcement is less than the minimum value, considered as 0,4% of the foundation beam's cross section, to be placed both in the upper and lower faces of the beam. Therefore, the minimum value of longitudinal reinforcement will be 5,4cm^2. Three bars with 16mm in diameter were placed in both the upper and lower faces of the beam, resulting in a total longitudinal reinforcement of 6,03cm^2. This reinforcement guarantees e resistant bending moment of M_{Rd}=73,06kNm.

The design shear force is calculated in a similar way as in the columns' design. Expression (6) was used to obtain this value.

$$V_{ed} = \frac{2M_{rd}}{L} \quad (6)$$

With L being the distance between two consecutive columns corresponding in the shorter façade, to 7,5m.

Once the overstrength factor was already considered in the computation of the design bending moment, it will not be considered in this expression.

The design shear force obtained from expression (6) was V_{Ed}=19,48kN.

The transverse reinforcement obtained from this value of shear force is below the minimum value. Thus, bars with 8mm in diameter, away from each other 250mm, were adopted. Despite a lower diameter for the transverse reinforcement was enough to exceed the minimum value, a higher diameter was chosen because these elements will be underground and, therefore, are subjected to more aggressive environmental conditions.

The maximum compressions on the concrete struts were also verified and were found to be extremely loose.

The axial force to be considered in these foundation beams is an aspect that is important to be underlined. Although the axial force calculated by SAP2000 is null, EC8's part 5 - article 5.4.1.2.(6) presents a methodology to obtain an axial force different from zero, to be considered in the design. According to this article, the axial force to be considered in the design of these beams is equal to N_{Ed}=12,87kN, both in tension and compression.

For this value of axial force, either in compression or tension, the resistance of the beam is more than enough to resist. Noted that for this axial force acting as tension, the resistance of the concrete is enough to resist to such force.

With all these aspects verified, the design of the foundation beam is concluded.

3.5.2. Design of connections

There are two types of connections approached in this thesis: the connection between column-beam and between column-foundation.

In relation to the connection column-beam, once column P2 was the one chosen to demonstrate the process of column design, the column-beam connection chosen was between column P2 and beam V2. This beam is connected to the short corbel through bolts and supported by a bearing pad.

To begin with the connection design, it is necessary to define which forces must be considered in the design. The most important ones are the shear forces in the bolts, corresponding to the axial force of beam V2 and one of the shear forces of this same beam, the torsional moment and the turnover moment of the beam and the other shear force of beam V2, which will produce a compression in the short corbel.

The last force is fundamental for the design of the short corbel. The other ones are necessary to design the bolts.

Once the three-dimensional model does not count with the eccentricity between V2 beam's axis and the short corbel, it is necessary to define the inertial force of beam V2 in order to calculate the overturning moment to be considered in the connection's design.

In order to do so, EC8's 4.3.5.2.(2) article approaches non-structural elements and the inertial forces that must be considered to be acting on them. However, EC8's 4.24 expression (expression (7)) can be used to calculate the inertial force in beam V2, even though this is a structural element.

$$F_a = (S_a W_a \gamma_a)/q_a \qquad (7)$$

To take into account that this beam is a structural element, the behaviour factor of this element, q_a, was considered equal to the behaviour factor of the whole structure and, in the definition of the seismic coefficient applicable to non-structural elements, S_a (EC8's 4.25 expression), the beam's fundamental period of vibration, T_a, was considered null, once this beam is rigidly connected to the structure.

The inertial force, or overturning force, obtained for beam V2 was 36,86kN. Once the lever arm of this force is equal to half height of the beam V2, where its axle is located, the overturning moment was equal to 16,59kNm.

With this overturning moment defined, all forces necessary to the design of this connection are known.

Starting with the bolt's design, according to CERIB's manual, it is necessary to define the shear force, G, and axial force, N, in these bolts, the number of them and their spacing.

Considering a 2 bolts connection, 16cm apart from each other, the axial force is obtained through the vector sum of half compression from beam V2, in the short corbel, with the axial force resultant of the sum of the turnover moment and the torsional moment of beam V2. Adopting the simplified rectangular diagram method, and considering that a torsional collapse mechanism is fragile by nature, an overstrength factor γ_{Rd}=1,2 was considered. With all this considerations, the axial force obtained for each bolt was 74,04kN.

The bolt's design shear force was equal to 27,49kN, being affected by a overstrength factor γ_{Rd}=1,2 as well.

