Numerical analysis of two pile caps reinforced concrete with partially embedded and shear key interface

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Abstract. On Precast concrete structures the column foundation connections can occur through the socket foundation, which can be embedded, partially embedded or external, with socket walls over the pile caps. This paper presents an experimental study about two pile caps reinforced concrete with partially embedded socket submitted to central load, using 1:2 scaled models. In the analyzed models, the shear key interface between the socket walls and column was considered. Additionally, a numerical analysis in the Abaqus software was performed, considering the physical non-linearity of the materials. The results are compared to a reference model that presents monolithic connections between the column and pile caps. It is observed that the ultimate load of pile caps with partially embedded socket present less magnitude than the reference model, and that the presence of additional reinforcement does not significantly change the behavior of pile caps.

.**Keywords:** *Pile caps. Socked foundation. Abaqus. Reinforced concrete.*

1 Introduction

Pile caps are foundation elements whose function is to transmit loads from the superstructure to a set of piles and, in turn, to the soil. It is one of the foundation solutions most used worldwide, especially in cases of soils with low load capacity. These elements are classified as volumetric and present a very complex behavior when compared to linear and internal elements. However, many university studies have been carried out to help understand its behavior and propose solutions that improve its performance when loaded.

A notable feature of pile caps is their high concentration of reinforcement. However, most of the sets of reinforcement present in these elements are complementary reinforcements that operate on the external surface of the blocks, whose main function is to reduce the cracking process of the concrete. In this context, this article has as main objective to study the influence of the inclusion of complementary vertical reinforcement in a two piles caps reinforced concrete tested by Barros [1]

2 Methodology

The study of this research was based on numerical simulations that included a non-linear elasto-plastic analysis via Finite Element Method (FEM) using the computational program ABAQUS 6.14. Its methodology was divided into two stages:

- First, a peak force calibration of the numerical model was performed with the experimental results of Barros [1]. Such calibration took place through the adjustments of the plasticity parameters of the concrete.

- Next, the numerical model containing the complementary vertical reinforcement was simulated and the influence of its inclusion in the pile cap was analyzed

3 Numerical Simulation

The pile caps used as a reference for numerical models were models M8, M11 and M12 tested and named by Barros [1]. Models M12 and M11 are models with partially embedded foundation socket with shear key interface, with and without complementary vertical reinforcement, respectively. In the case of the M8 model, its configuration corresponds to a pile cap with a monolithic connection with the column, without the presence of the socket or the complementary reinforcement, which is used as a reference pile cap for the others. Figure 1 illustrates models M12 and M8.

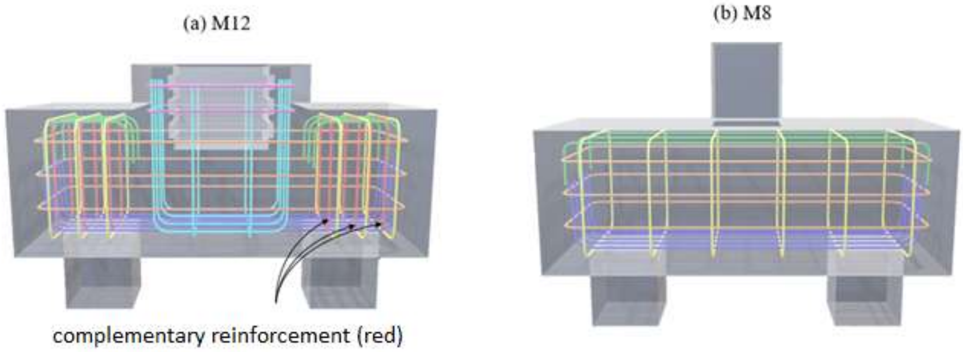

Figure 1 – Pile caps with reinforcement: (a) M12; (b) M8.

3.1 Geometry

The M8 model has a column measuring 15x15 cm and 22.5 cm high; 15x15 cm piles, 25 cm high plus 2.5 cm embedded in the block; and the block is 85 cm wide and 35 cm high and long, as shown in Figure 2.

