Analysis of Piles for Negative Skin Friction

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Abstract

Most of the high-rise structures have huge load transfers to the foundation which makes the adoption of raft foundation impossible especially when the bearing capacity of the supporting ground is not sufficient to sustain such amount of super structural load. As such, the piled foundation to come to action and would be the only choice as founding for the structure.

Negative skin friction is a well-known engineering phenomena that occurs in piles driven into soil been filled at earlier stage or would be filled in the future upon the completion of piling process.

In this study, we shall concentrate in studying the negative skin friction and calculate its value after Joseph E. Bowles. His approach shall be demonstrated through transferring the formulas he has suggested into a simple, friendly-user program and compare his output with certain g case studied.

This study would also shade the light on the effect of negative skin friction on the interrupting the piles capacity both geotechnically and structurally.

CHAPTER ONE

INTRODUCTION & THEORETICAL BACKGROUND

Introduction

1.1 Background

A relative movement between a pile and a soil produces shear stress along the interface of the pile and the soil. Such movement can be induced by a push-load on the pile pressing it down into the soil, or by a pull-load moving it upward.

A relative movement can also be induced when the soil settles in relation to the pile, or, in swelling soils, when the soil moves upward in relation to the pile. By definition, if the movement of the pile is downward, i.e., the shear stress induced in the pile is upward, the direction of the shear is positive.

*If the movement of the pile is upward, the shear stress direction is negative***;** *accordingly, the induced shear stress is called positive or negative.*

In older terminology, the induced shear along a pile was called 'skin friction'. In modern terminology, the term 'shaft resistance' is used and a distinction is made between on the one hand, positive and negative shaft resistance by which is meant shear stress induced by load on the pile in the form of push-load and pull-load, respectively, and, on the other hand, negative and positive skin friction, which is shear stress induced by settling or swelling soil, respectively.

Negative skin friction produces (accumulates to) a dragload which can be very large for long piles. Johannessen and Bjerrum

(1965), Bjerrum et al., (1969), and Bozozuk (1972) reported measurements of dragloads that exceed the allowable loads that ordinarily would have been applied to the piles. Bjerrum et al. (1969) also demonstrated the efficiency of coating the piles with bitumen to reduce the negative skin friction.

Fellenius and Broms (1969) and Fellenius (1969) presented measurement showing that a dragload can develop alone from the reconsolidation following the disturbance caused by the pile driving.

Walker and Darvall (1973) presented a comparison between bitumen coated and uncoated steel piles, and Clemente (1981) reported measurement of drag loads on coated and uncoated concrete piles. Fellenius (1975; 1979) discussed some practical aspects of bitumen coating of piles to reduce negative skin friction.

In all the papers referenced above, the emphasis is on the drag load. When the authors report observations of deformation and settlement, the main use of these is to calculate the loads in the pile.

When the consequence of the negative skin friction for design is included, these are discussed in terms of reduction of pile bearing capacity of allowable load.

In contrast, this paper suggests that the problem of negative skin friction is one of settlement and not of bearing capacity, i.e., the magnitude of the drag load is of no direct relevance to the geotechnical capacity of the pile, nor to the allowable load of the pile. Consequently, as recommended below, in the design for a down drag *condition, the calculation of the distribution of settlement is emphasized.*

1.2 Movement Necessary for Negative Skin Friction to Develop

The magnitude of the movement necessary for negative skin friction to develop has been reported in a few papers.

Walker and Darvall (1973) reported that a 35 mm settlement of the ground surface due to a 3 metre high surcharge placed around single piles was sufficient to develop negative skin friction down to a depth of 18 metre.

Settlement distribution with depth was not measured. Bjerin (1977) found that negative skin friction was fully mobilized to a depth of about 25 metre after a relative displacement of about 5 mm as measured at a short distance away from the pile (about 0.12 metre). At a distance of 5 metre, the relative displacement was about 8 mm. Bozozuk (1981) found that a reversal of direction of shear forces down to a depth of 20 metre occurred when loading a pile and generating a relative movement of about 5 mm at the pile head.

While Bjerrum et al. (1969) reported negative skin friction developing along piles at a site where the settlement under a recent fill amounted to 2 metre, they also reported that about the same magnitude of negative skin fraction developed on the same type of piles driven under an adjacent, 70 year old fill of the same height in *the same type of soil, which did not experience any new settlement after the pile installation.*

The reported observations indicate that no "slip" between the pile and the soil takes place and that extremely small movement is all that is needed to generate shear stress or to reverse the direction of shear along the pile-soil interface.

The pile material is immensely more rigid than the soil and, with time, there will always be small settlement in a soil generating a small relative displacement between a pile and the soil that is large enough to develop shear forces along the pile. The inescapable conclusion is that all piles experience drag loads!

*A consequence of the small displacement required to reverse the direction of shear forces in a pile is that live loads and dead load do not combine (Fellenius, 1972**;** Bozozuk, 1981).*

1.3 Magnitude Necessary for Negative Skin Friction to Develop

Johannessen and Bjerrum (1965) showed that negative skin friction is proportional to the effective overburden stress in the soil surrounding the pile. This was later confirmed by Bjerrum et al. 1969) and Bozozuk 1972).

*The constant of proportionality is called beta-coefficient, β, and it is a function of the earth pressure coefficient in the soil **K**s, times the soil friction, **tan** ϕ', times the ratio of the wall friction, **M** =*

tan *δ'/**tan** ϕ'. (Bozozuk, 1972). Thus, the negative skin friction, **q**n, follows the following relation.*

$$qn = \beta\, \sigma'v = M\, Ks \tan \phi'$$

*Bjerrum et al. (1969) found that the beta-coefficient in soft silty clay ranged between 0.2 and 0.3. Bjerin (1977) reported that in a clay of medium to firm consistency the beta-coefficient ranged between 0.20 and 0.25. Zeevaert (1959**;** 1972) presented a method of calculating the negative skin friction based on the reduction of the effective overburden stress caused by the soil "hanging" on the pile. DeBeer (1966) developed design charts based on Zeevaert's method.*

Axial pile-soil interaction is not simple. It is necessary to discuss it in some detail, as follows. The above defined separation of the terms for shear stress along a pile on negative, or positive, skin friction as different from negative, or positive, shaft resistance is justified by the different behavior of a pile under different types of axial loading. In fact, as illustrated in Fig. 1, a pile can be loaded axially in six different ways.

*Fig. 1 shows the upper portion of six piles marked **A** through **F**. The pile toes are at the elevation of a plane Z-Z. Pile **A**, or Mode **A**, indicates the behavior mode of a pile subjected to a push load applied at the pile head. The result is a downward deformation of the soil layers, as shown to the left of the pile, and an upward directed shaft resistance.*

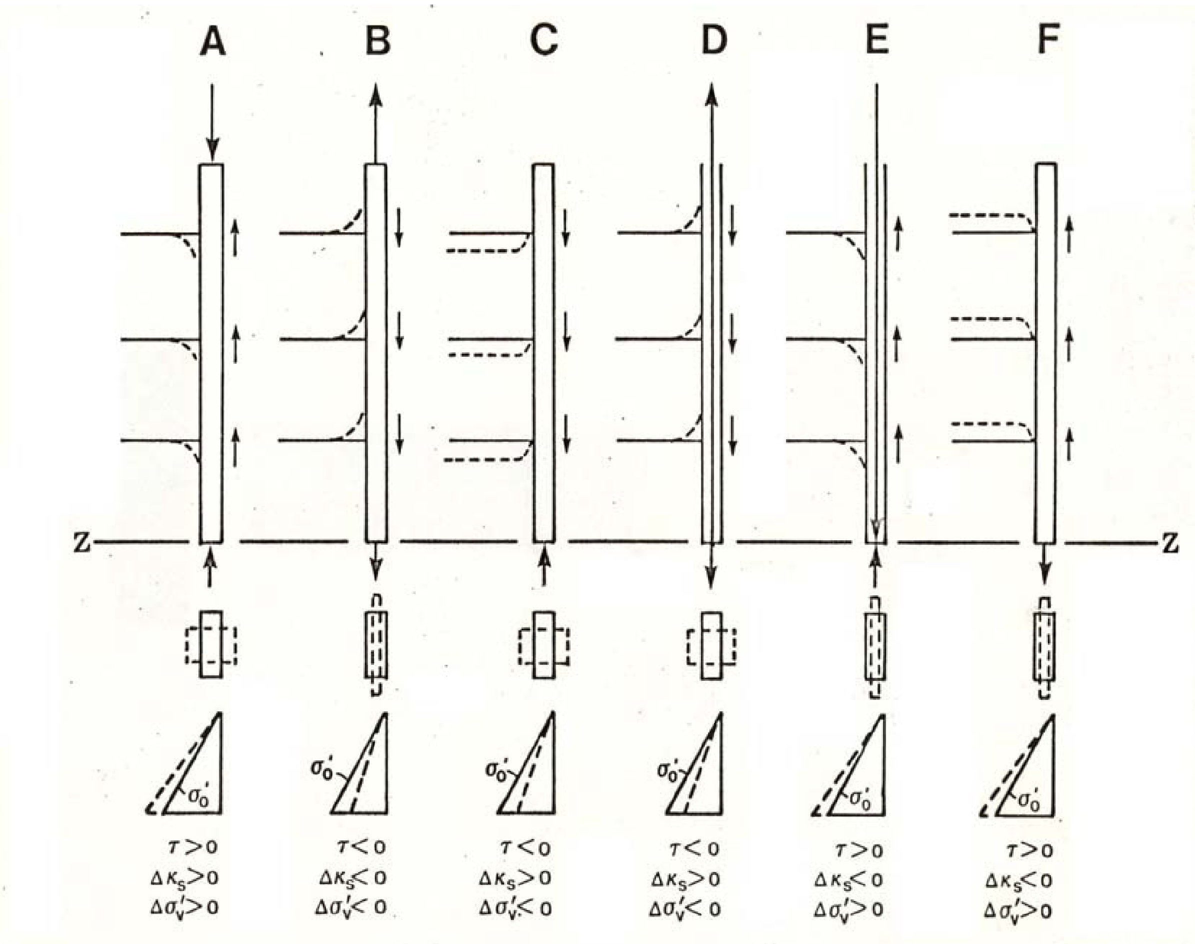


Fig.(1): Behavior modes of a pile subjected to different loading conditions

*As also indicated, the pile is in compression, which results, theoretically, in a Poisson's ratio effect (the pile diameter Increase) and the result is that the earth pressure coefficient **Ks**, increases.*

Furthermore, the positive shaft resistance transfers load from the pile to the soil, and because of this, the effective overburden stress increases in the soil. Both the Poisson's ratio effect and the increase of effective overburden stress will have the result of increasing the shaft resistance.