According to this axial and shear forces previously obtained, and the expression (1) of this document, the minimum section of the bolts, for a steel class A500NR, will be equal to 3,13 cm^2. Therefore, a 25mm in diameter bolt was adopted guaranteeing an area equal to 4,9 cm^2, above the minimum value: 3,13 cm^2.

CERIB's manual refers to another aspect that is not approached in EC8: the anchorage length for the bolts, both in the beam and in the short corbel and column. This length must be obtained for the sum of G+N.

This anchorage length can be obtained from expression (8):

$$G + N \le f_{bd} \pi \phi L_{amarr} \qquad (8)$$

The value of f_{bd} was calculated according to REBAP's 80th article. Considering that the penetration holes of the bolts were filled with grout, the anchorage length obtained for this case was 1,05m. However, this value can be much lower if a stronger binding agent is used to fill the bolt's holes.

The corbel's design was performed according to REBAP's 135th and 140th articles.

The thickness of the bearing pad is defined through expression (2), presented in CERIB's manual.

The relative rotation of beam V2 and column P2 can be calculated through expression (3) of this document, and it is approximately equal to 0,01269rad.

Therefore, the minimum thickness for the bearing pad will be 0,005m.

With all these values defined, the design of the connection beam V2-column P2 is concluded.

To begin with the design of the connection between the column P2 and its foundation, it is necessary to consider some simplifications.

Once EC8 presents two distinct methodologies for the footings and the connection itself, it is necessary to guarantee some kind of continuity between them because the reinforcement is defined, simultaneously, for both of them, through a strut-and-tie model.

Another important aspect is the connection of the foundation beam with this system. Once the insertion of the beam in this system would difficult the strut-and-tie model's definition, it was chosen to connect this beam in the column, directly above the socket.

Starting with the footings, their dimensions were designed with the effects obtained through EC8's 4.30 expression (expression (9) of this document). Note that, although this expression allows to take into account a design according to the *capacity design*, with a behaviour factor below 3, as it is in this case, it is not possible to consider it, once γ_{Rd}=1,0.

Since it is supposed to guarantee the continuity between the footings and the sockets, the effects obtained with this expression will be used only on the definition of the footings' dimensions and, consequently, in the definition of the rotational springs, to be applied on the three-dimensional model.

For the considering footing, the obtained dimensions were 3x3x1m^3. These dimensions were obtained from the effects on the pinned supports of the three-dimensional model, added with the footing's own weight and altering them with expression (9).

Table 5 presents the footing's dimensions' design forces considered:

Table 5: Footing's dimensions' design forces

Ned=	429,23kN
Med,x=	320,14kNm
Med,y=	350,96kNm

The effects obtained this way guarantee, for the support area of the footing, a maximum tension in the soil below the maximum tension allowed for it, equal to 400kPa.

The reinforcement's design of the set footing+socket were executed according to the strut-and-tie model proposed by Lúcio, V.[6]. This model also includes a strut-and-tie model in the entire column's height inside the socket, in order to consider the column's action on the socket and vice-versa.

Considering that both the bending moment and the shear force depend on the resistant bending moment of the column (EC8's 5.11.2.1.2 article), only the axial force will be different between the column's and footing+socket set's strut-and-tie models.

The axial force to be considered on the design of the column's zone inside the socket is the one obtained from the base of the respective column on the three-dimensional model. The axial force to be considered in the footing+socket set is the one obtained from the pinned support in the three-dimensional model, added with the own weight of the footing.

Table 6 presents the design values of the column's height inside the socket and the set footing+socket.

Table 6: Effects to be considered on the design of the column's height inside the socket and the foundations

Effects	Pillar	Foundation
N [kN]=	185,67	204,23
Footing's own weight [kN]=	0	225
Ned [kN]=	185,67	429,23
Med [kNm]=	401,75	401,75
Ved [kN]=	69,27	69,27

According to the expression for the computation of the column's penetration in the socket length, proposed by Lúcio, V.[6], the obtained value was around 0,62m. Therefore, a height of L=0,7m for the socket was considered.

The thickness on the upper part of the socket must be, at least, equal to half height of the socket. With this consideration, an upper thickness of 0,40m was used for the socket.

EC2 stipulates an upper and lower limit for the struts' steepness in the strut-and-tie models. Therefore, theoretically, there wouldn't be any transversal tie in the column's height inside the socket.

