Consolidating Soil-Pile Interaction

Interaction pieux-sol en cours de consolidation

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ABSTRACT: Soft clay is consolidated upon any slight change in the effective vertical stress. Consolidation causes a downdrag movement to the shaft piles that penetrate this layer. The downdrag movement adds additional loads to the already loaded pile. This force is expressed as a negative skin friction. Negative skin friction extends to a depth depending on the consolidating soil-pile interaction. This depth is referred to as the neutral plan.

This paper presents a study on the behavior of soil-pile interaction during soil consolidation. Experimental work was developed to analyze the negative skin friction and the location of the neutral plane on a single pile embedded in soft clay layer. The pile is ended either in clay or sand soil, or is floated. The clay layer was allowed to consolidate. The study concluded that the neutral plane is located closer to the end of the pile as the end bearing increases. The depth of neutral plane increases by increasing the embedded length of the pile in the clay layer. Closed form equations of Shong, 2002, predict the experimental results in very well agreement.

RÉSUMÉ : Une argile molle se consolide sous l'effet d'une augmentation de la contrainte verticale effective. Cette consolidation provoque un déplacement vertical des pieux qui pénètrent la couche de sol. Ce mouvement ajoute des charges supplémentaires au pieu déjà chargé. Cette force peut être exprimée en frottement latéral négatif et s'étendre en profondeur en fonction de l'interaction sol-pieux en cours de consolidation. Cette profondeur est définie comme étant le plan neutre.

Cet article présente une étude sur le comportement de l'interaction sol-pieux lors de la consolidation des sols. Un travail expérimental a été développé pour analyser le frottement négatif et l'emplacement du plan neutre sur un seul pieu fondé dans une couche d'argile. La couche d'argile peut se consolider. L'étude montre que le plan neutre se situe près de l'extrémité du pieu ou la charge augmente. Ainsi, la profondeur du plan neutre augmente en même temps que la longueur du pieu. Ces résultats sont en accord avec ceux de Shong, 2002.

KEYWORDS: soft clay consolidation, pile foundation, negative skin friction, down drag movement neutral plane, floating piles

1 INTRODUCTION

Deep foundations are installed in different soil stratification. Usually the penetrated soil stratifications offer considerable resistance for the pile shaft deformation upon loading. This resistance is called shaft resistance or skin friction. This soil behavior can be reversed drastically if the penetrated soil encompasses soft clay.

Soft clay is consolidated upon any slight change in the effective vertical stress. Consolidation and settlement of soft clay surrounding a pile usually drag the pile downward (Walker and Dravall 1973, Sharif 1998, Leung, et al. 2004). The downdrag movement adds additional loads, downdrag force, to the already loaded foundation. Dragload heavily depends on the interface properties (friction coefficient, and adhesion), surface loading, and axial load (Sangseom Jeong, et al. 2004).

This force is the most common problem in the design and construction of pile foundations in soft soil (Maugeri et al. 1997, Van Der Veen 1986). (Briaud 2010) pointed out situations where downdrag force should be considered in the design.

The downdrag force can be expressed as a negative skin friction (N.S.F) as it acts in the opposite direction for the normally skin friction. The negative skin friction will be mobilized in the upper portion starting from the pile head to a neutral depth, ND, after which positive skin friction is mobilized in the lower portion. ND can also be defined as the depth at which the relative displacement between the pile and the soil is zero (Fellenius 1989, 2004, and 2006). (Bozozuk 1972) suggested that ND depends only on the embedded length of the pile in the clay layer. However, (Poorooshasb et al. 1996) stated that ND is not highly influenced by the surcharge. But that the presence of a strong soil layer at the tip of the pile would have a significant influence.

(Shong 2002) proposed closed form equations to locate the neutral plane depending on the elasticity of the soil and the pile. According to this approach, the depth of neutral plane, ND, to pile penetration length, L is:

$$\frac{ND}{L} = \sqrt{\frac{Q_u}{2Q_s}\left[(1-\frac{Q_d}{Q_u})\right]} \tag{1}$$

Where

Q_u = ultimate pile capacity = Q_s+ Q_t (2)

Q_s, the pile shaft resistance over the whole shaft length, is:

$$Q_s = \int_0^L \beta\ (\pi D z)\sigma_z^{'} dz \tag{3}$$

L = length of the pile,
D = pile diameter,
Q_t, = the pile toe resistance:

$$Q_t = N_t.A_t.\sigma^{`}_{z=L} \tag{4}$$

A_t = the toe area of the pile,
Q_d = imposed load at pile top

Shong suggested a range of β varies between 0.25 - 0.35 for clay, and a value of 3 for N_t, toe bearing capacity coefficient.