*Mode **B** indicates a pile subjected to a pull load applied at the pile head. The pull load results in an upward deformation of the soil layers and a negative shaft resistance. This mode is characterized by a decrease of lateral pressure and a decrease of effective stress.*

*Mode **B**, the uplift testing mode, is often thought representative for a drag load mode. However, while the resistance appears in the negative direction and the effective overburden stress is reducing, in contrast to the pile in the drag load mode, Mode **C**, Pile **B** is in tension not compression. Therefore, an attempt to predict the drag load from static uplift testing might underestimate the drag load.*

*Were one to run an uplift test to study the magnitude of drag load, the arrangement should be as shown in Mode **D**, where the pull load is applied at the pile toe and, therefore, all three aspects are similar to the drag load mode.*

*Mode **E** indicates a push test with the load applied to the pile toe. The purpose of such a test would be to simulate the behavior mode of Pile **F**, which is affected by swelling soil above Plane Z-Z.*

To complete the modes of axial pile-soil interaction, a seventh mode that of a pile subjected to a torque could have been shown. However, this mode is of no practical significance.

1.4 Magnitude of Drag Load

Observations show that for piles bearing on very competent material, the negative skin friction along the pile surface can be the cause of very large dragloads. Bjerrum et al. (1969) measured drag loads of 4,000 KN on 0.5 metre diameter steel test piles installed to bedrock through 55 metre of soil settling under the influence of a recent surcharge.

It is obvious that if a pile is long enough and/or if the ratio of its unit circumferential area to its cross sectional area is large enough; the induced stress could exceed the material strength, i.e., the structural capacity of the pile. In the field tests reported by Bjerrum et al. (1969), the piles were driven to rock, and the induced drag load forced the pile to penetrate into the rock. Obviously, the toe force developed in the pile by the pile driving hammer must have been smaller than the drag load.

Immediately after a pile is installed in the soil, the soil reconsolidates from the disturbance caused by the installation of the pile, whether the pile was driven or otherwise. For example, Fellenius and Broms (1969) and Fellenius (1972) reported load measurements in 300 mm diameter concrete piles driven into a 40 metre thick clay deposit and into an underlying sandlayer.

Immediately after the driving, the load in the pile was small, about equal to the free standing weight of the pile prior to the driving. The reconsolidation of the clay after the driving took about 5 months. During this time, negative skin friction developed and the drag load *induced amounted to about 300 KN to 350 KN corresponding to a beta-coefficient of about 0.10 and to about one third of the maximum drag load measured later.*

The settlement of the ground surface associated with the reconsolidation was interpolated from measurements over a longer period of time and found to be about 1 mm. The distribution with depth of relative displacement between the pile and the clay was too small to be measured with the gages used in the test.

The pile test reported by Fellenius (1972) and Bjerin (1977) started in 1968 and measurements were taken until 1983, i.e., for 15 years. (The complete results are not yet published). The test is particularly interesting because it involves the effect of applying a static load to the pile head and not just observations of the development of the drag load in the pile.

Applying a static load to the pile head caused the drag load in the pile to be reduced by the magnitude of the load applied. As the load was kept on, however, becoming a permanent load (dead load) from having been a temporary load (live load), the negative skin friction built up again and the end effect was that the load applied to the pile head was added to the drag load in the pile. At the end of the test, the drag load was fully developed and the maximum load was 1,750 KN consisting of 800 KN dead load and 950 KN drag load.

1.5 The Distribution of Load in a Pile and the Neutral Plane

There must always be an equilibrium between the sum of the dead load applied to the pile head and the drag load, and the sum of the positive shaft resistance and the toe resistance.

The depth where the shear stress along the pile changes over from negative skin friction into positive shaft resistance is called the neutral plane. This plane is where there is no relative displacement between the pile and the soil. Provided the shear stress along the pile does not diminish with depth, the neutral plane lies below the mid-point of a pile.

*If the soil below the neutral plane is strong, the neutral plane lies near the pile toe. The extreme case is for a pile on rock, where the location of the neutral plane is at the bedrock elevation. For a pile with embedment length **D** floating in a homogeneous soil with linearly increasing shear resistance, the neutral point lies about the lower third point (assuming the negative skin friction is equal to the positive shaft resistance, that the toe resistance is zero, and that there is no load applied to the pile head). If the soil strength increases with depth, for instance, due to a transition from soft compressible soil to a dense competent soil, and if a toe resistance is present, the neutral plane moves deeper into the soil. If a dead load is applied to the pile head, the neutral plane moves up.*

*Fig. 2 illustrates the distribution of load in a pile subjected to a service load, **Q**d**,** and installed in a relatively homogeneous soil* *deposit, where the shear stress along the pile induced by a relative displacement is a function of the effective overburden stress. It is assumed that any excess pore pressure in the soil has dissipated and the pore pressure is hydrostatically distributed.*

For reasons of simplicity, the shear stress along the pile is assumed to be independent of the direction of the displacement, i.e., the negative skin friction, ***q****n, is equal to the unit positive shaft*

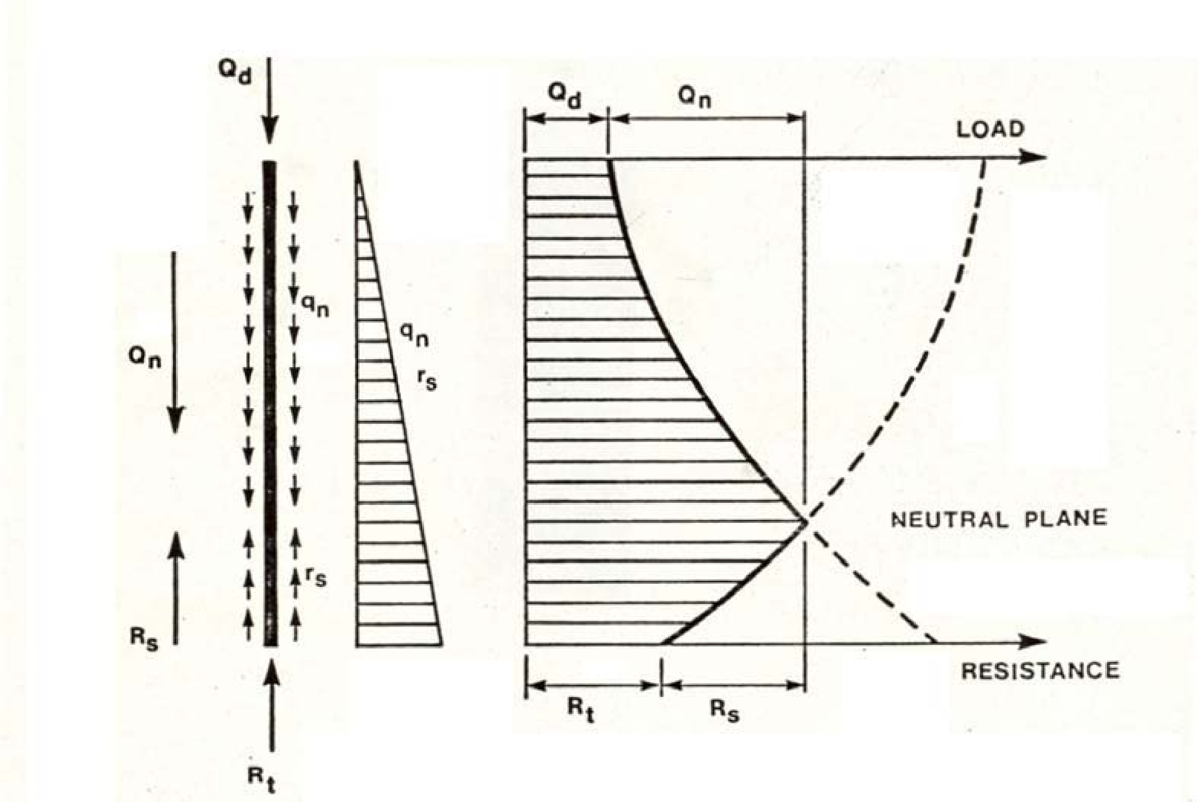


Fig.(2): Definition and construction of the neutral plane

resistance, ***r****s. Assume, also, that a toe resistance,* ***R****t, is available. The dragload,* ***Q****n, is the sum of the negative skin friction along the pile, and the total shaft resistance* ***R****s, is the sum of the unit shaft resistance. These conditions determine the location of the neutral plane as shown in the diagram.*

1.6 Settlement of a Pile

The neutral plane is, as mentioned, the location where there is no relative displacement between the pile and the soil. Consequently, whatever the settlement in the soil is as to magnitude and distribution, the settlement of the pile head is equal to the settlement of the neutral plane plus the compression of the pile caused by the applied dead load

