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Influence of column cross section and eccentricity of compression load in structural behaviour of two pile caps

Influência das dimensões da seção transversal do pilar e da excentricidade da força de compressão no comportamento estrutural de blocos sobre duas estacas


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Abstract

To design of the pile caps are utilized the strut and tie model and bending theory. The Model Blévot and Frémy [1] is one of the models used to design of pile-caps. Another method for the design of caps is Model Beam that used bending theory. In the model of French researchers to cross section of the column is proportional to the cross section of strut near the column. To check the influence of the column cross section in the structural behaviour of the pile caps, experimental tests of the two pile caps were realized, varying the dimensions of the column and the high of the pile caps. Comparative analysis of experimental results with the results found by the Model Blévot and Frémy [1] was realized. It was observed that the increase of the column cross section conferred the pile caps greater strength capacity.

Keywords: *foundations, pile caps, experimental analysis.*

Resumo

Para o dimensionamento de blocos sobre estacas utilizam-se geralmente os modelos baseados na analogia de escoras (bielas) e tirantes e teoria de flexão. O modelo de Blévot e Frémy [1] é um dos modelos de cálculo utilizado para o dimensionamento de blocos sobre estacas. Outro método para o dimensionamento de blocos sobre estacas é o modelo de viga que utiliza a teoria geral de flexão. No modelo dos pesquisadores franceses a seção transversal do pilar é proporcional à seção transversal da escora junto ao pilar. Para verificar a influência da seção transversal do pilar no comportamento estrutural dos blocos, realizaram-se ensaios experimentais de blocos sobre duas estacas, variando-se as dimensões da seção transversal do pilar e as alturas dos blocos. Análise comparativa dos resultados experimentais com relação aos resultados encontrados pelo modelo de Blévot e Frémy [1] foi realizada. Observou-se que o aumento da seção transversal do pilar conferiu aos blocos maiores capacidades resistentes.

Palavras-chave: *fundações, blocos sobre estacas, análise experimental.*

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1. Introduction

According to NBR 6118:2003 [2] caps can be considered rigid or flexible. If rigid, the Inequation [1] has to be satisfied.

$$h \geq (a - a_p)/3 \quad (1)$$

Being h the total height of the cap, a and a_p the dimensions of the cap and the column at a considered direction.
The authors of the present work understand that this expression is inadequate as there are situations that present results which differ from the lower limit to the angle of inclination of the strut established by Blévot e Frémy [1]. Figure [1] shows examples in which the expression of the Brazilian standard presents an angle of inclination of the strut (in relation to a horizontal plane) below the limit of (45º) established by Blevót e Frémy [1]. According to Brazilian standards, the pile cap B of Figure [1] can be considered rigid even having a height smaller than the pile cap A.
The Bulletin number 73 of CEB [3] establishes that for a cap to be considered rigid, its height must be bigger than one and a half fold the distance *c* from the more distant pile axis to the column face (see Figure [1]).
The Spanish standard EHE [4] accepts that for the cap to be considered rigid the height has to be bigger than two fold the distance *c* specified in Figure [1].
The Canadian Standard A23.3-04 [5] establishes that the cap must to have a minimum height equal to thirty centimeters and that the usable height of the cap in a section with distance d/2 from the column face must have shear strength higher or equal to Equation [2].

$$V_c = 038 \cdot \lambda \cdot \phi_c \cdot [(f'_c)^{1/2}] \quad (2)$$

Being λ a coefficient which takes into account the concrete density, φ_c the diminution coefficient of the concrete strength and β coefficient which considers the reduction of the shear strength in chinked sections with value equal to 0,21 if the distance *c* (see Figure [1]) is three-fold smaller than the height of the cap.
The American code ACI 318 [6] defines that the cap must have a height to bear the last shear strength, expression [3]

$$V_u \leq 2 \cdot (f'_c)^{1/2} \cdot b \cdot d \quad (3)$$

Where f_c is the characteristic concrete compression strength, b is the width of the cap and d is the usable height of the cap.
It also specifies that the pile caps must have a minimum height equal to 30 cm.
It was observed that there are different prescriptions between the Standards above mentioned, therefore it is suggested as criterion that the height of the cap be defined on the basis of the angle of

