



## Criteria for the design of friction piles subjected to negative skin friction and transient loads

## Criterios para el diseño de pilotes de fricción sujetos a fricción negativa y cargas accidentales

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### Abstract

In this paper, an analysis of the behavior of an individual friction pile subjected to negative skin friction and transient loads (seism or wind) for typical stratigraphic and piezometric conditions of Mexico City lacustrine zone is presented. This analysis is performed using a two dimensional finite element model (2D FEM) and taking as a reference the field and laboratory researchs on individual point bearing piles developed by Fellenius (1998), Bozozuk (1981) and Leung *et al.* (2004). A 2D FEM study of an internal pile within a pile group subjected to the weight of the structure and to negative skin friction is also presented. Finally, for the same conditions, 3D FEM analyses results are presented in order to assess the behavior of border and corner piles. Some criteria are proposed regarding design loads and negative skin friction estimation for internal, border and corner piles.

**Keywords:** Friction piles, negative skin friction, regional subsidence, numerical modelling, constitutive models, finite element method, design criteria, transient loads

### Resumen

En este artículo se presenta un análisis del comportamiento de un pilote de fricción individual sujeto a fricción negativa y cargas accidentales (sismo o viento), para condiciones estratigráficas y piezométricas típicas de la zona lacustre de la Ciudad de México. Dicho análisis que recurre al método de los elementos finitos bidimensional (MEF 2D) toma como referencia los estudios de campo y laboratorio de pilotes individuales desarrollados por Fellenius (1998), Bozozuk (1981) y Leung *et al.* (2004). Asimismo, se expone el estudio, también basado en el MEF 2D, de un pilote que forma parte de un grupo sometido al peso de la estructura y a fricción negativa. Finalmente, para las mismas condiciones, se presenta un análisis basado en el MEF 3D que permite evaluar el comportamiento de pilotes de borde y de esquina. A partir de los resultados de los distintos estudios mencionados, se proponen algunos criterios respecto a las cargas de diseño y a la estimación de la fricción negativa en pilotes internos, de borde y de esquina.

**Descriptores:** Pilotes de fricción, fricción negativa, hundimiento regional, modelado numérico, método de elementos finitos, modelos constitutivos, criterios de diseño, cargas accidentales

## INTRODUCTION

In the soft soils of Mexico City, friction piles have been used to reduce settlements (the design is in terms of deformation), most commonly as part of a box-type foundation. Less frequently, they have been used to support the total load of the structure and ensure the stability of the foundation (the design is in terms of bearing capacity). In all cases, a complex interaction develops between the subsoil, the piles and the structure. The subsoil is subjected to a double consolidation process: one of them is induced by the weight of the structure and the other is due to the decrease of pore water pressures caused by the intense pumping of the groundwater of the city that induces regional subsidence. The analysis and the design of foundations with friction piles in these difficult conditions is further complicated when the effect of transient loads such as seismic or wind loads is considered.

Worldwide, a substantial number of field studies related to the development of negative skin friction in individual piles have been performed (Plomp and Mierlo, 1948; Bjerrum *et al.*, 1969; Endo *et al.*, 1969; Walker and Darvall, 1973; Auvinet and Hanel, 1981; Bozozuk, 1981; Clemente, 1981; Fellenius, 1998), while only one publication with respect to laboratory studies of individual piles (Leung *et al.*, 2004) has been found in the literature.

Zeevaert (1957; 1963), Reséndiz and Auvinet (1973), Auvinet and Díaz-Mora (1981), Alberro and Hernández (2000) and Rodríguez (2011) have proposed analytical models for the analysis and design of friction piles foundations interacting with regional subsidence.

Nowadays, numerical models such as the *Finite Element Method* (FEM) offer new possibilities for the analysis of foundations subjected to negative skin friction (Jeong *et al.*, 1997; 2004; Auvinet and Rodríguez, 2001; 2002; Lee and Charles, 2004; Comodromos and Bareka, 2005; Rodríguez 2011; Rodríguez *et al.*, 2015). These models allow a detailed evaluation of the magnitude of the developed stresses at the piles tips and shafts, as well as the consequent deformations.

In this paper, the results of numerical analyses developed by the authors, based on two dimensional (2D FEM, axisymmetric) and three dimensional (3D FEM) finite element method, of friction piles in typical Mexico City soil and piezometric conditions, are presented. From these results some criteria for the design of this type of foundation have been developed and proposed to be included in the new Mexico City building code.

In the first part of this work, the analysis of the behavior of an individual friction pile subjected to negative skin friction and transient loads (seism or wind) using 2D FEM, is presented. This analysis reproduces field and laboratory research on individual point bearing piles developed by Fellenius (1998), Bozozuk (1981) and Leung *et al.* (2004). In the second part, the study of an internal pile within a pile group subjected to the weight of the structure and to negative skin friction using also 2D FEM, is shown. Finally, for the same conditions, some 3D FEM analyses are presented in order to describe the behavior of border and corner piles. Some design criteria are presented regarding design

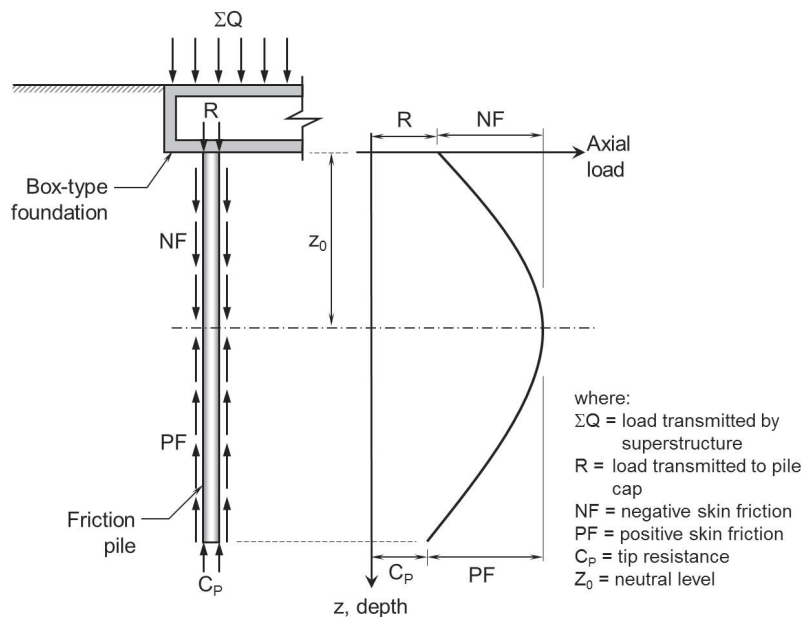


Figure 1. Loads developed on a friction pile subjected to  $NF$  due to pore pressure drawdown

loads and negative skin friction for internal, border and corner piles.

