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Combined pile-raft and raft foundation modelling and design for three distinct office buildings in Lisbon, Portugal

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ABSTRACT: In recent years many projects were designed using the Combined Pile-Raft Foundation (CPRF) concept. Combined Pile-Raft Foundations have a complex soil-structure interaction scheme including the pile-soil interaction, pile-pile interaction, raft-soil interaction, and finally the pile-raft interaction. Consequently, there is a need for 3D numerical models that can study this complex interaction. In this paper, several 3D models are presented and discussed for the foundation design of three office buildings, that were built using the Combined Pile-Raft and Raft Foundations solutions, near the right bank of the Tagus River in Lisbon, Portugal. The study was based on geotechnical information provided by the site investigation and by Static and Dynamic Load Tests on Driven Piles, both performed in several spots evenly distributed at the site. The developed 3D model was able to simulate the behaviour of the piled raft foundation system.

Keywords: 3D Modelling; Raft Foundation; Combined Pile-Raft Foundation; Shallow and Deep Foundations; Finite Element Method

1 INTRODUCTION

In this paper, the modelling and geotechnical design of the foundations for three office buildings in Lisbon, Portugal is presented.

These buildings were built close to the Tagus River, which brought geological and geotechnical restraints.

The three lots had different constraints regarding the number of basements, as well as the ground layer found at the level at which the building foundations were to be executed:

- Lot 1, despite being in the most unfavourable geotechnical zone, with thicker alluvial deposits, it had one more basement than the other lots (with a total excavation of 11m), therefor was designed with a raft foundation directly over the more compact Miocene sand and clay layer, with areas of greater thickness near the building cores and walls with a greater load level;
- Lot 2, the most geotechnically conditioned lot, with a total excavation of 6m. In order to control differential settlement, a pile-raft solution with a combination of barrettes, with the same diaphragm wall technology used for the retaining walls, as well as pre-cast driven piles, was studied;
- Lot 3, the Miocene Layer was outcropping, therefore, despite having the same number of basements as the lot 2(with a total excavation of 7m), it was possible to design a traditional raft foundation solution, directly over the most competent geotechnical layer, corresponding to the more compact Miocene sands and clays.

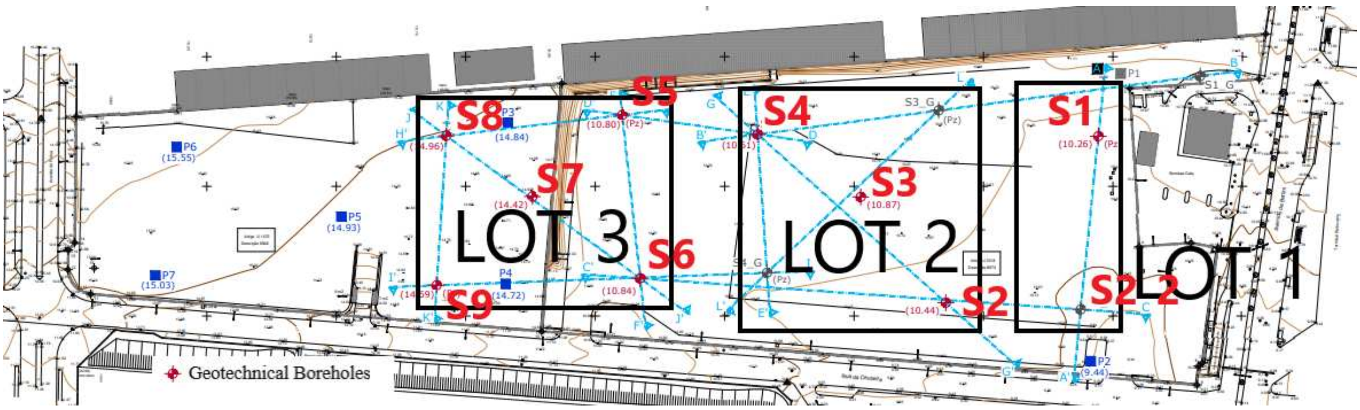


Figure 1. Building lots and boreholes for geotechnical investigation

2 GEOTECHNICAL CONSTRAINTS

This chapter presents the main geotechnical constraints that affected the solutions studied.