Figure 1 – Two pile caps, classification in relation to stiffness

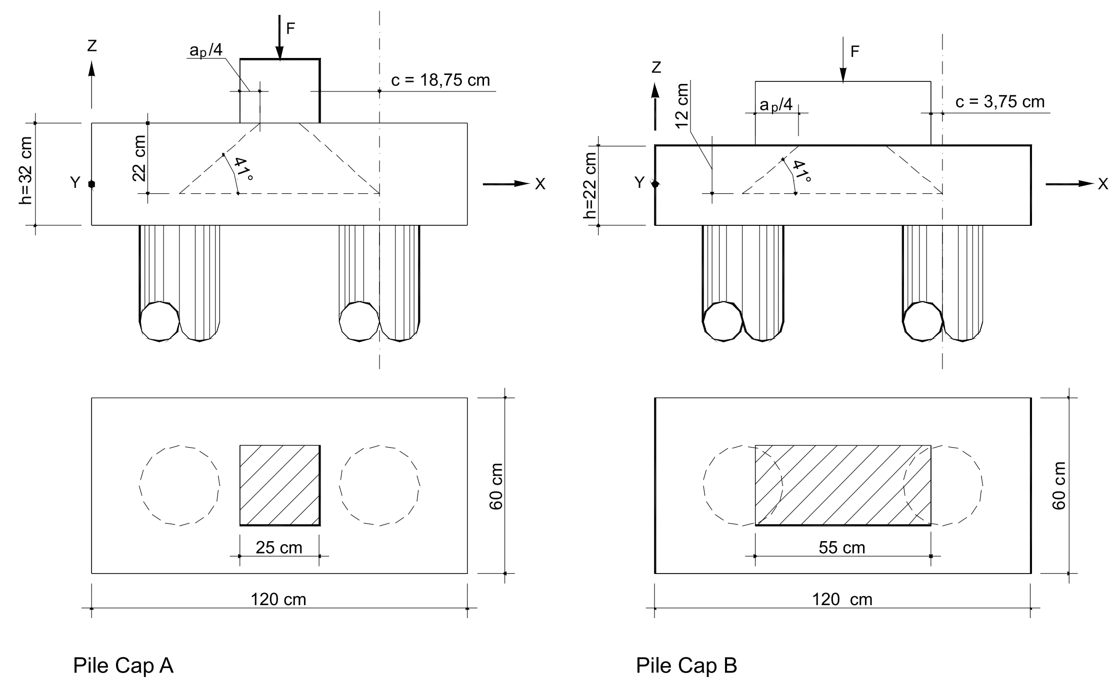


Table 1 – Classification of caps

Criteria	Pile cap A F_d = 1000 kN, M_d = 0, f_c = 25MPa			Pile Cap B F_d = 3500 kN, M_d = 0, f_c = 25MPa		
	Calculated	Existent	Classification	Calculated	Existent	Classification
Blévot (θ = 45º)	35	32	Flexible	28	22	Flexible
NBR 6118	32	32	Rigid	32	22	Flexible
CEB (1970)	38	32	Flexible	16	22	Rigid
EHE	38	32	Flexible	18	22	Rigid
CSA	98	32	-	145	22	-
ACI	40	32	-	40	22	-

Notes: F_D, design normalload; M_d, design moment; f_c,compression strength of the concrete.

inclination of the strut. It was also noted that the column cross section interferes directly in the calculation of the height of the cap. Table [1] presents the classification and minimum height of the cap shown in Figure [1], according to several standards and codes previously mentioned.

Among many researches with experimental emphasis developed in the last decades, any one has done reference to the influence that these dimensions of column cross section causes in the structural behaviour of pile caps.

Hobbs and Stein [7] studied the behaviour of two pile caps through theoretical and experimental analysis. Seventy models were assayed in a reduced scale at the ratio 1:3. Researchers developed an analytical solution based on the bidimensional elasticity theory and compared the theoretical results with the experimental results obtained through the assays. Up to that time caps were treated as beams, therefore there was no formulation for pieces with stress perturbation involving practically all the structural element so there was an evolution in the cap design method.

Blévot and Frémy [1] performed assays in one hundred pile caps with the purpose of studying the influence of different reinforcement arrangements. The experimental observations reported by Blévot and Frémy [1] are reference for most of the formulations for pile caps found in technical literature.

Mautoni [8], upon results of assays in two pile caps, established a method to determine the load and the failure mechanism through an expression which takes into account the critical reinforcement ratio. The models were developed with the purpose of analyze two pile caps but can be used for the corbel analysis without transverse reinforcement.

Rausch *et al.* [9] performed an assay with reinforced concrete above two pile caps in reduced scale (ratio 1:2). The authors obtained some interesting conclusions as: the strut method supposes that the lower bars (ties) are freely deformed but as the piles act limiting this deformation it was verified that when the bars cross the compression struts in the piles there was a significant reduction in these deformations; the assays show that tie reinforcement calculated through the strut and tie method is conservative, what in practice means that reinforcement area can be reduced; in the theoretical model it is accepted that the tie has constant deformations over its length but it was not observed in the assays as for values of load near the ultimate load of the pile cap the deformations obtained in the tie at pile section had values near to zero and in some cases negative values.