### NEGATIVE SKIN FRICTION

Negative skin friction (*NF*) can be defined as the downward traction that develops along the piles shaft when the surrounding soil consolidates due to piezometric drawdown or to surface loading. The forces that resist the penetration of piles are the *positive skin friction (PF)* and the tip resistance ( $C_p$ ), Figure 1. The depth where the relative displacements between pile and soil are nil and where skin friction changes from negative to positive is called neutral level ( $z_0$ ).

### SINGLE FRICTION PILE SUBJECTED TO *NF* AND TRANSIENT LOADS

#### FIELD TESTS RESULTS

Fellenius (1998) presents the results of more than seven years of measured *NF* developed in two 30cm diameter driven concrete piles (Figure 2a). These piles were driven through a 40m thick compressible layer formed by clays and silts. Their tips are located at a depth of 52m

in compact silty sand. The developed *NF* was due both to a slight regional subsidence and to an external load placed around the piles caps.

In Figure 2b the results of seven years of measurements made on one of the instrumented piles (pile II) for different stages of the tests, are shown. During pile installation (after driving,  $t=0d$ ) an increase of axial load due to the soil re-consolidation induced by the excess pore pressure dissipation was observed. After 495 days (Stage I,  $t=495d$ ) *NF* developed along the pile shaft due to regional subsidence down to gauge M5. At Stage II ( $t=520d$ ), *NF* between the surface and gauge M6 practically disappeared when a 440kN load was applied at the pile cap and a negligible increment of the maximum axial load (gauge M5) was observed. After 330 days ( $t=850d$ ) *NF* had recovered and a substantial increment of the maximum axial load was measured. At Stage III ( $t=850d$ ), when a new 360kN load was added on the pile cap, *NF* disappeared and even became positive from surface through gauge M6 and again no significant increment of the maximum axial load was observed. After 1072 days ( $t=1922d$ ), for a second time, *NF* recovered and the maximum axial load substantially increased.

Meanwhile, in 1981, in Canada, Bozozuk showed the results of a ten years field test developed in a 49m

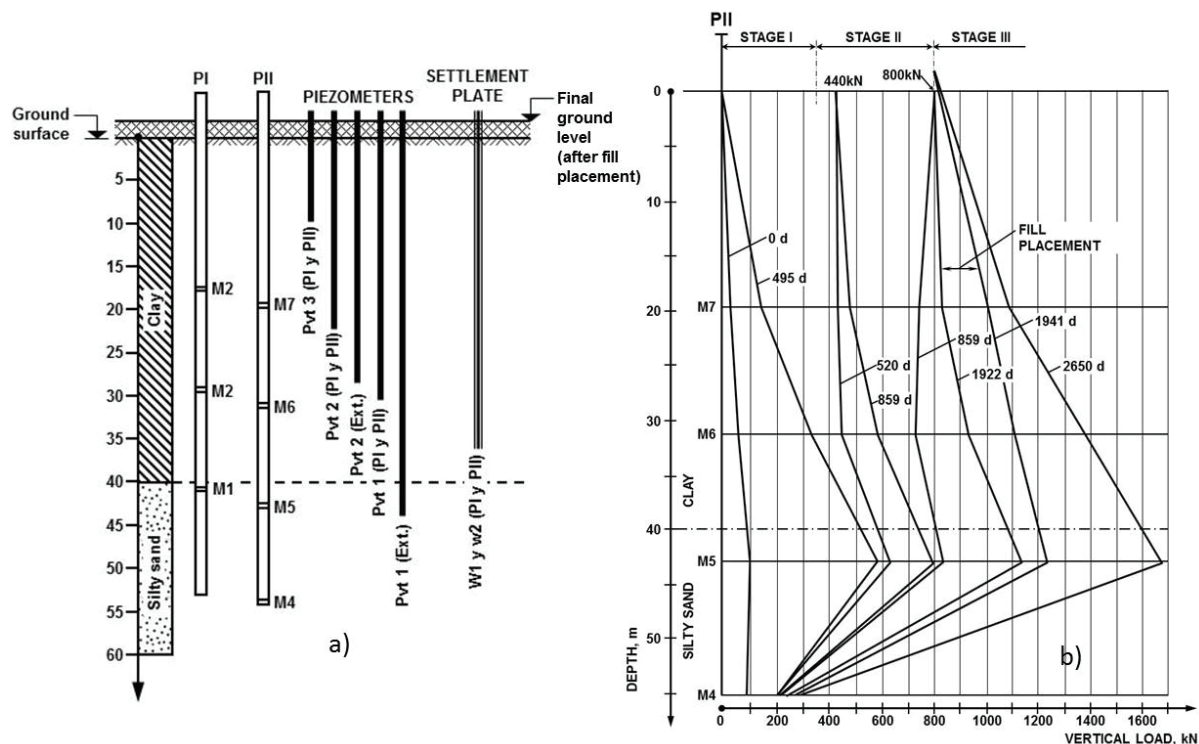


Figure 2. a) Soil conditions and pile and site instrumentation, and b) vertical distribution of load in Pile II (obtained and modified from Bjerin, 1977)

long and 32.4cm diameter steel pile, which was subjected to  $NF$  due to consolidation of a 73m thick marine clay, generated by a 10m high sandy embankment (Figure 3a).

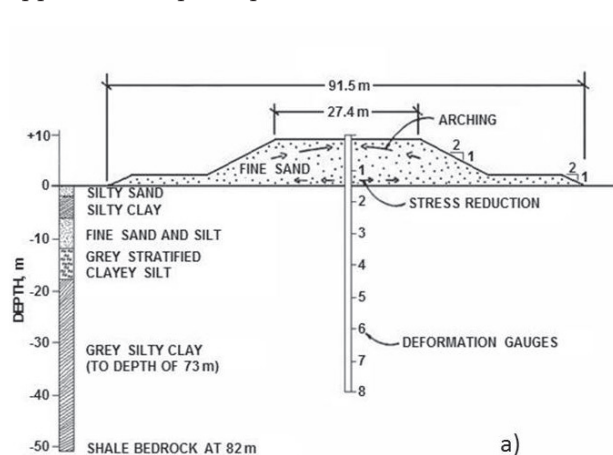
In Figure 3b the variation with depth of the developed axial load in the pile is presented. It can be seen that, after ten years of pile driving, the maximum developed axial load ( $Q_{max}$ ) was about 1.52MN. Then, a two stages bearing capacity test was performed. At the first stage an external load equal to  $Q_{max}$  was applied in three increments, in a total period of 11 days. After this period the load was removed, letting the pile to rest for three days. For the second stage the load was increased to  $2Q_{max}$  also in three increments but in a total period of seven days.