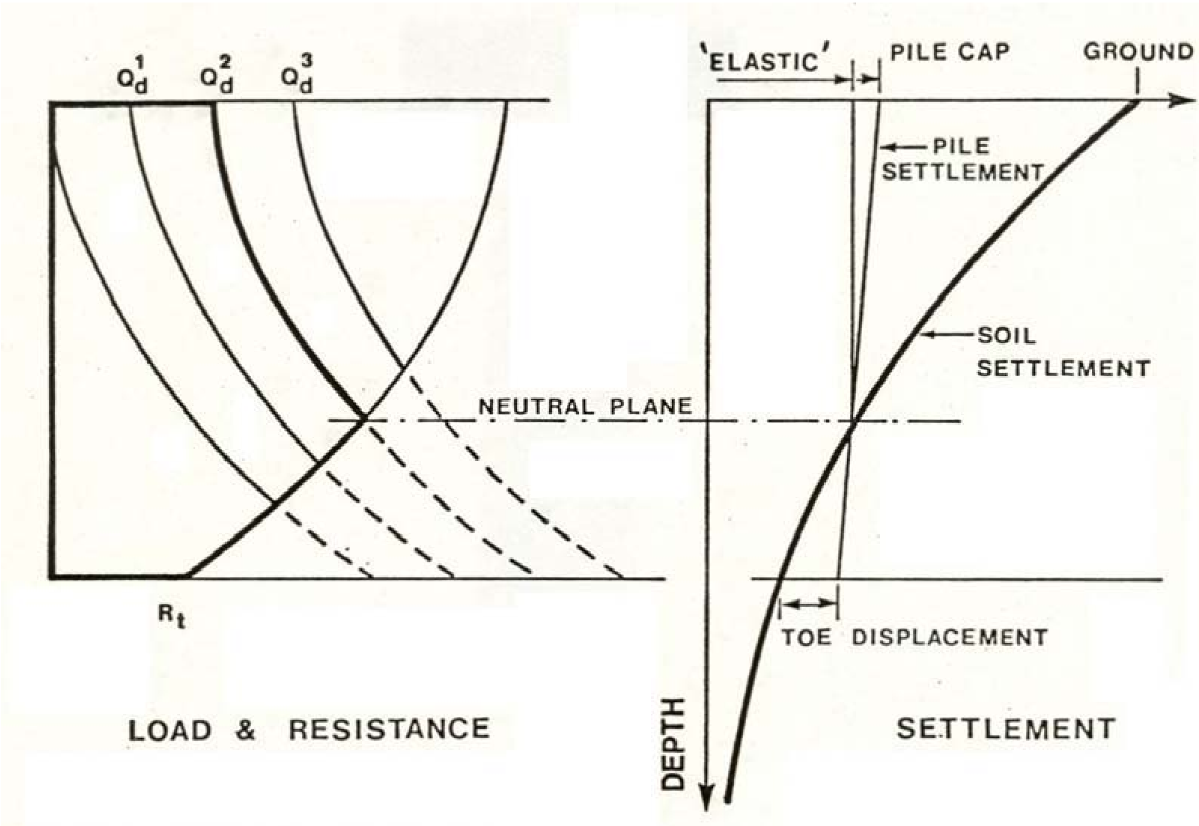


Fig. (3): Determination of the settlement of a pile plus the drag load.

The left hand side diagram in Fig. 3 illustrates how the location of the neutral plane for the pile in Fig. 2 changes with a variation of the load applied to the pile head. Notice also how the magnitude of the drag load changes as the service load, ***Q****d, increases.*

Assume that the distribution of settlement in the soil around the pile is known and follows the diagram on the right hand side in Fig. 3. As illustrated in the diagram for the case of the middle service load, by drawing a horizontal line from the neutral plane to intersection with the settlement curve, the settlement of the pile at the neutral plane can be determined. The settlement of the pile head is this settlement value plus the compression of the pile under the load.

The construction in the figure is made both for a small settlement that reduces quickly with depth and for a large settlement. If the settlement is small, it is possible that the toe movement is not large enough to mobilize the full toe resistance. In such a case, the neutral plane moves to a high location as determined by the particular equilibrium condition.

For a driven pile, the toe movement necessary to mobilize the toe resistance is about 1 % to 2 % of the pile-toe diameter. For bored piles, the movement is larger. However, in cases where the toe movement is too small for the full toe resistance to be mobilized, the settlement is normally not an issue.

Blanchet et al. (19870) reported measurements on a group of 27 shaft bearing piles supporting a bridge pier. The soil consisted of firm silty clay and the pile embedment depth was 15 metre.

The piles were of wood, had an average diameter of 280 mm, and were installed at a 5 diameter center-to-center spacing. During 260 days of observation, 40 mm settlement occurred in the soil outside the pile group. The pile cap settled 22 mm during the same time.

Settlement observations with depth are given in Fig. 4 and show that a neutral plane developed at a depth of about 12 metre. The observations confirm the qualitative behavior outlined in the foregoing.

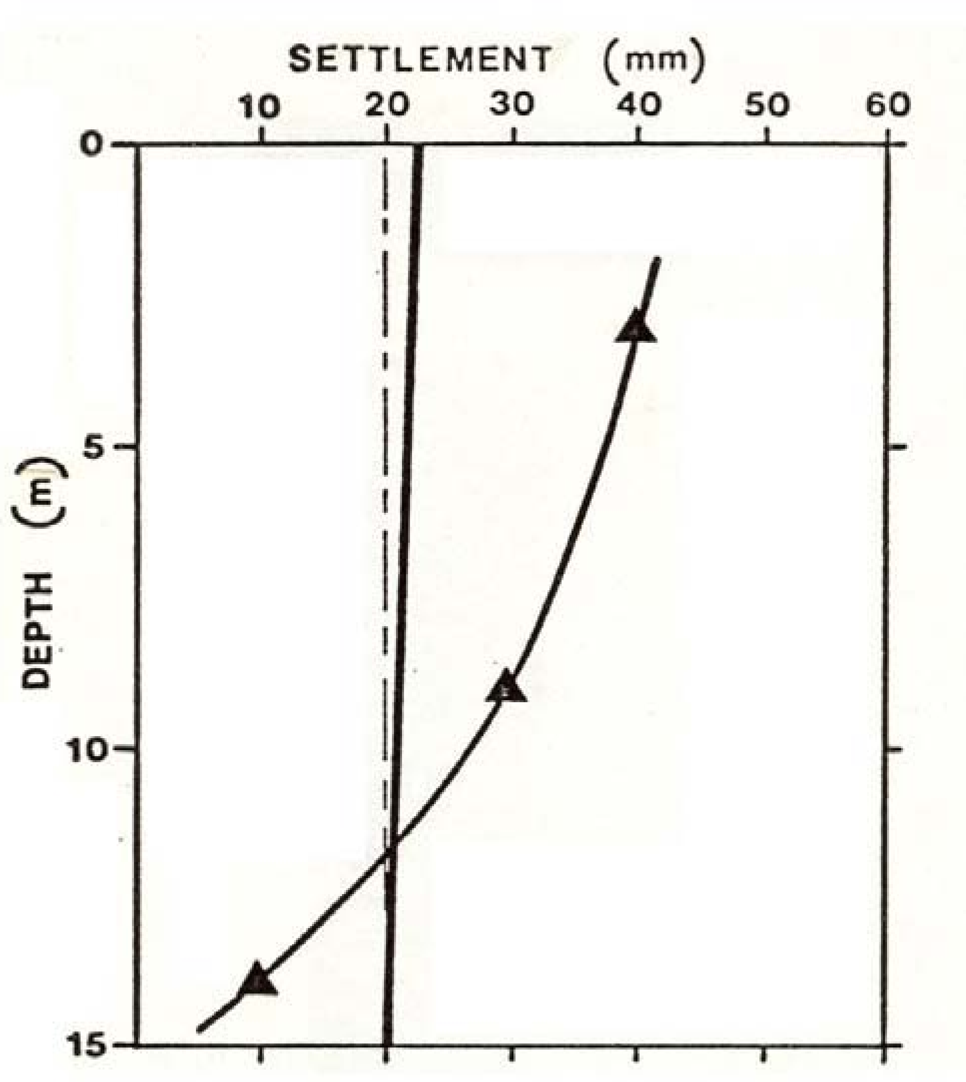


Fig. 4 Distribution with depth of settlement during reconsolidation of the clay for the instrumented pier at Riviere du Loup Bridge, Quebec. (after Blanchet et al, 1980)

1.7 Design of Piles Considering Negative Skin Friction

1.7.1 Fundamentals

The design principle outlined in the following is essentially the same for all piles, whether single or in a group, whether installed in a soil that settles significantly under the influence of a surcharge, groundwater lowering, or other cause, or installed in a soil that does not experience appreciable settlement, and whether they are essentially toe bearing, shaft bearing, or both toe and shaft bearing.

To understand the design principle, it is important to realize that the live load and the drag load do not combine and that two separate loading cases must be considered dead load plus drag load, but no live load and dead load and live load, but no drag load. Furthermore, a rigid, high capacity pile will experience a large drag load, but small settlement, whereas a less rigid smaller capacity pile will experience a smaller drag load, but larger settlement.

Moreover, the drag load is caused by settlement, or, rather, relative displacement, but the drag load does not generate settlement, and no pile will settle more than the ground surface nearest the pile, indeed no more than the soil settlement at the location of the neutral plane.

The design has to consider three aspects separately: The structural strength of the pile, the settlement, and the geotechnical capacity (the bearing capacity).

1.7.2 Neutral Plane

As a first step in the design, the neutral plane must be determined.

The neutral plane is located where the negative skin friction changes over to positive shaft resistance (the point of equilibrium). Its location is determined by the requirement that the sum of the applied dead load plus the drag load is in equilibrium with the sum of the positive shaft resistance and the toe resistance of the pile. It can be found at the intersection of two load distribution curves construed as follows.

First, as illustrated in Fig. 2, above, a load distribution curve (forcing load curve) is drawn from the pile head and down with the load value starting with the applied dead load and increasing with the load due to negative skin friction taken as acting along the entire length of pile. Second, a load distribution curve (resistance curve) is drawn from the pile toe up starting with the value of the ultimate toe resistance and increasing with the positive shaft resistance.

The determination of the load distribution in a pile is subject to large uncertainty. To correctly determine the distribution requires reliable information on the soil strength parameters. The theoretical analysis using the above mentioned method of beta-coefficient on the effective overburden stress is preferred over any total stress method. The analysis should be supplemented with information from static cone penetrometer tests.

For driven piles, the analysis should be combined with results from analysis of dynamic monitoring data aimed toward the calculation the distribution of resistance along the pile.