Miguel and Giongo [10] performed experimental analysis of reinforced three pile caps submitted to the action of centered force. The authors concluded that the strut method developed by Blévot and Frémy [1] is conservative, indicating a safety margin of 12%. All models assayed had failure by chinking of the struts because of the expansion of the stress flow, followed by the yield of the principal reinforcement bars.

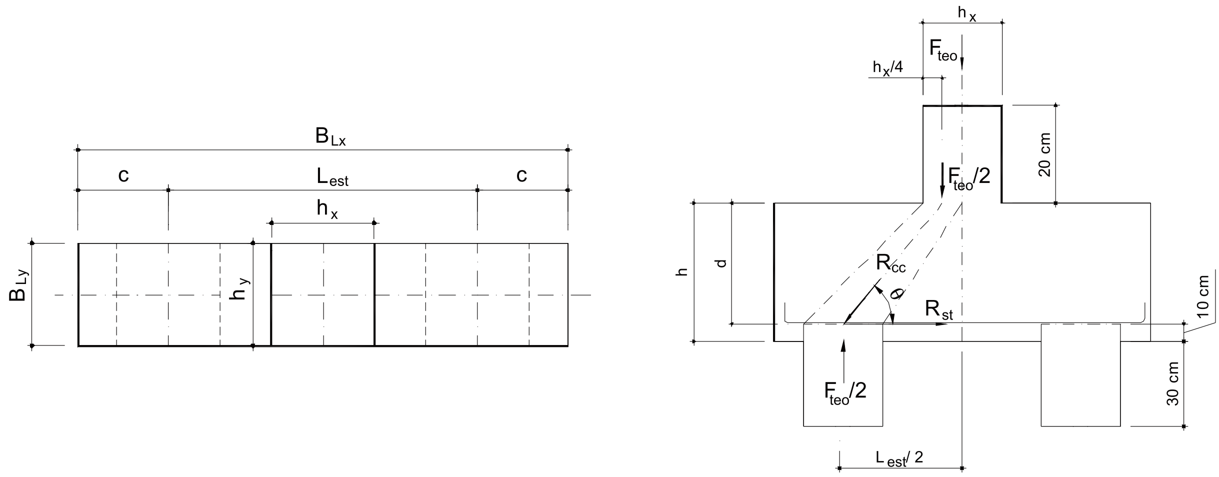
Chan and Poh [11] performed an experimental study about the behaviour of precast pile caps. Three caps were assayed until failure. One of them was cast-in-place and the other two were precast. The purpose of the research was to compare the behaviour between precast pile caps and cast-in-place pile caps. It was observed that the methodology used for pile cap design can be applied to precast caps whereas in some cases the results were conservative.

Adebar [12] upon the results of assays in twenty three caps transition caps (caps with large spacing between the upstands) proposed a change in canadian standard for the design of this kind of structural element. CSA Standard A23.3-04 [5] restricts the compression stresses in lower and upper nodal zone. Adebar [18] suggests the use of stirrups in pile caps in order to avoid the rupture caused by the action of shear stresses.

Souza and Bittencourt [13] discussed the classification of caps as rigid or flexible proposing the Strut Method and the Beam Model as viable solutions for the design issue. Complementing the work, the results of a nonlinear three-dimensional analysis of a rigid two pile rigid cap were presented with the purpose of confronting numeric results to experimental and analytical answers.

Through bibliographical review, it is verified that all pile caps experimentally analyzed were built with column of square cross sec-

Figure 2 – Dimensions of two pile caps



tion and only centered load. In light of the above-mentioned and the different existing criteria for the design of pile caps, it was performed an experimental analysis of fourteen pile caps with different reinforcement arrangements, heights, cross sections of columns and also centered and eccentric loads were considered. Information regarding the assays can be obtained in Delalibera [14]. In this work are presented the experimental results of six out of fourteen caps assayed.

2. Materials and experimental program

In relation to the caps analyzed in this work, four had column cross section with dimensions equal to 25 cm x 50 cm. The other two had square section measuring 25 cm. The distance between the axes of the piles had 62.50 cm and the pile inlay in the cap was of 10 cm. Figure [2] shows the dimensions of caps experimentally analyzed. The heights of the columns were adopted as 20 cm and the heights of the piles as 30 cm with the purpose of making possible the assays because if they were considered with the real length the assays would be unachievable in the environment of the Structures Laboratory of Escola de Engenharia de São Carlos – USP. In order to verify the influence of the length of pile shaft in the caps, Delalibera [14] performed several numerical simulations and observed that there is a change in the intensities of stress in the struts but their trajectories were kept. More information regarding these analyses can be obtained in Delalibera & Giongo [15].

In Table [2] the geometric properties of the analyzed models are presented and Figures [3] and [4] show the reinforcement arrangements used in the caps.