2.1 *Geotechnical investigation*

A site investigation campaign consisting of eleven boreholes was carried out with SPT tests. The location of the boreholes is shown in Figure 1.

In general, the area where the lots are located is at a low topography level and alluvial zone, corresponding to old water courses that formerly flowed into the Tagus River, with a general orientation WNW-ESE (main water course) and SW-NE (secondary water course). Under these alluvial deposits layer is the Miocene layer, characterized by the "Areolas de Braço de Prata" which is formed by compact sands and clays (as illustrated in Figures 2 and 3).

Throughout the intervention area were identified landfill deposits, resulting from various anthropic actions developed over time and of diverse nature.

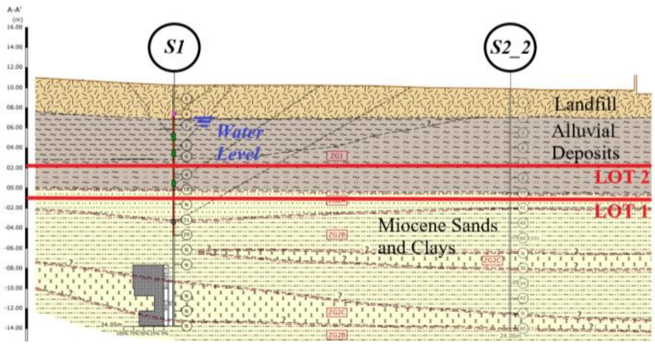


Figure 2. Site investigation geotechnical profile (Section Cut AA' from S1 to S2_2 at lot 1)

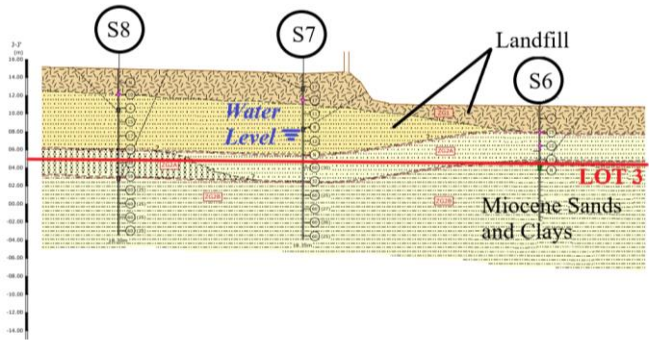


Figure 3. Site investigation geotechnical profile (Section Cut JJ' from S8 to S6 at lot 3)

2.2 *Hydrology and permeability*

The study of the ground permeability, as well as of the water lines crossing the site area proved to be very important for the design and execution of the foundation solutions.

These permeabilities were evaluated based on laboratory analyses of samples taken during the boreholes drilling.

The Miocene Layers in the investigated area are mainly composed of fine silty-clayey sands, sandy silts, and silty clays, sometimes with the intercalation of very fossiliferous levels. In general, this Miocene complex has porosity permeability (primary), but locally it may present fissure permeability in the more carbonate/fossiliferous layers (secondary permeability).

This Miocene complex, due to its fine sandy, silty-sandy, and clayey-sandy composition, has overall moderate to low permeability, providing unfavourable conditions for water percolation with moderate to not very significant flows. The retaining wall reached this layer with lower permeabilities, which drastically reduced the water inflow into the excavation pit. In general, the foundations for the 3 lots were several meters below the investigated ground water level.

2.3 *Geotechnical parameters*

The geotechnical parameters related to the identified geotechnical zones, presented in Table 1, were estimated, considering the geotechnical zones defined during the site investigation as well as specialised bibliography, and on the experience in similar works in nearby sites.

Table 1. Geotechnical Parameters from the site investigation

Geotechnical Zone	Lithology	NSPT	Cu (kPa)	φ'	E' (MPa)	Eu (MPa)
ZG1	Landfill and other materials that may have been displaced	1-17	-	30	10	-
	Alluvial Deposits	0-8				
ZG2A	Fine silty-clayey sands and sandy silts	10-41	-	33	50	-
ZG2B	Silty clays, clayey silts and sandy silts	24-60	200	-	-	100
ZG2C	Fossiliferous lumachelic/calcarenite levels	>60	-	38	150	-

3 FOUNDATION SOLUTIONS

Two types of foundation solutions were adopted for the three lots.