1.7.3 Structural Strength

The structural capacity is the structural strength of the pile material at the neutral plane for the combination of dead load plus drag load - live load is not be included. (At or below the pile cap, the structural strength of the embedded pile is determined as a short column subjected to dead load plus live load, but drag load is not included).

At the neutral plane, the pile is confined and it is suggested that the limiting value of maximum combined load be determined by applying a safety factor of 1.5 on the pile material strength (steel yield and/or concrete 28-day strength and long term crushing strength of wood).

It should be realized that if both the negative skin friction and the positive shaft resistance as well as the toe resistance values are determined assuming soil strength values "erring" on the strong side, the calculated maximum load in the pile will be on the conservative side (and the neutral plane located deep down in to the soil).

As illustrated in Fig. 3, above, a reduction of the dead load on the pile will result in a lowering of the location of the neutral plane, but have proportionally smaller effect on the magnitude of the maximum load in the pile.

1.7.4 Settlement

As demonstrated in Fig. 3, above, the settlement of the pile head is determined by first calculating the distribution of settlement and, then, drawing a horizontal line from the neutral plane to intersection with the settlement curve.

The settlement of the pile is equal to the settlement of the soil at the elevation of the neutral plane plus the elastic compression of the pile due to the dead load and the drag load in combination.

A condition for the analysis is that the movement at the pile toe must be equal to or exceed the movement required to mobilize the ultimate toe resistance of the pile. In most soils, this required movement is about 1 % to 2 % of the pile toe diameter of driven piles and about 5 % to 10 % of the toe diameter for bored piles. If the movement is smaller than this, the toe resistance will not be fully mobilized and the neutral plane will move to a higher location in the settlement diagram.

In a design case where the toe resistance value is difficult to estimate or where it is variable, for instance, in the case of toe-jetted piles, a conservative estimate of the settlement is obtained by disregarding the toe resistance when construing the neutral plane.

The settlement calculation is carried out according to conventional methods for the effective stress increase caused by the dead load on the pile(s), surcharge, groundwater lowering, and/or any other aspect influencing the stress in the soil. The settlement *calculations must include the compression of silt and sand layers in the soil profile, in particular, those located below the neutral plane.*

This makes it important to carry the investigation of the soil conditions at a site to a sufficiently large depth and to include a representative amount of sampling and laboratory testing of the soils located below the pile toe. As a minimum, an investigation should include static cone penetrometer tests and sampling of all layers encountered with undisturbed samples taken of all cohesive soils.

The settlement calculation of non-cohesive soils should not be based on the use of a constant 'elastic' modulus, but on the tangent modulus approach, which considers that the compression of soil is not linearly increasing with the increase of stress. The Canadian Foundation Engineering Manual (1985) details the use of the Janbu unified theory for both cohesive and non-cohesive soils and gives reference values of moduli to use.

It should be realized that if both the negative skin friction and the positive shaft resistance as well as the toe resistance are determined assuming soil strength values "erring" on the weak side, the calculated location of the neutral plane will be located higher up in the settlement diagram, i.e., the settlement of the pile will be calculated on the conservative side. As illustrated in Fig. 3, a reduction of the dead load on the pile will result in a lowering of the neutral plane, and, therefore, a reduction of the settlement of the pile.

1.7.5 Geotechnical Capacity

In opposition to what the author has recommended in the past, the drag load must not be included in the consideration of the geotechnical capacity. Consequently, it is incorrect to reduce the dead load by any portion of the drag load (unless required by insufficient structural strength of the pile at the location of the neutral plane).

Consideration of the geotechnical capacity in the design of a pile, or of a group of piles, amounts to making a check of the safety against plunging failure of the pile.

In such a case, the pile moves down along its entire length and the negative skin friction is eliminated. Therefore, the load applied on the pile in the design effort is the combination of the dead and live loads. The drag load must not be included.

When the pile capacity has been determined by static loading test or by the analysis of data from dynamic monitoring, a factor of safety of 2 or larger ensures that the neutral plane is located below the mid-point of the pile. When the capacity is calculated from soil strength values, the factor of safety should not be smaller than 3.

1.7.6 Special Considerations

All piles will be subjected to negative skin friction and experience drag load.

However, unless the structural strength of the pile is exceeded, piles where the soil settlement is small will not constitute a problem.

Where the settlement is large, the maximum drag load induced in a straight and vertical pile is not going to be significantly different to the drag load where the settlements are small.

However, large settlement will cause an inclined pile to bend. For this reason, it is advisable to avoid inclined piles in the foundation, or, at least, to limit the inclination of the piles to values which can follow the settlement without excessive bending being induced in the piles.

Piles which are bent, doglegged or damaged during the installation will have a reduced ability to support the service load in a down drag condition. Therefore, the design according to the above approach postulates that the pile installation is subjected to stringent quality control directed toward ensuring that the installation is sound and that bending, cracking, and local buckling does not occur.

1.7.7 Means for Reducing Negative Skin Friction

When the design calculations indicate that the pile settlement could be excessive, solutions such as increasing the pile length or decreasing the pile diameter, could improve the situation.

When the calculations indicate that the pile structural capacity is insufficient, solutions such as increasing the pile section, or increasing the strength of the pile material could improve the situation.

When such methods are not practical or economical, the negative skin friction can be reduced by the application of bituminous coating or other viscous coatings to the pile surfaces before the installation (Fellenius, 1975**;** 1979, and Clemente, 1981).

For cast-in-place piles, floating sleeves have been used successfully.

CHAPTER TWO

EXPERIMENTAL WORK

EXPERIMENTAL WORKS

The aim of this study as it has mentioned above is to estimate the negative skin friction in piles after Joseph E. Bowles.

Bowels have identified in brief the cases where the negative skin friction developed in piles and this has been outlined in detail in Appendix “A” attached.

What we have done in this study, is converting his theoretical formulas to a program that could calculate the negative skin friction for piles embedded in one layer of soil.

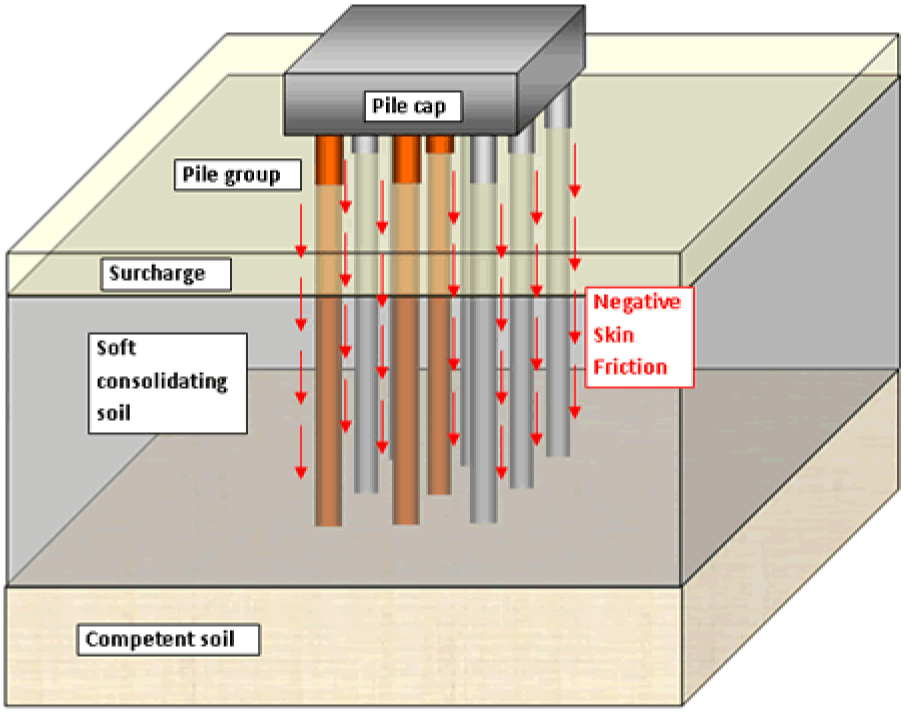
Definitely, this is considered as a noticeable restriction to the calculation process as the pile usually penetrate several layer of soil but nevertheless, his approach is widely followed and considered practical especially in our region (Middle East, Especially Iraq) as most of the geological stratum are almost the same at different areas.

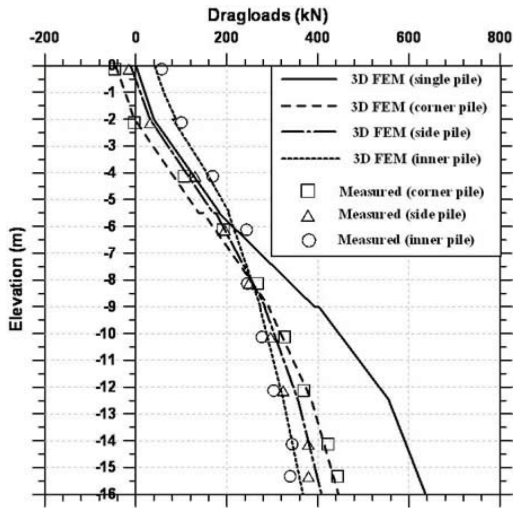
In his analysis he has presented his theory through two examples in his reference and we have in this study keyed in the data of his working examples into our program and the results were identical.

