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# Evaluation of Negative Skin Friction on Precast Piles

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**Abstract:** This study aims to better understand negative skin friction. The phenomenon is conceptualized, some situations in which it takes place are presented as well as some factors that contribute to the nature, magnitude and ways to minimize it. The main forecasting methods in the literature review are discussed. Estimation of negative skin friction is performed for a hypothetical situation on driven single piles or pile groups (when it is feasible through the methodology) embedded within a soil profile made up of soft to very soft organic clay layer, which is superposed on a compact sand layer. Underlying this sand layer there is a residual soil. It is considered a 3 meter deep embankment. The type of pile is precast concrete piles with a closed end. For the pile group it is considered six precast piles connected to a pile cap. The values of the estimated negative skin friction ranged from 68 to 497 kN. Significant differences were expected using several methods, since they are developed based on different considerations. It was noted the relevance of studies on instrumentation regarding this field, particularly in Brazil, where there is a lack of studies on the subject.

**Key words:** negative skin friction, driven piles, precast piles, soft soil

## 1. Introduction

Worry and concern in regard to city climate has been increasing during the most recent decades, especially since the twentieth century. This is due in part to the overall well-being of the planet's population being threatened by climatic changes and variabilities. A climate's dynamics directly impact people's daily lives, resulting in readjustments for human beings in relation to their environment, in accordance with this said variability. This influence is better perceived in urbanized areas, either through the amplitude of different temperatures that exist throughout the day, the humidity of the air, or the frequency and intensity of rainfall.

In this respect, points out that there are some changes in the local climate (microclimate) in urban areas, such as the increase of temperatures in central areas, forming heat islands, for example, and the increase of rainfall. These alterations can occur due to anthropic interference such as soil sealing, civil construction projects, and the removal of vegetation, as well as the channeling of streams [5].

Studies related to this subject are mainly done in large urban centers, but currently there are some works that call attention to small and medium sized cities [2]. Therefore, research in these places is also important and necessary, in an attempt to understand the dynamics of the climate in these cities — whether there are or if there have been changes, as well as what they are. Thus, climatic and/or meteorological analyses carried out in these urban centers can help prevent possible changes in local climate dynamics [6].

This is the city profile addressed in this study and the methodology chosen for the collection of data: to define where to position the equipment in fixed transects, following the Northwest-Southeast and Southwest-Northeast orientation. The positions were determined according to a preceding cartographic mapping of the urban area of the Guarapuava. The data collection took place from December 2015 to November 2016. The choice of this period made it

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possible to observe the four seasons of the year (summer, fall, winter and spring), for the monthly and seasonal analysis.

The main objective of this study was to analyze the temporal and seasonal variability of the temperature in the urban area of the city of Guarapuava, Paraná. In order to do so, possible relations between climatic variability and altitude were identified, as were the types of use and occupation/cover of the soil.

Due to the process of urbanization, complex and global transformations took place, as it is in the city that environmental problems are much more accelerated. The intensive use of urban territory by great concentrations of human activities such as industrialization, transportation, and urban services, all conspire to provoke the creation of great amounts of pollutants in the air, soil degradation, and water pollution [8].

Guarapuava does not classify as a city that conforms to the patterns headed by the great metropolises, which arise from industrialization. Alternatively, its development comes from agriculture and agribusiness, activities that are still dominant in the municipality, mainly because of its great extension of rural area.

However, the process of urbanization, independent of its origin, is quite significant in terms of changes to the urban landscape. It is considered an altering factor in the atmosphere, thus creating an environment with specific characteristics of temperature and relative humidity. In other words, changes authored by human activities in urban environments can create a local climate this is noticeably different from its surroundings [3].

Moved by concerns generated by urbanization, Carlos Augusto de Figueiredo Monteiro, one of the pioneers of geographic climatology in Brazil, directed his studies to the geography of cities. Monteiro (2003) says that studying the urban climate, for him, was a geographic concern of someone who was aware of the urban universe and widely open to all interdisciplinary works.