The main criteria for choosing the solution was the control of differential settlements. In cases where the foundation level was directly over the more compact Miocene layer, a raft solution was recommended. When the foundation level intersected alluvial deposits, a combined pile-raft foundation solution (CPRF) was studied to control deformations.

3.1 Raft foundation

The raft foundation solution consists of a continuous slab, connected to the retaining walls by bolts. In the areas where there are higher loads, as for example under the lifts and stairs boxes, greater thicknesses were considered in order to better control differential settlements.

This type of solution allows the hydrostatic vertical uplift pressures, which can be mobilized in the long term, to be accommodated, without having to provide an extensive drainage solution.

3.2 Combined pile-raft foundation

The combined pile-raft foundation (CPRF) solution consists of a continuous foundation slab connected to the peripheral walls by bolts. In areas where there is a greater concentration of loads, such as at the lifts and stairs boxes, the execution of rigid, approximately rectangular barrettes was considered, using the same diaphragm wall technology used for the execution of the retaining walls. At areas under structural columns, thicker areas of the slab were adopted, with driven precast piles resting over the more compact layer (Miocene sands and clays), in order to reduce settlement in these areas and to homogenise displacements throughout the foundation (Figure 4).

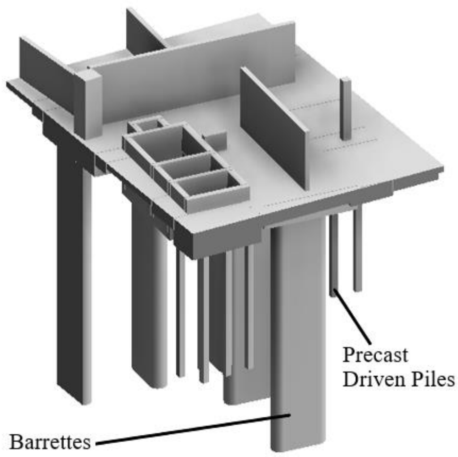


Figure 4. Pile-raft foundation solution at lot 2

4 STRUCUTRAL DESIGN

4.1 Raft foundation design

The design of a raft foundation consists of the distribution of the loads that come from the structure to the slab, which in turn will transmit them to the ground.

The key issue to this type of design is to minimise the differential settlements derived from the heterogeneous spatial distribution of structural loads.

In this particular case, there is also the action of uplift hydrostatic pressures, which were taken into account in the long-term design of this slab.

To reduce the differential settlements, thicker areas of the foundation slab were adopted under the structural columns and walls.

The design of raft foundation solutions considered the following main design states:

- Maximum total settlements;
- Differential settlements;
- Bending moments and shear forces at the raft.

4.2 Combined pile-raft foundation design

There are several design methods for combined piled-raft foundations, with guidelines from Poulos (2000, 2001), Poulos et al. (2011), Fellenius (2015), among others. The main goal is that the load is transferred to the ground by both the raft and piles. The loads are resisted by the bearing capacity between the raft and soil and by the lateral friction and end bearing capacity of the piles. The design method should comprise 4 phases:

- Preliminary study where the viability of the solution is analysed;
- Mapping of the piles and definition of their general characteristics;
- Detailed study of the optimal number of piles;
- Estimation of settlements, bending moments, and shear forces at the raft, as well as axial loads and bending moments at the piles.

The design of CPRF solutions considers, according to Poulos (2001):

- Ultimate bearing load for axial, shear stresses, and bending moments;
- Maximum total settlement;
- Differential settlement;
- Bending moments and shear forces at the raft;
- Bending moment and axial loads at the piles.

5 IN SITU TESTS

5.1 Pile Driving Analyser

To estimate the real bearing capacity of the precast driven piles, 9 load dynamic load tests were performed using the "Pile Driving Analyser (PDA)" method. The tests were performed on a square prefabricated reinforced concrete piles of section 300x300mm2 (Figure 5).