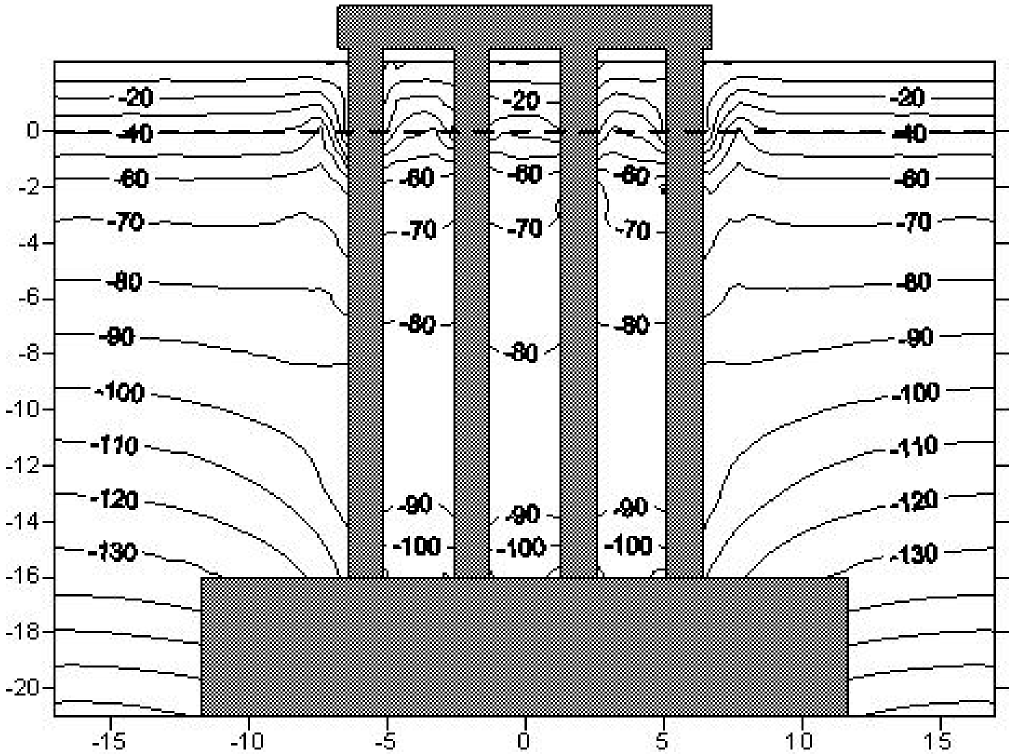
Upon ensuring the reliability of our program, certain case studies has been obtained from university and companies whom were interested in measuring experimentally the values of the negative skin frictions.

The results of their experimental tests were forwarded to us as a courtesy and we would like to take this opportunity to express our sincere appreciations to the University of Malaya (UM) of Malaysia and ESa Jurutera Perunding Sdn. Bhd. Company (Malaysia) for their support in accomplishing this study.

The following figures shade lights on the site experimental works.



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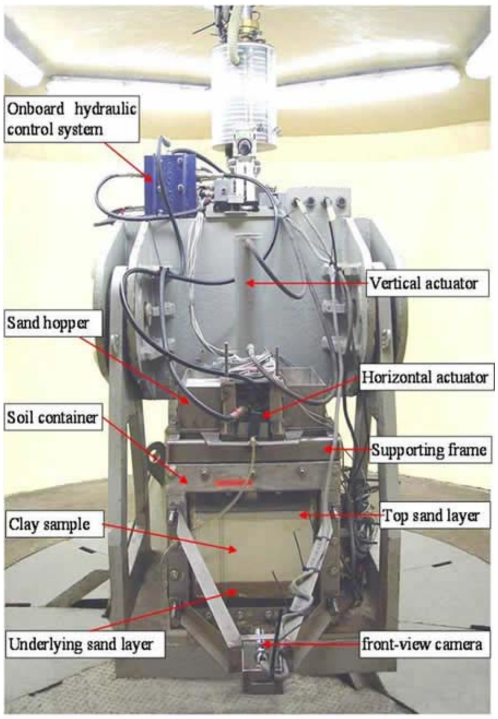

Onboard hydraulic control system
Vertical actuator
Sand hopper
Horizontal actuator
Soil container
Supporting frame
Top sand layer
Clay sample
Underlying sand layer
front-view camera

CHAPTER THREE

RESULTS AND THEIR DISCUSSION

RESULTS & THEIR DISCUSSION

In the following pages, over 15 case study has been reviewed and the negative skin friction were calculated following Joseph E. Bowels theories using the program prepared by the students arranged for this study.

Frankly speaking, it was our great surprise when we find out that the experimental figures for the down drag forces are very much alike to those obtained from programs.

This was clearly noticed for the piles known to be penetrating a single layer of soil.

The difference in results tends to be higher when the tested pile penetrated more than one different soil layer.

We shall recommend in the conclusions and suggestions chapter some measures to overcome this issue.

Negative Skin Friction in Piles

This work sheet is dealing with bored piles embedded in non-cohesive soil over-laid by cohesive fill

A. Negative skin friction for single pile

Pile diameter (mm) =	500
No. of bars in the pile =	10
Bars Diameter (mm) =	16
Concrete compressive strength (Mpa) =	35
Pile Structural Capacity (Kn) =	2060
Angle of friction Φ =	30
Coefficient related to applied pressure ά =	0.3851
Pile Perimeter (m) =	1.570795
Density of the filling soil (kn/m^3) =	17.29
Coefficient of lateral earth pressure (K) =	0.500
Height of Fill (m) =	3
Negative skin friction (Pnf), Kn =	**23.53**

B. Negative skin friction for overall pile cap (Group Action)

Pile cap length (m) =	3
Pile cap width (m) =	3
Pile cap area (m^2) =	9
Pile cap Perimeter (m) =	12
No. of piles in the pile cap =	9
Negative skin friction per pile for group action (Qn), Kn =	**71.84**

The governing value, Kn =	***71.84***
Net Pile Capacity for Structural Load, Kn =	***1988.02***

Case Study No.	**1**
Experimental Drag Load (Kn) =	**79**
Theoretical Drag Load (Kn) =	**72**
Percentage of Difference (%) =	**8.86%**

Negative Skin Friction in Piles

This work sheet is dealing with bored piles embedded in non-cohesive soil over-laid by cohesive fill

A. Negative skin friction for single pile

Pile diameter (mm) =	600
No. of bars in the pile =	12
Bars Diameter (mm) =	16
Concrete compressive strength (Mpa) =	40
Pile Structural Capacity (Kn) =	3238
Angle of friction Φ =	25
Coefficient related to applied pressure ά =	0.3110
Pile Perimeter (m) =	1.884954
Density of the filling soil (kn/m^3) =	18.5
Coefficient of lateral earth pressure (K) =	0.577
Height of Fill (m) =	2.75
Negative skin friction (Pnf), Kn =	**23.68**

B. Negative skin friction for overall pile cap (Group Action)

Pile cap length (m) =	3.6
Pile cap width (m) =	3.6
Pile cap area (m^2) =	12.96
Pile cap Perimeter (m) =	14.4
No. of piles in the pile cap =	6
Negative skin friction per pile for group action (Qn), Kn =	**140.04**

The governing value, Kn =	***140.04***
Net Pile Capacity for Structural Load, Kn =	***3097.56***

Case Study No.	**2**
Experimental Drag Load (Kn) =	**158**
Theoretical Drag Load (Kn) =	**140**
Percentage of Difference (%) =	**11.39%**

Negative Skin Friction in Piles

This work sheet is dealing with bored piles embedded in non-cohesive soil over-laid by cohesive fill

A. Negative skin friction for single pile

Pile diameter (mm) =	300
No. of bars in the pile =	10
Bars Diameter (mm) =	12
Concrete compressive strength (Mpa) =	35
Pile Structural Capacity (Kn) =	811
Angle of friction Φ =	18
Coefficient related to applied pressure ά =	0.2167
Pile Perimeter (m) =	0.942477
Density of the filling soil (kn/m^3) =	17.7
Coefficient of lateral earth pressure (K) =	0.691
Height of Fill (m) =	2.25
Negative skin friction (Pnf), Kn =	**6.32**

B. Negative skin friction for overall pile cap (Group Action)

Pile cap length (m) =	1.5
Pile cap width (m) =	1.5
Pile cap area (m^2) =	2.25
Pile cap Perimeter (m) =	6
No. of piles in the pile cap =	5
Negative skin friction per pile for group action (Qn), Kn =	**25.97**

The governing value, Kn =	***25.97***
Net Pile Capacity for Structural Load, Kn =	***784.79***

Case Study No.	**3**
Experimental Drag Load (Kn) =	**30.86**
Theoretical Drag Load (Kn) =	**26**
Percentage of Difference (%) =	**15.75%**

Negative Skin Friction in Piles

This work sheet is dealing with bored piles embedded in non-cohesive soil over-laid by cohesive fill

A. Negative skin friction for single pile

Pile diameter (mm) =	750
No. of bars in the pile =	16
Bars Diameter (mm) =	16
Concrete compressive strength (Mpa) =	45
Pile Structural Capacity (Kn) =	5517
Angle of friction Φ =	21
Coefficient related to applied pressure ά =	0.2560
Pile Perimeter (m) =	2.3561925
Density of the filling soil (kn/m^3) =	18.6
Coefficient of lateral earth pressure (K) =	0.642
Height of Fill (m) =	3.1
Negative skin friction (Pnf), Kn =	**34.59**

B. Negative skin friction for overall pile cap (Group Action)

Pile cap length (m) =	4.5
Pile cap width (m) =	4.5
Pile cap area (m^2) =	20.25
Pile cap Perimeter (m) =	18
No. of piles in the pile cap =	9
Negative skin friction per pile for group action (Qn), Kn =	**159.10**

The governing value, Kn =	***159.10***
Net Pile Capacity for Structural Load, Kn =	***5357.88***

Case Study No.	**4**
Experimental Drag Load (Kn) =	**172**
Theoretical Drag Load (Kn) =	**160**
Percentage of Difference (%) =	**6.98%**

Negative Skin Friction in Piles

This work sheet is dealing with bored piles embedded in non-cohesive soil over-laid by cohesive fill

A. Negative skin friction for single pile

Pile diameter (mm) =	1000
No. of bars in the pile =	15
Bars Diameter (mm) =	25
Concrete compressive strength (Mpa) =	40
Pile Structural Capacity (Kn) =	9106
Angle of friction Φ =	30
Coefficient related to applied pressure ά =	0.3851
Pile Perimeter (m) =	3.14159
Density of the filling soil (kn/m^3) =	18.5
Coefficient of lateral earth pressure (K) =	0.500
Height of Fill (m) =	2.9
Negative skin friction (Pnf), Kn =	**47.06**

