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Geotechnical Design and Structural Evaluation of End-Bearing Piles Subjected to a Double Consolidation Process

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Abstract. The regional subsidence that occurs in Mexico City derives from the overexploitation of water for the supply of the city. This phenomenon has a consequence on the foundation elements, such as piles. The effect that is generated is known as negative skin friction. The mechanical elements are quantified with special attention to the axial load that is generated in a pile group subjected to a double consolidation process. Models were developed base on the Finite Element Method in three dimensions (FEM 3D) based on end-bearing piles using the PLAXIS 3D program. Based on the results obtained, the structural evaluation of the piles was made to know the influence of negative skin friction.

Keywords. Regional subsidence, negative skin friction, end-bearing pile.

1. Introduction

Given the accelerated growth that Mexico City has presented in recent years, urbanization plans have evolved to vertical growth, making the constructions heavier. Considering the precarious characteristics of the lacustrine clays of the city, whose high compressibility and low resistance to shear stress are widely known [1-3], it is necessary to resort to deep foundations to ensure the stability of the structure.

In Mexico City, the design of foundations must take into account the regional subsidence. This phenomenon is related to the reduction of the piezometric loads due to the overexploitation of aquifers for water supply. One of the main effects of the regional subsidence on deep foundations is the negative skin friction that can cause the increase of axial loads on piles, the development of excessive differential settlement or its structural failure as reported by various authors [4-6]. In this work the axial load generated by negative skin friction is quantified in a pile group subjected to a double consolidation process (weight of the building and regional subsidence) in the lacustrine zone of Mexico City. The foundation system consists of a slab with end-bearing piles. The system was analyzed using three-dimensional models with the Finite Element Method (FEM 3D) using the PLAXIS 3D program that allowed to simulate the regional subsidence to quantify the negative skin friction developed in driven piles. The interaction of different piles was studied by their location in the group (interior, side and

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corner). Finally, the structural evaluation of the piles considering the increase of load on the piles due to negative skin friction is presented.

2. Study in piles groups subjected to negative skin friction

2.1. Geotechnical model

The site stratigraphy corresponds to a typical "Lake Zone" according to Mexico City Geotechnical Zoning [4]. It consists of soft clays interspersed with seams and layers of harder clayey sands. The soil profile can be described as follows: (a) the surface crust (from 0 to 5 m) that for analyzing purposes is subdivided into two sub-strata the dry (DCR) and the moisture crust (MCR); (b) the upper clay formation (UCF) located between 5 and 29 meters that according to its water content and preconsolidation level was subdivided into 3 sub-strata; (c) the Hard Layer (HL) located from 29 to 31 m deep and consisting of clayey sands with blow count higher than 50 and an average water content of 50%; and (d) the Lower Clay Formation (LCF) composed of highly compressible clays with an average water content of approximately 150% and a superior resistance to UCF.

2.2. Initial stress state, piezometric conditions and soil properties

Figure 1a shows the initial stress state for the study site. The groundwater level (NAF) is 2 m depth. The initial pore water pressure profile was obtained from the piezometric measurements in permeable lenses located at different depths in the UCF and the HL. This initial value was considered as a representative pore water pressure drawdown of the area. Additionally, Figure 1b presents two hypotheses of future water drawdown that represent a moderate (Hypothesis #1) and extreme but possible scenarios (Hypothesis #2), respectively.

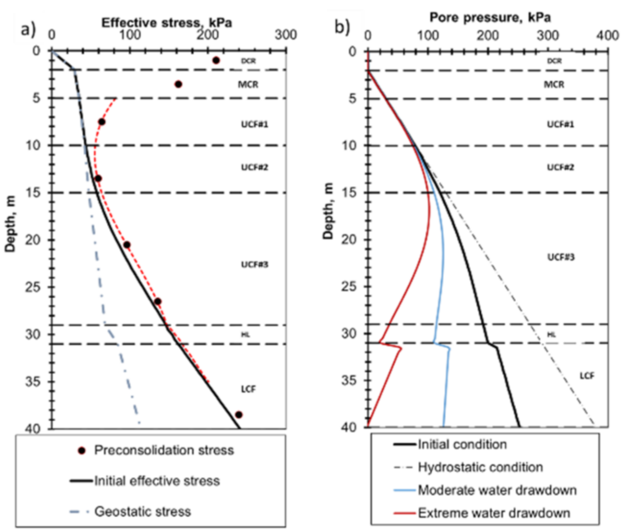

Figure 1. Initial stress state and piezometric conditions.