The results obtained from the dynamic load test indicate several layers of low to medium strength up to a depth of approximately 2.9 to 13.1 meters, where the shaft resistance is lower than 90kN/m². Under this layer, a zone of higher resistance was found, with a shaft resistance ranging from 128 to 524 kN/m². The tip resistance for these piles was between 14.6 and 18.2 MPa.

These values (both for the shaft and tip resistance) are an underestimation since low energy was being transferred during these tests to guarantee pile integrity.

Figure 5. Pile Driving Analyser preliminary tests being executed at lot 2

6 MODELLING

The modelling was performed using software PLAXIS 3D. For the pile-raft foundation of lot 2, PLAXIS 2D axisymmetric analysis were also used for the calibration of the modelled inclusions with empirical methods widely used in the past.

6.1 FE models for structural elements

Different finite element (FE) models were chosen for each of the structural elements to simulate their behaviour in both the short and long term.

6.1.1 Foundations slabs

The foundation slabs were modelled with plate elements (PLAXIS, 2023).

6.1.2 Precast driven piles and barrettes

Both the precast driven piles and the barrettes were simulated using the embedded beam element in PLAXIS 3D. These elements describe the interaction between the piles and the surrounding soil. The interaction at the shaft and at the tip is described by means of embedded interface elements. The pile is considered as a beam which can cross a volume element at any place with any arbitrary orientation. Due to the existence of the beam element three extra nodes are introduced inside the volume element.

6.1.3 Connection between the foundation slab and the retaining wall

The ground retaining wall and the foundation slab are connected by a linear element which allows shear forces to be transmitted, however rotations are free. This behaviour allows the simulation of the bolted connection between the two elements.

6.1.4 Structural columns and walls

The structural loads for all relevant combinations, are simulated in the model by the means of point loads applied at the gravity centre of the structural columns and walls.

The structural wall behaviour was simulated by implementing a rigid beam along the wall centreline, which will redistribute the point load along the walls path, and there for avoiding any numerical problems that may appear due to the high concentrated loads (applied at the walls gravity centres).

6.1.5 Soil elements

The soil was modelled using Hardening Soil, with volume elements in the 3D models. The Miocene sand layers were set to respond with a drained behaviour and the Alluvial deposits, and the Miocene clay layers were set to respond with an Undrained(A) behaviour (effective stress analysis) (Tables 2, 3 and 4).

Both short term and long-term situations were accounted for in the analysis.

Table 2. Material properties in the 3D Model

	Properties	
	Material model	**Drainage type**
ZG1	HS	Undrained (A)
ZG2A	HS	Drained
ZG2B	HS	Undrained (A)
ZG2C	HS	Undrained (A)

Table 3. Material stiffness in the 3D Model

	Stiffness		
	E_{50}^{ref} (kPa)	**E_{oed}^{ref} (kPa)**	**E_{ur}^{ref}(kPa)**
ZG1	10000	10000	30000
ZG2A	50000	50000	150000
ZG2B	100000	100000	300000
ZG2C	150000	150000	450000

Table 4. Material resistances in the 3D Model

	Resistance	
	c'^{ref} (kPa)	**φ' (°)**
ZG1	1	28
ZG2A	1	38
ZG2B	40	36
ZG2C	20	39

6.2 *PLAXIS models*

6.2.1 *Raft foundation models for lots 1 and 3*

The raft foundation models for lots 1 and 3 were obtained by modelling plate elements with different thicknesses (Figure 6). The geotechnical layers were modelled according to the parameters presented at Table 1.