It is possible to observe that the obtained results are similar to those reported by Fellenius, since when the incremental external load is applied,  $NF$  decreases and even turns to positive and  $Q_{max}$  remains almost the same.

In both field tests it was shown that piles who developed negative skin friction behaved as preloaded elements that are able to carry on external loads (such as transient loads due to seism or wind) with no increase of the developed maximum axial pile load.

#### LABORATORY TEST RESULTS

In 2004, Leung *et al.* present the results obtained from a scale centrifuge model developed in order to investigate the  $NF$  in piles installed in soft clays which consolidate due to their own weight and the weight of a shallow sand fill. Also, the behavior of the pile under the combined effect of the developed  $NF$  and an external load applied on the pile cap was studied.



The study consists in modeling the behavior of an instrumented hollow pile 18.5m long and 1.6m in diameter, placed through a 16m thick soft clay layer and resting on a compacted sand (Figure 4a).

Figure 4b presents part of the published results of the tested model in which the combined effects of  $NF$  and external load on the pile cap were evaluated. The first curve from left to right represents the measured axial load along the pile at the end of the consolidation of the clay by own weight. The following six curves show the developed axial load due to an external load of about 4.750kN, equivalent to 50% of the ultimate strength of the pile, applied in six increments. Immediately afterwards, the sand fill was placed and left in the hopper until the end of the consolidation of the clay (83 months).

The results obtained in the field tests were confirmed: the developed  $NF$  is gradually replaced by the applied external load and becomes positive. Additionally, it can be observed that the elevation of the neutral level changed when the sand fill was placed, because the initial load conditions were modified.

#### NUMERICAL MODELING

In order to simulate conditions analogous to the field and laboratory studies presented above, a numerical axisymmetric FEM model was developed using PLAXIS 2D finite element code, to analyze the behavior of a single friction pile subjected to  $NF$  and transient loads (seism or wind). The study was conducted considering typical stratigraphic and piezometric conditions of Mexico City lacustrine zone.

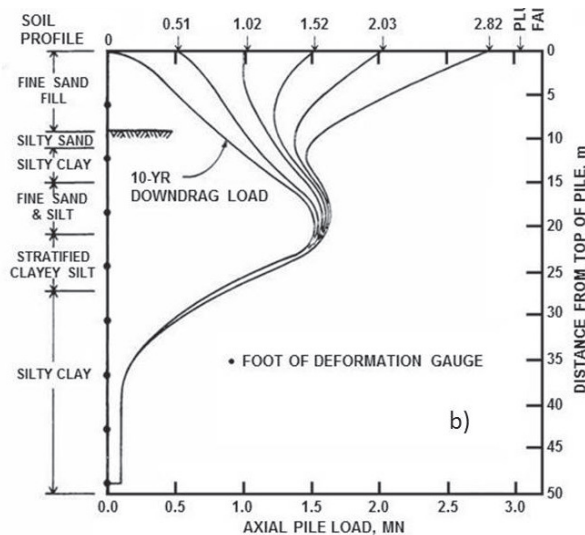


Figure 3. a) Installation of steel-pipe pile through sand embankment and b) load distribution in teste pile (Bozozuk, 1981)

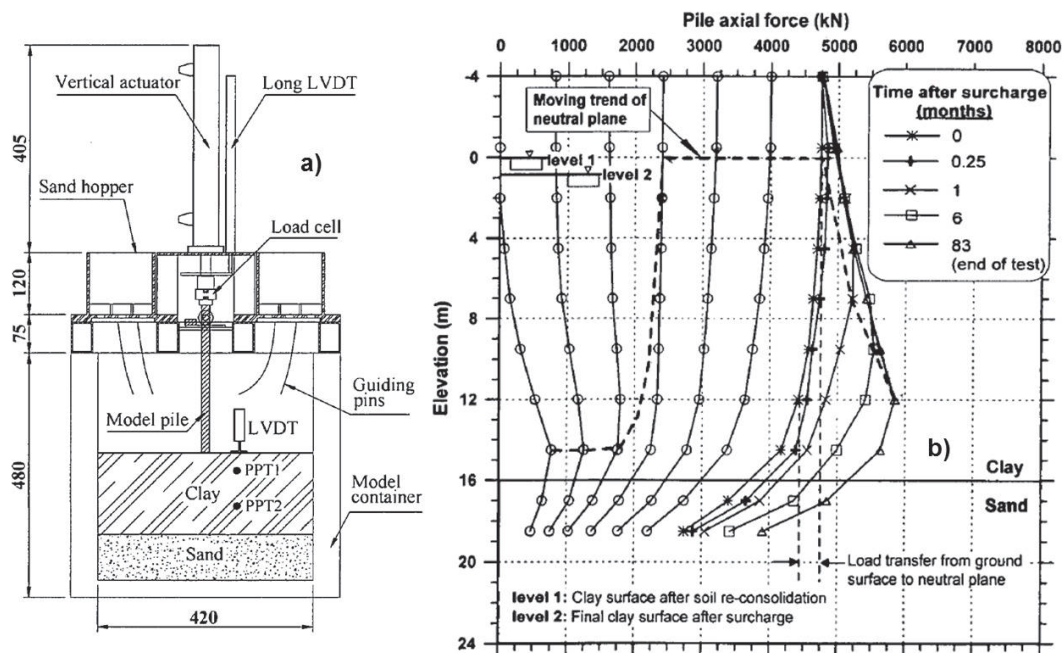


Figure 4. a) Centrifuge model setup (units in mm), b) effect of applied load on the shifting of neutral plane (Test A1) (Leung *et al.*, 2004)

A 50cm diameter and 25m long friction pile was modeled. The problem was discretized using a finite element mesh (Figure 5) with fifteen-noded triangular elements. The pile-shaft interaction was simulated and discretized using five pair-noded interface elements. The lateral boundary was placed at 10m from pile axis and fixed in the horizontal direction, and the bottom boundary was placed at 31m depth (beneath the hard layer) and fixed in both directions. Sensitivity studies showed that the mesh was dense enough to give accurate results.