Because of Monteiro's studies, others began to research geographic climatology in Brazil as well. Points out that the studies of urban climate in Brazil intensified during the 1970s, which was when several small, medium, and large cities were surveyed [6]. Due to concerns of urban environments, especially with their degradation and dwindling quality of life, urban climatologists improved their studies and raised the field of Climatology to a direct interaction with urban planners, integrating urban climate and urban planning.

Studies on urban climate in medium cities to small cities, with similar characteristics to those of Guarapuava are still few, but it is worth mentioning that some authors already worry about this subject. Researched Climatology and urban space management in a small town, discussing how events of precipitation could cause floods, due to problems that result from urbanization [4].

On the other hand, the changes in temperatures that occur in urban environments, which cause some discomforts, such as the formation of heat islands, is the most concrete expression of the shift in the balance of energy in urban environments [1]. These are characterized as domes of warm air that cover the city.

Thus, these authors conclude that an urban heat island (UHI) is the manifestation of the increase of temperatures caused by a high density of buildings, concentration of construction materials of high emissive energy potential, reflectance and, still, human activity. When the soil surrounding the pile consolidates more than the pile, an overload known as negative skin friction takes place at the pile. Some situations in which negative skin friction is manifested are as follows: (i) Density of disturbed clay minerals due to pile driving. (ii) Density of clay due to disposal in embankments, for example. During the process of densification, soft clay undergoes settlement and negative skin friction develops along the embankment layers and soft clay. (iii) Clay densification when the ground water table is lowered in sand layer above the soft clay or when the pressure on sand layer below the soft clay is relieved. (iv) Piles driven in under consolidated soils, in the process of densification because of its own load are also subjected to negative skin friction. (v) Piles driven in collapsible soils which, when saturated, result in the process of densification [1].

Negative skin friction can develop gradually over time and it tends to increase until it reaches a maximum value [2].

There is a certain depth in which there is no relative displacement between the pile and the ground, defined as a neutral point position. It may be determined establishing settlement profiles of the soil as a result of the densification and the expected settlement of the pile. The point where both profiles intersect is assumed to be the neutral point.

Some factors may contribute to the nature and magnitude of negative skin friction: (i) Constituent material that forms the pile. (ii) Method of installing the pile (driven or bored). (iii) Type of pile (end bearing, friction, open and closed-ended). (iv) Pile slope. (v) Cross-section of piles. (vi) Group effect. (vii) Type of soil. (viii) If the soil is under consolidated, normally consolidated or over consolidated. (ix) Degree of soil sensitivity. (x) Water table (related to level variation). (xi) Soil compressibility. (xii) Compressible layer thickness in relation to overburden (embankment, sediment, etc). (xiii) Characteristics of tension curves versus soil deformation [3].

Brazilian Standard for Design and Execution of Foundations, the occurrence of negative skin friction should be included in the project when there is a chance that it will happen [4].

Considering the problems imposed for the staking, negative skin friction can be quite costly for the foundation process as it reduces the load on piles and consequently increases the quantity and dimensions.

There are several ways to minimize effects of negative skin friction on piles: (i) Preloading of the compressible layer prior to pile installation. It should be remarked that the preloading must be sustained for a certain period, that is, this measure can affect the timetable of the work. (ii) Reinforce the piles with large diameter pipe. (iii) Use of frustoconical piles — the soil separates from the pile as the pile suffers settlement so as not to happen negative skin friction. (iv) Use of small diameter piles. (v) Pile painting — in Brazil, bituminous layer has been applied to the surface of piles subjected to negative skin friction [1].

## 2. Material and Methods

*2.1 Method of Terzaghi and Peck (1948)*

Terzaghi and Peck proposed a method considering the group effect. They assert that after driving piles the embankment material situated in the upper portion of the pile group may no longer freely settle since its downward movement is affected by the lateral resistance between the embankment and the piles. They consider that the weight of the embankment on piles is transferred to the piles themselves. The load P' in each pile due to the weight of the embankment is [5]:

$$P' = \frac{\left(AH_1\gamma\right)}{n} \tag{1}$$

where:

$A$ = area of a horizontal section included within the boundary of the pile group;

$H_1$ = thickness of the embankment layer;

$\gamma$ = specific weight of the embankment material;

$n$ = number of piles in the group.

