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Deformations in the strut of two pile caps

Deformações nas diagonais comprimidas em blocos sobre duas estacas


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Abstract

There are several models for the design of pile caps, one of them, is the strut and tie model. Struts are graphical representations of the flows of the compression principal stress, while, the tie represents the flows of the tensile principal stress. The NBR 6118:2003 [1] recommends the use of strut and tie model for the design and verification of pile caps. The use of simple strut and tie models is much used by engineers. Basically, the model consists in determining steel areas based on the traction tie and verifying the crushing of the concrete in the inferior and superior nodal zones (the interface between the inferior face of the pile cap and the pile is the inferior nodal zone and, the interface between the superior face of the pile cap and the column, is the superior nodal zone). The strut integrity is guaranteed when the stress intensities in the superior and inferior nodal zones are lower than the limit value. This limit value is determined in function of the mechanical properties of the concrete pile cap and the stresses in the strut are determined in function of the internal forces and the area of the inferior and superior nodal zones. The correct form of the diagonal compression inside the pile cap is unknown. In function of the experimental analysis of the principal compression strain, the geometric form of the strut to two pile caps submitted the centered and eccentric load is shown along with the intensity of the stress in the inferior nodal zone.

Keywords: Reinforcement concrete, strut and tie model, experimental analysis, pile caps.

Resumo

Existem vários modelos para o dimensionamento de blocos sobre estacas, sendo que um deles é o modelo de escoras (bielas) e tirantes. Escoras são representações gráficas dos fluxos das tensões principais de compressão de um dado elemento, enquanto que os tirantes representam os fluxos das tensões principais de tração deste. A NBR 6118:2003 [1] recomenda a utilização de modelos de escoras e tirantes para o dimensionamento e verificação de elementos estruturais de volume, idéia que se aplica aos blocos sobre estacas. Os conceitos de modelos de escoras e tirantes são simples e muito utilizados na análise de estruturas de concreto. Basicamente, o modelo consiste em determinar às áreas das barras de aço da armadura principal de tração por meio das tensões de tração obtidas para o tirante e verificar a ruptura do concreto da diagonal comprimida junto à zona nodal inferior e superior (entende-se por zona nodal superior e inferior as interfaces pilar-bloco e estaca-bloco respectivamente). A integridade da diagonal comprimida é garantida quando as intensidades das tensões nas zonas nodais superior e inferior são menores que um valor último. Esse valor último é determinado em função das propriedades mecânicas do concreto do bloco e as tensões nas zonas nodais inferior e superior são determinadas em função da força resultante das tensões atuantes na diagonal comprimida e de suas áreas. A forma correta da diagonal comprimida no interior do bloco é desconhecida. Por meio de uma análise experimental obtiveram-se as deformações principais de compressão e a forma geométrica das diagonais comprimidas em blocos sobre duas estacas submetidos à ação de forças centrada e excêntrica, como também as intensidades das tensões atuantes na zona nodal inferior.

Palavras-chave: Concreto armado, modelo de escoras e tirante; análise experimental; blocos sobre estacas.

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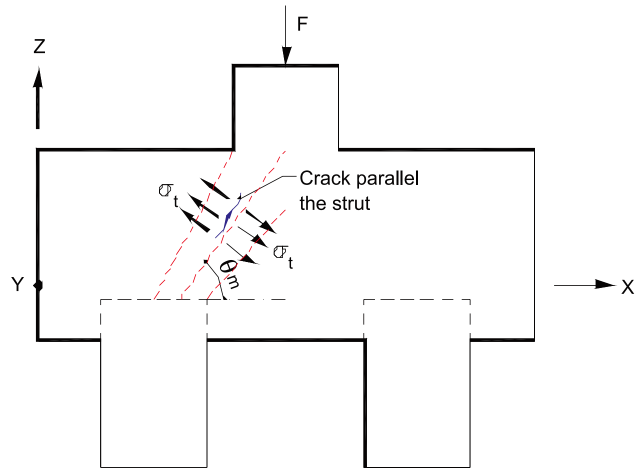
1. Introduction

The foundation type for a certain construction is chosen after the study that considers the technical and economical conditions of the work. Through the knowledge of the soil parameters, the stress intensity, the neighboring buildings and the types of foundations available on the market, the engineer can choose the best alternative following technical and economical criteria.
The pile foundations are adopted when the soil in its superficial layers is not capable to support stresses originating from of the superstructure; therefore it's necessary to look for strength in deep layers. When it is necessary, the pile foundation is indispensable in the construction of another structural element, the pile caps.
Pile caps are important structural elements whose function is to transfer the stresses of the superstructure to a group of piles. Despite its importance to the safety of the superstructure, these structural elements don't usually allow the visual inspection when in service, so it's important to know the behavior in relation to the ultimate limit state and the serviceability limit state.
The developed researches regarding this theme in the last years have concentrated on two types of analysis: the linear theory of elasticity consisting of the strut and tie model and the beam theory and the analysis of the experimental results. The beam theory is used in the flexible pile caps and the strut and tie models in the rigid pile caps. It is evidenced that the previous knowledge of its dimensions is necessary for the design and verification of these structural elements.
The structural behavior of pile caps can be defined through the strut and tie models due to the discontinuous areas, where the Bernoulli hypotheses are not valid. In the strut and tie model the compression verifications in the strut are similar to the Model of Blévot & Frémy [2], however the stress in the nodal zones have values, which are different from the stress limits suggested by Blévot. The Model Code of CEB-FIP [3] suggests geometries for the nodal zone, enabling therefore the stress verifications in these areas.
The strut and tie model can be adopted considering the stress flow in the structure, using the process of the minimum load, suggested by Schlaich & Schafer [4]. These stresses can be obtained through

Figure 1 – Cracks provoked by tensile stress perpendicular to the strut, splitting, Delalibera (14)



linear-elastic analysis, using numerical methods, for example the Finite Element Method.
According to NBR 6118:2003 [1], "pile caps are volume structures used to transfer to the pile the loads of the superstructure for the foundation ", in other words, all the external dimensions have the same order of greatness. They are treated as special structural elements, that don't respect the hypothesis of the plane sections that remain plane after the deformation, because it isn't sufficiently long to dissipate the located disturbances. NBR 6118:2003 [1] classifies the structural behavior of the pile caps as rigid or flexible. In the case of rigid pile caps the structural model adopted for the design can be three-dimensional, linear or not, and three-dimensional strut and tie model. The strut and tie models are preferred for defining the stress distribution better in the struts and ties.
The uncertainty in the geometric form of the stress flow that form the strut in pile caps, submitted to centered and eccentric loading, give this

Figure 2 – Strut and tie model proposed by Adebar et al. (6)

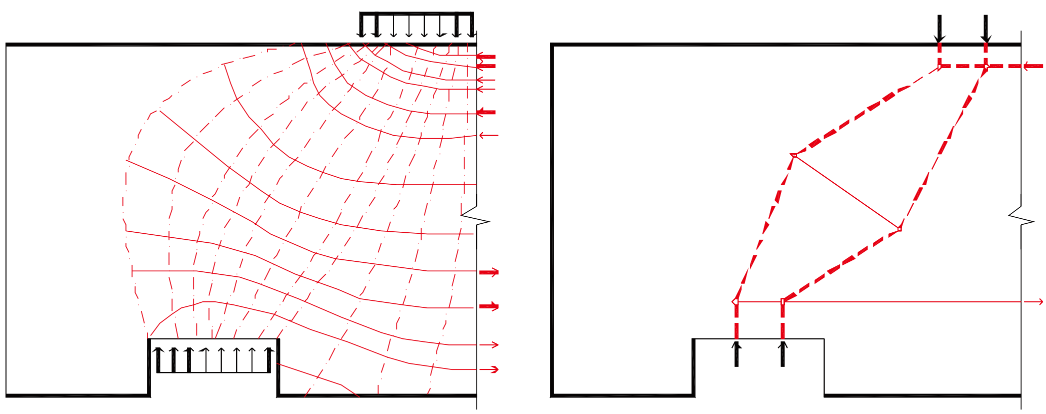
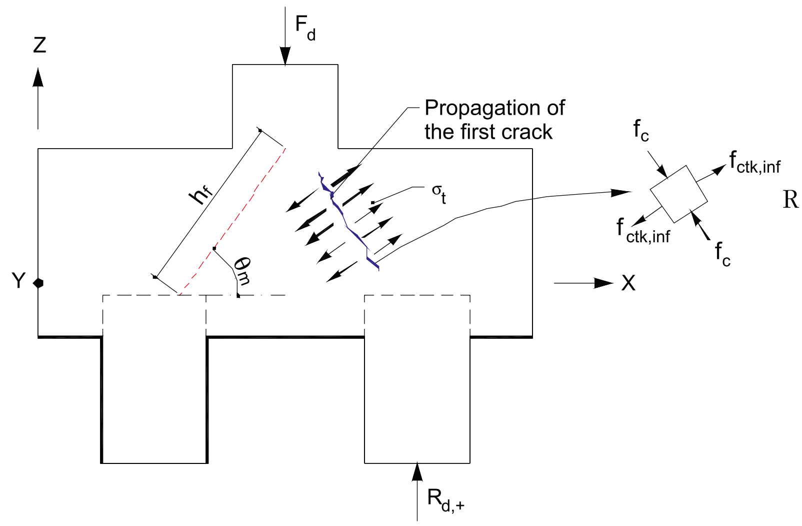


Figure 3 – Minimum Load splitting, Delalibera & Giongo (9)



$$R_{ct,mín} = \frac{2}{3}.h_f.h_y.f_{ctk,inf} \geq (0{,}15).R_{d,+}$$

$$A_{sf,mín} = \frac{R_{ct,mín}}{f_{yd}}$$

article the opportunity to contribute to the scientific society, because with the knowledge in the strut, the compression stress flow developed in the pile caps can be analyzed with better accuracy, therefore improving the structural project and establishing criteria for the verification of the stress in the struts. Besides, the hypotheses adopted by NBR 6118:2003 [1]: "in the case of rigid pile caps, with spacing of 2,5.ø to 3.ø (where ø is the diameter of the pile), it can be admitted plane (it means that the cross section of the head of the pile is stressed uniformly) the load distribution in the pile"; and " for flexible pile caps or extreme cases of short piles, with pile supported in very rigid substratum, the previous hypothesis needs to be reviewed"; need to be studied, because in analysis through numeric models of rigid pile caps it was observed that the load distribution in the pile is not uniform, being necessary, therefore, the adaptation of the used hypotheses.