For simulation of the clay behavior (Upper Clay Formation, UFC, Figure 5) the Soft Soil model (a isotropic hardening elasto-plastic model included in PLAXIS code) was used and the crust, dry crust and hard layer were simulated with the Mohr Coulomb model. The stages of the analysis were as follows:

- Stage 1: Consolidation due to pile self-weight.
- Stage 2: Consolidation due to a typical future pore pressure drawdown (Hypothesis #1, Figure 6).
- Stage 3: A total load of 500 kN was placed on the pile cap in undrained conditions (transient load) and in five increments.
- Stage 4: Consolidation due to the load of 500 kN.
- Stage 5: Consolidation due to a second pore pressure drawdown (extreme but possible condition, Hypothesis #2, Figure. 6).

As part of the obtained results, Figure 7 shows the variation with depth of the axial load transmitted by the pile ( $Q_{pile}$ ) during the different stages of the analysis. From these results, it can be observed that:

1. For Stage 1, the pile self-weight induces an axial load that increases with depth with a maximum value of 90kN.
2. In Stage 2, for the different consolidation degrees of the pore pressure drawdown ( $U = 20$  to 100%), the neutral level remains at a depth of about 20m (neutral level #1).
3. As the transient load is gradually placed (load applied in undrained conditions, Stage 3) the  $NF$  is progressively replaced by the applied load, even for values greater than 300 kN, and a second neutral level develops at a depth of 4m (neutral level #2).
4. As the second pore pressure drawdown develops (Step 5), the positive skin friction generated at the pile top due to the transient load tends to disappear and the neutral level stabilizes at a depth of 16.5m (neutral level #3).
5. There is a significant shift of neutral level depth from Step 2 (neutral level #1) to Stage 5 (neutral level #3), because the initial loading conditions for both steps are different.

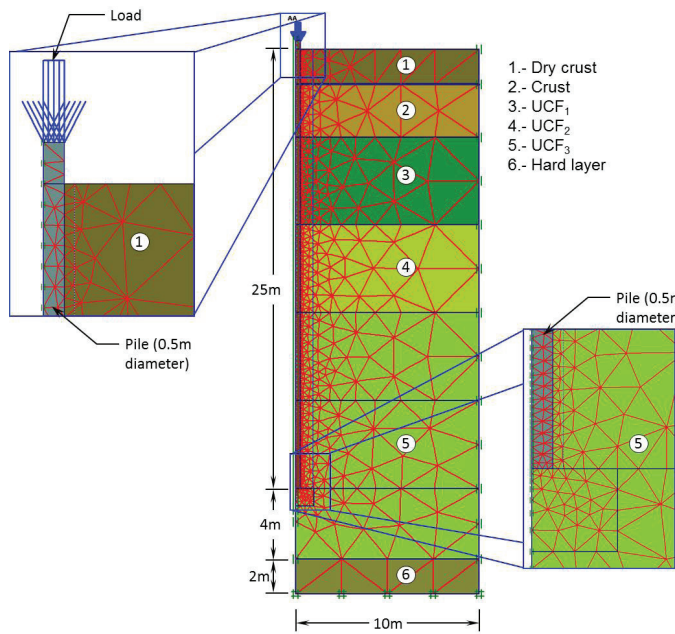


Figure 5. 2D FEM mesh (axisymmetric) for a single friction pile

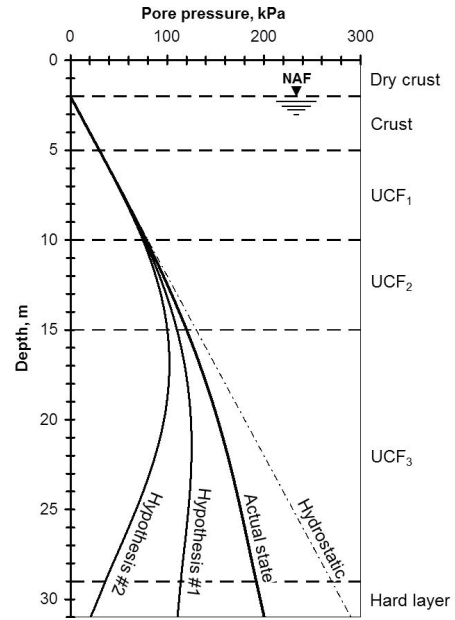


Figure 6. Pore pressure conditions

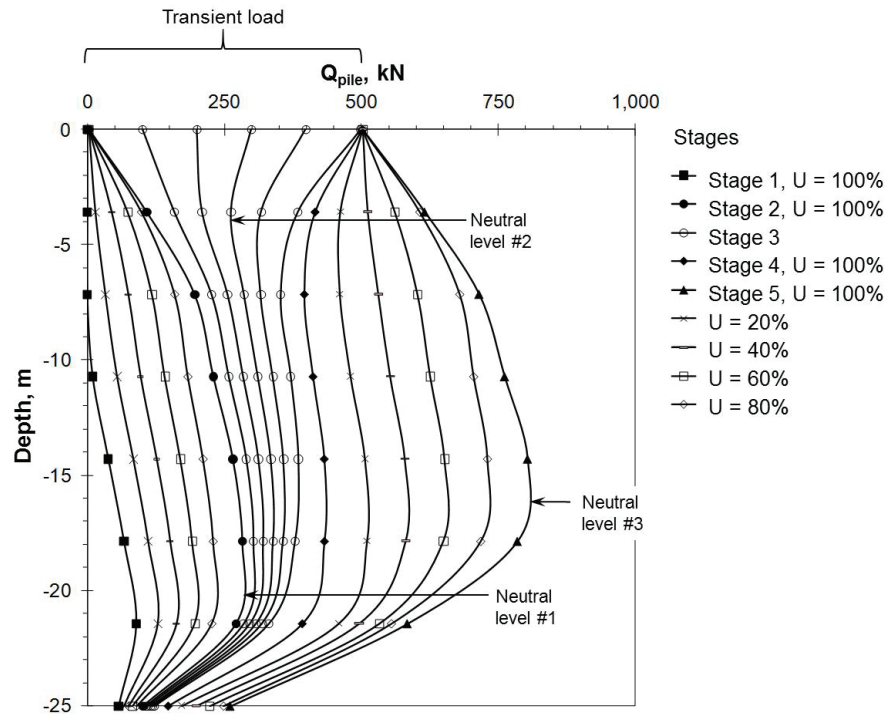


Figure 7. Axial loads obtained for the different stages of the analysis

Important similarities are found between the results obtained for friction piles by FEM with those reported by Bjerin, Bozozuk and Leung for point bearing piles in the field. One of the most important is that a huge part of the developed  $NF$  can be replaced by transient loads and  $NF$  can even become positive. Therefore, for the design of piles under these conditions, it would be too conservative to add the effects of the  $NF$  and transient loads.