In the spaces between groups, the weight of the embankment produces progressive settlements. If there are end bearing piles in the group, they do not create the downward movement, thus the surrounding soil descends in relation to the soil itself and allows dragging to take place.

This drag force increases with the densification process and it depends on the settlement of clay surface.

The value of the minimum drag force depends on the value of settlement of the compressible layer. It reaches values almost null for small-scale settlements, it increases with the settlement, and may not be larger than the result of the thickness of the clay layer ($H_2$) by the perimeter ($U$) of the pile group and by the undrained shear strength ($S_u$). Thus:

$$P^{"} = \frac{(UH_2Su)}{n} \tag{2}$$

The total value of negative skin friction in each pile is:

$$Q_n = P^{'} + P^{"} \tag{3}$$

*2.2 Method of Moretto and Bolognesi (1959)*

Moretto and Bolognesi estimate negative skin friction based on the following expression:

$$Q_n = A_L Su \tag{4}$$

where:

$A_L$ = lateral area of the pile;

$Su$ = undrained shear strength in clay.

The authors do not take into account the existence of a neutral point. They consider an isolated single pile and the mobilization of undrained shear strength along the entire shaft [6].

*2.3 Method of Johannessen and Bjerrum (1965)*

Johannessen and Bjerrum proposed an expression to estimate negative skin friction from the steel piles driven instrumentation in a thick clay layer on which a ten meter high embankment was placed [7]:

$$Q_n = Ktg\phi_a^{'} \int_0^z \sigma_v^{'} dzU \tag{5}$$

where:
$K$ = coefficient of soil thrust;
$\phi_a^{'}$ = adhesion angle between pile and soil;
$\sigma_v^{'}$ = vertical effective stress;
$U$ = pile perimeter.

*2.4 Method of De Beer and Wallays (1968)*

De Beer and Wallays assume that part of the initial vertical effective stress of the soil is transferred to the pile. Thus, we have [8]:

$$p_{v,z} = \left(\frac{\gamma}{m_1}\right)\left\{1 - e^{[(-m_1)H]}\right\} + p_0 e^{[(-m_1)H]} \tag{6}$$

where:

$p_{v,z}$ = final vertical effective stress;

$\gamma$ = specific soil weight;

$m_1 = \frac{(K_0 tg\phi U)}{A_t}$;

$K_0$ = coefficient of thrust at rest;
$\phi$ = angle of internal friction in soil;
$U$ = pile perimeter;
$A_t$ = effective area for each pile;
$e$ = Naperian logarithms (base $e$);
$p_0$ = initial vertical effective stress;
$H$= thickness of the layer.

The negative skin friction is:

$$Q_n = A_t \left\{ \gamma H + \left[ \frac{p_0 - \gamma}{\left(\frac{U}{A_t}\right) K_0 tg\phi} \right] \left[ 1 - e^{\left(\frac{-U}{A_t} K_0 tg\phi H\right)} \right] \right\} \tag{7}$$

Considering soil with two layers made up of distinct soils, the authors propose:

• In layer 1:

$$AN_1 = AN_{0,1} + AN_{\gamma,1} \tag{8}$$

where:
$AN_{0,1}$ = negative skin friction corresponds to the overload $p_0$;
$AN_{\gamma,1}$ = negative skin friction corresponding to layer 1 weight.