B. Negative skin friction for overall pile cap (Group Action)

Pile cap length (m) =	6
Pile cap width (m) =	3
Pile cap area (m^2) =	18
Pile cap Perimeter (m) =	18
No. of piles in the pile cap =	9
Negative skin friction per pile for group action (Qn), Kn =	**137.26**

The governing value, Kn =	***137.26***
Net Pile Capacity for Structural Load, Kn =	***8968.44***

Case Study No.	**5**
Experimental Drag Load (Kn) =	**148**
Theoretical Drag Load (Kn) =	**137**
Percentage of Difference (%) =	**7.43%**

Negative Skin Friction in Piles

This work sheet is dealing with square precast piles embedded in non-cohesive soil over-laid by cohesive fill

A. Negative skin friction for single pile

Pile diameter (mm) =	300
No. of bars in the pile =	4
Bars Diameter (mm) =	12
Concrete compressive strength (Mpa) =	40
Pile Structural Capacity (Kn) =	977
Angle of friction Φ =	19
Coefficient related to applied pressure ά =	0.2297
Pile Perimeter (m) =	1.2
Density of the filling soil (kn/m^3) =	18.4
Coefficient of lateral earth pressure (K) =	0.674
Height of Fill (m) =	2
Negative skin friction (Pnf), Kn =	**6.84**

B. Negative skin friction for overall pile cap (Group Action)

Pile cap length (m) =	1.8
Pile cap width (m) =	1.8
Pile cap area (m^2) =	3.24
Pile cap Perimeter (m) =	7.2
No. of piles in the pile cap =	9
Negative skin friction per pile for group action (Qn), Kn =	**17.81**

The governing value, Kn =	***17.81***
Net Pile Capacity for Structural Load, Kn =	***959.10***

Case Study No.	**6**
Experimental Drag Load (Kn) =	**22.3**
Theoretical Drag Load (Kn) =	**19**
Percentage of Difference (%) =	**14.80%**

Negative Skin Friction in Piles

This work sheet is dealing with square precast piles embedded in non-cohesive soil over-laid by cohesive fill

A. Negative skin friction for single pile

Pile diameter (dimension) (mm) =	600
No. of bars in the pile =	4
Bars Diameter (mm) =	16
Concrete compressive strength (Mpa) =	45
Pile Structural Capacity (Kn) =	4187
Angle of friction Φ =	24
Coefficient related to applied pressure ά =	0.2970
Pile Perimeter (m) =	2.4
Density of the filling soil (kn/m^3) =	17.8
Coefficient of lateral earth pressure (K) =	0.593
Height of Fill (m) =	2.3
Negative skin friction (Pnf), Kn =	**19.91**

B. Negative skin friction for overall pile cap (Group Action)

Pile cap length (m) =	2.7
Pile cap width (m) =	1.35
Pile cap area (m^2) =	3.645
Pile cap Perimeter (m) =	8.1
No. of piles in the pile cap =	6
Negative skin friction per pile for group action (Qn), Kn =	**36.07**

The governing value, Kn =	***36.07***
Net Pile Capacity for Structural Load, Kn =	***4150.65***

Case Study No.	**7**
Experimental Drag Load (Kn) =	**42**
Theoretical Drag Load (Kn) =	**36**
Percentage of Difference (%) =	**14.29%**

Negative Skin Friction in Piles

This work sheet is dealing with square precast piles embedded in non-cohesive soil over-laid by cohesive fill

A. Negative skin friction for single pile

Pile diameter (dimension) (mm) =	600
No. of bars in the pile =	4
Bars Diameter (mm) =	16
Concrete compressive strength (Mpa) =	45
Pile Structural Capacity (Kn) =	4187
Angle of friction Φ =	24
Coefficient related to applied pressure ά =	0.2970
Pile Perimeter (m) =	2.4
Density of the filling soil (kn/m^3) =	17.8
Coefficient of lateral earth pressure (K) =	0.593
Height of Fill (m) =	2.3
Negative skin friction (Pnf), Kn =	**19.91**

B. Negative skin friction for overall pile cap (Group Action)

Pile cap length (m) =	5.4
Pile cap width (m) =	5.4
Pile cap area (m^2) =	29.16
Pile cap Perimeter (m) =	21.6
No. of piles in the pile cap =	12
Negative skin friction per pile for group action (Qn), Kn =	**114.41**

The governing value, Kn =	***114.41***
Net Pile Capacity for Structural Load, Kn =	***4072.31***

Case Study No.	**8**
Experimental Drag Load (Kn) =	**124**
Theoretical Drag Load (Kn) =	**115**
Percentage of Difference (%) =	**7.26%**

Negative Skin Friction in Piles

This work sheet is dealing with square precast piles embedded in non-cohesive soil over-laid by cohesive fill

A. Negative skin friction for single pile

Pile diameter (dimension) (mm) =	200
No. of bars in the pile =	4
Bars Diameter (mm) =	10
Concrete compressive strength (Mpa) =	40
Pile Structural Capacity (Kn) =	453
Angle of friction Φ =	30
Coefficient related to applied pressure ά =	0.3851
Pile Perimeter (m) =	0.8
Density of the filling soil (kn/m^3) =	18.2
Coefficient of lateral earth pressure (K) =	0.500
Height of Fill (m) =	2.75
Negative skin friction (Pnf), Kn =	**10.60**

B. Negative skin friction for overall pile cap (Group Action)

Pile cap length (m) =	0.6
Pile cap width (m) =	0.6
Pile cap area (m^2) =	0.36
Pile cap Perimeter (m) =	2.4
No. of piles in the pile cap =	4
Negative skin friction per pile for group action (Qn), Kn =	**12.45**

The governing value, Kn =	***12.45***
Net Pile Capacity for Structural Load, Kn =	***440.95***

Case Study No.	**9**
Experimental Drag Load (Kn) =	**14.8**
Theoretical Drag Load (Kn) =	**12.5**
Percentage of Difference (%) =	**15.54%**

Negative Skin Friction in Piles

This work sheet is dealing with square precast piles embedded in non-cohesive soil over-laid by cohesive fill

A. Negative skin friction for single pile

Pile diameter (dimension) (mm) =	150
No. of bars in the pile =	4
Bars Diameter (mm) =	10
Concrete compressive strength (Mpa) =	45
Pile Structural Capacity (Kn) =	307
Angle of friction Φ =	16
Coefficient related to applied pressure ά =	0.1913
Pile Perimeter (m) =	0.6
Density of the filling soil (kn/m^3) =	17.8
Coefficient of lateral earth pressure (K) =	0.724
Height of Fill (m) =	3
Negative skin friction (Pnf), Kn =	**6.66**

B. Negative skin friction for overall pile cap (Group Action)

Pile cap length (m) =	0.45
Pile cap width (m) =	0.45
Pile cap area (m^2) =	0.2025
Pile cap Perimeter (m) =	1.8
No. of piles in the pile cap =	4
Negative skin friction per pile for group action (Qn), Kn =	**7.70**

The governing value, Kn =	***7.70***
Net Pile Capacity for Structural Load, Kn =	***298.83***

Case Study No.	**10**
Experimental Drag Load (Kn) =	**8.6**
Theoretical Drag Load (Kn) =	**7.7**
Percentage of Difference (%) =	**10.47%**

This work sheet is dealing with square precast piles embedded in non-cohesive soil over-laid by cohesive fill

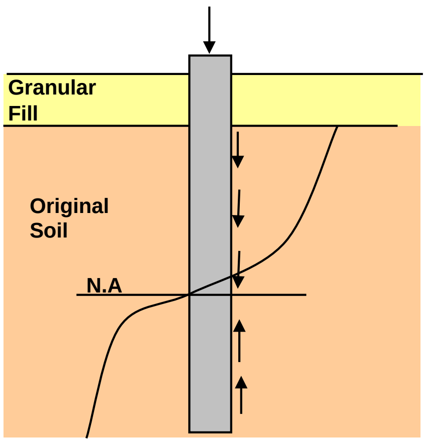
Borehole Reference :	
Height of Fill (m) =	*3.8*
Estimated Length of Pile (m) =	*25*
Effective Pile Length (L, m) =	*21.2*
Fill Density (Kn/m^3) =	*18.6*
Total Pressure Due to Fill (q_o, Kn/m^2) =	*70.68*
Underlying Soil Density (γ', Kn/m^3) =	*9*
Internal Angle of Friction (Φ, Deg.) =	*20*
K Value = 1 - SIN (Φ) =	*0.657980134*
*ά Value = 0.667 * TAN (Φ) =*	*0.243*
Pile Dimension (mm) =	*250*
Pile Perimeter (P', m)	*1.000*
Concrete Compressive Strength (Mpa) =	*45.0*
No. of Bars in the Pile =	*4*
Bars Diameter (mm) =	*10.0*
Pile Structural Capacity (Kn) =	756.53
Position of Neutral Axis (m) =	13.43
Load Increase Due to -ve Skin Friction (Kn) =	281.21
Pile's Structural Capacity Excluding -ve Skin Friction Load (Kn) =	*475.32*

Formula for the Position of the Neutral Axis is:

$$L_1 = \frac{L}{L_1}\left(\frac{L}{2} + \frac{q_o}{\gamma'}\right) - \frac{2q_o}{\gamma'}$$

Formula for the Load Due to -ve Skin Friction:

$$P_n = \alpha' p' \left(q_o + \frac{\bar{\gamma}' L_1}{2}\right) L_1 K$$



Case Study No.	11
Experimental Drag Load (Kn) =	312
Theoretical Drag Load (Kn) =	282
Percentage of Difference (%) =	9.62%

This work sheet is dealing with square precast piles embedded in non-cohesive soil over-laid by cohesive fill