Based on the experimental results, other strut geometry is suggested with base on the distribution of the principal compression stress flow and on the collapse forms observed during the tests.

1.1 Research significance

In function of the existent divergences in the analytic methods used for the design of pile caps it was decided to study the structural behavior of these elements. In this work, fourteen pile caps with several reinforcements were tested. The objective was to analyze the deformations in the superior and inferior nodal zones, to determine the most stressed regions in the interface pile-block.

2. Method for design

The NBR 6118:2003 [1], version of 2004, doesn't specify any criterion for the design of pile caps, however, it indicates the use of the strut and tie model, that represents the internal structural behavior of the caps better. The strut and tie model applied to the design of pile caps use the criteria developed by Blévot & Frémy [2]. The inclination angle of the strut is limited between 45º to 55º and the stress verifications are proceeded in the nodal zones in relation to the limit values of the concrete compression stress for these areas. The necessary amount of tie bars is calculated through the static balance of the truss, maids starting from the application of the strut and tie model. These bars of steel, that represent the tie in strut and tie model, are positioned in the inferior face of the cap, above the piles. Finally, the verifications of the anchorages of these bars are carried out.

Another criterion much used in the technical society for the design of pile caps is the method proposed by the Technical Bulletin Number 78 of CEB [5]. The reinforcements are calculated in function of the internal balance of normal stresses in the specified cross section. The verification of the crushing of the strut is done in an indirect way, the values of shell forces are verified and comparing these values with the values of limit shell stresses in two specified sections. Finally, the anchorages of the tie are verified, in an area delimited based on the diameter of the pile.

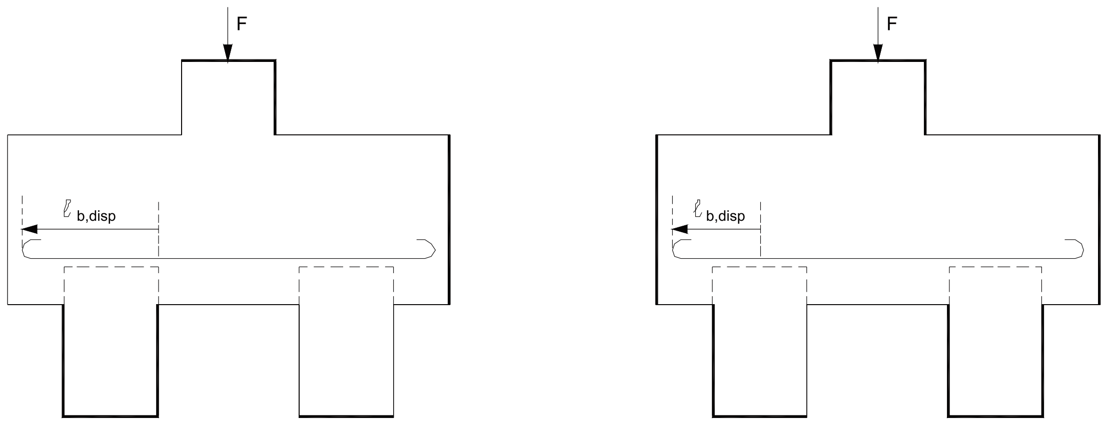
What is evident in the previous analysis of the two methods for the design of the caps is that the tensile stress along the strut is not considered. These tensile stresses provoke fissures parallel to the axis of the stress, reducing, therefore, the concrete compression strength of that area, see Figure [1].

Adebar et al. [6] carried out experimental tests of four and six pile caps presenting important contributions in relation to the design of pile caps. One of them is the use of diagonal tie; see Figure [2], positioned perpendicular to the strut. The Canadian Code CSA Standard A23.3-94 [7] incorporated in its text some of the indications suggested by Adebar et al. [6].

The Spanish Code EHE [8] presents reinforcement similar to those suggested by Adebar et al. [6] for pile caps submitted to load of "great magnitudes", because the existent tensile stress in the pile caps can be important.

Delalibera & Giongo [9] with base on experimental results propose a criterion for the design of minimum splinting reinforcement, see Figure [3]. This reinforcement is composed by a diagonal tie positioned perpendicular to the strut.

Figure 4 – Measure of the anchorage length, Delalibera & Giongo (15)



a – hypothesis I   b – hypothesis II

According to NBR 6118:20031, the basic anchorage length is the straight length of a bar necessary to anchor the ultimate load Rst = $A_{st} \cdot f_{yd}$ in this bar, admitting along the length uniform adherence. The NBR 6118:20031 doesn't present indication for the position of the cross section at the beginning of the anchorage in the cases of bars in pile caps.

Some authors suggest that the basic anchorage length is measured starting from the geometric center of the pile (Hypotheses I – Figure [4a]), while others suggest that this measure is initiated from the farthest side of the pile in relation to the edge of the cap (Hypotheses II – Figure [4b]).

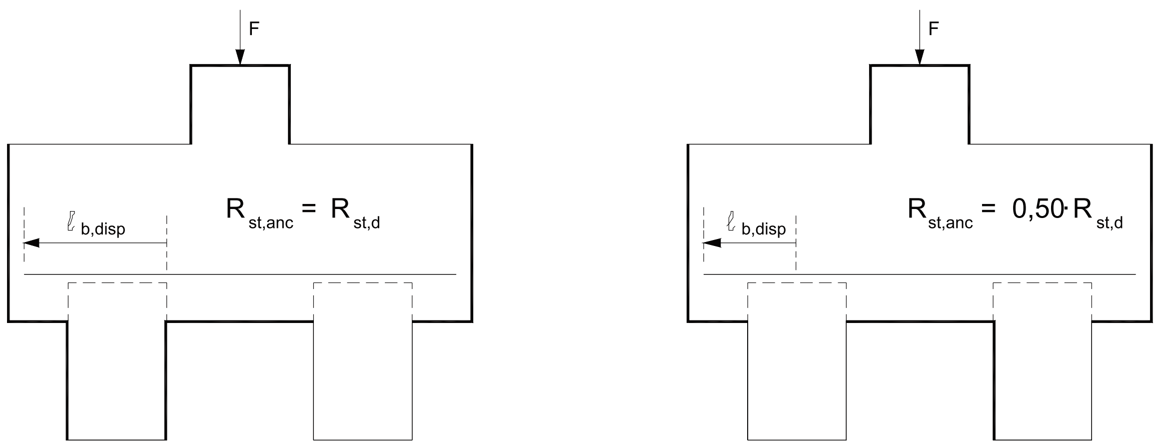
Besides, there is a divergence in the calculation of the force to anchor. It is known that based on the favorable action of the strut compression stress, the value of the force to anchor can decrease. Fusco [10] suggests that this reduction is twenty percent. Taylor & Clarke [11] suggest that the reduction is a function of the compression stress of the reinforcement rate of the tie and of the concrete compressive strength. Results of Adebar et al. [6], Miguel et al. [12], Iturrioz et al. [13] and Delalibera [14] show that the force in the tie is not constant, presenting abrupt reduction in the region of the pile. Besides, it was verified that the deformation in the extremity of the tie (and in some models, in the hook) was equal to zero.

Delalibera & Giongo [15], with base on experimental results, present suggestion for the anchorage of the tie in rigid two pile caps, in function of measures of the anchorage length. Figure [5] presents the anchorage suggestions.

3. Mechanical and geometric properties of the models

Fourteen pile caps with two piles were tested; four pile caps had cross sections of the column with dimensions equal to 25 cm for 50 cm. The others had square section with side measures of 25 cm. The distance between the piles were 62,50 cm and the piles were 10 cm embedded in the block.. Figure [6] shows dimensions of the experimentally analyzed pile caps.

Figure 5 – Suggestion for anchorage of the tie, Delalibera & Giongo (15),
a) starting from the moved away face of the pile; b) starting from the geometric center of the pile