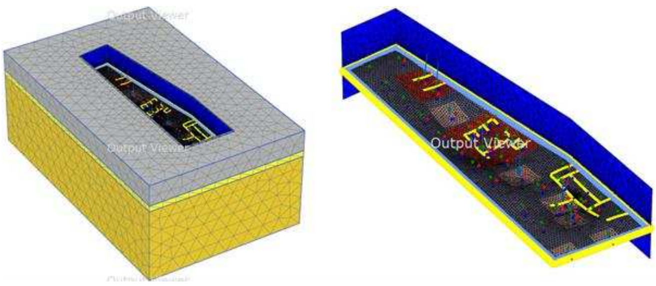
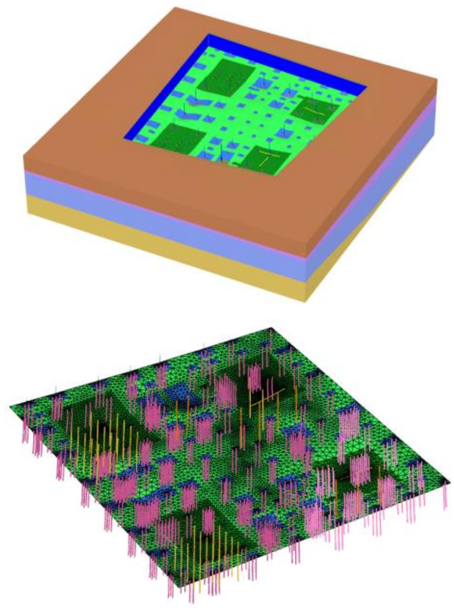


Figure 6. PLAXIS 3D model for lot 1 (a) with soil elements; (b)without the soil elements

6.2.2 *Combined pile-raft foundation model for lot 2*

The calculation model for Lot 2 proved to be quite complex to build, however, with the aid of some homemade programs, and due to the easy integration of python in PLAXIS it was possible to effectively model all the 622 driven precast piles, the 41 barrettes and the 204 point loads, with the 20 relevant structural combinations (Figure 7)



a)

b)

Figure 7. PLAXIS 3D model for lot 3: (a) with soil elements (b) without soil elements

6.2.3 *PLAXIS 2D axisymmetric modelling for embedded beam behaviour calibration*

The three-dimensional model for lot 2 was calibrated using the axisymmetric mode of PLAXIS 2D, where the response of the embedded beams was compared with the real pile behaviour.

The results (illustrated on Figure 8) show that the loads, for the failure criterion, estimated using the 2D axisymmetric model match the maximum service load calculated using empirical methods (Bustamante and Gianeselli, 1993, 1998).

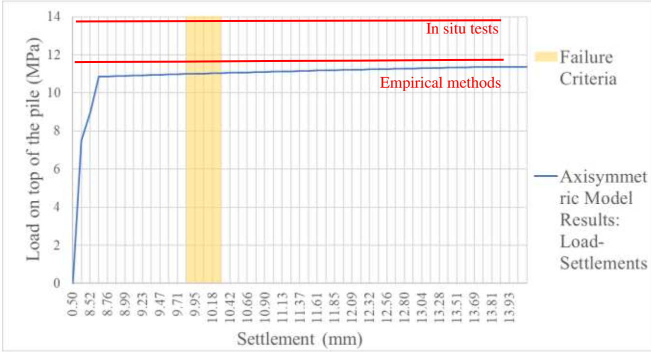


Figure 8. Load-Settlement obtained in the 2D model. An ultimate load of 990kN was obtained for the failure criteria, for the precast driven piles

These results were lower than the values obtained in the PDA tests. This proves that the empirical methods used, that are based on a collection of data from numerous field trials, take on assumptions so that the values obtained for the theoretical bearing capacity are, in most cases, conservative in relation to their actual capacity.

7 FINAL DESIGN

For the safety verification against relevant limit states, partial safety factors relative to both the actions and the materials, were adopted according to the European standard regulations.

7.1 *Ultimate limit states*

The final design of the foundation slab and the deep foundation elements was made using the results of the three-dimensional models, duly calibrated.

A load envelope was considered for each structural element, based on the most relevant combinations.

It should be noted that the lots in question are located in a seismic zone, and therefore there are, in all the buildings, sets of stairs and lift boxes and walls that have severe loads that need to be transmitted to the foundations.