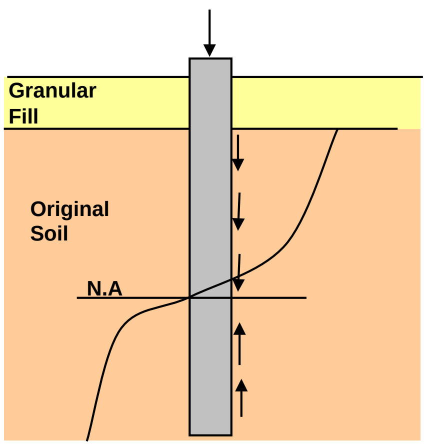
Borehole Reference :	
Height of Fill (m) =	*2.8*
Estimated Length of Pile (m) =	*20*
Effective Pile Length (L, m) =	*17.2*
Fill Density (Kn/m^3) =	*17.7*
Total Pressure Due to Fill (q_o, Kn/m^2) =	*49.56*
Underlying Soil Density (γ', Kn/m^3) =	*8*
Internal Angle of Friction (Φ, Deg.) =	*17*
K Value = 1 - SIN (Φ) =	*0.707628535*
*ά Value = 0.667 * TAN (Φ) =*	*0.204*
Pile Dimension (mm) =	*300*
Pile Perimeter (P', m)	*1.200*
Concrete Compressive Strength (Mpa) =	*45.0*
No. of Bars in the Pile =	*4*
Bars Diameter (mm) =	*12.0*
Pile Structural Capacity (Kn) =	1089.41
Position of Neutral Axis (m) =	10.92
Load Increase Due to -ve Skin Friction (Kn) =	176.26
Pile's Structural Capacity Excluding -ve Skin Friction Load (Kn) =	***913.15***

Formula for the Position of the Neutral Axis is:

$$L_1 = \frac{L}{L_1}\left(\frac{L}{2} + \frac{q_o}{\gamma'}\right) - \frac{2q_o}{\gamma'}$$

Formula for the Load Due to -ve Skin Friction:

$$P_n = \alpha' p' \left(q_o + \frac{\bar{\gamma}' L_1}{2}\right) L_1 K$$



Case Study No.	12
Experimental Drag Load (Kn) =	189
Theoretical Drag Load (Kn) =	175
Percentage of Difference (%) =	7.41%

This work sheet is dealing with square precast piles embedded in non-cohesive soil over-laid by cohesive fill

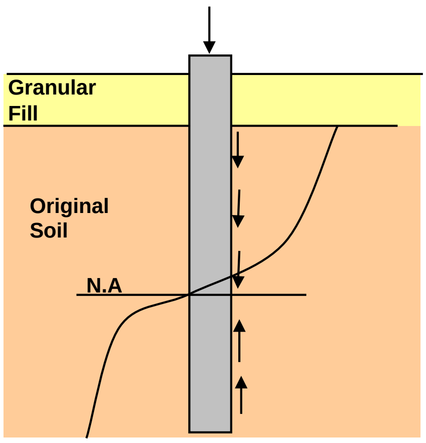
Borehole Reference :	
Height of Fill (m) =	*3.7*
Estimated Length of Pile (m) =	*30*
Effective Pile Length (L, m) =	*26.3*
Fill Density (Kn/m^3) =	*16.9*
Total Pressure Due to Fill (q_o, Kn/m^2) =	*62.53*
Underlying Soil Density (γ', Kn/m^3) =	*9*
Internal Angle of Friction (Φ, Deg.) =	*11*
K Value = 1 - SIN (Φ) =	*0.809191164*
*ά Value = 0.667 * TAN (Φ) =*	*0.130*
Pile Dimension (mm) =	*400*
Pile Perimeter (P', m)	*1.600*
Concrete Compressive Strength (Mpa) =	*45.0*
No. of Bars in the Pile =	*4*
Bars Diameter (mm) =	*16.0*
Pile Structural Capacity (Kn) =	1936.72
Position of Neutral Axis (m) =	17.07
Load Increase Due to -ve Skin Friction (Kn) =	399.27
Pile's Structural Capacity Excluding -ve Skin Friction Load (Kn) =	*1537.45*

Formula for the Position of the Neutral Axis is:

$$L_1 = \frac{L}{L_1}\left(\frac{L}{2} + \frac{q_o}{\gamma'}\right) - \frac{2q_o}{\gamma'}$$

Formula for the Load Due to -ve Skin Friction:

$$P_n = \alpha' p' \left(q_o + \frac{\bar{\gamma}' L_1}{2}\right) L_1 K$$



Case Study No.	13
Experimental Drag Load (Kn) =	428
Theoretical Drag Load (Kn) =	400
Percentage of Difference (%) =	6.54%

This work sheet is dealing with square precast piles embedded in non-cohesive soil over-laid by cohesive fill

Borehole Reference :	
Height of Fill (m) =	*4.2*
Estimated Length of Pile (m) =	*36*
Effective Pile Length (L, m) =	*31.8*
Fill Density (Kn/m^3) =	*17.5*
Total Pressure Due to Fill ($q_{o,}$ Kn/m^2) =	*73.5*
Underlying Soil Density (γ', Kn/m^3) =	*9*
Internal Angle of Friction (Φ, Deg.) =	*14*
K Value = 1 - SIN (Φ) =	*0.758078305*
*ά Value = 0.667 * TAN (Φ) =*	*0.166*
Pile Dimension (mm) =	*400*
Pile Perimeter (P', m)	*1.600*
Concrete Compressive Strength (Mpa) =	*45.0*
No. of Bars in the Pile =	*4*
Bars Diameter (mm) =	*20.0*
Pile Structural Capacity (Kn) =	2013.63
Position of Neutral Axis (m) =	20.68
Load Increase Due to -ve Skin Friction (Kn) =	694.68

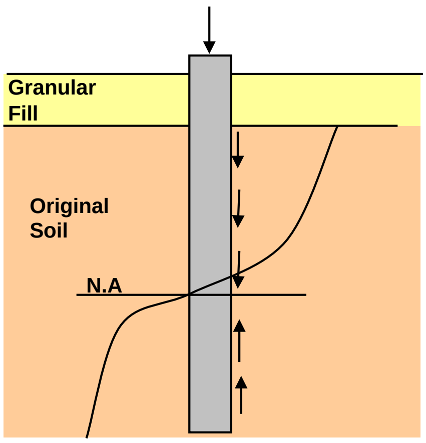
Pile's Structural Capacity Excluding -ve Skin Friction Load (Kn) = ***1318.95***

Formula for the Position of the Neutral Axis is:

$$L_1 = \frac{L}{L_1}\left(\frac{L}{2} + \frac{q_o}{\gamma'}\right) - \frac{2q_o}{\gamma'}$$

Formula for the Load Due to -ve Skin Friction:

$$P_n = \alpha' p' \left(q_o + \frac{\bar{\gamma}' L_1}{2}\right) L_1 K$$



Case Study No.	14
Experimental Drag Load (Kn) =	770
Theoretical Drag Load (Kn) =	695
Percentage of Difference (%) =	9.74%

This work sheet is dealing with square precast piles embedded in non-cohesive soil over-laid by cohesive fill

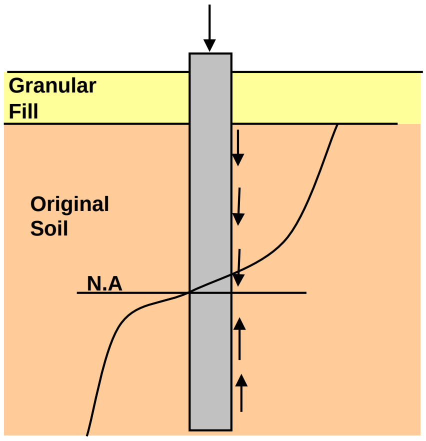
Borehole Reference :	
Height of Fill (m) =	*2*
Estimated Length of Pile (m) =	*18*
Effective Pile Length (L, m) =	*16*
Fill Density (Kn/m^3) =	*18*
Total Pressure Due to Fill (q_o, Kn/m^2) =	*36*
Underlying Soil Density (γ', Kn/m^3) =	*8*
Internal Angle of Friction (Φ, Deg.) =	*12*
K Value = 1 - SIN (Φ) =	*0.792088482*
*ά Value = 0.667 * TAN (Φ) =*	*0.142*
Pile Dimension (mm) =	*200*
Pile Perimeter (P', m)	*0.800*
Concrete Compressive Strength (Mpa) =	*40.0*
No. of Bars in the Pile =	*4*
Bars Diameter (mm) =	*10.0*
Pile Structural Capacity (Kn) =	453.41
Position of Neutral Axis (m) =	10.34
Load Increase Due to -ve Skin Friction (Kn) =	71.87
Pile's Structural Capacity Excluding -ve Skin Friction Load (Kn) =	***381.54***

Formula for the Position of the Neutral Axis is:

$$L_1 = \frac{L}{L_1}\left(\frac{L}{2} + \frac{q_o}{\gamma'}\right) - \frac{2q_o}{\gamma'}$$

Formula for the Load Due to -ve Skin Friction:

$$P_n = \alpha' p' \left(q_o + \frac{\bar{\gamma}' L_1}{2}\right) L_1 K$$



Case Study No.	15
Experimental Drag Load (Kn) =	84
Theoretical Drag Load (Kn) =	72
Percentage of Difference (%) =	14.29%

CHAPTER FOUR

CONCLUSIONS & SUGGESTIONS

Conclusions

1. *Any pile driven in fill would be subjected to negative skin friction.*
2. *The amount of the negative skin friction and considerable and have to be cater for it in design process.*
3. *The negative skin friction would be more in groups of piles rather than single piles for cases where non-cohesive soil overlaid by cohesive soils.*
4. *The negative skin friction would vary in value depend on the soil layer properties.*
5. *In this study the negative skin friction is calculated base on single soil layer the pile penetrates through.*
6. *Many studies have considered the negative skin friction and lots of those studies has suggested different approaches to estimate the negative skin friction. In this study the approach suggested by Joseph E. Bowels has been adopted.*

Suggestions

7. *This study shades the light on the necessity of considering negative skin friction in the foundation design especially in the southern Part of Iraq where piles are widely used.*
8. *Most piles penetrates more than a layer, thus more studies are required to estimate the negative skin friction.*
9. *Many approaches has been suggested to estimate the negative skin friction and in this study one approach only discussed.*

Further studies are recommended for assessing the other approaches reliability.