The numerical analyses were carried out in terms of effective stress considering drained parameters of the soil strata. This is because deferred settlements are of interest in this work. The dry crust, the moist crust and the hard layer were modeled as isotropic elastoplastic materials with a Mohr-Coulomb failure criterion (Mohr-Coulomb model, MC), whereas for the clayey strata the Soft-Soil model (SS) was considered. Table 1 and 2 summarize the parameters used for each stratum.

Table 1. Properties of the Mohr-Coulomb model.

Stratum	γ kN/m³	E' kPa	v' -	φ' °	k_0 -
DCR	14.5	4,825	0.25	55	1.17
MCR	12	3,444	0.25	47	0.82
HL	18	10,000	0.33	45	0.29

Table 2. Properties of the Soft-Soil model.

Stratum	γ kN/m³	λ* -	κ* -	POP kPa	k_0^{NC} -
UCF #1	11.4	0.35	0.021	25	0.32
UCF #2	11.1	0.30	0.021	5	0.36
UCF #3	11.5	0.33	0.018	10	0.36
LCF	13.3	0.22	0.015	10	0.36

2.3. Foundation analysis

In the analyses a 30 m square foundation slab was considered, displaced at level of natural terrain ($D_f = 0$) added with end-bearing piles supported on the CD. The piles consisted on reinforced concrete elements with a length (L) of 29 m and a circular cross section of 0.5 m diameter. The separation between piles (S) was defined in terms of the bearing capacity, using the calculations proposed by [6] and [9]. A theoretical separation of 3.8 m between piles was obtained. For practical purposes, a separation of 3 m between piles was considered. Table 3 shows the number of piles for the proposed dimensions. The design of the piles was carried out within the "Design in terms of bearing capacity" philosophy. A total load of 100kPa was considered.

Table 3. Piles characteristics.

B=L (m)	SA m²	S m	NP --
30	900	3	100

3. Results and discussion

Given the symmetry of the problem and to speed up calculation times, a quarter of the complete physical problem was modeled. Figure 2 shows the model used. The stages of analysis were the following:

- Stage 1: application of a uniformly distributed load on the slab, with magnitude of 100 kPa.
- Stage 2: the load is maintained and the soil is subjected to a moderate pore water pressure drawdown.
- Stage 3: the load from stage 1 is maintained and the soil is subjected to an extreme pore water pressure drawdown.

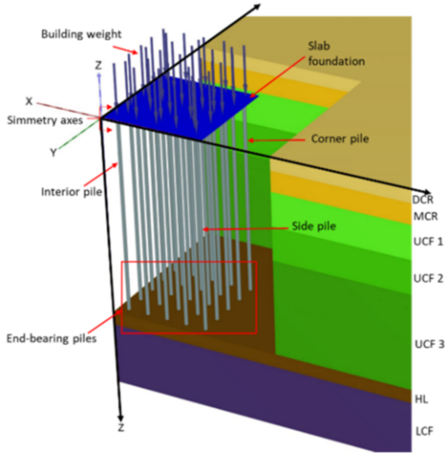

Figure 2. Foundation slab model with end-bearing piles in a FEM 3D program.

The analysis of the results was carried out in terms of the vertical displacements in the foundation slab and the axial load (Q_p) developed in the end-bearing piles. Likewise, the regional subsidence induced by different levels of pore water pressure drawdown was estimated. The settlement for the stage 2 (moderate) was 1.8 m and the settlement for the stage 3 (extreme) was 3.6 m.