a   b

Figure 6 – Dimensions of the two pile caps

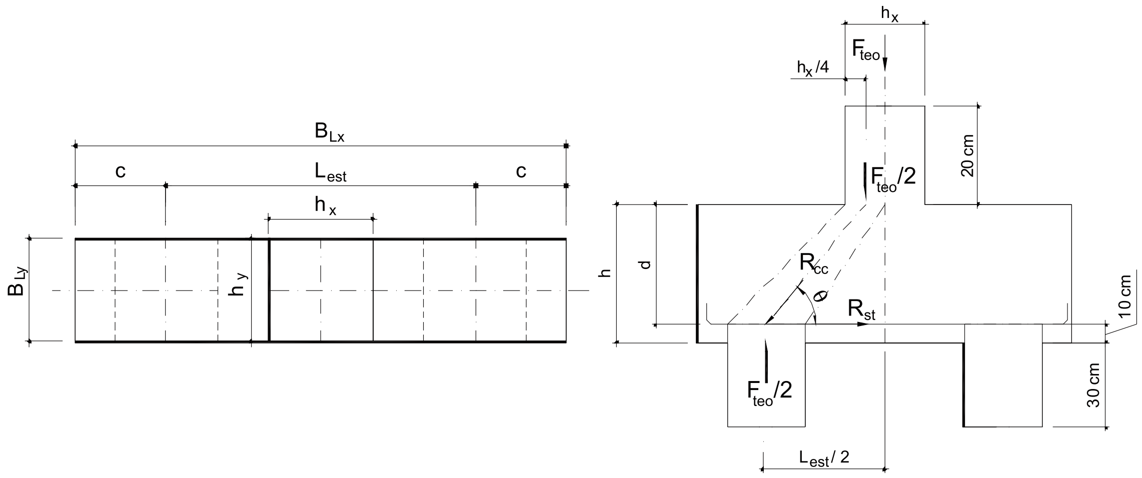
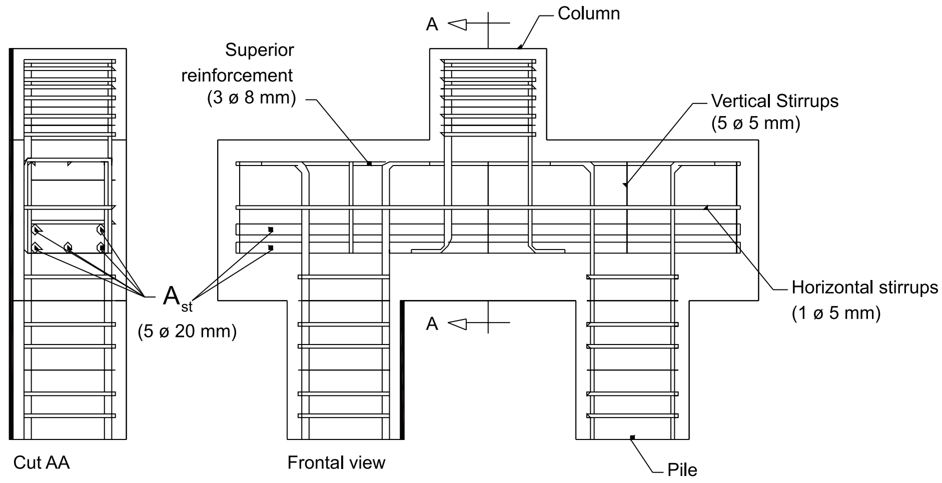


Table 1 – Geometric properties of the models analyzed experimentally

SERIES	Pile Caps	Section of the pile (cm)	Section of the column (cm)	B_{Lx} (cm)	B_{Ly} (cm)	h_x (cm)	h_y (cm)	L_{est} (cm)	c (cm)	e_{adot} (cm)	h (cm)
B35P25	B35P25E25e0	25 x 25	25 x 25	117,5	25	25	25	62,5	27,5	0	35
	B35P25E25e2,5	25 x 25	25 x 25	117,5	25	25	25	62,5	27,5	2,5	35
	B35P25E25e0Asw,C	25 x 25	25 x 25	117,5	25	25	25	62,5	27,5	0	35
	B35P25E25e0Asw,0	25 x 25	25 x 25	117,5	25	25	25	62,5	27,5	0	35
	B35P25E25e0CG	25 x 25	25 x 25	117,5	25	25	25	62,5	27,5	0	35
B45P25	B45P25E25e0	25 x 25	25 x 25	117,5	25	25	25	62,5	27,5	0	35
	B45P25E25e5	25 x 25	25 x 25	117,5	25	25	25	62,5	27,5	5,0	45
	B45P25E25e0Asw,C	25 x 25	25 x 25	117,5	25	25	25	62,5	27,5	0	45
	B45P25E25e0Asw,0	25 x 25	25 x 25	117,5	25	25	25	62,5	27,5	0	45
	B45P25E25e0CG	25 x 25	25 x 25	117,5	25	25	25	62,5	27,5	0	45
B35P50	B35P50E25e0	25 x 25	25 x 50	117,5	25	50	25	62,5	27,5	0	35
	B35P50E25e12,5	25 x 25	25 x 50	117,5	25	50	25	62,5	27,5	0	45
B45P50	B45P50E25e0	25 x 25	25 x 50	117,5	25	50	25	62,5	27,5	12,5	35
	B45P50E25e12	25 x 25	25 x 50	117,5	25	50	25	62,5	27,5	12,5	45

Notes: B_{Lx} e B_{Ly} are the lengths of the pile caps in directions x (longitudinal) e y (transversal), e_{adot} is the eccentricity of the compression load applied in the column, h is the height of the pile cap; h_x e h_y are the dimensions of the column; L_{est} is the distance among the axes of the piles; c, is the distance of the center of the pile to the border of the block.

Figure 7 – Arrangement of the models: B35P25E25e0, B35P25E25e2,5, B45P25E25e0, B45P25E25e5, B35P50E25e0, B35P50E25e12,5, B45P50E25e0 e B45P25E25e12,5



The heights of the column were adopted with cross section equal to 20 cm and the heights of the piles equal to 30 cm. This was done with the intention to make the tests possible the tests, because if these were adopted with actual lengths, the tests would be not possible in the Laboratory of Structures of the São Carlos Engineering School. In order to verify the influence of the pile length in the pile cap, Delalibera [14] carried out several numerical simulations and observed that there is a change in the stress intensities of the struts, however, its paths were maintained. More information can be obtained in Delalibera & Giongo [16].
In Table [1] are presented the geometric properties of the test models and the Figures [7], [8], [9] and [10] shows the reinforcement arrangements used in the blocks.
Fourteen pile caps with two pile were tested with variations in the arrangements of the reinforcement, the inclination angle of the strut (height of the block), the cross section of the column and in the position of the compression load (equal eccentricity and different to zero). These factors were chosen based on a statistical analysis developed firstly.
Each model had an acronym, whose meaning is presented in the example, B35P25E25e0:
– B35: pile cap with height equal the 35 cm;

Figure 8 – Arrangement of the models: B35P25E25e0$A_{sw,C}$ e B45P25E25e0$A_{sw,C}$

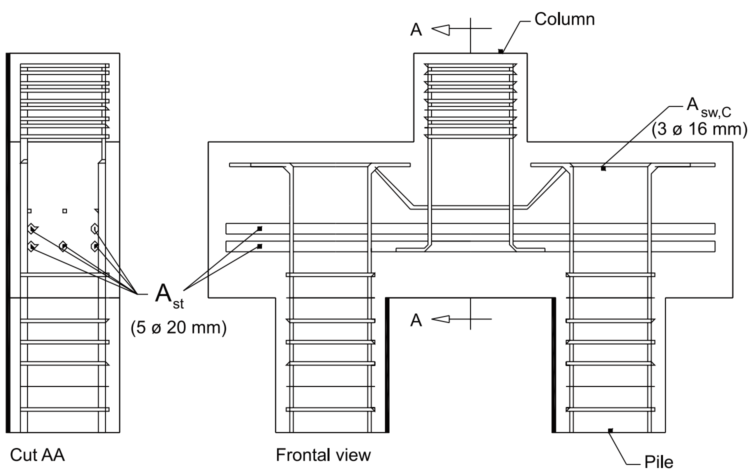
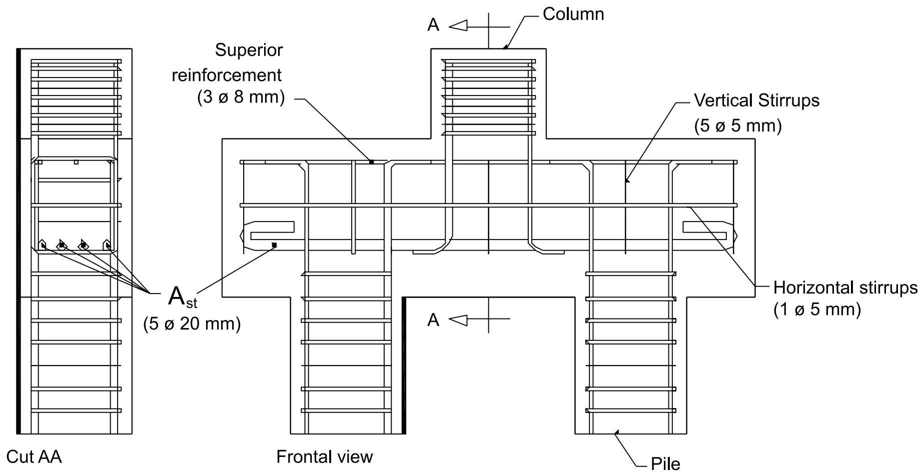


Figure 9 – Arrangement of the models: B35P25E25e0CG e B45P25E25e0CG



– P25: column with cross section equal to 25 cm x 25 cm;
– E25: piles with cross section equal to 25 cm x 25 cm;
– e0: eccentricity of compression load equals the zero.

Besides the acronyms already presented, the models still have the following denominations: $A_{sw,C}$ and Asw,0, whose meanings are: $A_{sw,C}$, area of the cross section of the traverse reinforcement of pile cap, calculated through analytic process destined to absorb the stress originating from of the diagonal tensile stress of the strut; and Asw,0, area of the traverse reinforcement of the pile caps equal the zero.

The covers of the reinforcement were: column and pile: 25 mm; pile cap: 40 mm.

The tie reinforcement (A_{st}) used five bars with diameter equal the 20 mm. The pile caps B35P25E25e0$A_{sw,C}$ e B35P25E25e0$A_{sw,C}$, consisted of three bars with diameter equal the 16 mm.

In the longitudinal reinforcement of the pile of the series B35P25 four bars were used with diameter equal the 10 mm and stirrups with diameter equal to 6.3 mm and space between them equal the 5 cm. For the piles of the series B45P25bars were used with diameter equal the 12.5 mm and space between them equal to 5 cm.

The columns of series B35P25 were built with eight bars with diameter equal to 10 mm and the column of series B45P25 with eight bars with diameter equal to 12.5 mm.