The analysis also confirms that (Reséndiz and Auvinet, 1973) the depth of the neutral level tends to stabilize as the consolidation process due to pore pressure drawdown develops; it is also demonstrated that the depth of such level depends significantly on the initial pile load conditions.

Additionally, in Figure 8 the variation of the pile cap displacement with the applied transient load for a pile previously subjected to  $NF$  due to pore pressure drawdown and for another pile not subjected to this condition is shown. It can be seen that, in the first case, the pile develops an elastic behavior (linear load-displacement relationship) until the 500 kN, i.e., until the applied force exceeds the maximum  $NF$  developed in the pile. This can be compared to the behavior a pre-loaded element (Fellenius, 1998). Accordingly, the displacements obtained are considerably lower than those obtained when pore pressure drawdown is not considered.

#### INTERNAL PILE WITHIN A PILE GROUP

The numerical modeling of the behavior of an internal pile within an infinite pile group can be carried out con-

sidering that approximately axisymmetric conditions prevail around each pile, Figure 9 (Auvinet and Rodriguez, 2001). For finite pile groups, this assumption is not valid for border and corner elements but for large pile groups the boundary condition loses relevance.

The analysis was conducted considering the same stratigraphic and piezometric conditions of the single pile study presented before (Figures 5 and 6), but in this case the analysis stages were as follows:

- Stage 1: Consolidation due to a 75kPa external load (representing the weight of the structure), applied directly under the foundation slab.
- Stage 2: Consolidation due to typical future pore pressure drawdown (Figure 6).
- Stage 3: Consolidation due to a second pore pressure drawdown (extreme but possible condition, Figure 6).

Figure 10 presents the increments of the vertical effective stresses developed around the pile, for Stages 2 and 3 and for 1.5 and 4m pile spacing. It can be seen that, in all cases, the higher values of the effective stress increment develop near the pile tip.

Figure 11a shows the increment of the effective vertical stresses developed in the medium in presence ( $\sigma'_{WP}$ ) and absence ( $\sigma'_{WOP}$ ) of piles with 1.5m pile spacing and for Stage 3, while Figure 11b shows the increment of effective stress obtained from  $\sigma'_{WP} - \sigma'_{WOP}$ . The  $\sigma'_{WP} - \sigma'_{WOP} < 0$  condition represents the unloading of the reinforced soil with respect to the surrounding medium. It is possible to see that, above  $z_0$  the value of the

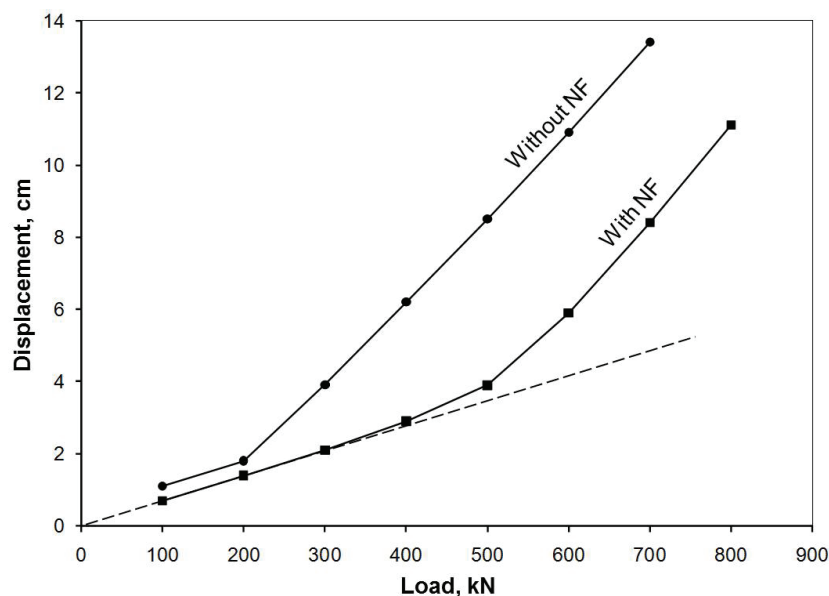


Figure 8. Load- displacements curves at the pile cap subjected and not subjected to  $NF$  due to pore pressure drawdown

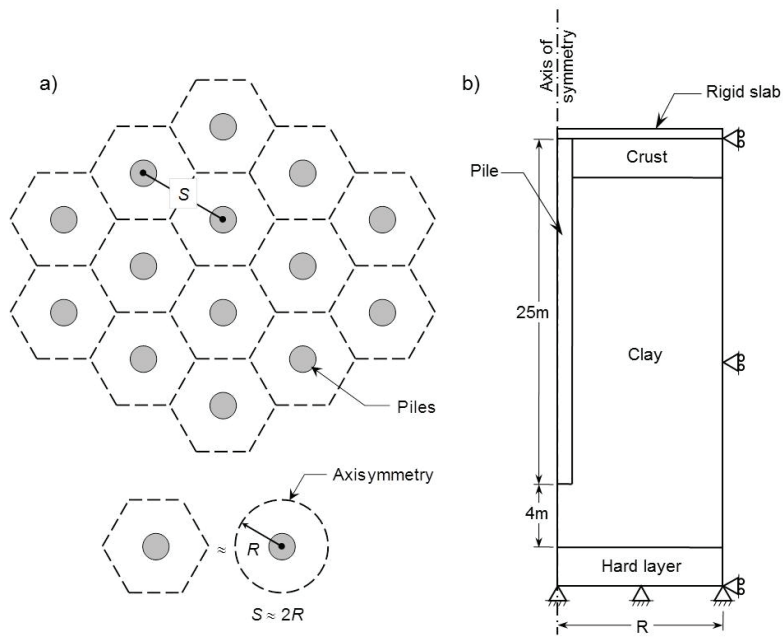


Figura 9. Model of a pile raft, a) infinitely large pile raft supported by friction piles, b) numerical model (not at scale) (modified from Auvinet and Rodriguez, 2001)