Table 2 – Geometric properties of models experimentally analyzed

Caps	Dimension of the pile (cm)	Dimension 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)
B35P25E25e0	25 x 25	25 x 25	117.5	25	25	25	62.5	27.5	0	35
B45P25E25e0	25 x 25	25 x 25	117.5	25	25	25	62.5	27.5	0	35
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
B45P50E25e0	25 x 25	25 x 50	117.5	25	50	25	62.5	27.5	12.5	35
B45P50E25e12,5	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 caps in the directions x (longitudinal) and y (transversal), e_{adot} is the eccentricity of the compression load applied in the column, h is the height of the cap; h_x e h_y are the dimensions of the column; L_{est} is the distance between the pile axes; c, is the distance from the pile center to the cap border.

Figure 3 – Reinforcement arrangement of the model: B35P25E25e0 and B45P25E25e0

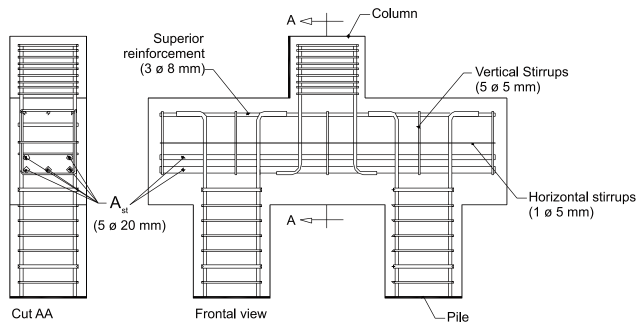
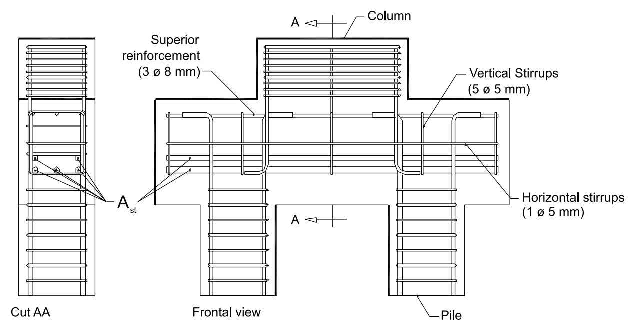


Figure 4 – Reinforcement arrangement of model: B35P50E25e0, B35P50E25e12,5, B45P50E25e0 e B45P50E25e12,5



Each model has an acronym whose the meaning is presented in example B35P25E25e0:

- B35: cap with height equal to 35 cm;
- P25: lengths of the edges of the column cross section equal to 25 cm;
- E25: lengths of the edges of the pile cross section equal to 25 cm;
- e0: eccentricity of the compression load equal to zero.

The reinforcement cover in the columns was equal to 25 mm and in the caps was equal to 40 mm.
The principal tensile reinforcement (A_{st}) of the caps was built by five bars with diameters equal to 20 mm.
The pile frames of the caps of the sequence B35 were built by four bars with diameters of 10 mm (longitudinal) and stirrups with diameter of 6.3 mm spaced each five centimeters.
For the piles of the caps of sequence B45 the gauges of the longitudinal bars were equal to 12.5 mm and the stirrups were equal to the models of the previous sequence.
The columns of the sequence B35 were built with eight bars of 10 mm and those of sequence B45 were built with eight bars of 12.5 mm. At the top of the columns were placed reinforcements composed by bars of 6.3 mm. All care with concrete launching and compacting during the molding of caps was considered.
The mechanical properties of the steel bars used in the building of the models are presented at Table [3].
The longitudinal strain modulus of the steel (E_s) experimentally determined presented mean value of 200 GPa.
The compression strength of the concrete in piles and columns were greater than the compression strength of the concrete in caps for assuring that there are no failures in piles and columns. It was adopted a compression strength of the concrete in columns and piles equal to 50 MPa and concrete strength of the concrete for caps equal to 25 MPa. The mass proportions for concretes with characteristic compression strength of 25 MPa were equal to 1:2,55:3,54:0,68 with cement CP-II-F32. For a concrete with characteristic compression strength of 50 MPa, the mass proportion was 1:2,66:3,66:0,49 using cement CP-V and 1% (in mass) of super plasticizer. The models were assayed with 28 days old.
As one of the purposes of the research was to obtain information regarding the geometric shape of the compression strut, one of the faces was instrumented through steel pellets placed so to form a rectangular rosette which worked as measure points for concrete strains. The relative displacement measures (strains) between the steel pellets were obtained through mechanical strain gauge. Through the relative displacements between steel pellets it was also possible to accompany the development of cracks over the assay. Details regarding the strains in cap faces can be obtained in Delalibera e Giongo [15].
The intensities of the loads applied were measured through the load cells. Three load cells were used, one with a capacity equal to 5000 kN and two with capacities equal to 2000 kN. The cell with higher capacity was installed on the columns and the other on the piles so it was possible to obtain the value of the load applied in the column and the reactions in the piles.
The force applied at the pillar top was exercised through a hydraulic piston with capacity of 5000 kN and maximum stroke of 160 mm. The oil required to move the piston was injected through an electrical pump with control for the application of force.
As reaction structure it was used a steel frame with capacity of 5000 kN and a reaction slab of Structures Laboratory of the Engineering School at São Carlos, São Paulo University. Figure [5] presents the arrangement of the assays.
The load data acquisition system was the System 5100, of Mea-