Shong approach is adapted in the analysis of the results in this study.

This paper represents a study to investigate the behavior of soil-pile interaction during soil consolidation. Since full-scale testing of the influence of the large number of variables involved is economically unreasonable, a simulated laboratory experiment has been designated. A special rig was designed and constructed for this purpose. In order to carry out the investigations, experimental program was developed.

2 EXPERIMENTAL WORK

(Nasser 2010) arranged the experimental rig as shown in Figure 1. Three P.V.C circular model piles with different diameters of 1.5, 2 and 2.5 cm were chosen to model the pile in this study. Compression tests were carried out on a specimen of this P.V.C pipes to determine its modulus of elasticity. The modulus of elasticity of the pile material; P.V.C is 1500000 KN/m^2. The surface of the pile is smooth.

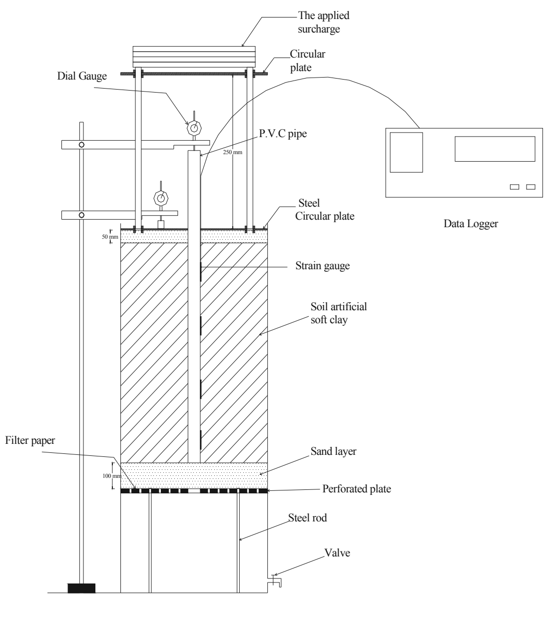

Fig. 1 The experimental rig

Two layers of sand with depth 5cm for each layer. One of these layers placed on the surface of soft soil and the second layer under the soil.

Bentonite soil has been adapted for the experimental investigation in this study. 85% water content provides a soft consistency of bentonite. For this condition initial void ratio is 2.38, bulk density is 16 KN/m^3.

A surcharge has to be applied to the soil layer for consolidation. Steel and lead plates of 1.2kN total weight were arranged for the surcharge. These loads simulate a surcharge of 1m fill of unit weight 17kN/m^3.

The vertical displacement of the model pile and the soil surface were measured using dial gauges. The displacement of the model pile is measured at its center. The soil settlement is measured at a point located at the mid distance between the pile shaft and the container wall.

Strain gauges are fixed at various depths of the length of the pile to measure the strains occurred on the pile during soil consolidation. Number and locations of the strain gauges are designed depending on the thickness of the clay layer.

Table (1) illustrates the experimental program. Three cases of boundary conditions were considered in this study. The first case is the pile ended in the clay layer. This represents clay end bearing case. The second case is the pile ended in the sand layer. And the last case represents floating pile. That is, the pile passes through the lower plate and not resting on any soil. This case is aimed to investigate the pure shaft resistance without the interference of the end condition.

Table (1) Different cases with code for each test

No.	Cases	Pile Diameter (cm)	L/d	Pile Length (cm)	Code	No. of Strain gages
1	CASE I (end bearing clay EC)	1.5	10	15	L15EC	3
2			15	22.5	L22.5EC	3
3			20	30	L30EC	4
4		2	10	20	L20EC	3
5			15	30	L30EC	4
6			20	40	L40EC	4
7		2.5	10	25	L25EC	4
8			15	37.5	L37.5EC	4
9	CASE II (end bearing sand ES)	1.5	10	15	L15ES	3
10			15	22.5	L22.5ES	3
11			20	30	L30ES	4
12		2	10	20	L20ES	3
13			15	30	L30ES	4
14			20	40	L40ES	4
15		2.5	10	25	L25ES	4
16			15	37.5	L37.5ES	4
17	Floating (F)	2.5	10	25	L25F	4
18			15	37.5	L37.5F	4
19			20	50	L50F	4

3 RESULTS AND ANALYSIS

3.1 Time–strain behavior of pile model

Figures 2 shows the axial strain with time of applying the surcharge, for pile model (L22.5Ec). From the Figure, it can be seen that the top strain is higher and occurs earlier than the middle and the bottom strains. However, it declines just after reaching the early peak. The other strains continue increasing until the end of the test.