The only needed reinforcement in the column's height inside the socket is a longitudinal one and corresponds to, approximately, 20,71cm^2 which can be accomplished with an extra bar of 16mm in diameter in the entire column's penetration's height. It is extremely important to guarantee that this extra bar does not overlaps the height L=0,7m because if this happens, the resistant bending moment above the socket (critical zone of the column) is higher than the one considered on the design. This fact can lead to a shear collapse mechanism (fragile) on the column's base, which should be avoided.

The reinforcement to be used on the footing+socket set is obtained from another strut-and-tie model.

The first step should be to verify, with the considered effects, if the maximum compression on the soil continues to be below the maximum allowed. Based on the axial force (with the footing's own weight added) and the bending moment obtained from the pinned support of the three-dimensional model, the eccentricity was 0,936m, which leads to a maximum compression of 338kPa, below the maximum 400kPa.

With this aspect verified, the next step should be to determine the forces on the struts and the ties on the strut-and-tie footing+socket set's model proposed by Lúcio, V.[6].

Another important aspect on the design of this connection is the punching shear. To verify this aspect, EC2 presents the article 6.4.

For the case-study presented here, once the axial force is relatively low, the minimum punching shear's resistant tension was used as comparison. This minimum punching shear's resistant tension depends only on the concrete's class and the footing's, the column's and the control perimeter's dimensions.

This way, the maximum punching shear tension is 0,02408MPa and the minimum resistant punching shear tension is 0,312MPa, much higher than the previous one. Therefore, this aspect is automatically verified.

With all these aspects verified, the designs of these connections are concluded.

4. Conclusions

Nowadays, the design of precast structures to seismic actions is performed according to EC8, which presents, also, some aspects on the design of precast concrete structures. However, there are many aspects that are inexistent or poorly developed.

The importance class of a precast structure used to support a commercial building is not properly defined. It should be considered as belonging to the class II. However, a commercial building is often crowded and, therefore, could be considered as belonging to the importance class III.

The case-study presented in this document was classified as possessing an inverted pendulum structural behaviour because there are some columns that are not connect through the two horizontal directions. However, this classification leads to a decrease in the behaviour factor, which, by itself, leads to an increase in the design effects. It would be interesting to investigate if the fact of guaranteeing a rigid diaphragm behaviour on the roof could lead to a different structural behaviour than an inverted pendulum type. Noted that, in this case, CERIB's manual presents a higher behaviour factor than the one considered by EC8.

EC8 and CERIB's manual have little information about how to impose a rigid diaphragm behaviour on the roof and floors.

Another dubious aspect of EC8 is the design of the foundations. When the foundation type adopted in the structure is a set of footing+socket, there is a problem of compatibility between the effects to be considered on the design of the footings and the socket, because EC8 presents two different methodologies to obtain these effects.

Note that CERIB's manual approaches many aspects that are neglected by EC8, as the existence of a judiciousness of the basic behaviour factor due to quality control on the manufacture and construction phases, a special basic behaviour factor for each type of precast structures, an expression to obtain the relative rotation between beams and columns and another one to obtain the thickness of the bearing pad.

Based on these conclusions, it would be interesting to develop more cautious studies referring to the classification of the importance class of these structures, the identification of the structural system and the consequent determination of precast structures' behaviour factor.

Another aspect of extreme importance is the correct design of column-foundation connection. According to the available elements of EC8 and CERIB's manual, the design of a connection footing+socket type is badly developed and leaves room for mistaken assumptions.

The existence of a foundation beam on this structure indicates that the effects that these elements are subjected to are minimum. Therefore, it would be interesting to evaluate how effective is their adoption. Another important aspect is to verify if the design presented here for these elements are the correct, once EC8 does not present a specific design procedure for foundation beams, since it presents expression 4.30 for the computation of the design effects but also refers, on article 5.4.2.2, that these beams should be designed according to *capacity design*. It is obvious that expression 4.30 takes into account this *capacity design*, once it considers an overstrength factor, γ_{Rd}. However, on this case, since $q<3$, this overstrength factor is equal to one, and, therefore, a *capacity design* is not considered.

In the end, it is assumed that is necessary and of extreme utility to elaborate a manual, similar to CERIB's manual, with which Portuguese engineers could use to design precast structures according to EC8. Nevertheless, contrary to CERIB's manual, the proposed manual should be based in a modal response spectrum analysis and not in a lateral forces analysis, once this last one is no longer common in Portugal.

5. References

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