The mechanical properties of the steel bars used in the construction of

Figure 10 – Arrangement of the models: B35P25E25e0$A_{sw,0}$ e B45P25E25e0$A_{sw,0}$

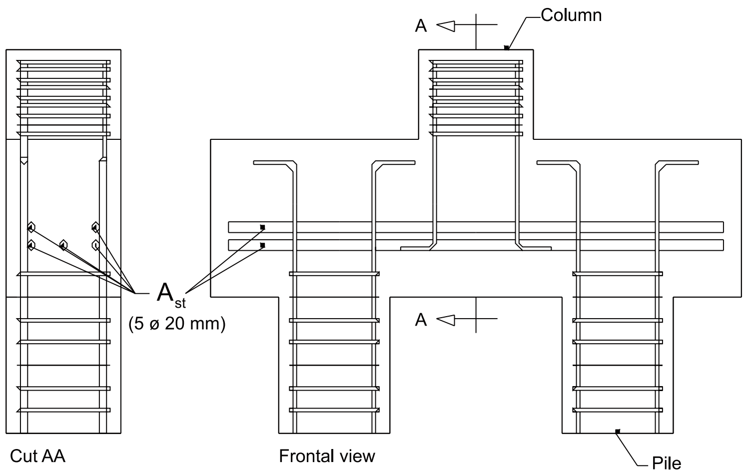


Table 2 – Properties of reinforcements				
$\varnothing_{nom}$ (mm)	f_y (MPa)	e_y (‰)	f_u (MPa)	Class
5,0	681	4,05	755	CA-60
6,3	597	2,99	733	CA-50
8,0	581	2,91	674	CA-50
10,0	549	2,75	658	CA-50
12,5	578	2,89	740	CA-50

the models are presented in Table [2], where: $\varnothing_{non}$, the nominal diameter of the reinforcement; f_y, the yield strength of reinforcement; ε_y, the tensile reinforcement strain; and f_u is the ultimate strength in reinforcement.
The module of elasticity of reinforcement was determined experimentally, presenting medium value of 200 GPa.
The concrete compressive strength of the pile and columns were larger than the concrete compressive strength of the pile caps in order to guarantee that ruins didn't take place in the piles and in the columns. The concrete compressive strength adapted to the piles and column, were equal the 50 MPa and the concrete compressive strength of the pile caps equal to 25 MPa.
The mixture in mass for concrete of 25 MPa was equal the 1:2.55:3.54:0.6, with common Portland cement (CP-II-F32). For a concrete strength equal the 50 MPa, the mixture in mass was 1:2:2.66:3.66:0.49 with cement of the high initial resistance (CP-V) and 1% (in mass) of superplasticizer.

4. Experimental investigation

The objective of the research was to obtain information about the geometric form of the strut. The face of the pile caps was scored through steel tablets with rectangular rosette form. These points served as measure points for the concrete strain. The measures of the relative displacements (deformations) among the steel tablets were obtained through mechanical strain-gage.

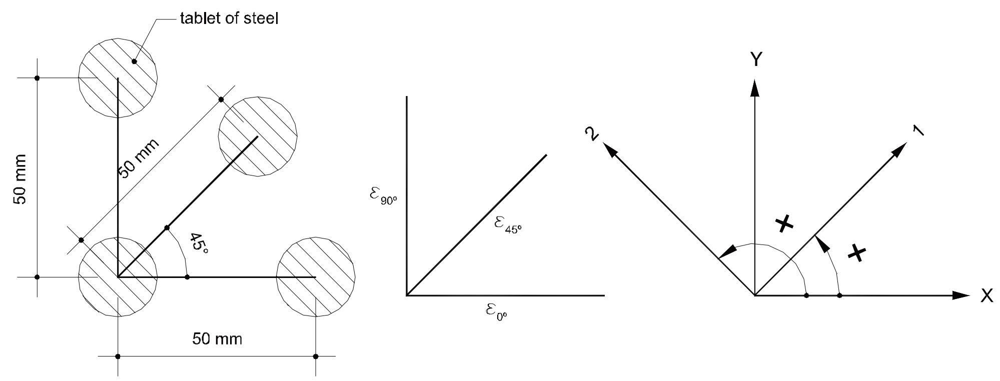
The principal strain was calculated through the Expression [1] and the principal directions were calculated with the Expression [2], with the experimental strain measured according to rectangular rosette, according to Figure [11].

$$\begin{matrix}\varepsilon_1\\ \varepsilon_2\end{matrix} = \frac{1}{2}\left[\varepsilon_{0°} + \varepsilon_{90°} \pm \sqrt{\left(\varepsilon_{0°} - \varepsilon_{90°}\right)^2 + \left(2\varepsilon_{45°} - \varepsilon_{0°} - \varepsilon_{90°}\right)^2}\right] \quad (1)$$

$$\alpha_{1,2} = \frac{1}{2}.\arctan\frac{2\varepsilon_{45°} - \varepsilon_{0°} - \varepsilon_{90°}}{\varepsilon_{0°} - \varepsilon_{90°}} \quad (2)$$

Where:
– ε_1 e ε_2, principal stress;
– $\alpha_{1,2}$, principal directions;
– $\varepsilon_{0°}$, $\varepsilon_{45°}$ e $\varepsilon_{90°}$, strain in the direction 0°, 45° e 90°.
Ten readings of relative displacements among the steel tablets were stipulated for each model. The ultimate force in each pile cap was divided in ten same parts, and, for each load increment applied through the cylindrical hydraulic, a reading was carried out in all the points. The Figures [12], [13], [14] and [15] present the positions of the points where the relative

Figure 11 – Position of the tablets of steel in the lateral faces of the pile caps

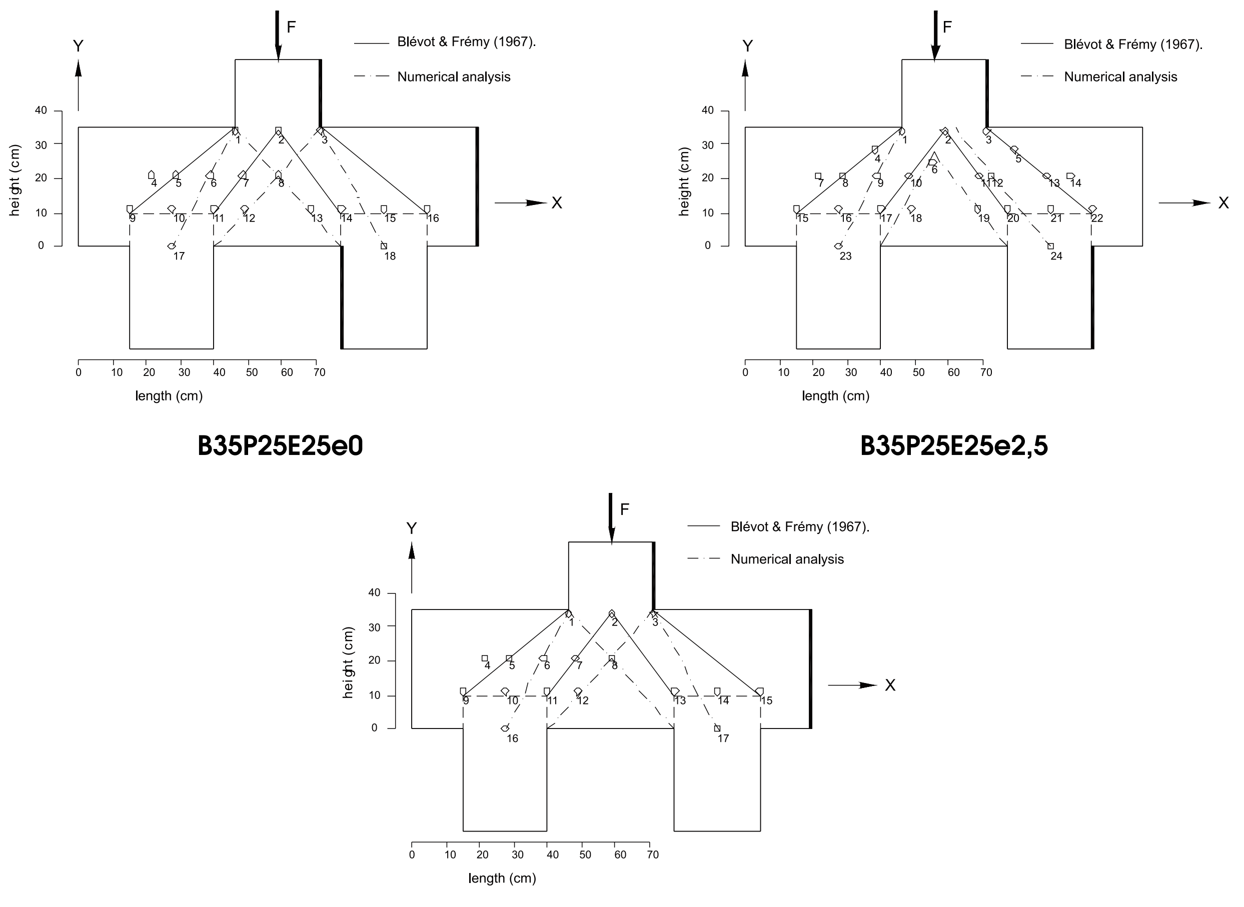


displacements measurements among the tablets of steel were carried out. In each point three measurements were made by load step. It is verified through these figures, that in the models with compression eccentric load, (symmetrical), there are less points of readings. This was done this way to reduce the reading time during the test. Figure [16] shows the tablets of steel glued on the face of the tested pile caps.
It is observed in Figure [6] and [16] that the experimentally analyzed pile caps were built without the concrete brim (dimension "c" of Figure [6], orthogonal to the longitudinal axis of the block), because it was necessary to measure the relative displacements among the tablets of steel inside the block, in other words, near to the column-block and block-pile connections.
The procedures and the equipments used in the research can be seen in Delalibera [14].

