BEHAVIOR OF PILE SUBJECT TO NEGATIVE SKIN FRICTION AND AXIAL LOAD


ABSTRACT
Centrifuge model tests have been conducted to investigate the effect of negative skin friction on piles installed through soft clay and founded in underlying dense sand. The first test series involved the study of fundamental mechanism of piles subject to negative skin friction only. In these tests, negative skin friction was induced by self-weight consolidation of clay followed by in-flight sand surcharge placement on the clay. The second test series examined the behavior of piles subject to simultaneous negative skin friction and axial load. The effects of axial load on the load-transfer characteristics along a pile experiencing locked-in negative skin friction induced by consolidating clay are investigated in detail in this paper. The effects of pile tip condition (end-bearing or socket pile), pile socket length and magnitude of applied load on the pile are also studied.

Key words: axial load, centrifuge modelling, consolidation, load transfer, negative skin friction, surcharge (IGC: E4/E14/H2)

INTRODUCTION
Negative skin friction occurs in piles installed through soft soil that undergoes consolidation caused by surcharge loading, lowering of piezometric level and soil re-consolidation after pile driving. It would induce additional vertical loading and settlement on a pile and in severe conditions may lead to structural failure of the pile (see for example Inoue et al., 1977; Kog, 1987). Although many studies had been carried out to examine pile behavior subject to negative skin friction, the mechanism is still not well understood and existing design methods are mainly empirical in nature. Some researchers have conducted field tests to study the behavior of pile subject to negative skin friction, see for example, Endo et al. (1969), Fellenius and Broms (1969), Fellenius (1972) and Indraratna et al. (1992). Although these field tests provided valuable information on pile behavior subject to negative skin friction, the huge cost involved rendered the number of field studies to be limited. In addition, the changing ambient conditions such as the fluctuation of groundwater level and temperature variations during the long test duration required for the development of negative skin friction inevitably affects the quality of test data. Recently, a number of centrifuge model studies have been carried out to study negative skin friction (see for example, Lee et al., 1998; Tomas et al., 1998; Tomisawa and Nishikawa, 2000). However, all these studies examined the pile behavior subject to negative skin friction only. In practice, piles are usually subjected to simultaneous applied load and dragload caused by negative skin friction. The effects of applied load on the load-transfer characteristics along a pile subject to negative skin friction deserve an in-depth study.

In the present study, centrifuge model tests have been conducted to investigate the development of negative skin friction on piles installed through consolidating soft clay. The negative skin friction is induced by self-weight consolidation of clay and then by in-flight placement of sand surcharge on the clay. After the clay consolidation has been completed, axial load is applied to the pile to study the behavior of pile subject to simultaneous axial load and negative skin friction. The test results are presented in detail in this paper.

EXPERIMENTAL SETUP AND PROCEDURE
The experimental set-up and test procedure are briefly described here and more details are given in Shen et al. (2002). All the centrifuge model tests were conducted at 100 g on the National University of Singapore Geotechnical centrifuge. This 2-m radius centrifuge comprises a balanced arm with dual swing platform with a capacity of 40 g-tonnes. Details of the facility are given in Lee et al.
The soil container has an internal dimension of 400 mm in length, 420 mm in width and 480 mm in height, as schematically shown in Fig. 1. The model pile is fabricated from a 270 mm long cylindrical aluminum tube with an outer diameter of 12 mm and an internal diameter of 10 mm. A total of 9 groups of semi-conductor strain gauges (in full Wheatstone bridge configuration) are bonded on the external surface of the model pile at appropriate levels to measure the load transfer along the pile shaft during the tests. The semi-conductor strain gauges have a gauge factor of at least 760 times that of normal strain gauges. Such high gauge factor enables the loads along the pile shaft to be measured with high accuracy. In addition, the full bridge configuration at each section immunes the measuring system from any temperature shifting effects. The strain gauges are protected by a thin layer of epoxy, which results in a final model pile external diameter of 16 mm. Thus, the completed instrumented model pile would simulate a 27 m long prototype pile with an external diameter of 1.6 m at 100 g.

An 80-mm thick sand layer is first formed by raining silicon sand into the soil container. Kaolin clay slurry with water content of 1.5 times its liquid limit is then transferred into the container. The container is then mounted on the centrifuge swing platform and spun up to 100 g to consolidate the clay for about 6 hours resulting in a soil profile of 160-mm thick soft clay overlying 80-mm thick dense sand. The centrifuge was then stopped and the assembly of vertical actuator and sand hopper was placed on top of the container. The instrumented model pile was then jacked into the kaolin sample and embedded into the underlying dense sand using the vertical load actuator. For the test involving end-bearing piles, the underlying sand layer is replaced by a thick acrylic plate at the base of the model container. In such case, the pile is only installed through soft clay and rested on the rigid acrylic plate.