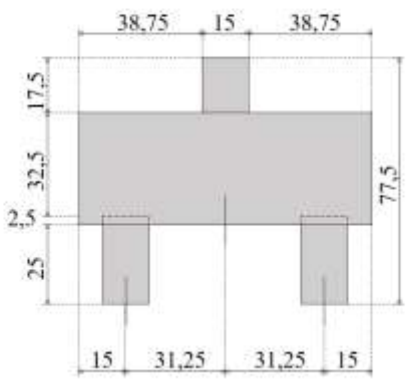

Figure 2 – M8 model dimensions. (values in cm)

3.2 Material Properties

To define the physical properties of the numerical model, the experimental results obtained by Barros[1] during his characterization of the materials were used. For the reinforcement steel, the values of the modulus of elasticity (Es) and yield stress (fy) were considered, assuming perfect elastic-plastic behavior. The value of Poisson's coefficient (ν) was adopted based on typical values for this material. Table 1 shows a summary of the adopted steel properties.

Table 1 – Stell Properties

	Ø5mm	Ø6,3mm	Ø8mm	Ø10mm
Es (GPa)	195	206	203	198
fy (MPa)	667	597	569	574
εs (‰)	3.42	2.90	2.80	2.90
ν_s	0.3	0.3	0.3	0.3

The same attribution logic was applied to the concrete parameters. From the experimental results of Barros[1] the values of the modulus of elasticity (E_c), average compressive strength (f_{cm}) and average resistance to traction (f_{tm}) were extracted. The value of Poisson's coefficient (ν) was adopted based on typical values for concrete. Table 2 brings together these properties.

Table 2 – Concrete Properties

	Column	Grout	Pile cap	Pile
E_c (GPa)	26.6	39.5	22.1	41.2
f_{cm} (MPa)	37.7	64.2	33.1	70.5
f_{tm} (MPa)	2.79	3.20	2.22	4.40
ν_c	0.2			

4 Numerical model calibration

The calibration procedure was based on the M11 experimental pile caps from Barros [1], and was divided into three steps. Initially, a parametric study was performed on the CDP variables. The objective of this study was to vary the plasticity parameters of concrete within the limits recommended in the literature and to identify how such variations affected the model.

Then, based on the results of the parametric study, one of the parameters was chosen to adjust, so that the peak force results of the M11 block simulation would approximate the experimental results obtained by Barros [1]

Finally, the simulation of the M8 model was carried out based on the settings adopted in the previous steps. This procedure aims to validate both calibrations of the M11 model.

The parameters adopted for the CDP were adjusted based on a simplified parametric study and the experimental results of the blocks tested by Barros et al [2], whose properties adopted for concrete are shown in Table 3. Of all the variables that make up the CDP, the dilation angle was the one that had the greatest influence on the results. Other information regarding calibration can be obtained in Morais Neto [3].

Table 3 – CDP Parameters of concrete

ψ	e	k_c	σ_{b0}/σ_{c0}	μ
39°	0.1	0.667	1.16	0.001

4.1 Calibration validation

In order for the calibration to be validated, it is necessary to verify if, when assigning the same parameters adopted for a new structure, we will also obtain results close to the experimental ones. For this, a numerical model of the M8 model was simulated and compared with the experimental values obtained by Barros [1]. To facilitate the comparison of peak forces, this analysis was done under displacement controlled loading.

The numerical model presented a peak force of 753.5 kN, a difference of only 3.4% in relation to the 729.0 kN presented experimentally. Such results allow us to conclude that the calibration obtained from the numerical model is valid for the different blocks under study.

5 Numerical Analysis

After obtaining the calibrated and validated numerical model of the M8 block, a simulation of the M11 and M12 model is then carried out so that the influence of the inclusion of the complementary vertical reinforcement on the peak force can be analyzed, as well as several characteristics regarding stresses, cracks and flow of loads presented by the models.

5.1 Peak force and deformation in the main longitudinal reinforcement

The results of the numerical simulation of the M12 model are presented in Figure 3. As shown in the graph, the M12 block presented a maximum load of 884 kN, an increase of only 0.03% in relation to the 883.8 kN of the numerical model M11. This demonstrates that, according to numerical simulations, the inclusion of complementary vertical reinforcement does not have a significant influence on the maximum force resisted by the block. This result can be observed in Figure 3

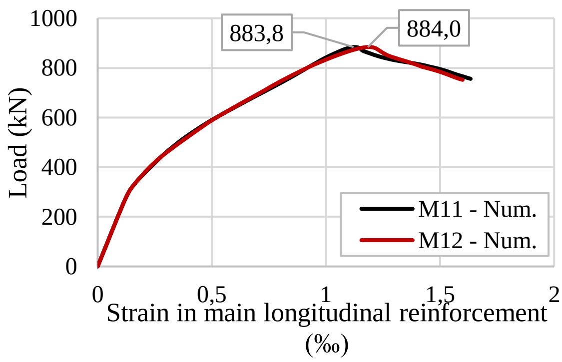

Figure 3 - Comparison between numerical models M11 and M12

5.2 Cracking

With regard to cracking, the simulation results showed a cracking panorama similar to the two numerical models, M11 and M12, and convergent with the experimental results obtained by Barros [1], where cracks were presented in the lower part of the block and on the sides following the direction of the compressed connecting rod. Figure 4 illustrates the cracking obtained in the M12 model.