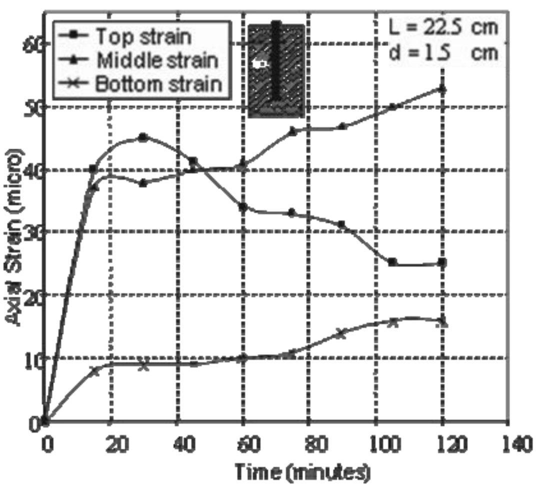

Fig. 2 Time-Axial strain curves of pile model (L22.5Ec)

It is clear that the top portion of the deposit go through consolidation due to the nearby surcharge faster than the remaining deposit. By the time being, the rate of water dissipation decreases, and, hence, the axial strain decreases. The middle axial strains continue increasing with lower rate than the initial one. The bottom strain is much less than the upper and middle ones. The stress increase, and hence the water dissipation and the soil consolidation, is moderate at the bottom strain compared with the other two locations.

Figure 3 illustrates the pile strains for pile model (L22.5Ec). It can be seen from the Figure that the bottom strains are generally larger when compared with the previous case of pile ended in clay. This can be attributed to the increased dissipation of pore water pressure through the bottom layer. Moreover, sand layers offer resistance to the pile movement; end bearing. This increases stresses in the bottom portion of the pile. Hence, there is increase in the monitored strain.

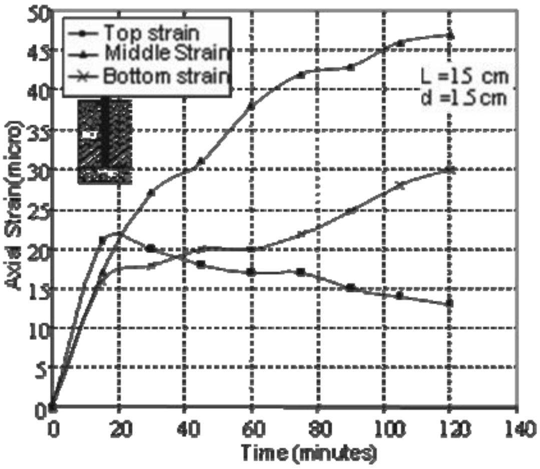

Fig. 3 Time- Axial strain curves of pile model (L15ES)

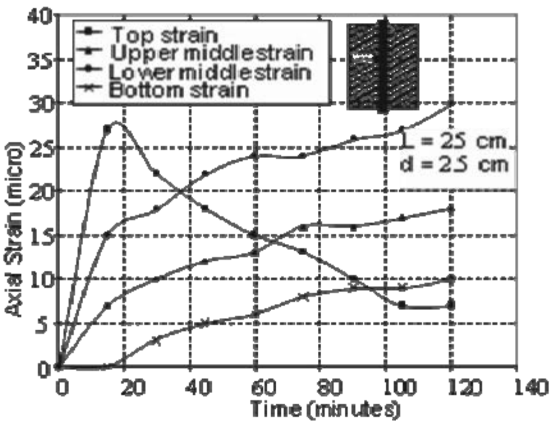

Fig.4 Time- Axial strain curves of pile model (L25F)

Figure 4 shows the pile strains for pile model (L25F). Here, the boundary condition at the pile tip does not allow for developing stresses at the bottom of the clay layer. Hence, the bottom strains are the least among all locations along the pile.