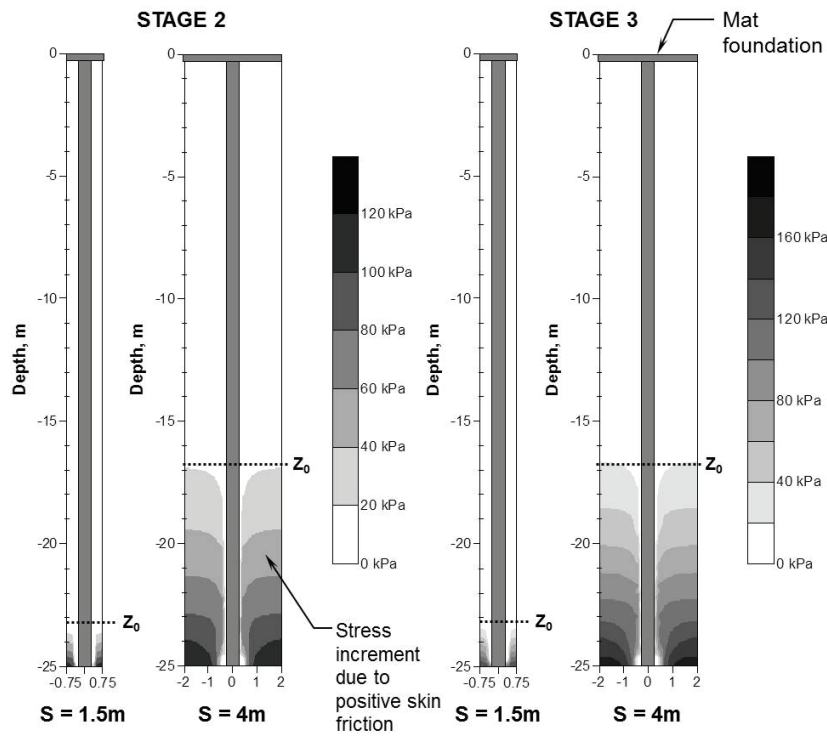


Figura 10. Predicted effective vertical stress increment in soil mass for Stages 2 and 3 and for spacing  $S = 1.5$  and  $4.0$ m



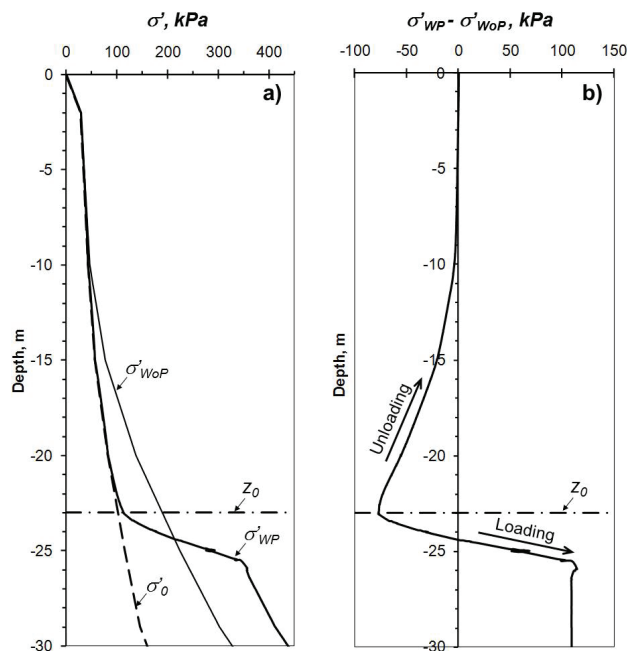


Figure 11. Predicted effective stresses in the soil mass for with and without pile conditions ( $\sigma'_{WP}$  and  $\sigma'_{WoP}$ , respectively), for 1.5m pile spacing and for Stage 3

developed effective stress without piles due to pore pressure drawdown ( $\sigma'_{WoP}$ ) decreases almost to zero when the piles are included in the analysis ( $\sigma'_{WP}$ ), i.e. above  $z_0$  the soil practically stops consolidating because  $\sigma'_{WP}$  has almost the same value as the initial effective stress ( $\sigma'_0$ ). This is due to the fact that, the effective stress increment is transmitted almost completely to the pile when the soil hangs from its shaft ( $\sigma'_{WP} - \sigma'_{WoP} = \sigma'_0 - \sigma'_{WoP}$ ). Below  $z_0$  the soil keeps consolidating due to the  $\sigma'_{WP} - \sigma'_0$  term in which are included the loads due to the pore pressure drawdown and the weight of the structure.

The pile-soil interaction analysis demonstrates that  $NF$  on central piles in consolidating soils cannot be larger than the apparent increase of the submerged weight of the mass of soil surrounding the pile above the neutral level, induced by seepage forces associated to the head gradient caused by pore pressure drawdown.

### BORDER AND CORNER PILES

To study the behavior of border and corner piles a three-dimensional model has been developed using PLAXIS 3D finite element code. A square mat foundation of 20m side, representing a typical 5 to 10 stories building surface located in the lacustrine zone of Mexico City, with a total of 100 friction piles uniformly distributed with a 2m spacing, was considered. Due to the

symmetry of the problem, it was only necessary to model a quarter of the building surface, so only 25 piles were considered, allowing an adequate mesh refinement, Figure 12. The problem was discretized using a finite element mesh with fifteen-noded wedge elements. The pile-shaft interaction was simulated and discretized using eight pair-noded interface elements. The lateral boundaries were placed at 40m from the symmetric axes and fixed in the horizontal direction, and the bottom boundary was placed at 31m depth (beneath the hard layer) and fixed in both directions. Sensitivity studies showed that the mesh was dense enough to give accurate results. The considered stratigraphy, pore pressure conditions, constitutive soil models and analysis stages were the same as those of the internal pile study presented above.

The variation with depth of the axial load developed in the center, border and corner piles are shown in Figure 13. For Stage 1, the load transmitted by the border piles is larger than for central ones (1.4 times  $\Sigma Q$ ), while in the corner values up to 1.74 times  $\Sigma-Q$  are attained. Due to the mat stiffness, the transmitted loads tend to concentrate on the borders, and further, at the corners of the mat foundation.

For Stages 2 and 3, the settlement developed by pore pressure drawdown has a greater influence on the behavior of the corner piles than on the border and center ones. This can be attributed to the fact that the area of influence of the corner piles is considerably larger than for the others leading to higher  $NF$  values.

For Stage 3 (Figure 13), small differences in the developed axial loads can be appreciated between border and corner piles, this is due to the fact that, as illustrated in Figure 14, limit shear conditions for  $NF$  have been developed for all of the perimeter piles of the group. These limit conditions were not reached for central piles even for stage 3 (extreme pore pressure drawdown).