The vertical tension at the base layer 1 after pile driving is:

$$p_V(H_1)=\frac{\gamma_{K,1}}{m_{1,1}}\left\{1-e^{\left[(-m_{1,1})H_1\right]}\right\}+p_0e^{\left[(-m_{1,1})H_1\right]} \quad (9)$$

Thus, in layer 1:

$$AN_1=A_1\left[p_0(H_1)-p_v(H_1)\right] \quad (10)$$

where:

$$p_0(H_1)=p_0+\gamma_{K,1}H_1$$

• In layer 2:

The overload due to the external load and the overlying layer is:

$$(p_0)_2=p_v(H_1) \quad (11)$$

The vertical tension at the base layer 2 after pile driving is:

$$p_v(H_2)=\frac{\gamma_{K,2}}{m_{1,2}}\left\{1-e^{\left[(-m_{1,1})H_1\right]}\right\}+(p_0)_2\,e^{\left[(-m_{1,1})H_1\right]} \quad (12)$$

In layer 2 the result is:

$$AN_2=A_2\left[p_0(H_2)-p_v(H_2)\right] \quad (13)$$

where:

$$p_0(H_2)=(p_0)_2+\gamma_{K,2}H_2$$

*2.5 Method of Johnson and Kavanagh (1968)*

Johnson and Kavanagh presented a method that admits non-uniform mobilization. As there are greater displacements in the upper portion of the pile, the mobilization will be greater in this section. When approaching the pile tip, the mobilization tends to zero, since the relative displacements are practically zero in this region.

They admit that the force caused by negative skin friction in the shaft is equal to the force distributed in the ground that would produce the same pressure.

Following this method the first step is to calculate the settlement that would occur in the soil if there was no pile. For this purpose, Terzaghi's One-Dimensional Density Theory is used. The settlement rate $y$ is estimated. Then it is calculated the lateral pile surface which produces the same settlement $y$ when applied to the ground. This force corresponds to $f_x$, at a depth z( = $x$) at the top of the clay layer. Considering a linear distribution along $H$ (thickness of the clay layer), we have:

$$f_x=f_0\left(1-\frac{x}{H}\right) \quad (14)$$

where:

$f_0$ = force relative to negative skin friction per unit length acting at the top of the clay layer.

Dividing $H$ into $n$ segments, we have:

$$\Delta x=\frac{H}{n} \quad (15)$$

where:

$\Delta x$ = thickness of each segment.

The force relative to negative skin friction in any segment can be written:

$$AN_x=f_x\Delta x \quad (16)$$

Thus:

$$AN_x=f_0\left[1-\frac{x}{(n\Delta x)}\right]\Delta x \quad (17)$$

The authors accept that in each segment $AN_x$ is distributed to the soil above each segment, forming the angle $\phi$.

Based on Terzaghi's One-Dimensional Consolidation Theory, for segment $i$, we may conclude that the settlement caused by the friction force acting on this segment in the overlying clay is:

$$y_i=\left[\frac{C_c}{1+e_0}\right]x\log\left[p_{0(i)}+\frac{\Delta_p\left(AN_{(i)}\right)}{p_0(i)}\right] \quad (18)$$

The initial vertical stress $P_{0(i)}$ at the middle of the layer above depth $x$, due to the weight of the clay is:

$$p_{0(i)}=\frac{\gamma x}{2} \quad (19)$$

where:

γ = weight of soil.

But:

$$x = i\Delta x - \frac{\Delta x}{2} = \left(\frac{\Delta x}{2}\right)(2i-1) \tag{20}$$

Thus:

$$p_{0(i)} = \frac{\left[\gamma\left(i\Delta x - \frac{\Delta x}{2}\right)\right]}{2} \tag{21}$$

At the same point, the increased stress caused by the friction force acting on the segment is:

$$\Delta_p\left(AN_{(i)}\right) = \frac{AN(x_i)}{y_i} \tag{22}$$

As the soil distribution $AN(x)$ occurs at an angle $\phi$, the result is:

$$y_i = \pi\left[\frac{(xtg\phi)}{2}\right]^2 \tag{23}$$

Thus:

$$\Delta_p\left(AN_{(i)}\right) = \frac{\left\{f_0\left[1-\frac{x}{(n\Delta x)}\right]\Delta x\right\}}{\left\{\pi\left[xtg\frac{\phi}{2}\right]^2\right\}} \tag{24}$$