3.2 *Distribution of normal strain and shear stress along the normalized depth of the pile*

Figure 5 depicts the distribution of normal strain and shear stress along the normalized depth of the pile models (L15EC and L15ES). The shown strains are those monitored at the end of the test. As can be seen from the Figure, small values of strains are indicated in the upper part where the excess pore pressure had dissipated, and hence, consolidation process had decayed. The axial strain increases by the presence of a sand layer at the pile tip (case II). Strain increases until it reaches a peak value at an intermediate depth. Then, it decreases. Strain decrease reflects a decrease in the dragging force. That is the developing of positive shear resistance along the pile shaft. Hence, the zone of the peak strain is a transition zone from negative skin friction to positive skin friction. Obviously, the neutral plan is located at this peak point.

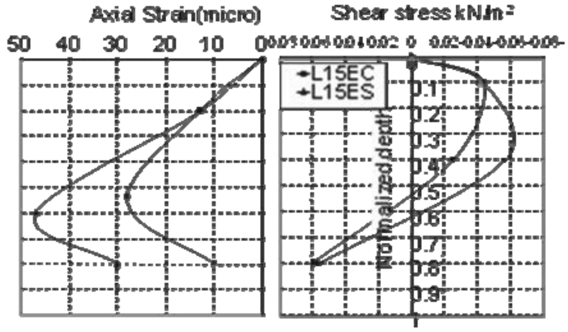

Fig. 5 Axial strain and shear stress distribution along pile length for cases (L15EC and L15ES)

The developed shear stress along the pile shaft is calculated from the monitored strain. Shear stress starts from zero value at the surface of the soil and increases until it reaches a peak negative value at an intermediate depth, then it decreases down to zero at the elevation of the neutral plane where the positive skin friction develops. Neutral plane is determined where shear stress changes from negative to positive. That is at the intersection of the curve with the vertical axis. As can be seen from the Figure the location of the neutral plane is matched from these two approaches.

As can be seen from Figure 5, the transition of pile ended in clay is located near to the middle of the pile. The transition zone of pile ended in sand is located near the pile toe. The neutral plane is located at the peak point of the transition zone. In addition, it can be seen that the normalized neutral depth is (0.53) and (0.61) for the cases of end bearing on clay and sand, respectively. That is the neutral plane is located closer to the end of the pile as the base layer gets stiffer.

This observation can be explained based on the simple equilibrium of vertical forces. (Accumulative negative skin friction = accumulative positive skin friction + bearing resistance). Since small-bearing resistance is available for case I, positive skin friction should be large enough to resist negative skin friction. Hence, negative skin friction will be reduced with the neutral plane being located further from the pile tip.

3.3 *Effect of the pile length embedded in the clay layer on the location of neutral plane*

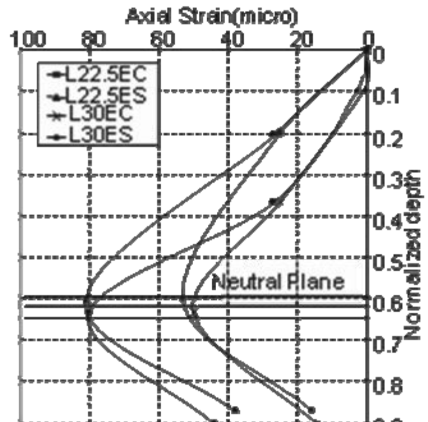

Fig. 6 Normalized neutral plane for piles ended in sand and clay (same L/d)

Figure 6 illustrates the normalized neutral depth as a function to the pile length embedded in the clay layer. The pile diameter is 1.5cm. Two cases; I, and II of tested piles are shown in the Figure. As can be seen from the Figure, the normalized neutral depth increases with the increase of the pile length embedded in the clay layer.

The explanation of the effect of the pile length on the neutral plane is due to the compressibility of the pile length. When negative skin friction is induced in a short pile, most of the downdrag is transmitted to the pile tip in the form of penetration to the bearing layer. Whereas, for long pile the downdrag is partly taken by the pile compressibility, and partly transmitted to the tip.

3.4 Determination of the Neutral Plane

Shong approach is adapted to locate the neutral plan. This approach is outlined in Section 1. The factor of safety of pile capacity against ultimate pile capacity, Fs is considered two, and β is taken 0.3.