Table 3 – Mechanical Properties of steel bars

$Ø_{nom}$ (mm)	f_y (MPa)	ε_y (‰)	f_u (MPa)
5.0	681	4.05	755
6.3	597	2.99	733
8.0	581	2.91	674
10.0	549	2.75	658
12.5	578	2.89	740

Notes: $Ø_{non}$ is the nominal diameter of steel bars; f_y is the yield strength of steel bars; ε_y is the tensile strain in steel bars; and f_u is the ultimate strength of steel bars.

Figure 5 – Assay scheme

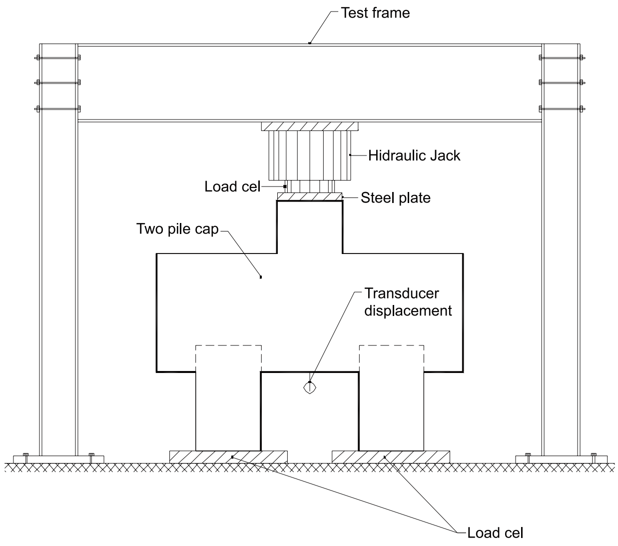
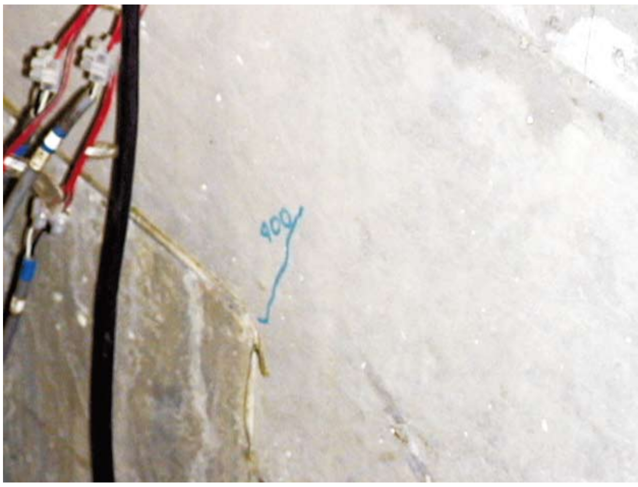


Figure 6 – Position of the first crack (near the pile)



surements Group, and the computer program used was the StrainSmart of the same manufacturer.
The measures of the displacements were obtained through displacement transducers with a maximum stroke of 50 mm.

3. Description of the Experimental Analysis

The assays were performed in increasing steps of load application until failure. It was done an anticipation of the strength capacity of caps and the theoretical ultimate load was split in ten equal parcels. Admeasurements of the relative displacements between the steel pellets were performed for each load increasing.
Usually, all models presented similar behaviour. The first chink came out in the lower face of the cap next to the pillar and was propagated until the upper face of the cap next to the column (see Figure [6]). Other cracks came out in other load application steps with inclination similar to the first crack. Although the cracks presented large openings (w > 0,4 mm) the cap only stop resisting the applied load at the moment the cracking process of the compressed diagonal (splitting) started, after that there was a rupture of the concrete next to the upper nodal zone and in the models of sequence B35P50E25e0, B35P50E25e12,5, B45P50E25e0 and B45P50E25e12,5 next to the lower and upper nodal zone concomitantly.
All models presented failure by splitting and rupture of the concrete, i.e., after the rupture by traction of the concrete of the section delimited by the strut there was rupture of the concrete next to the column and in some cases next to the pile with the formation of a rupture plane along the cap height.
The table [4] presents the results of the ultimate loads and relative

Table 4 – Values of the ultimate load and the first chink obtained in the assays

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
B35P25E25e0	40.6	1821	465	1776	761	0.26	0.98	0.42	0.61
B45P25E25e0	31	2276	465	1796	770	0.20	0.79	0.34	0.60
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
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

Notes: f_{cm}, median compression strength of the concrete, obtained through assays of cylindrical concrete specimens; F_u, ultimate force obtained experimentally; F_r, force that caused the first chink; F_{teo}, theoretical force, calculate using the criteria of the French researchers (1), limiting the stress in lower nodal zone equal to f_{ck} and the stress in upper nodal zone equal to $1{,}40 \cdot f_{ck}$; F_d, design load, calculated using criteria of the French researchers, limiting the stresses in upper and lower nodal sections equal to $0{,}85 \cdot f_{cd}$.