Replacing the value $x$ in the expression (24):

$$\Delta_p\left(AN_{(i)}\right) = \frac{\left\{f_0\left[1-\frac{x}{(n\Delta x)}\right]\Delta x\right\}}{\left\{\pi\left[xtg\frac{\phi}{2}\right]^2\right\}} \tag{25}$$

The value of settlement in clay caused by each segment $i$ is as expressed:

$$y_i = \left[\frac{C_c}{1+e_0}\right]\left(i\Delta x - \frac{\Delta x}{2}\right)\log\left\{1+\frac{\frac{\left[8f_0(2n-2i+1)\right]}{\left[n\pi tg^2\phi(2i-1)^2\Delta x\right]}}{\frac{\left[y\left(i\Delta x-\frac{\Delta x}{2}\right)\right]}{2}}\right\} \tag{26}$$

where:

$C_c$ = compression index for clay;

$e_0$ = initial void index of the clay;

$\Delta x$ = thickness of each segment;

γ = weight of soil;

$n$ = number of segments in which the clay layer was divided;

$\phi$ = angle of distribution of negative skin friction force in the soil.

The following expression is achieved:

$$y_i = f(i, f_0) \tag{27}$$

where:

$y_i$ = clay settlement caused by each segment;

$i$ = segment number;

$f_0$ = force relative to negative friction per unit length acting at the top of the clay layer.

When values are assigned to $i$, we find the corresponding settlements values for many $f_0$ estimation.

Comparing the settlements obtained through the Eq. (27) to the initial settlement estimation, the $f_0$ value is obtained and this can lead to overloading the area under the shear force diagram in the soil due to negative skin friction [9].

### *2.6 Method of Bowles (1968)*

Bowles proposed some expressions for single piles or pile groups at all types of overload (clay or granular embankment).

Taking the case of isolated piles in clay soil, we have [10]:

$$AN = \Gamma H_f Su \tag{28}$$

where:

$\Gamma$ = pile perimeter;

$H_f$ = thickness of embankment;

$Su$ = undrained shear strength of the clay in the zone $H_f$.

For single piles in granular soil:

$$AN = \Gamma\left(H_f\right)^2 \gamma Kf \tag{29}$$

where:

$\Gamma$ = pile perimeter;

$H_f$ = thickness of embankment;

$\gamma$ = unit weight of the soil embankment;

$K$ = thrust coefficient;

$f$ = coefficient of friction.

Two possibilities must be analyzed for pile group:

(i) It is performed negative skin friction calculation and addition for each pile in order to obtain the total overload capacity in the group:

$$AN_{group} = nAN_{piles} \tag{30}$$

(ii) The pile group action is considered, in such a way that shearing occurs along the perimeter of the group ($\Gamma$), adding the soil weight contained in pile group. Thus, we have:

For layers of clay soils:

$$AN_{group} = SuH_f\Gamma + \gamma H_f A \tag{31}$$

For layers of granular soils:

$$AN_{group} = \frac{(H_f)^2\Gamma\gamma f}{2} + \gamma H_f A \tag{32}$$

where:

$\Gamma$ = pile perimeter;

$A$ = area of pile group.

*2.7 Method of Endo et al. (1969)*

Based on measurement of skin friction on four kinds of steel pipe piles carried out for more than two years in Tokyo, Endo *et al*. proposed as expressed below aiming to calculate the maximum value of negative skin friction (that occurs in the neutral point) [2]:

$$AN_{\max.} = \eta\Gamma\alpha\int_0^{\beta l}\sigma'_{vz}dz \tag{33}$$

where:

$\eta$ = coefficient depending on the pile tip;

$\Gamma$= pile perimeter;

$\alpha = Ktg\phi$ = 0.30 for embankment and 0.20 for clay;

$K$ = thrust coefficient;

$\beta = \frac{l_n}{l}$ = relative depth of the neutral point (the authors suggest $0.73 < \beta < 0.78$;

$l_n$ = depth of the neutral point in relation to pile tip;

$l$ = length of the pile in the compressible soil.