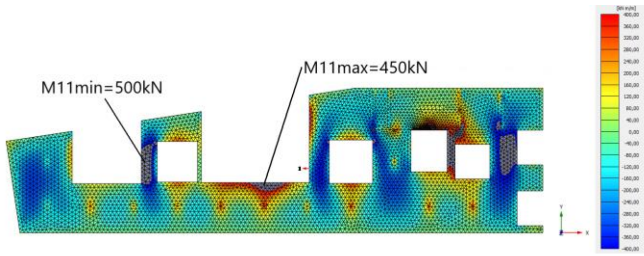


Figure 9. Bending moments M11 at the raft plate for a single combination in the lot 1 model

Figure 9 shows the distribution of the bending moments in the lowest thickness slab. The distribution of loads on the slab leads to cylindrical bending between the spans where there are no deep foundation elements. The areas over deep foundations will be stiffer, and therefore behave as supports for the hydro-static uplift pressures.

For the punching verification, the column that had the highest vertical load was checked according to Eurocode 2. The dimensions varied between pile cap and raft due to the variation in thickness of both solutions. Both solutions were designed against punching.

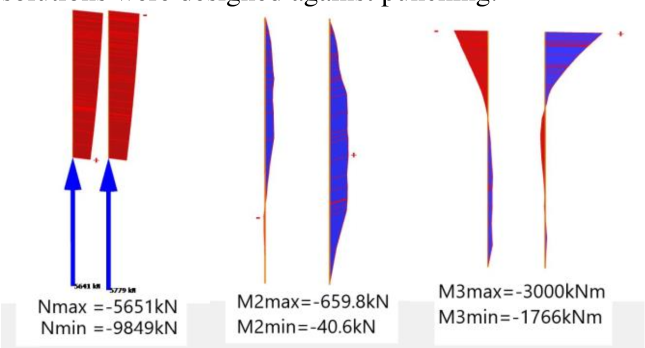


Figure 10. Forces in a pair of barrettes embedded beams in the model, for a single combination at lot 2

All the precast driven piles and the barrettes were checked for combined axial force and biaxial bending, adopting the results from the model (Figure 10).

7.2 Service limit states

For the serviceability limit state checks, several analyses were carried out.

7.2.1 Verification of the maximum settlements

The maximum settlement for each model was accessed and checked, for the relevant combinations. The maximum values of 6.7mm, 20.3mm and 16.3mm were obtained, respectively, for lots 1, 2 and 3.

7.2.2 Verification of differential settlements (distortions)

To check the differential settlements, the distortions were checked for each model.

Equation (1) shows the formula used to check the distortions at lot 1.

$$\tan\left(\theta_{rel,max}\right) = \tan\left(\frac{(\delta_{max}-\delta_{min})\times 10^{-3}}{distance}\right) = \tan\left(\frac{(6.7-2)\times 10^{-3}}{15}\right) = 5.5\times 10^{-5} \leq \frac{1}{2000} = 5\times 10^{-4} \qquad (1)$$

7.2.3 Verification of the global settlements near the façade, due to architectural limitations.

Due to architectural limitations, it was necessary to verify the differential settlements at the lots peripherical zones. The models provided the ability to quickly access this information, as shown in Figure 11, for lot 2.

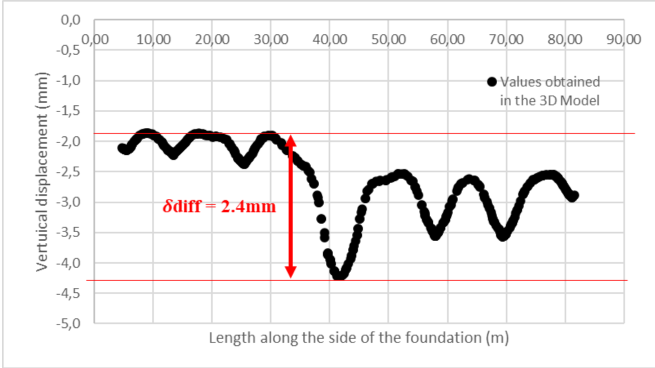


Figure 11. Total settlements from two corners at the lot 2 model

8 CONCLUSIONS

Higher processing power and access to smaller and greater data storage units are allowing more complex models to be built and more quickly. The described type of model can now be built, calibrated, and ran within most foundation projects time frame and enables the study of crucial issues in this type of project, such as soil-structure interaction, soil permeability, different settlements caused by different foundation stiffness zones, geological and geotechnical changes throughout the area site and much more, to be simulated effectively.

9 ACKNOWLEDGEMENTS

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