Table 5 – Value of the cracks openings

Models	F_r/F_u (%)	Maximum Opening (mm)	Angle of inclination of the chink	w_{max} NBR 6118:2003
B35P25E25e0	67.28	0.32	57°	0.3
B45P25E25e0	74.69	0.50	60°	0.3
B35P50E25e0	72.22	0.37	65°	0.3
B35P50E25e12,5	46.84	0.26	50°	0.3
B45P50E25e0	74.25	0.46	60°	0.3
B45P50E25e12,5	67.94	0.30	50°	0.3

Notes: F_r, load that caused the first chink; F_u, ultimate load, w_{max}, chink maximum opening.

Figure 7 – Cracks overview

load (F_r) for the first chink, theoretical force (F_{teo}), calculated using the criteria of French researchers[1], design load (F_d) and the mean compression strength of the concrete of caps.

In Table [4] it is noted that the first chink came out for a force of approximately twenty per cent of the last ultimate load and approximately fifty per cent of the design force.

It is possible to verify that the results obtained by the authors of the present work are close the results noted in the assays of Blévot and Fremy [1], i.e., the value of $1.4{\cdot}f_{ck}$, related to the stress in the strut next to the column. These results took place in the assays of sequences B35 e B45.

It should be remembered that the value of $1.4{\cdot}f_{ck}$ is valid only for models where there were ruptures near the pillar. In models where the ruptures of caps occurred simultaneously near the piles and the column (models B35P50E25e0, B35P50E25e12,5, B45P50E25e0 and B45P50E25e12,5), the values are different from the value f_{ck} presented by Blévot and Frémy [1] as the stress near lower and upper nodal zones were approximately equal to $1{,}4{\cdot}f_{ck}$. Therefore, the results of the models B35P50E25e0, B35P50E25e12.5, B45P50E25e0 and B45P50E25e12.5 presented stress values of the strut near the pile which were above the values defined by Blévot and Frémy [1].

4. Analysis of the experimental results

4.1 Cracks opening

The cracks openings were determined according to relative displacements between the steel pellets. For safety concerns of the assay operators, the cracks openings were not measured for loads close to the ultimate force. In average the cracks openings were evaluated until 70% of the ultimate load.

Table [5] shows the opening of cracks measured during the assays, the inclinations in relation to a horizontal plane (for example, the principal

Table 6 – Stress near the upper and lower nodal zone

Models	f_c (MPa)	F_u (kN)	$\sigma_{b,zs}$ (MPa)	$\sigma_{b,zi}$ (MPa)	$\sigma_{b,zs/fc}$	$\sigma_{b,zi/fc}$
B35P25E25e0	40.6	1821	61.6	30.8	1.52	0.76
B45P25E25e0	31	2276	57	28.5	1.84	0.92
B35P50E25e0	35.8	3877	49.5	49.5	1.38	1.38
B35P50E25e12,5	35.8	4175	43.9	43.9	1.23	1.23
B45P50E25e0	35.10	3202	53.2	53.2	1.52	1.52
B45P50E25e12,5	35.10	3386	48.6	48.6	1.38	1.38

Notes: f_c, compression strength of the concrete, obtained through assays of cylindrical concrete specimen; F_u, ultimate load experimentally obtained; $\sigma_{b,zs}$, stress near the upper nodal zone (near the column); $\sigma_{b,zi}$, stress near the lower nodal zone (near the pile).

Figure 8 – Load vs. displacement curve, B35P25E25e, B35P50E25e0 and B35P50E25e12,5