*2.8 Method of Poulos and Davis (1975)*

As proposed by Poulos and Davis, the maximum force acting on the pile, when shearing occurs at the soil-pile interface, is [11]:

$$AN = P_{NFS}N_R N_T + P_a \tag{34}$$

where:

$P_{NFS}$ = maximum overload caused by negative friction when mobilization of shear strength occurs; $N_R$ = correction factor used when mobilization of shear strength does not occur;

$N_T$ = correction factor used when the pile driving did not take place at the time of embankment placement, but rather after a certain time $t$;

$Pa$ = axial load acting on the pile at the compressible top layer.

$P_{NFS}$ results from the expression:

$$P_{NFS} = \pi d\int_0^H \tau_a dz \tag{35}$$

where:

$d$ = pile diameter;

$\tau_a$ = soil-pile adhesion;

$H$ = thickness of the compressible layer.

*2.9 Method of Combarieau (1985)*

Combarieau presents a method for isolated piles and pile groups [12].

For a single pile, it is accepted the ultimate unit skin friction $\tau_n$ expressed as:

$$\tau_n = ktg\delta q'(z) \tag{36}$$

where:

$q'(z)$ = effective vertical stress at soil-pile contact with depth;

$tg\delta$ = coefficient of soil-pile friction;

$K$ = thrust coefficient.

If $h_c$ is the length of the pile along which negative skin friction is acting, we have:

$$AN = 2\pi R \int_0^{h_c} \left(Ktg\delta\right) q^{'}\left(z\right) dz \quad (37)$$

where:

$$R = \frac{\Gamma}{\left(2\pi\right)}$$

Γ = pile perimeter.

Some other experiences have already shown that it is not possible to determine analytically the term *Ktg*δ. For its measurement on-site testing is recommended in major engineering works. In order to make approximate calculations, values of *Ktg*δ are provided in Table 1.

The calculation method of Combarieau does not consider the soil settlement value, assuming that it presents enough compressibility. On the other hand, it takes into account the impact the presence of the pile has on tensions acting adjacent to the pile shaft. Its principle is based on the fact that negative skin friction results from the transfer of forces from the soil to the pile. But first there must be a reduction of the vertical tension in the soil nearby existing pile: this reduction attains maximum value adjacent to the pile and it decreases at a certain distance. We have [12]:

σ′(z) = effective vertical stress in the soil at the initial situation, before installation of piles;

σ′(*z, r*) = effective vertical stress in the soil after becoming overloaded without considering the piles, equal to $q_0 + \sigma_0^{'}\left(z\right)$, when there is an overload $q_0$;

*q*′(*z, r*) = actual vertical effective stress which is *q*′(*z, r*) adjacent to the pile, determining $\tau_n = Ktg\delta q'(z, r)$.

For the equation *q*′(*z, r*) the expression for r ≥ R is proposed:

$$q^{'}\left(z,r\right) = q^{'}\left(z,R\right) + \left[\sigma^{'}\left(r,z\right) - q^{'}\left(z,R\right)\right]\left\{1 - e^{\left[-\lambda\frac{(r-R)}{R}\right]}\right\} \quad (38)$$

where:

$\lambda$ = coefficient showing the soil hanging up on the pile.

To determine the value of *q*′(*z, R*), a balance of forces is achieved through a vertical soil slice with thickness *dz* around the pile. The following equation is obtained:

$$\frac{dq^{'}\left(z,R\right)}{dz} + m\left(\lambda\right) q^{'}\left(z,R\right) = \frac{d\sigma^{'}\left(z,R\right)}{dz} \quad (39)$$

$$\text{With } m\left(\lambda\right) = \frac{\lambda^2}{\left(1+\lambda\right)} \frac{Ktg\delta}{R} \quad (40)$$

Taking an interval on which $\frac{d\sigma^{'}}{dz}$ is assumed as constant, the previous equation is then integrated:

$$q^{'}\left(z,R\right) = \frac{1}{m}\frac{d\sigma^{'}}{dz} + e^{-mz}\left[\sigma^{'}\left(0,R\right) - \frac{1}{m}\frac{d\sigma^{'}}{dz}\right] \quad (41)$$