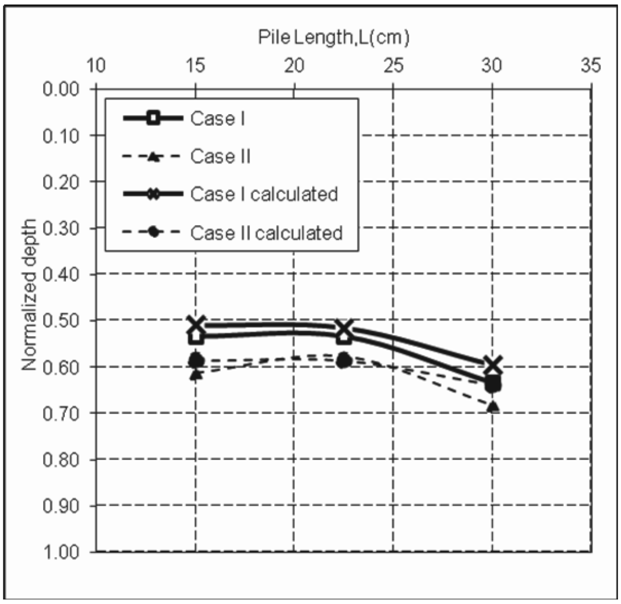

Fig. 7 Monitored and Calculated Location of the Neutral Plane

Figure 7 depicts the normalized neutral depth from both the monitored data and Shong approach. The pile diameter is 1.5cm. Two cases; I, and II, of tested pile are shown in the Figure. As can be seen from the Figure, the calculated normalized neutral depth is having the same trend as the monitored ones. The approach results into very good agreement with the normalized neutral depth.

4 CONCLUSION

This paper presents a study on the behavior of pile during soil consolidation. In order to carry out the investigations, experimental program was designated. A special rig was designed and constructed for this purpose. Based on test results the following conclusions are reached:

- Consolidation of the soil surrounding the pile due to surcharge loads induces negative skin friction on piles.
- The monitored strains along the pile length reflect the consolidation process with time. Strain increases from the pile top until it reaches a peak value at the location of the neutral plane.
- Sand layers, at the pile tip, offer resistance to the pile movement; end bearing. This increases stresses in the bottom portion of the pile. Hence, there is increase in the monitored strain, when compared with piles ended in clay, or floated.
- Shear stress is initiated from zero value at the surface of the soil and increases until it reaches a peak negative value at an intermediate depth. Then, it decreases down to zero at the elevation of the neutral plane where the positive skin friction develops.
- The neutral plane is located closer to the end of the pile as the base layer getting stiffer.
- The normalized neutral depth increases with the increase of the pile length.
- Shong approach for ND results into very good agreement with the monitored ones.

5 REFERENCES

Bozozuk M. 1972. Downdrag measurements on a 160-ft floating pipe test pile in marine clay. Canadian Geotechnique J. 9(2), 127-136.

Briaud J. L. 2010. Designing for downdrag on uncoated and coated piles. Piling and Deep Foundation. Middle East.

Fellenius B. H. 1989. Unified design of piles and pile groups. Transportation Research Record. 1169, 75-82.

Fellenius B. H. 2004, Unified design of piled foundations with emphasis on settlement analysis. ASCE. Geotechnical Special Publication. GSP 125, 253 - 275.

Fellenius B. H. 2006. Piled foundation design – clarification of a confusion. Geotechnical News Magazine. 24 (3), 53-55.

Leung C. F., B. K. Liao, and Chow Y.K. 2004. Behavior of pile subject to negative skin friction and axial load. Japanese Geotechnical society.44 (6), 17-26.

Maugeri M., Amenta G., Castelli F. and Motta E. 1997. Settlement of a piled foundation due to negative skin friction. Proc. 14th. int. Conf. on SMFE Hamburg. (2), 1111-1114.

Nassar A. E. 2010. Geotechnical Behavior of Pile during Soil Consolidation. Msc. Zagazig University. Zagazig. Egypt.

Poorooshasb H. B., Alamgi M. and Miura N.1996. Negative skin friction on rigid and deformable piles. Computers and Geotechnics, 18 (2), 109-126.

Sangseom J., Leea J. , Leeb C. J. 2004. Slip effect at the pile–soil interface on dragload. Computers and Geotechnics 31, 115.126.

Sharif, A. 1998. Negative skin friction on single piles in clay subjected to direct and indirect loading. M.Sc. thesis. Concordia university, Montreal. Quebec. Canda.

Shong Ir L. S. 2002. Pile design with negative skin friction, Geotechnical Engineering. 27 Puteri Pan Pacific Hotel.

Van Der Veen C. 1986. A general formula to determine the allowable pile bearing capacity in case of negative friction. International conference on deep foundations. Beijing. China, pp. 2.138-2.147.

Walker L.k., and Darvall,P. 1973. Downdrag on coated and uncoated pile. Proc. 8th int. conf. in soil mech. Eng. Moscow. 2(1), 257-262.