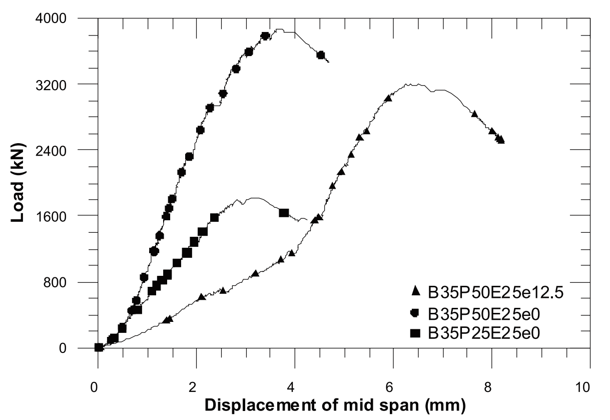
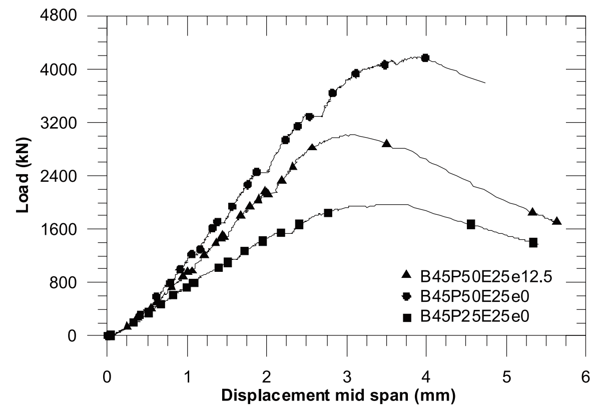


Figure 9 – Load vs. Diplacement curve, B45P25E25e, B45P50E25e0 and B45P50E25e12,5



tensile reinforcement) and compares the value of cracks openings in relation to the maximum value established by the Brazilian standard, considering the moderate environmental aggressiveness class and the constant combination of actions, whose the value is equal to 0.3 mm.
Through Table [5] it is verified that the chink opening values are bigger than the maximum value established by the Brazilian standard. Figure [7] presents the overview of cracks of one of the experimentally analyzed models. It is noted in Figure [7] that the shape of the cracks shows appropriately the geometric shape of struts and ties applied to two pile caps.
Influence of the dimensions of the column cross section
It was verified in Table [6] that the column cross section changed the structural behaviour of caps. The caps with bigger dimensions of column cross section presented greater resistance capacity in the assays. It is clear that the resistance capacity of caps is related to the increase in the strut cross section near the column.
It is noted, in Table [6], that the relation between stress near the column ($\sigma_{b,zs}$) and compression strength of the concrete f_c, is equal to 1.48 in average ($\sigma_{b,zs}/f_c$ = 1.48). This value is approximately equal to the value presented by Blévot and Frémy [1], i.e, $\sigma_{b,zs}/f_c$ = 1.40.
The relation between the stress calculated near the pile and the concrete strength for piles with square cross section with side equal to 25 cm was equal to 0.84. This value is below the limit established by Blévot and Frémy [1], i.e., f_c.
For caps with piles with rectangular cross section whose sides are equal to 25 cm and 50 cm, the relation between the stress calculated near the pile and the concrete resistance was equal to 1.38. It is clear that the column cross section interferes directly in the strength capacity of the cap.
Figures [8] and [9] show correlations between the models: B35P25E25e0 with the model B35P50E25e0 and of the model B45P25E25e0 with the model B45P50E25e0.

4.3 Influence of the eccentricity of the normal force

Through the assays it became clear that in the models with

Figure 10 – Proposed Model, Delalibera and Giongo (14)

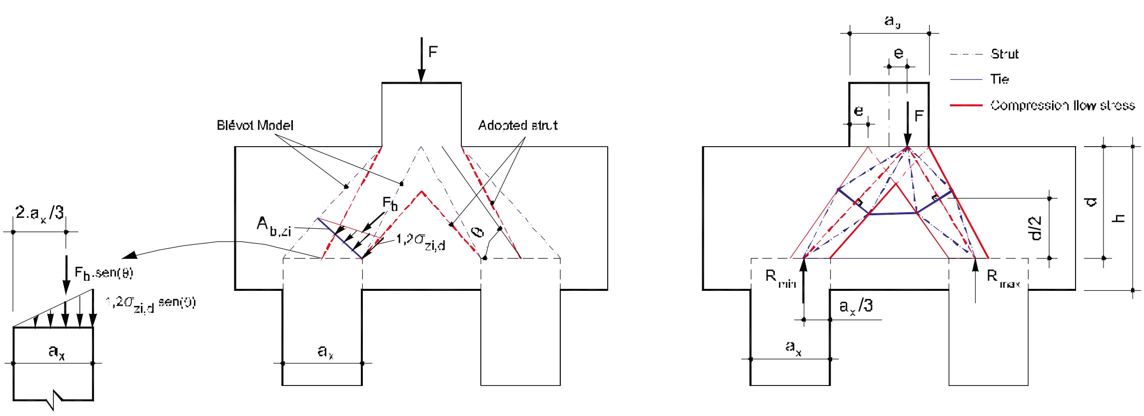


Table 7 – Reactions in piles

Models	Experimental reactions in the piles		e_{real} (cm)	Analytical reactions in the piles		$R_{max,exp}/R_{max,alt}$	$R_{min,exp}/R_{min,alt}$
	$R_{max,exp}$ (kN)	$R_{min,exp}$ (kN)		$R_{max,alt}$ (kN)	$R_{min,alt}$ (kN)		
B35P25E25e0	962	859	1.8	971	850	0.99	1.01
B45P25E25e0	1180	1096	0.7	1188	1088	0.99	1.01
B35P50E25e0	1980	1897	10.3	1989	1888	0.99	1.01
B35P50E25e12,5	2127	1075	1.2	2211	991	0.96	1.08
B45P50E25e0	2131	2044	0.7	2141	2034	0.99	1.01
B45P50E25e12,5	2358	1028	12.3	2462	924	0.96	1.11