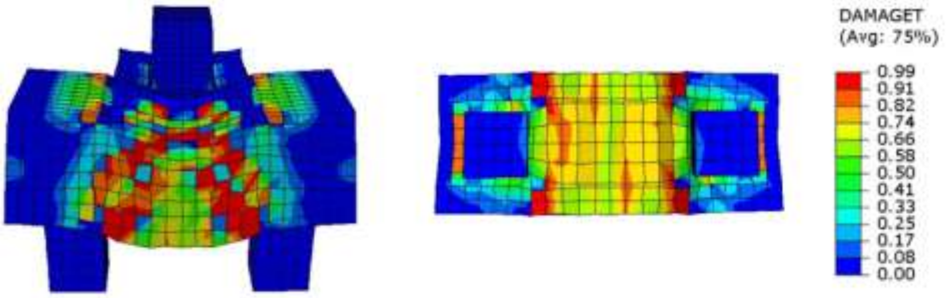

Figure 4 – Cracking view of block M12. Deformations magnified 100x. (a) side view; (b) bottom view;

Through numerical simulation it is still possible to analyze the cracking panorama within the concrete mass. Figure 5 shows the cracks formed in the longitudinal section of the block between the axis of the piles. It is shown in the image that the region covering the main tie is quite cracked, showing that the main longitudinal reinforcement is absorbing the tensile stresses and reducing the opening of cracks in this location.

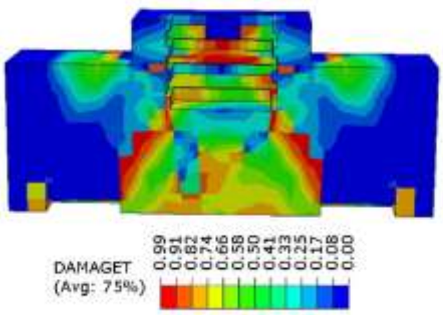

Figure 5 – Panorama of cracking in the longitudinal section of block M12 in its ultimate force

In Figure 6, it is also possible to observe two main cracks connecting the base of the column connection with the edges of the piles. These cracks are caused by the high shear stress acting between the socket and the piles. Due to the low presence of armor cutting these openings, they tend to form freely and evolve considerably. This fact is evidenced by the cracking value in this location being higher than in the region of the main tie. When analyzing the extension of the surface of these cracks in Figure 6, it can be seen that it extends throughout the entire thickness of the model, generating a high tendency to separate the central massif from the block by punching.

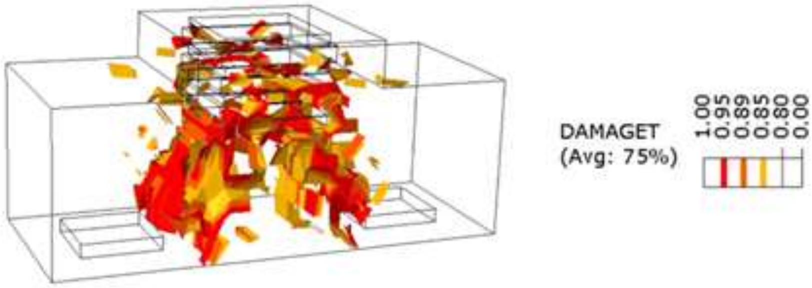

Figure 6 – 3D surface of the main cracks in the M12 model.

5.3 Stresses in concrete

The failure mode of both numerical models M11 and M12 were characterized by the rupture of the strut compressed in the connection together with the piles and without following the flow of the main longitudinal reinforcement. It was also found that one of the segments of the connection was broken by a shear key.

To study the stresses inside the block, it is interesting to define boundary limits for the compressed connecting rod. This setting makes it easier to visualize the variation of the results, such as the dimensions and inclinations of the connecting rod. However, because it is a stress gradient, defining the limits and area of a connecting rod is a complex task.