10. *A comparison is always required between the theory part and practical part. More experiments are required to fully understand the nature of negative skin friction.*
11. *There is ways to reduce or even neutralize the negative skin friction. These ways have to be studied further and implemented in practical aspect with special care for their effect on the pile's structural and geotechnical capacity.*

CHAPTER FIVE

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CHAPTER SIX

APPENDICES

- ## APPENDIX “A” – Joseph E. Bowles Approach in Estimating the Negative Skin Friction

Appendix "A"
Joseph E. Bowles Approach in Estimating the Negative Skin Friction

When a fill is placed on a compressible soil deposit, consolidation of the compressible material will occur. When a pile is driven through (or into) the compressible material (either before or after fill placement) before consolidation is complete, the soil will move downward relative to the pile. This relative movement will develop skin friction between the pile and the moving soil termed negative skin friction.

According to measurement reported by Bjerrum et al. (1969), Bozozuk (1972) and Bozozuk et al. (1979), the negative skin friction can exceed the allowable load for pile section. Fellenius (1972) has also reported large values of measured negative skin resistance.

The principal effect of negative skin resistance is to increase the axial load in the lower fixed portion of the pile. It may results also in increased pile settlements due to the axial shortening and/or additional point penetration of the pile under the increased axial load. Note that in figure (1), the fill settlement may be such that a gap forms between the bottom of the pile cap and the soil. This will transfer the full cap weight to the piles and may change the bending stresses in the cap.

Negative skin friction can produce large tension stresses when the effect is from expansive soils – especially if no, or insufficient, gap is left between soil and pile cap and the soil expands against both the pile and the cap.

Negative skin friction can be developed from the following:

1. *A cohesive fill placed over a cohesionless soil deposit. The fill develops shear resistance (adhesion) between the soil and pile from lateral pressure/flow effects, so that the pile is pushed downward as the fill consolidates. Little effect is produced in the underlying cohesionless soil except that the weight of fill increases the lateral pressure. This provides additional skin resistance against further pile penetration and raises the center of resistance nearer the cohesive fill for point-bearing pile.*
2. *A cohesionless fill placed over a compressible, cohesive deposit. In this case there will be some down-drag in the fill zone, but the principal down-drag will occur in the zone of consolidation. For point-bearing piles any settlement of the group will be due to axial shortening of the pile. For floating pile, additional penetration with matching settlement will occur unless the pile is sufficiently long that the bottom portion can develop enough positive skin resistance to balance the additional load developed by negative (or downward) skin resistance. In this can, an approximation of the location of the balance, or neutral, point can be made.*
3. *Lowering of the ground water table with resulting ground subsidence.*

4. *Pile-driving (and Load-test) operations that produce negative stresses in the upper shaft when the load is released and the pile shaft expands upward. The resulting slip and negative skin resistance must be balanced by a positive skin resistance in the lower shaft and/or point load [Vesic' (1977)].*

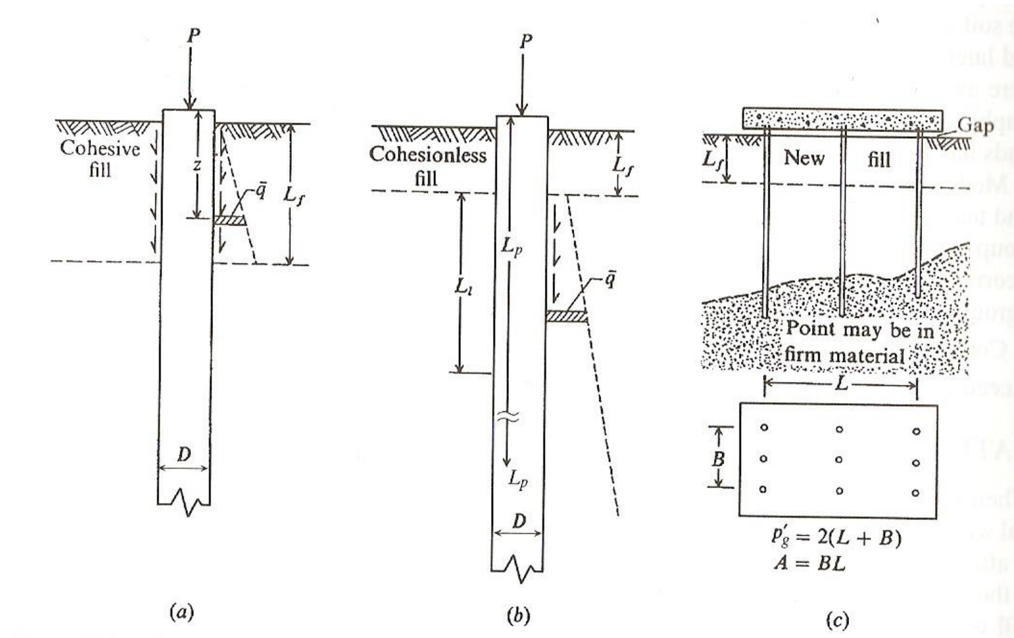

Fig. (1): Development of negative friction forces on a single pile from a cohesive fill or on a pile group in a cohesive fill

For negative skin resistance forces to develop significantly, a portion of the pile must be fixed against vertical movement, such as the point bearing on rock or the lower part being in a dense sand. If the entire pile moves down with the consolidation effect no negative skin resistance forces develop. For a single pile, the negative skin resistance force can be estimated as follows:

I. *For cohesive fill overlying cohesionless soils in figure (1-a):*

$$P_{nf} = \int_0^{L_f} \alpha' p' \bar{q} K dz$$

Where: $\alpha^{'}$ *= coefficient relating the effective lateral pressure* $\bar{q}K$ *to the shearing resistance about the pile perimeter,* $\alpha^{'}$ *= tanδ where δ≈0.5 to 0.9ø,* S_u *is replaced by* $\bar{q}K$ *as this is somewhat of a drained case.*

$p^{'}$ *= Pile Perimeter*

K *= Lateral earth- pressure coefficient; use k =* k_o*=1-sin ø*

$\bar{q}$*= effective overburden pressure at any depth z*

II. *For cohesive soil underlying cohesionless fill, take the origin of coordinates at the bottom of the fill (see figure (1-b):*

$$P_{nf} = \int_0^{L_1} \alpha^{'} p^{'} \bar{q} K dz$$

Below the neutral point (refer to figure 2), if there is one, positive friction is developed to the bottom of effective pile length L:

$$P_{nf} = \int_{L_1}^{L} \alpha_2^{'} p^{'} \bar{q} K dz + p_{np}$$

Where p_{np} *= amount of negative skin resistance carried by the point where point-bearing piles are used and other terms as previously defined.*

Note that the general form of $\bar{q}$*is:*

$\bar{q}$=$\bar{q}_o + \gamma^{'} z$

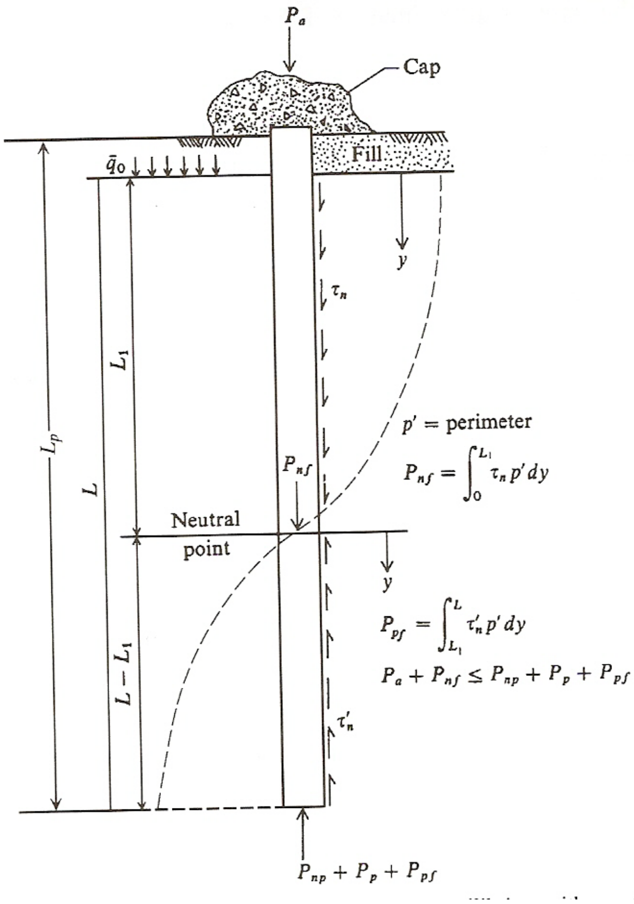

Fig. (2): Location of neutral point to satisfy statics of vertical equilibrium with negative skin friction acting on pile

Also it may be necessary to adjust the integration limits if the soil is stratified to obtain a summation of negative skin contribution.

If we take $\alpha' = \alpha_2'$, *and a floating pile where* p_{np} *=0, and if we equate the above equations upon integration for their limits, we obtain:*

$$L_1 = \frac{L}{L_1}\left(\frac{L}{2} + \frac{\overline{q_o}}{\gamma'}\right) - \frac{2\overline{q_o}}{\gamma'}$$

Note L is the effective pile length in the embedment zone and usually is not L_P.

When the piles are spaced at small S/D ratios, the negative friction force may act effectively on the block perimeter rather than on individual piles to obtain two modes of stressing requiring investigation:

1. *The total group negative skin resistance as the sum from individual piles,*

   $$Q_n = \sum p_{nf}$$

2. *The "block" skin resistance based on shear resistance on the block perimeter + weight of block trapped between the piles,*

   $$Q_n = f_s L_f P_g' + \gamma L_f A$$

   Where:

   γ *= unit weight of soil enclosed in pile group to depth of* L_f

   A *=area of pile group enclosed in perimeter* P_g'

   $f_s = \alpha' \bar{q} k$ *= effective skin resistance on the group perimeter*

   P_g' *=Perimeter of pile group*

Some evidence exists that coating the pile shaft down-drag zone with special bitumen mixture will substantially reduce negative skin friction force.