3.1. Vertical displacements in the foundation slab with end-bearing piles

- Stage 1: The maximum displacements were located in the center of the foundation slab. The rigidity of the slab it is important, because it affects the magnitude and distribution of the displacements. These results agree with those obtained by [9].
- Stage 2. A change in the location of the maximum displacements was observed, which are now located at the edges and corners of the foundation slab. This change is attributed to the appearance of negative skin friction, which develops to a greater extent in the corner and side piles with respect to the interior piles.
- Stage 3. A similar behavior to stage 2 was observed, with differences in magnitude of displacements due to the increase in negative skin friction caused by extreme pore water pressure drawdown. Figure 3 shows the displacement contours mentioned. Table 4 presents the maximum displacements ($\delta_{máx1,2,3}$) for the different stages, as well as the net vertical displacements ($\delta_{n2,3}$). The net displacement is defined as the difference between the displacement by regional subsidence without foundation ($\delta_{HR2,3}$) and the maximum displacement with foundation either for Stage 2 or 3.

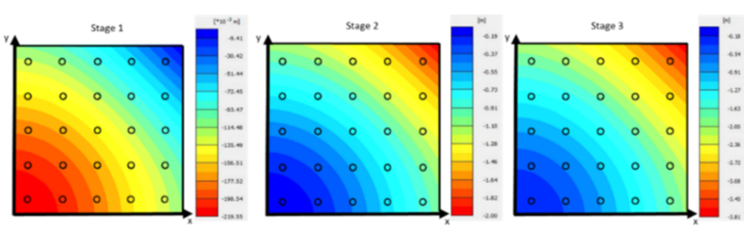

Figure 3. Contour of verticals displacements on the foundation slab for the different stages.

Table 4. Maximum and net displacements in the different stages.

Foundation	Displacements δ (m)				
	Stage 1	Stage 2		Stage 3	
	$\delta_{máx1}$	$\delta_{máx2}$	δ_{n2}	$\delta_{máx3}$	δ_{n3}
End-Bearing Piles	-0.06	-1.44	0.36	-2.32	1.28

3.2. Axial load developed in the interior, side and corner piles

3.2.1. Stage 1

Figure 4 shows the variations in axial load due to the location of the different piles and the rigidity of the foundation slab. The transmitted stresses tended to concentrate on the edges, and to a greater extent on the corners. The axial load on the head of the piles was greater for those located in the corner. And lower for the interior ones. The load transfer from the pile to the ground through the shaft was also observed, since a smaller tip load was noticed compared with that in the head.

3.2.2. Stage 2

The sinking generated by the reduction of the piezometric levels allowed the development of negative skin friction, exerting greater axial load on the corner piles than on the side and interior ones. This can be attributed to the fact that the area of influence was considerably greater in this pile.

3.2.3. Stage 3

A considerable increase in the axial load was observed due to negative skin friction, with a change of location of the neutral axis of the different piles. There was a substantial development of the pile toe bearing resistance. A loading and unloading area were also observed at the head of the piles, this was attributed to the slab's stiffness and negative skin friction. Negative friction drag force tried to vertically displace the corner and side piles; while the slab tried to keep them in their location until they are released and discharged, contrary to interior piles that tend to increase their load.

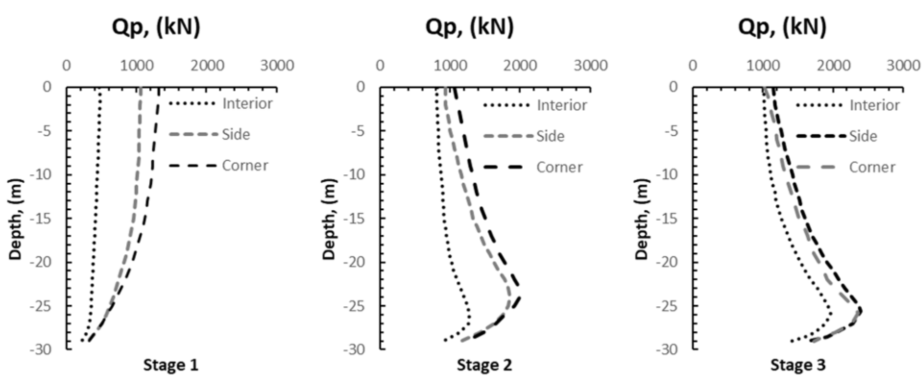

Figure 4. Axial load (Qp) for the different stages in end-bearing piles.