For this research, the definition of the minimum tension limit of the compressed connecting rod was made through the study of the minimum tensions presented in the longitudinal section of the block. A stress scale of 1.0 MPa was used, , and then the first value that met the following criterion was chosen: in general, the thickness covered by the scale interval from the chosen value to the value immediately after it should be approximately equal to the thickness of subsequent scale intervals. This criterion was adopted due to the fact that the stresses at the boundary of the strut present a greater variation of stresses than in the regions external to it.

It is important to emphasize that the minimum tension limit of the connecting rod varies according to the load level in which the model is. However, to be able to more effectively analyze the variation from one model to the other, it is interesting that both models present the same scale limits for certain load states. Thus, following the criteria defined in this section, the minimum tension value of the connecting rods for blocks M11 and M12 was defined according to their loading stages, being 6 MPa at peak force and 5 MPa at force limit. MPa

5.4 Stresses in complementary reinforcement

The study of tensions in the complementary vertical reinforcement aims to analyze which regime of efforts is acting on the reinforcement so that its contribution to the structure can be understood. The maximum and minimum stresses of these elements were analyzed under the instant of maximum load of the model. Figure 18 presents the results of this study. Due to the symmetry of the model, the image only illustrates the reinforcement located to the left of the column.

As shown in Figure 7, the complementary vertical reinforcement did not reach its yield stress of 667 MPa during the instant of peak force. Regarding the bars most external to the socket, it was identified that they presented tensions of the order of 5% of their maximum capacity. The maximum stress values occurred in the bars located closest to the column, with efforts in the order of 38% of the stress of.

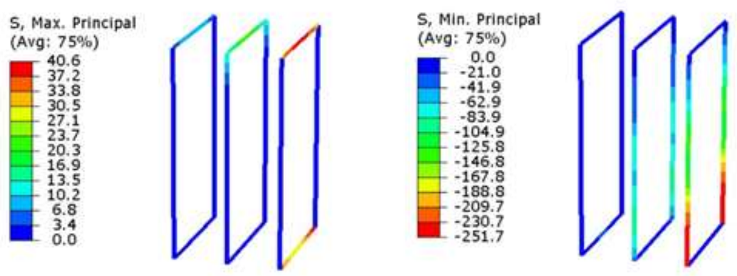

Figure 7 – Panorama of cracking in the longitudinal section of M12 model in ultimate force stage

Based on the stress values obtained, the complementary vertical reinforcements are mostly working in compression. This compression effort is concentrated in the lower region of the vertical spans of the bars close to the center of the block, exactly in the regions where the reinforcement and the connecting rod intersect. In this way, it is possible to affirm that the complementary vertical reinforcements are performing the function of reinforcing the compression efforts in the connecting rod. However, the magnitude of this reinforcement is low, so that the reinforcement does not reach its maximum resistive capacity and cannot produce significant changes in the model, as discussed in the previous items. In addition, the position of the reinforcements proved to be inefficient, since, during the maximum capacity resisted by the block, the outermost bar was stressed in only 4% of its yield stress, the intermediate one in 14% and the one next to the column in 38% . These results corroborate those obtained by Delalibera [4].

6 Conclusion

The present research carried out numerical simulations to evaluate the influence of considering a complementary vertical reinforcement in capping blocks with sockets partially embedded with shear key tested by Barros [1].

Pile caps M8, M11 and M12 were used as a reference. Models M12 and M11 being blocks with socket partially embedded with shear key, with and without complementary vertical reinforcement, respectively; and the M8 model, a block with a monolithic connection to the column, without the presence of the socket or the complementary reinforcement.

The simulation was carried out using the ABAQUS finite element software and included different analyzes in order to calibrate the model to obtain more consistent results.

From the analysis of the influence of the properties of Concrete Damaged Plasticity (CDP) it was identified that the most influential parameter in the variation of the force resisted by the block is the Dilatance Angle. Thus, this was the main parameter chosen to adjust the peak force of the numerical model with the experimental results obtained by Barros [1].

After calibrating, different characteristics of the simulated blocks were analyzed. The results pointed to very similar data with regard to shape, stress levels and strut inclination, cracking, failure mode and resistive capacity. The peak force variation between the models was only of the order of 0.03%.

Furthermore, it was identified that the complementary reinforcements are being underused. During the peak force stage of the models, the complementary reinforcements more external to the column presented maximum stress modules in the order of 4% of their yield stress, the intermediate reinforcements, 14%, and those closest to the column, in the order of 38%.

Thus, it can be concluded that the inclusion of complementary vertical reinforcement was not able to produce significant changes in the numerical models in any of the parameters analyzed by this research.

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