5. Obtained results

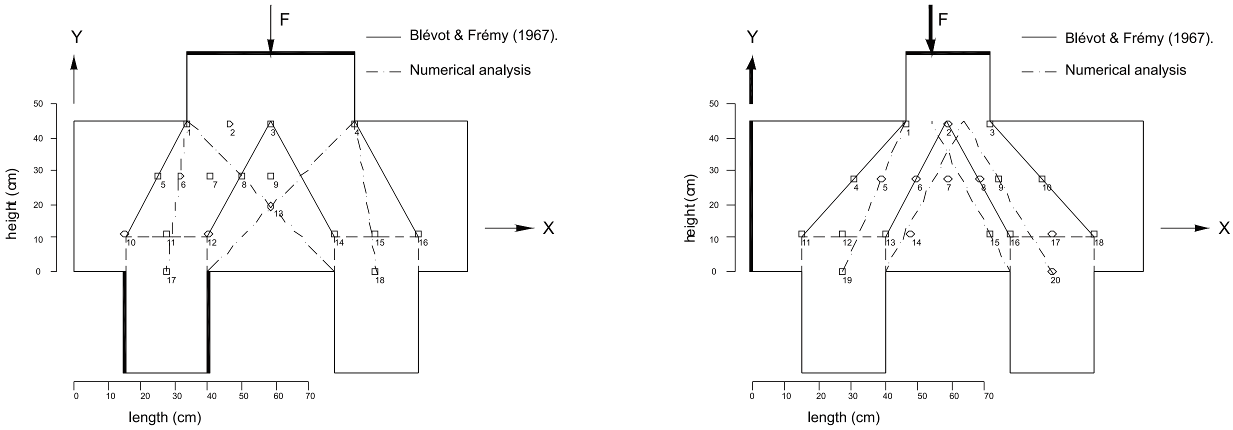
In general, all the models presented similar behavior. The first crack appeared in the inferior face of the pile cap and spread to the superior face of the block near the column (see Figure [17 - a]). The others cracks appeared along the load with similar inclination to the first crack. Despite the great openings presented by the cracks, the block only stopped resisting the applied load when the cracking process of the splitting load started, after that, the crushing of the concrete in superior nodal zone and in the models of the series B35P50 and B45P50 in the inferior nodal zone took place.
All the models presented defined collapse for splitting and crushing of the concrete, in other words, after the crushing of the concrete near the column and in some cases near the pile, a rupture plane was formed along the height of the block provoked by the shell stress (Figure [17 - b]). This phenomenon didn't occur in the models B35P25E25e0A$_{sw,C}$ and B45P25E25A$_{sw,C}$ due to the presence of splint reinforcement (seam).
Table [3] shows results of the ultimate load (Fu), the load that provoked the first crack (F_r), the theoretical load (F_{teo}) - calculated with the criteria of the French researchers [2] - ignoring the maxim tensile load suggested by Moraes [17], the design load (F_d) and the average concrete compressive strength of the pile caps.

Figure 12 – Position of the tablets of steel in the lateral faces of the pile caps of the series B35P25



B35P25E25e0A$_{sw,C}$, B35P25E25e0A$_{sw,0}$ e B35P25E25e0CG

Figure 13 – Position of the tablets of steel in the lateral faces of the pile caps of the series B45P25



B45P25E25e0, B45P25E25e0A$_{sw,C}$, B45P25E25e0A$_{sw,0}$, B45P25E25e0CG | **B45P25E25e5**

It is observed in Table [3] that the first crack appeared for a load of approximately twenty percent of the ultimate load and approximately fifty percent of the design load.
It is possible to see that the results obtained by the authors are close to the results observed in the tests of Blévot & Fremy [2] (F_{teo}/F_u = 0,87, on average), in other words, the value of 1,4·fck, relative to the compressive strength of the strut close to the column observed by the French researchers, were repeated in the tests of the series B35P25 and B45P25.
However, considering that the whole cross section of the pile was submitted by this stress is not correct, because it was observed that only part of the pile was submitted more intensely. Remembering that the value of 1,4·f_{ck} is only valid for the models where the collapses took place close to the column . For the models where the ruins of the blocks occurred near the piles (series B35P50 and B45P50), these values are reduced to f_{ck}.
Despite the fact that the elasticity theory for the study of pile caps cannot be used in function of the cracks and the heterogeneity of the concrete material, the principal strains were calculated because they are calculated just for rotation of axes in the considered point and in areas where there were not any cracks. When, a crack went by a

Figure 14 – Position of the tablets of steel in the lateral faces of the pile caps of the series B35P50

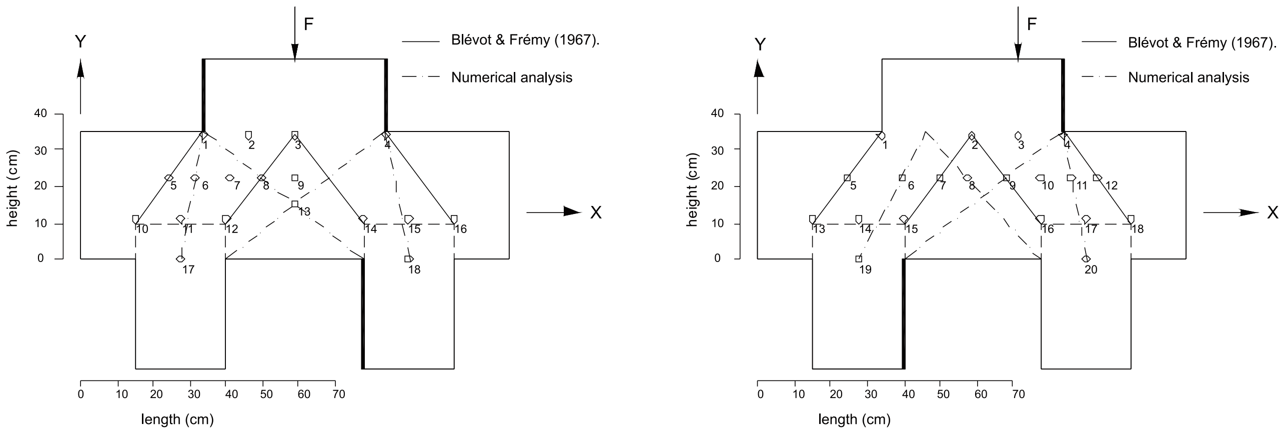
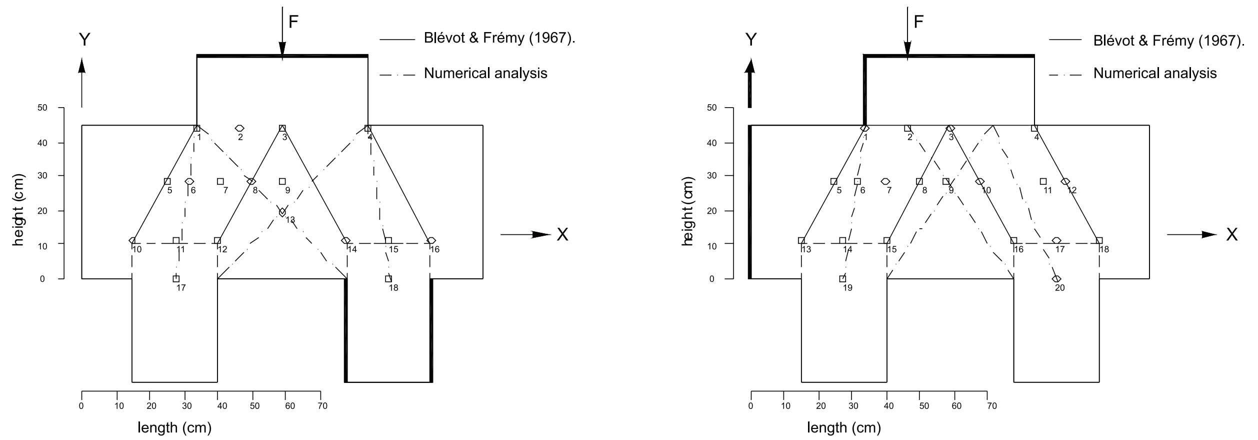


Figure 15 – Position of the tablets of steel in the lateral faces of the pile caps of the series B45P50



measure point the determination of the strain in a certain direction was affected, because there was a rupture to the tensile stress of the concrete, modifying the stress distribution and the stress paths. When this occurred, the principal tensile stresses were affected, but as the angle of inclination of the cracks was practically parallel to the direction of the principal compressive strain, the value of the strain in the concrete confronted in the direction $\varepsilon_{45°}$ was taken as reference, since the fissures didn't cross to the bases of the measures.

Through the deformation intensities and the principal directions diagrams were produced, that evidenced which areas of the pile caps had more deformations. Figure [11] shows the convention adopted for the angles of the principal strain, being positive in the counterclockwise sense measured from the axis x. Figure [18] and [19] show the diagrams of intensities of the principal compression strain in the faces of the pile caps (lines in the red color).

Through the previous diagrams the areas of the blocks, the more submitted areas are verified. It was evidenced that the pile sections closer to the extremities of the blocks had small strains, therefore, considering that the section of the pile presents the same intensities of stress is not adequate.

Figures [20] to [23] show the curves load vs. displacement of the series B35P25, B45P25, B35P50 and B45P50.

It is noticed in Figure [20] to [23] that the eccentric compression stress reduced the strength capacity of the pile caps, and that the pres-

Figure 16 – Tablets of steel fastened in the surface of the pile caps

Figure 17 – a) Position of the first crack; b) Rupture along the height of the pile cap

a b

ence of the splitting reinforcement (models B35P25E25e0Asw,C and B5P25E25Asw,C) increased its strength capacity. Even the model B35P25E25e0, that had a value very superior to the concrete compressive strength, fc = 40,6 MPa, presented the ultimate force smaller than the model B35P25E25e0Asw,C.
All the models were built with the same mixture for the concrete, however, in function of the delays of the tests of some models (B35P25E25e0 and B35P25E25e2,5) an increase of the compressive strength of concrete occurred. Table [4] presents the stress results based on the criteria of Blévot & Frémy [2], close to the superior and inferior nodal zones.