When: λ = 0

$$q^{'}\left(z,R\right) = \sigma^{'}\left(0,R\right) + z\frac{d\sigma^{'}}{dz} = \sigma^{'}\left(z,R\right) \quad (42)$$

Critical height $h_c$ is determined assuming that negative skin friction only occurs if *q*′(*z, R*) is higher than the initial stress $\sigma_0^{'}\left(z\right)$, that is:

**Table 1 Values of *Ktg*δ.**

| Type of material | *Ktg*δ |
|---|---|
| Bitumen-coated clay piles | 0.02 |
| Bentonite ring layer | 0.05 |
| Driven piles in very soft to soft clay soils and organic soils | 0.20* |
| Bored piles in very soft to soft clay soils and organic soils | 0.15 |
| Bored piles with loss of coating in very soft to soft clay soils and organic soils | 0.10 |
| Driven piles in stiff to very stiff clay soils | 0.30** |
| Bored piles in stiff to very stiff clay soils | 0.20 |
| Bored piles with loss of coating in stiff to very stiff clay soils | 0.15 |
| Driven piles in sensitive clay soils with negative skin friction caused by the pile driving | 0.10 |
| Soft sand and gravel | 0.35*** |
| Medium compacted sand and gravel | 0.45*** |
| Compacted sand and gravel | 0.5 a 1.0 and more*** |

* Reduce to 0.15 in open-ended driven piles.

** Reduce to 0.20 in open-ended driven piles.

*** Higher values correspond to driven piles; lower values correspond to bored piles.

$$q'(h_c, R) = \sigma'_0(h_c) \quad (43)$$

The total negative skin friction overloading the pile can then be calculated using one of these expressions:

$$AN = 2\pi R\int_0^{h_c} Ktg\delta q'(z,R)\,dz \text{ , if } h_c \langle H \quad (44)$$

$$AN = 2\pi R\int_0^{H} Ktg\delta q'(z,R)dz \text{ , if } h_c \rangle H \quad (45)$$

For pile group, we found a greater effect of soil suspension around the pile and it increases with decreasing the spacing between piles.

Given an infinite pile group of cross-section type $A_e$ and equivalent radius $R = \frac{\Gamma}{(2\pi)}$, with the perimeter Γ, considering there is regular spacing between piles, the problem for inner pile $e_i$ can be solved as an annular ground area of outer radius:

$$d = \sqrt{\frac{ab}{\pi}} \quad (46)$$

$$\text{and } A_i = \pi d^2 - A_e \quad (47)$$

The calculation is performed considering a single pile, restricting the analysis to the interval ($R, d$) instead of ($R, \infty$).

We finally obtain a differential equation:

$$\frac{dq'(z,R)}{dz} + m(\lambda,d)q'(z,R) = \frac{d\sigma'(z)}{dz} \quad (48)$$

With:

$$m(\lambda,d) = \frac{\lambda^2}{1+\lambda-\left(1-\frac{\lambda d}{R}\right)e^{\left[(-\lambda)\frac{(d-R)}{R}\right]}}\frac{Ktg\delta}{R} \text{ , if } \lambda \neq 0 \quad (49)$$

Or

$$m(0,d) = \frac{2}{\left(\frac{d}{R}\right)^2 - 1}\frac{Ktg\delta}{R} \text{ , if } \lambda = 0 \quad (50)$$

Thus the case of isolated piles appears as a borderline case in the group when $d$ tends to infinity, with $m(\lambda, \infty) = m(\lambda)$.

To determine the critical height and to calculate negative skin friction, the procedure is the same for an isolated single pile.

## 3. Results and Discussion

Estimations of negative skin friction are presented for a hypothetical situation involving a single pile and pile group (when it is feasible through the methodology) of piles driven in a soil profile consisting of a nine-meter layer of soft to very soft dark gray organic clay ($\gamma_{NAT.}$ = 13 kN/m³, ϕ = 15°, Su = 10 kN/m²), overlying a three-meter layer of compact sand ($\gamma_{NAT.}$ = 20 kN/m³, ϕ = 36°). Underlying this sand layer there is a residual soil. It is considered a three-meter high embankment in the surface ($\gamma_{NAT.}$ = 18 kN/m³, ϕ = 30°) and the water level is located at a depth of 1 meter in the organic clay layer.