It is possible to conclude that limit shear conditions for  $NF$  can develop for border and corner piles but very unlikely for central ones.

### DESIGN CRITERIA FOR FRICTION PILES

#### CONSIDERATIONS FOR DESIGN LOADS

This research established that, when pore pressure drawdown occurs, large values of  $NF$  can develop. However, under the effect of transient loads (seism or wind) a large part of the developed  $NF$  disappears and  $NF$  can even become positive. Therefore, for the design of piles under these conditions, it would be too conser-

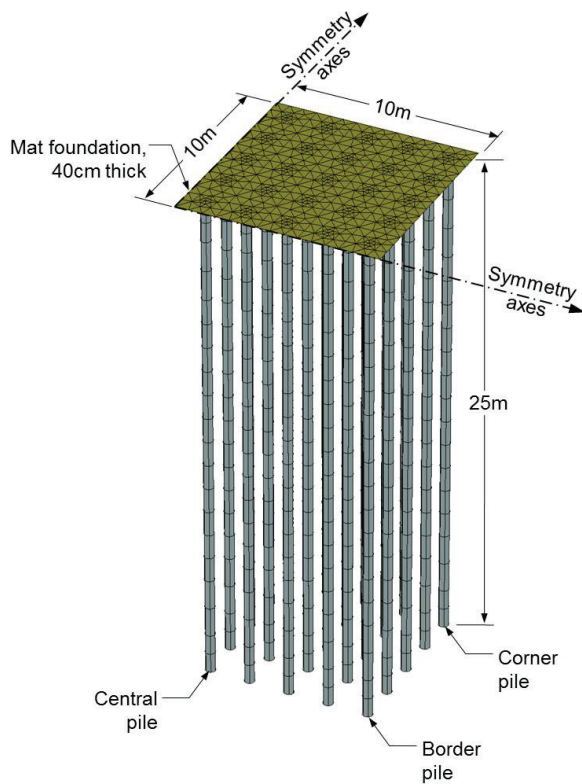


Figure 12. Details of the 3D FEM mesh, piles and mat foundation

vative to add the effects of *NF* and transient loads. Also, for long term conditions, the stiffness of the pile increases because the pile behaves like a preloaded element. Hence, the effect of the *NF* should be considered only for the structural safety review and for the estimation of the long term displacements (settlements or apparent emersion) of the foundation.

According to the above and adopting the definitions established in the Mexico City standards for foundations (GDF, 2004), for the structural safety review and for the estimation of the long term displacements of friction pile foundations, the loads to be considered should be: permanent loads (*AP*), live loads with medium intensity ( $AV_{med.}$ ) and the effect of the *NF* developed on piles shafts.

As the *NF* estimation is a complex problem, it is recommended to have recourse to the explicit numerical modeling of the problem. Conservative assumptions should be adopted regarding the future evolution of the pore pressure drawdown. Alternatively, this estimation can be done as proposed below.

#### NF FOR INTERNAL PILES

As shown before, for large pile groups, the *NF* above the neutral level ( $z_0$ ) cannot be greater than the apparent increment of the submerged weight of the soil

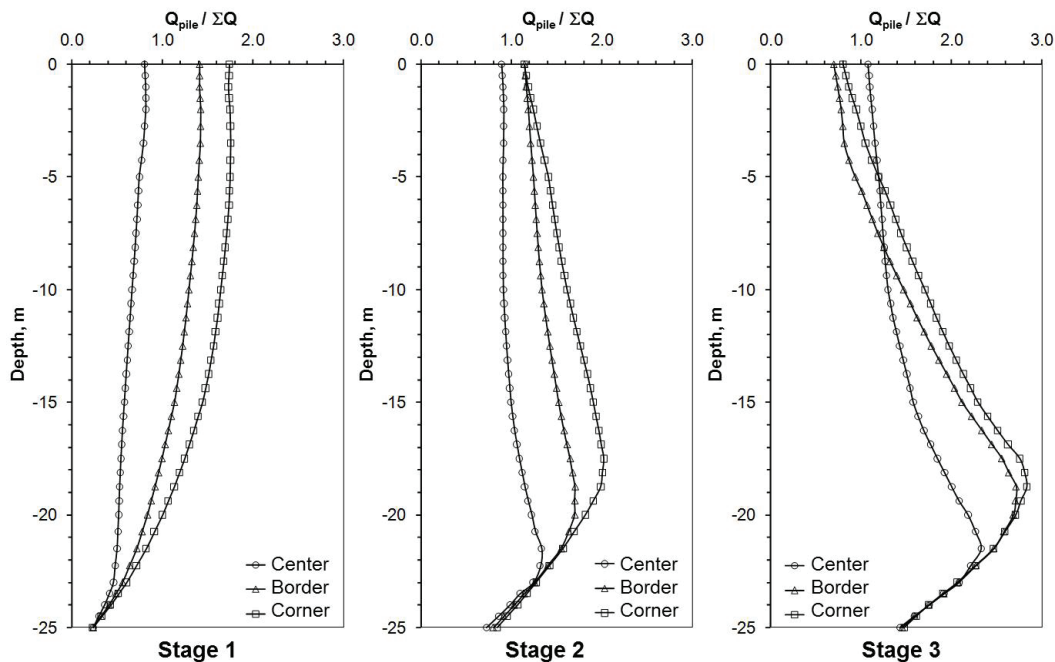


Figure 13. Predicted axial loads in center, border and corner piles, for the three stages of the analysis