6. Discussion of the results

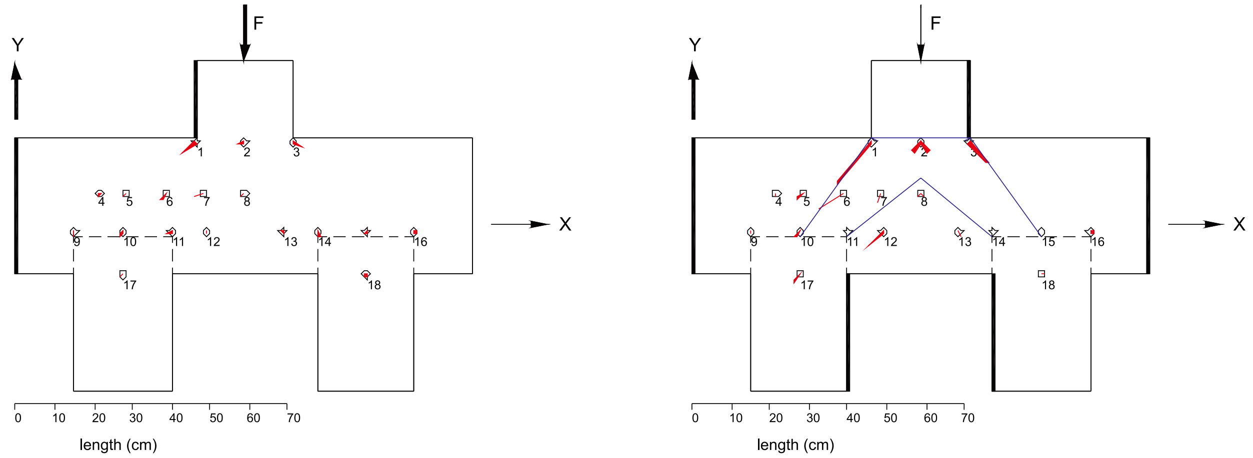
It is observed through the results that the existent stress in the experimentally tested models made some sections of the pile caps

Table 3 – Values of the ultimate load and of the first fissure

SERIES	PILE CAPS	f_{cm} (MPa)	F_u (kN)	F_r (kN)	F_{teo} (kN)	F_d (kN)	F_r/F_u	F_{teo}/F_u	F_d/F_u	F_r/F_d
B35P25	B35P25E25e0	40,6	1821	465	1776	761	0,26	0,98	0,42	0,61
	B35P25E25e2,5	40,6	1688	445	1776	761	0,26	1,05	0,45	0,58
	B35P25E25e0$A_{sw,C}$	32,8	1880	270	1435	615	0,14	0,76	0,33	0,44
	B35P25E25e0$A_{sw,0}$	32,8	1406	266	1435	615	0,19	1,02	0,44	0,43
	B35P25E25e0CG	28,9	1263	315	1264	542	0,25	1,00	0,43	0,58
B45P25	B45P25E25e0	31	2276	465	1796	770	0,20	0,79	0,34	0,60
	B45P25E25e5	31	1972	522	1796	770	0,26	0,91	0,39	0,68
	B45P25E25e0$A_{sw,C}$	32,4	3055	482	1877	805	0,16	0,61	0,26	0,60
	B45P25E25e0$A_{sw,0}$	32,4	2090	305	1877	805	0,15	0,90	0,39	0,38
	B45P25E25e0CG	28,9	2270	473	1674	718	0,21	0,74	0,32	0,66
B35P50	B35P50E25e0	35,8	3877	450	2864	1718	0,12	0,74	0,44	0,26
	B35P50E25e12,5	35,1	3202	585	2808	1685	0,18	0,88	0,53	0,35
B45P50	B45P50E25e0	35,8	4175	851	3477	2092	0,20	0,83	0,50	0,41
	B45P50E25e12,5	35,1	3386	477	3409	2045	0,14	1,01	0,60	0,23
Average values		33,8	-	-	-	-	0,19	0,87	0,42	0,49

Notes: f_{cm}, average compressive strength of concrete, obtained through cylindrical specimens of concrete; F_u, experimental ultimate load; F_r, load first crack; F_{teo}, analytic load; F_d, design load.

Figure 18 – Intensities of the principal compressive stress and geometric of the strut model, model B35P25E25e0

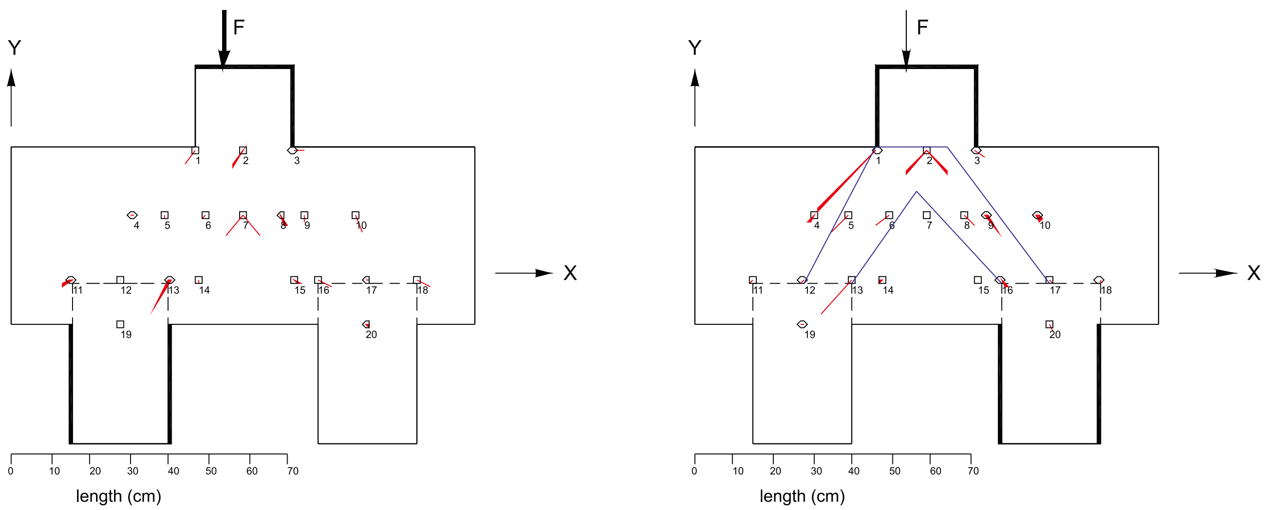


F = 435 kN, before the beginning of the first crack

F = 1225 kN, before of the rupture

present rupture of the concrete (compression-compression-compression, near to the superior nodal zone) and rupture due to tensile stress of the concrete (tensile-compression-compression, near to the inferior nodal zone). In spite of the fact that some regions of the models presented rupture, other they still remained in elastic regime, as it can be observed mainly in the inferior nodal zones, because the sections of the blocks closest to the extremities presented strain and stress of small intensity.

Figure 19 – Intensities of the principal compressive stress and geometric of the strut model, model B35P25E25e2,5



F = 290 kN, before the beginning of the first crack

F = 1260 kN, before of the rupture

Figure 20 – curve load vs. displacement, series B35P25

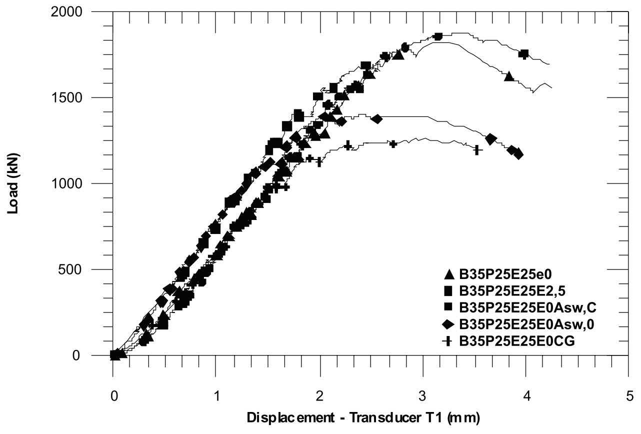
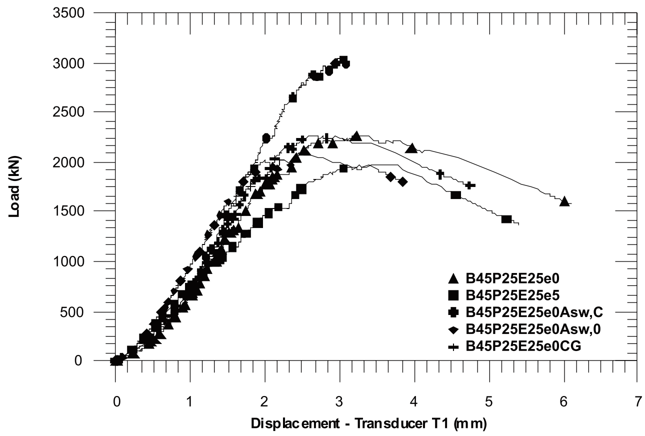


Figure 21 – curve load vs. displacement, series B45P25



The values of the effective stress (σ_{ef}) in the models were determined in function of the criteria of Blévot & Fremy [2] that considers that the area of the strut is same for the area of the whole pile in a tilted plan whose angle is 90º - θ, being θ the angle of inclination of the strut in relation to the horizontal. Those stress are described in Table [4], close to the inferior nodal zone ($\sigma_{zi,B}$) and close to the superior nodal zone ($\sigma_{zs,B}$). Most of the sections of the pile caps didn't present stress that exceeded to the plasticity stress of the concrete (f_c and f_t). The pile caps were still capable to support loads, until the collapse, when the external load produced stress higher than the plasticity stress of the material – Theory of the Inferior Limit. As reinforced concreted is studied, where the steel is responsible for absorbing to the existent tensile stress in the models, the ruptures of the models were then restricted to the crushing of the concrete and the yield of the steel bars.
In function of these results presented in Table [4] it is possible to understand why the Model Code CEB-FIP [3] restricts the value of the stress in the strut by sixty percent of the compressive strength of concrete. But, the σ_{zi} value = $0,60 \cdot [1-(f_{ck}/250)] \cdot f_{cd}$, in megapascal, presented by the Model Code is very conservative, because in the models where the rupture occurred close to the superior and inferior nodal zone simultaneously (models of the series B35P50 and B45P50), these stresses were on the average the same as the value of the strength $1,17 \cdot f_{cm}$. The average concrete strength of the pile caps for the models of these series was equal to 35,45 MPa.
Therefore, considering the reduction coefficient of concrete (γ_c) equal to 1,5, the effective stress close to the pile was determined, $\sigma_{zi,Bd}$ = 27,65 MPa. The established limit stress for CEB-FIP [3] was equal to 12,17 MPa. So, the limit value presented by CEB-FIP [3] is about 27 % in favor of the safety.
Considering the models of the series B35P25, the relationship $\sigma_{zi,B}/f_{cm}$ = 0,70, disregarding the relationships of the model