The pile is a precast reinforced concrete pile with closed end, 15.5 meters in length and 50 centimeters in diameter. For pile groups six precast piles (closed end, length of 15.5 meters and diameter of 50 centimeters) interconnected by a crowning block are considered.

Fig. 1 illustrates a scheme with the soil profile adopted for the study.

Table 2 shows the summary of results obtained. The estimated negative friction coefficients ranged from 68 (Endo et al., 1969) to 497 kN (Poulos and Davis, 1975), which means high variation. The average value considering all the methods studied was 232 kN. The standard deviation was 149.

High values of negative skin friction obtained using the method of Poulos and Davis (1975) may be related to the consideration proposed by this method, that is: the pile deformations necessary to mobilize negative skin friction have occurred along the entire length of the pile.

In any case, significant differences were expected in the values of negative skin friction through the different methods, since they are developed based on different aspects such as the existence of a neutral point or not, specific application for singles piles and pile

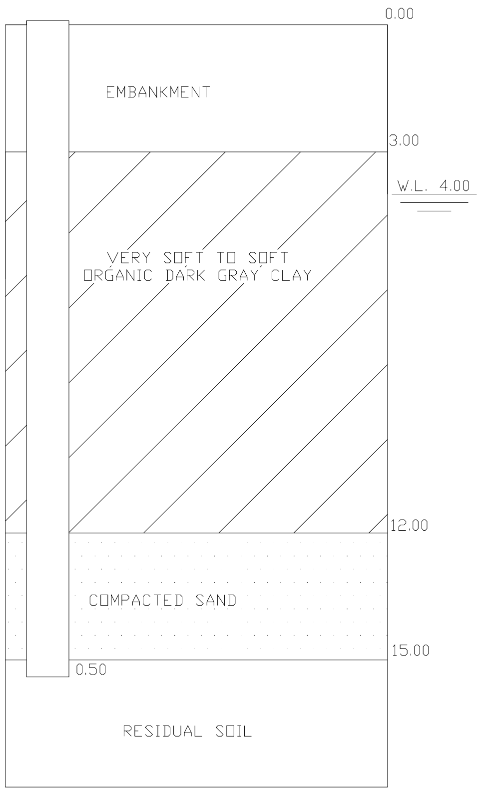


**Fig.1 Scheme of the soil profile (adapted from [13]).**

**Table 2 Summary of the values obtained for negative skin friction (adapted from [13]).**

| Method | Negative skin friction (kN) |
|---|---|
| Terzaghi and Peck | 228 |
| Moretto and Bolognesi | 171 |
| Johannessen and Bjerrum | 460 |
| Beer and Wallays | 172 |
| Johnson and Kavanagh | 92 |
| Bowles | 189 |
| Endo *et al.* | 68 |
| Poulos and Davis | 497 |
| Combarieau | 209 |
| Mean | 232 |
| Standard Deviation | 149 |

groups etc. From these results, the importance of studies involving the instrumentation in the field of negative skin friction is verified, particularly in Brazil, where there is a shortage of studies on the subject.

## 4. Conclusion

The following conclusions are drawn:

(1) The estimated negative friction coefficients ranged from 68 to 497 kN, which means high variation. Significant differences were expected in these coefficients, since different methods studied are developed based on many aspects such as the existence or not of neutral points, specific application for single piles and pile groups etc.

(2) The average value among several methods focused was 232 kN and the standard deviation was 149.

(3) High values of negative skin friction obtained from the method of Poulos and Davis (1975) may be related to the consideration proposed by this method, that is: the pile deformations necessary to mobilize negative skin friction have occurred along the entire length of the pile.

(4) The importance of studies involving the instrumentation in the field of negative skin friction, particularly in Brazil, where this issue is not thoroughly studied.

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