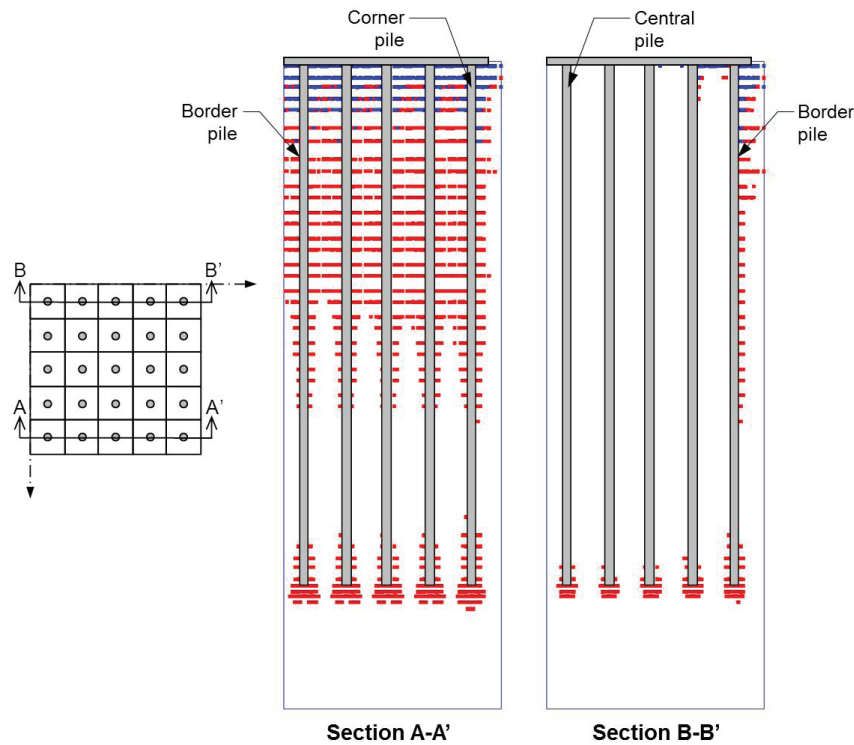


Figure 14. Plastic points in the soil mass (around the piles)

mass that surrounds the pile. Hence, the magnitude of the  $NF$  can be estimated by trial and error, varying the depth of  $z_0$ , until the following equation is satisfied (Figure 15)

$$\frac{\sum Q}{N_p} + NF = PF + C_p \quad (1)$$

where

$\sum Q$  = permanent loads plus the live loads with medium intensity

$N_p$  = number of piles

$\bar{P}F = C_f \Big|_{z_0}^{D_f+L_p}$  = positive skin friction, equal to the lateral bearing capacity developed from  $z_0$  to the depth of the pile tip ( $D_f + L_p$ ), considering a unitary strength factor

$C_p$  = point bearing capacity, considering a unitary strength factor

$L_p$  = pile length

$D_f$  = depth of the mat or box-type foundation

$NF$  = negative skin friction

equal to

$$NF = \min \left\{ \begin{array}{l} C_f \Big|_{D_f}^{z_0} \\ \Delta\sigma_{z_0} A_T \end{array} \right. \quad (2)$$

where

$C_f \Big|_{D_f}^{z_0}$  = lateral bearing capacity developed from  $D_f$  thru  $z_0$ , considering a unitary shear strength factor

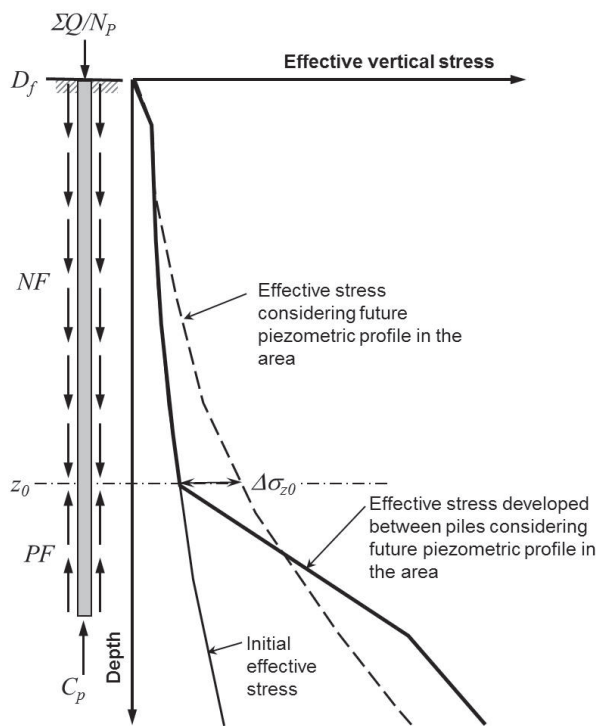
$\Delta\sigma_{z_0}$  = effective stress increment at  $z_0$  (without piles) developed by the potential future pore pressure drawdown

$A_T$  = tributary area between piles

$NF$  FOR BORDER AND CORNER PILES

Due to the fact that, as mentioned before, limit shear conditions for  $NF$  can develop on the shaft of border and corner piles,  $NF$  can be estimated following the criteria established for internal piles (ec. 1), but considering that

$$NF = C_f \Big|_{D_f}^{z_0} \quad (3)$$

Figure 15. Considerations for  $NF$  developed on a pile

### CONCLUSIONS

The results of numerical analyses developed by the authors, based on two dimensional and three dimensional *Finite Element Method* (FEM), of friction piles in typically Mexico City soil and piezometric conditions, are presented.

From the analysis of the behavior of an individual friction pile subjected to negative skin friction ( $NF$ ) and transient loads using 2D FEM, it is concluded that:

- There are important similarities between the results obtained for friction piles using FEM with those reported by Bjerin, Bozozuk and Leung for point bearing piles field. One of the most important results is that a large part of the developed  $NF$  can be replaced by transient loads and  $NF$  can even become positive. Therefore, for the design of piles under these conditions, it would be too conservative to add the effects of the  $NF$  and transient loads.
- The depth of the neutral level tends to stabilize as the consolidation process due to pore pressure drawdown develops, also it is demonstrated that the depth of such level depends significantly on the initial pile load conditions.

- The behavior of a friction pile, when the pore pressure drawdown is considered, can be compared to a pre-loaded element and therefore, the displacements obtained are considerably lower than those obtained when the pore pressure is not taken into account.

The 2D FEM study of an internal pile within a pile group subjected to the weight of the structure and to negative skin friction shows that  $NF$  on central piles in consolidating soils cannot be greater than the apparent increase of the submerged weight of the mass of soil surrounding the pile above the neutral level.

On the other hand, from the 3D FEM analyses it was concluded that limit shear conditions for  $NF$  can develop on border and corner piles but are very unlikely to occur for the central ones.

Finally, based on these conclusions, some design criteria were presented in order to be included in the updated (2017) Mexico City building code (complementary standards for foundations):

- For the structural safety review and for the estimation of the long term displacements of friction pile foundations, the loads to be considered should be: permanent loads, live loads with medium intensity and the effect of the  $NF$  developed on piles shafts.
- As the  $NF$  estimation is a complex problem, it is recommended to have recourse to the explicit numerical modeling of the problem. Conservative assumptions should be adopted regarding the future evolution of the pore pressure drawdown.
- Alternatively to the above, the magnitude of the  $NF$  for internal, border or corner piles can be estimated by the equations proposed in this paper.

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