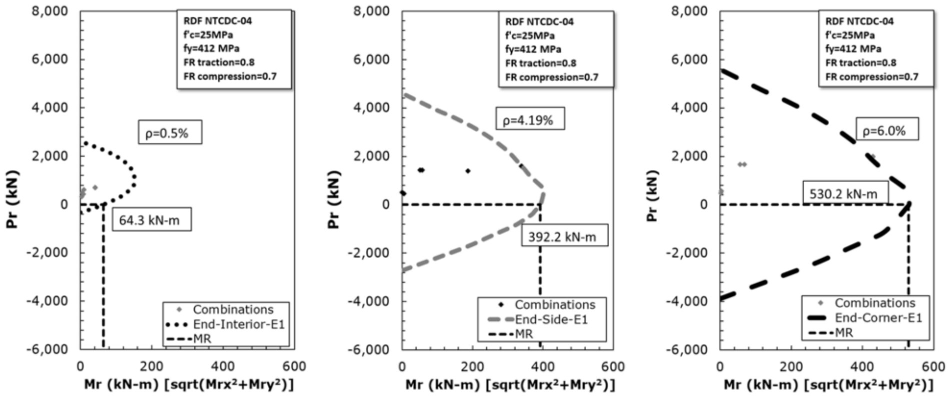

Figure 5. Interaction diagrams for Stage 1 on end-bearing piles.

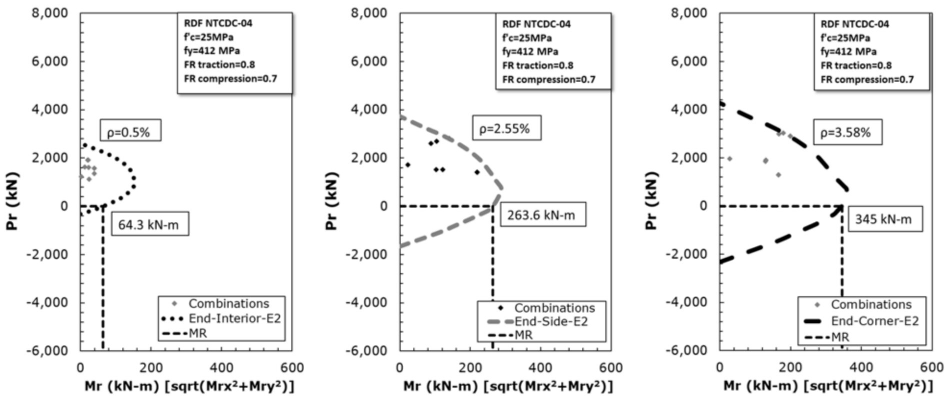

Figure 6. Interaction diagrams for Stage 2 on end-bearing piles.

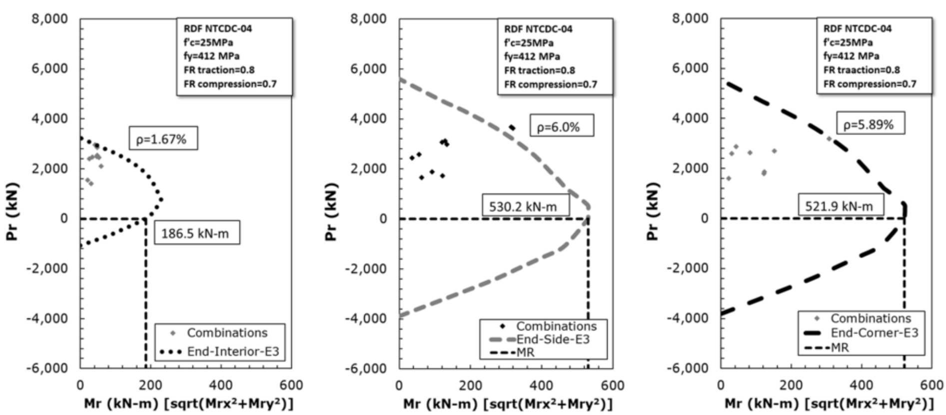

Figure 7. Interaction diagrams for Stage 3 on end-bearing piles.