Notes: $R_{max,exp}$, maximum experimental reaction; $R_{min,exp}$, minimum experimental reaction; $R_{max,exp}$, maximum analytical reaction; $R_{min,alt}$, minimum analytical reaction.

eccentricity of the load, the strength capacity of the caps was lower due to different strains in the compression struts. The eccentricity caused concentration of stresses in one of the cap sides, modifying the distribution of the flow of compression and tensile stresses.
Generally, the design of pile caps with acting of normal moment and load in the column is done is the following way: the reactions in the piles are determined; it is checked if there are tensioned piles, if so, particular models for the design and detailing of reinforcements have to be used; for compressed piles only, the number of piles is multiplied by the value of the reaction of the more compressed pile; finally, it is applied the value in the interface pile-cap as being a equivalent centered normal force.
The criterion previously presented it is not judged to be the more correct as it considers that the struts have the same strains, what does not happen in fact. An interesting criterion for checking the stresses in nodal sections would be the application of the strut and tie model which considers the eccentric compression load (see Figure [10] – in the case of two pile caps, supported in rigid substrates). This model was based on the original model by Adebar et al. [13]. In Delalibera and Giongo [16] it is showed the model idealized in Figure [10].
In Figures [8] and [9] it is observed that the eccentricity of the compression load decreases the strength capacity of the caps. It occurred due to the failure by diagonal tensile of the more subjected strut and, possibly the rupture of the concrete near the lower and upper nodal zone simultaneously. Through the assays it was clear that the struts were not subjected by the same load.
Figure [7] shows one of the models after failure. It was observed that the cracks delimit the struts and ties positions.
Table [7] presents the pile reactions in assays and pile reactions calculated by the model illustrated in Figure [13]. Analyzing the previous table, it is observed that the criterion used for the distribution of loads in piles and rigid caps is satisfactory once it presents good correlation between the experimental and theoretical results.

5. Conclusions

All caps presented the same failure modes, i.e., rupture of the strut near the column or the pile. It was observed that the rupture of the concrete happened only after intense chinking towards the struts along the height of the caps. These cracks influenced negatively in the compression strength of the concrete in the strut. Therefore, the use of steel bars placed in perpendicular direction to the strut would limit the opening of cracks, conferring to them greater resistance. In summary, the crushing of the strut took place after their chinking.
The cracks openings were measured until approximately 70% of the ultimate load of each model. Notwithstanding, taking into account that the element pile cap in most of cases is backfilled after its building, it is necessary to limit the chink opening making difficult the penetration of water in the interior of the concrete and avoiding the oxidation of reinforcement steel bars. Usually the opening is limited to 0.3 mm. However in all the prototype assayed the limit value established by NBR 6118:2003 [3] was reached, i.e., there were results against the safety in relation to the Cracks Opening Limit State.
The results obtained through experimental assays determined that the columns of bigger cross section presented higher resistance capacity in relation to the caps built with a smaller cross section. It was verified that the stress near the column presented analogue values to the limits established by Blévot and Frémy [1]. In piles there were significant differences. In the case of caps with column square cross section which were assayed the calculated stresses were lower than the limit presented by Blévot and Frémy [1]. As regards to the columns with rectangular cross section, the stress values were higher. As criterion for design it is suggested a limit value for stress near the piles of $0.85 \cdot f_{cd}$ and near the columns f_{cd}, once the stresses obtained in the assays were: near the piles: $0.84 \cdot f_c$ and near the columns $1.48 \cdot f_c$.
Through the results of experimental assays it was observed that the angle of inclination of the strut which is the function of the cap height and the

distance between the pile axes has basic importance in the resistance capacities of the caps. The more rigid models (with greater height and, therefore, greater angle of inclination of the strut) present higher strength capacity when compared to the models of lesser stiffness.

The assayed caps with eccentric vertical force presented lesser strength capacity. As the struts are subjected in different manner, the deformations in lower and upper nodal sections (near the piles and the column) also presented different stress values, hence there was a rupture of the concrete in the more lengthy tie with lesser intensity of load applied in the column when compared to models with centered load.

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