Figure 22 – curve load vs. displacement, series B35P50

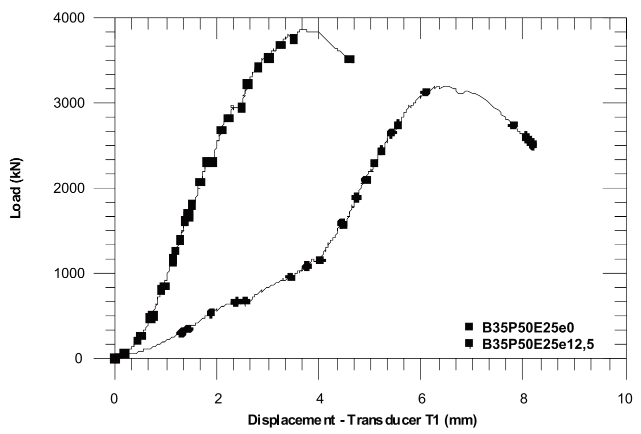


Figure 23 – curve load vs. displacement, series B45P50

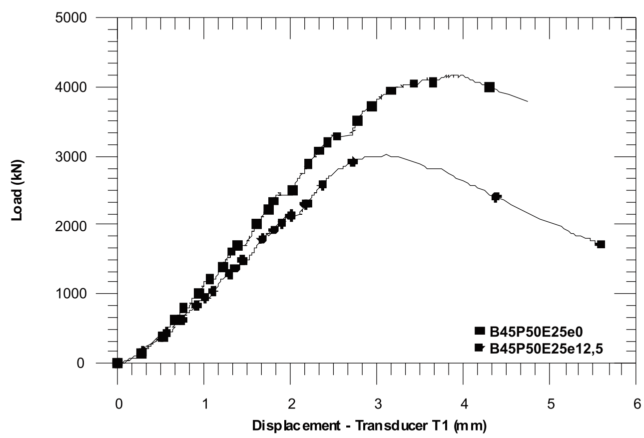


Table 4 – Stress calculated in function of the criteria of Blévot & Frémy (2)

SERIES	PILE CAPS	f_{cm} (MPa)	F_u (kN)	θ (graus)	$\sigma_{zni,B}$ (MPa)	$\sigma_{zns,B}$ (MPa)	$\sigma_{zni,B}/f_{cm}$	$\sigma_{zns,B}/f_{cm}$
B35P25	B35P25E25e0	40,6	1821	45	29,14	58,27	0,72	1,44
	B35P25E25e2,5	40,6	1688	45	27,01	54,02	0,67	1,33
	B35P25E25e0$A_{sw,C}$	32,8	1880	45	30,08	60,16	0,92	1,83
	B35P25E25e0$A_{sw,0}$	32,8	1406	45	22,50	44,99	0,69	1,37
	B35P25E25e0CG	28,9	1263	45	20,21	40,42	0,70	1,40
B45P25	B45P25E25e0	31	2276	54,5	27,47	54,94	0,89	1,77
	B45P25E25e5	31	1972	54,5	23,80	47,61	0,77	1,54
	B45P25E25e0$A_{sw,C}$	32,4	3055	54,5	36,87	73,75	1,14	2,28
	B45P25E25e0$A_{sw,0}$	32,4	2090	54,5	25,23	50,45	0,78	1,56
	B45P25E25e0CG	28,9	2270	54,5	27,40	54,80	0,95	1,90
B35P50	B35P50E25e0	35,8	3877	53,1	48,46	48,46	1,35	1,35
	B35P50E25e12,5	35,1	3202	53,1	40,03	40,03	1,14	1,14
B45P50	B45P50E25e0	35,8	4175	61,8	43,00	43,00	1,20	1,20
	B45P50E25e12,5	35,1	3386	61,8	34,88	34,88	0,99	0,99
Average values		33,8	-	-	31,15	50,41	0,92	1,51

Notes: f_{cm}, average compressive strength of concrete, F_u, experimental ultimate load; θ, angle of inclination of the strut; σ_{zni},B and $\sigma_{zns,B}$ are the stress near to inferior and superior nodal zones, calculated Blévot & Frémy (2).

B35P25E25e0Asw,C, because the experimental ultimate load was higher than the other models of its series due to the fact that these models were built with splitting reinforcement. It is valid to state that the rupture of the models of the series B35P25 occurred due to splitting and after crushing of the concrete along the superior nodal zone. The average concrete compressive strength of the pile caps of the models of this series, disregarding the model B35P25E25$A_{sw,C}$, was equal to 35,73 MPa. Therefore, using the limit stress close to the inferior nodal zone of CEB-FIP [3], the value σ_{zi} = 13,12 MPa was obtained. Considering that the reduction coefficient of the concrete (c) is equal to 1,5 (established by CEB), the effective design stress ($\sigma_{zi,Bd}$) was determined close to inferior nodal zone, as equal to 16,67 MPa. Finally, it can be concluded that the suitable value for CEB-FIP [3] is about 27% in favor of the safety.

The same method was applied for the models of the series B45P25, disregarding the value of the stress close to the pile for the same reason already exposed, it was determined that the relationship $\sigma_{zi,B}/f_{cm}$ is equal to 0,85. The reduction coefficient to concrete (γ_c = 1,5) was used and the effective design stress close to the inferior nodal zone was calculated, $\sigma_{zi,Bd}$ = 17,5 MPa. The limit stress established by CEB-FIP [3] presents a result equal to 10,81 MPa. The limit stress presented by CEB indicates safety of approximately 61%.

Considering the superior nodal zone, the Model Code of CEB-FIP [3] restricts the value of the stress in the strut in $\sigma_{zs} = 0,85\cdot[1-(f_{ck}/250)]\cdot f_{cd}$ [same value suggested by Machado [18] - $0,85\cdot f_{cd}$]. Using the same method for the determination of the effective stress of the support close to the pillar, in other words, considering that the area of the support in the superior nodal zone is half of the area of the pillar in a tilted plane whose angle in relation to the horizontal is 90º - θ, the relationship $\sigma_{zs,B}/f_{cm}$ was determined with a value on the average of 1,51. In this case, the Model Code introduces conservative results. The $\sigma_{zn,B}/f_{cm}$ value = 1,51 evidence the action of confinement of the concrete in the superior nodal zone provoked by the strut.

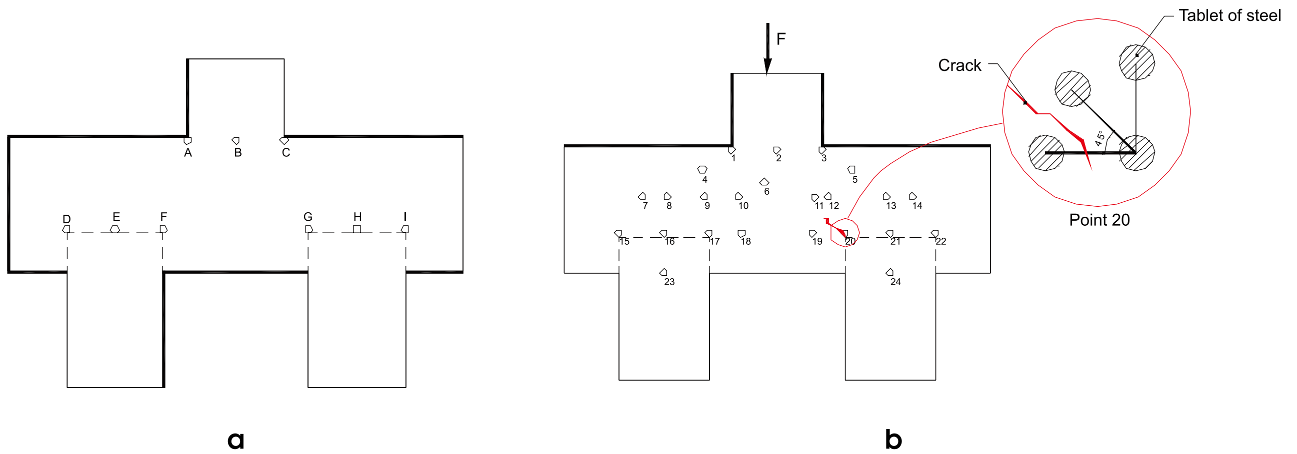
Analyzing only the models of the series B35P25, disregarding the model B35P5e0$A_{sw,C}$, it was verified that the relationship $\sigma_{zn,B}/f_c$ was equal to 1,38. The average concrete compressive strength of the models analyzed in the series was equal to f_{cm} = 35,73 MPa, the average stress close to the column was 49,30 MPa. Using γ_c = 1,5, the limit stress established by CEB-FIP[3]was determined (σ_{zs} = 17,35 MPa). Finally, it was verified that the method of CEB, presents 106% of safety.

Repeating the same method for the pile caps of the series B45P25, excluding the model B45P25E25e0$A_{sw,C}$, it was determined that the relationship $\sigma_{zn,B}/f_{cm}$ was equal to 1,69 and that the average concrete compressive strength of these models was equal to 30,83 MPa. So, the average stress close to the columns was equal to 52,10 MPa. The stress limit established by CEB was equal to 14,96 MPa, therefore, the stress limit of CEB presents safety of 106%.