3.3. Structural evaluation of end-bearing piles

For the structural design of the piles, it was considered that the soil that confine them does not allow buckling due to load application. Thus, they could be analyzed as short columns. In this work, the design process focused on the axial load by weight of the building and negative skin friction. In addition, The Complementary Technical Standards for Design and Construction of Foundations 2017 [8] stipulates that the buckling on piles should be checked when they have a diameter less than or equal to 40 cm. A concrete compressive strength (f'_c) of 25MPa and a yield strength of reinforcement steel (f_y) of 412 MPa was considered. This data, as well as the mechanical elements obtained from the numerical models made with the PLAXIS 3D program (all the mechanical elements can be found in [6]) were entered into a program that incorporates the RDF standards NTCDC-2004[10] and ACI 318-14. Figures 5, 6 and 7 show the interaction diagrams for the design of short columns in flexo-compression.

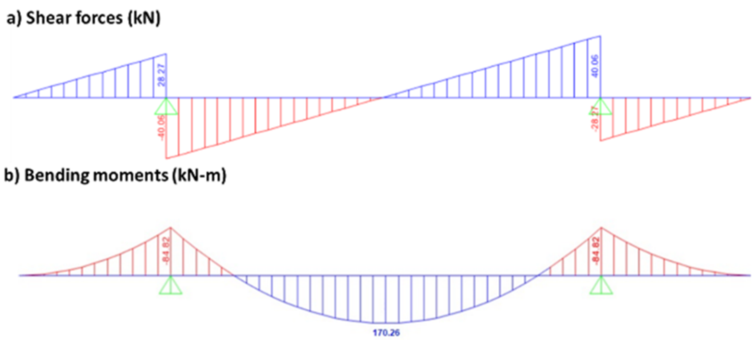

Figure 8. Shear (a) and bending moments (b) diagrams for a configuration of 2 lifting points.

The most important effect in the design of a pile is the axial load to which it will be subjected during its performance, however, sometimes it is required to design them for actions that are generated by its handling (shear forces and bending moments). The dimension of the cross-section as well as the reinforcing steel of the piles are commonly governed by the stresses derived from its handling. As a result of the elements own weight the value of the shear forces and bending moments will depend on the lifting form, the number of supports and their distribution along the piles. Lifting points were used that are considered in professional practice as shown in [11]. Figure 8 shows the shear forces and bending moments of the piles using two lifting points to compare them with the mechanical elements developed due to the weight of the building and negative skin friction (Table 5).

Table 5. Comparison of resistant bending moments and last moment of the section due to its installation.

Pile	Bending moments (kN-m)			
	Stage 1	Stage 2	Stage 3	$Mu_{lifting}$
Interior	64.0	64.0	186.0	
Side	392.0	263.0	530.0	255.0
Corner	530.0	345.0	521.0	

4. Conclusions

In end-bearing piles it is expected that only negative skin friction exists when supported by a competent layer, but due to small displacements in the HL and the LCF have a behavior very similar to the friction piles.

There is a compensatory phenomenon of the vertical displacements for the foundation slab since when the negative skin friction is generated in the piles within the group, the maximum displacements pass from the center to the periphery thereof.

More negative skin friction develops in the corner piles than in the side and interior piles. In the interior piles, less negative skin friction load develops due to the fact that it is protected by the other piles. In the side and corner piles a discharge was observed in the head, this attributed to the negative skin friction drag force tries to displace them vertically and due to the stiffness of the slab tries to restrict this displacement, thus producing the load reduction. This can be corroborated when observing the increase of the load on the head of the interior piles.

In the structural part, for the end-bearing piles a behavior was observed where the pile of interior needs smaller amount of steel of reinforcement that those of side and corner.

It is concluded that it is necessary to take into account the axial load by negative skin friction in end-bearing piles since the combination of failure takes into account the axial load generated by negative skin friction.

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