For the models of the series B35P50 and B45P50, the relationship $\sigma_{zn,B}/f_{cm}$ was equal to 1,17 and the average concrete compressive strength of the pile caps for these models was equal to 35,45 MPa. So, the value of the average stress close to the superior nodal zone was determined as equal the 41,48 MPa. The value of the stress limit established for CEB-FIP [3], was equal to 17,24 MPa, indicating safety of 140%

Something very important, that agrees with the numerical analyses developed by Delalibera [14], relates the stress and the strain in the inferior nodal zones. Analyzing the results of the diagrams presented in Figures [18] and [19], it is verified that in the sections D and I (see Figure [24 - a]) the strains have small intensity, unlike the sections E, F, G and H (see Figure [24 - b]). Even without being aware of the value of the stress in the sections F and G, it could be concluded that these sections are more submitted than the others (because this section presents cracking in concrete - see Figure

Figure 24 – a) Position of the inferior and superior nodal zones; b) Crack that crosses a point of measure of relative displacement among the steel tablets



[24 - a]), therefore, considering that the load in the strut is distributed uniformly in the cross section, in a tilted plane of (90º - θ) is not adequate. The results of the tests showed that the sections away from the extremity of the pile caps were more submitted by the compression load of the strut.
Based on the exposed, it is noticed that the compression stress in the strut is eccentric in relation to the pile. Therefore, it is verified that the pile is subjected to the action of the normal load and bending moment.
Figures [25] to [26] present the fissures developed during the tests of the models according to Delalibera [14]. In all the models the fissures grew more intensely in the area delimited by the red line. Through the positions of these fissures it is possible to verify, approximately, the flow of the principal compression stress, determining the geometric form of the strut.

In function of the strain in the faces of the pile caps close to the inferior nodal zones (sections F and G presenting larger strain in relation to the sections E, H, D and I) and in function of the design of the flows of the principal compression stress, a criterion for the design of the piles is suggested considering a eccentric load originating from of the strut.
It is admitted that part of cross section of the pile is submitted, because of this a stress diagram close to the inferior nodal zone was created, that, in an indirect way, introduces an eccentric load on the pile head. Figure [27] shows the proposed model.
On the face more distant from the pile in relation to the edge of the block, the stress is admitted equal to $1{,}2 \cdot \sigma_{zi,d} \cdot \mathrm{sen}(\theta)$ and in the section of the pile closer the border of the block, it is supposed, for the design of the piles, that the value is qual the zero. The average stresses in the piles were equal to $0{,}60 \cdot \sigma_{zi,d}$ and need to be smaller than the given ultimate stress.

Figure 25 – Model adopted to flow of principal compression stress, model B35P25E25e0

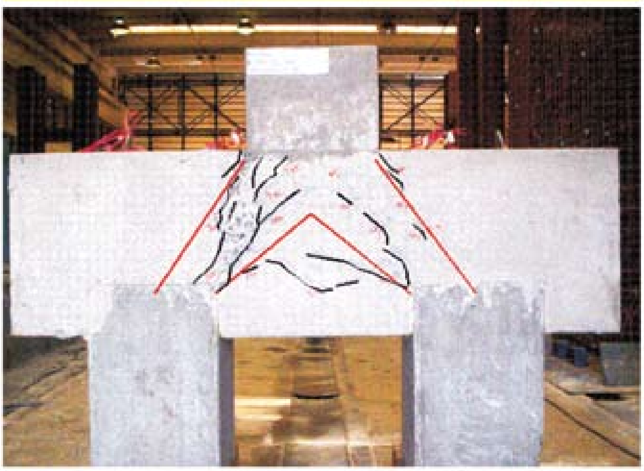


Figure 26 – Model adopted to flow of principal compression stress, model B35P25E25e0$A_{sw,C}$

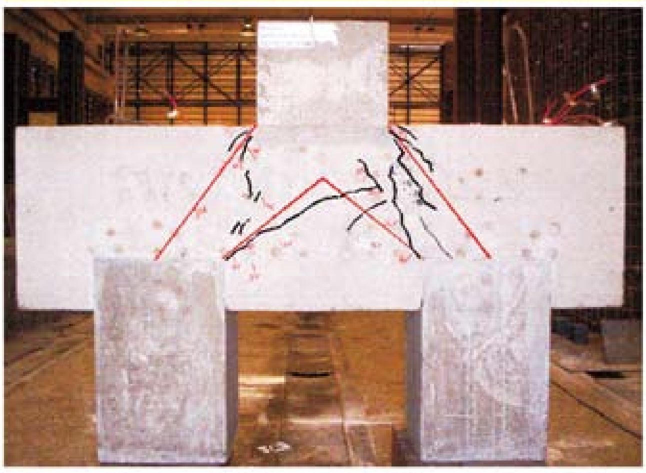
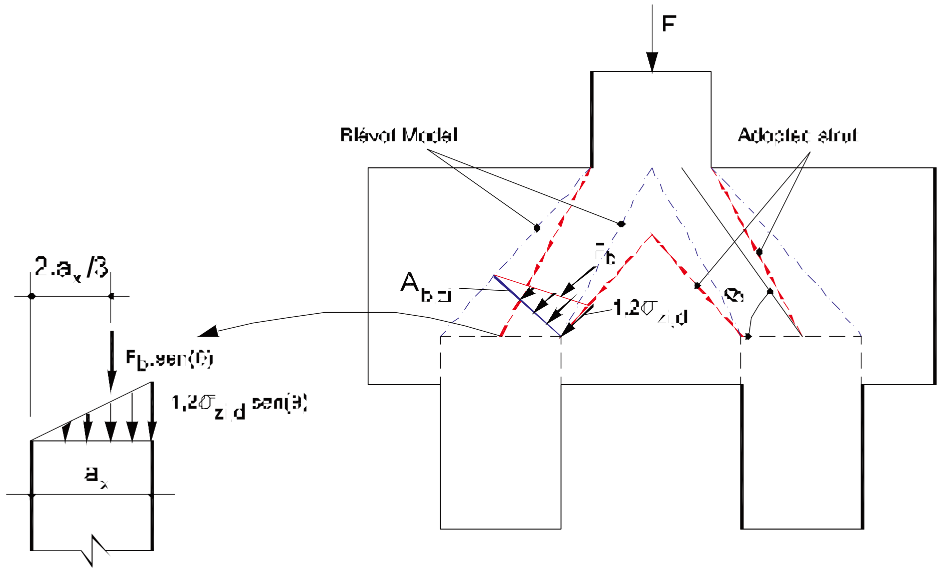


Figure 27 – Model proposed for the consideration of the eccentric load close to the pile



These values are valid for pile caps without splitting reinforcement, because the introduction of this reinforcement increased the value of the strength capacity of the pile cap. That evidences that the ultimate capacity of the pile cap is not related to the crushing of the strut close to the pile and to the column, but, it is related to the splinting of the strut.

7. Conclusion

The tested models with splint reinforcement (B35P25E25e0A$_{sw,C}$ and B45P25E25e0A$_{sw,C}$) presented values of strength capacity 21,7 % higher than the pile caps of the series B35P25 and 41,9 % higher than the pile caps of the series B45P25.

The presence of bending moment in the pile caps reduces its strength capacity.

The increase of the cross section of the column, keeping the mechanical and geometric properties of the pile caps constant, increased the strength capacity of the pile caps.

With the results obtained through the strain in the faces of the pile caps, it was possible to identify the geometric form of the compression stress flow (Figure [18] and [19]).

The strain intensities of the pile caps, in the sections less distant from the pile are smaller than the strains in the sections of the pile more distant away. It can be considered that only part of the pile is more submitted. (this model is a more realistic, because it doesn't not consider that the whole cross section of the pile is submitted to the same compression load of the strut).

In the superior nodal zone, there is great concentration of stress below the column. In function of this, considering that the half of the area of the cross section of the column is submitted by the half of the applied load on the top of the column is the good procedure.

Unlike what Adebar et al. [6] affirms in his refined strut and tie model didn't occur in the tests, the expansion of the compression stress flow along the height of the pile caps. the opposite occurred, the strain were larger in the sections of the pile caps close to the extremities of the column and in the section away from the extremity of the pile cap.

Finally, a method based on experimental observations was presented. This is method consider the compressive eccentric load in the strut close to the inferior nodal zone. It is supposed that the pile needs to be designed to resist compression load and bending moment.

During the test, the horizontal displacements of the piles weren't limit, only the attrition between the base of the pile and the plate of steel positioned on the load cells was considered. It was verified, through displacements transducers positioned to obtain values of displacements in the longitudinal sense of the pile caps, that displacements were between 1 mm to 2,5 mm. Of course the behavior of pile caps with piles with great shaft lengths is different from the pile caps tested at the laboratory, because the piles had small shaft length. However, this factor was also analyzed before the tests, through numerical analysis. Pile caps with long shafts were analyzed. The soil was modeled as half continuous. The results were similar to the results of the tests and more details can be seen in Delalibera [14] and Delalibera & Giongo [16].

The results of the effective stress, calculated in function of the criteria of Blévot & Frémy [2], presented superior results to the stress limits close to the superior and inferior zones presented by the criteria of CEB-FIP [3].

It can be observed that in the models of the series B35P50 and B45P50, the stress close to the piles (B35P50E25e0 - $\sigma_{zni,B}$ = 48,46 MPa; B35P50E25e12,5 - $\sigma_{zni,B}$ = 40,03 MPa; B45P50E25e0, $\sigma_{zni,B}$ = 43,00 MPa; and B45P50E25e12,5 - $\sigma_{zni,B}$ = 34,88 MPa), calculated with the criterion of Blévot & Frémy [2], presented higher values than the average compressive strength of the concrete of

the pile cap. Therefore, in situations of pile caps, where there are prolonged columns, it is necessary to analyze carefully the stress in the inferior nodal zones.
It is suggested, as a safe criterion, that only part of the cross section of the pile (half) is considered for the verification of the stress in the inferior nodal zone, limiting the stress in the inferior nodal zone the f_{cd}.

8. Acknowledgments

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