The completed model setup is then transported onto a swinging bucket of the centrifuge which is then spun up to 100 g. At this stage, the clay undergoes self-weight re-consolidation as it had swelled by a certain extent due to stress relief from 100 g to 1 g during the centrifuge spun-down earlier. After re-consolidation of the clay has been completed, the horizontal actuator is activated. This enables the alignment of the holes at the base of the sand hopper and the underlying plate to coincide thus facilitating the sand from the hopper to fall onto the clay. The guiding flaps below the sand hopper are used to overcome the Coriolis Effect and direct the falling sand to form an uniform surcharge on the soft clay surface. At this stage, the clay undergoes further consolidation settlement due to surcharge loading. For the tests in the second series, axial load is applied on the pile using the vertical load actuator prior to the placement of surcharge. The responses of the instrumented model pile as well as the pore water pressure and surface settlement of the soil during the entire test process are monitored by the data acquisition system in the centrifuge control room at regular intervals.

SOIL PROPERTIES AND STRENGTH PROFILE

After soil re-consolidation, the bulk unit weight of the soft clay is 14.6 kN/m³ at the clay surface. The unit weight increases approximately linearly with depth to 16.4 kN/m³ at 160 mm depth (16 m in prototype scale) below the clay surface. After the consolidation under surcharge, the unit weight increases to 15.4 kN/m³ at the clay surface and increases linearly with depth to 17.4 kN/m³ at 16 m depth below the clay surface. The ground water table is maintained at the clay surface level throughout the test by the provision of an overflow standpipe. Oedometer tests were also carried out on the remolded clay samples and the compression index of the clay is established to be 0.533.

In the present study, vane shear and T-bar penetrometer tests (Stewart and Randolph, 1991) were carried out to evaluate the soil strength profile. The T-bar adopted in the present study has a cross bar 25 mm long and 5 mm in diameter attached perpendicularly to the end of a vertical shaft to form a T. The penetration resistance was monitored by a load cell mounted immediately behind the T-bar. After self-weight consolidation of the clay is completed, the T-bar is pushed into the clay in-flight at a constant penetration rate of 20 mm/s. At this rate, this method determines the soil strength in an undrained condition. The T-bar tests were conducted at both 100 g and 1 g conditions while vane shear tests were conducted at 1 g only. By comparing the T-bar test data against vane shear test data at 1 g, a bearing capacity factor of 10.5 is obtained for converting the measured T-bar data to undrained soil strength. The results shown in Fig. 2 reveal that the undrained shear strength of the clay essentially increases linearly with depth, as expected. However, the
soil shear strength measured at 1 g is approximately 30% lower than that measured at 100 g. This is not unexpected as the clay would swell and soften due to the release of high overburden pressure from 100 g to 1 g.

Unfortunately, in-flight T-bar tests could not be conducted to determine the soil strength profile after soil consolidation upon surcharge. As such, vane shear tests were conducted at 1 g after the test beneath the centre of the embankment. Assuming the same difference of 30% applies to the clay between 1 g and 100 g under surcharge loading, the projected soil strength profile after surcharge loading at 100 g can be extrapolated using the measured vane shear data at 1 g. The projected soil strength profile is also plotted in Fig. 2.

PILE SUBJECT TO NEGATIVE SKIN FRICTION ONLY

Two series of centrifuge model tests were conducted in the present study. The first test series involved the study of negative skin friction on a free-head single pile during self-weight soil re-consolidation and subsequent sand surcharge. This test series is aimed at studying the basic mechanism of pile subject to negative skin friction only. The second test series is aimed at investigating the pile behavior subject to simultaneous vertical load and negative skin friction. Table 1 summarizes the test parameters and conditions of the two test series.

Test N1 in the first test series was used to illustrate the pile behavior subject to negative skin friction only. In this test, the model pile penetrated through the 160-mm thick soft clay (16 m at 100 g) and socketed 25 mm (2.5 m at 100 g) into the underlying sand. Although the clay has been normally consolidated at 100 g during the clay sample preparation stage, the model setup needs to be spun down from 100 g to 1 g to facilitate the installation of model pile and placement of sand hopper. The clay has swelled due to the release of overburden pressure from 100 g to 1 g. For the test proper, the model setup is spun up to 100 g again and the soil would reconsolidate and settle. This self-weight soil re-consolidation approximately simulates the condition of a pile installed in clay that is still consolidating under self-weight.

Unless otherwise stated, all the test results are presented in prototype scale in this paper. Figure 3(b) shows that the excess pore water pressure due to self-weight soil re-consolidation has completely dissipated after a period of about 83 months. During this period, the soil settlement keeps on increasing, but the rate of increase in settlement decreases with time as shown in Fig. 3(c). A final soil settlement of about 1060 mm was recorded at the end of self-weight re-consolidation. The large soil settlement drags the pile down and a final pile settlement of about 13 mm is observed at the end of this stage; see Fig. 3(d). However, unlike soil settlement that increases continuously throughout the entire stage, the pile settlement essentially stabilizes after 30 months. At this time, the soil settlement is about 960 mm. Although thereafter the soil continues to settle by another 100 mm, this further soil settlement does not induce any additional pile settlement.

The axial force profiles along the pile shaft during self-weight soil re-consolidation are represented by the left hand four curves in Fig. 4. It can be seen that the dragload at various elevations increases gradually with time. The dragload reaches its respective maximum magnitude after a period of 30 months which is the identical time for the maximum observed pile settlement presented earlier. Some researchers (for example Fellenius, 1972; Lambe and Whitman, 1979) reported that a small magnitude of relative settlement in the order of 6 mm to 25 mm between the soil and the pile is
sufficient to cause full development of negative skin friction on a pile. However, other field studies (for example Lee and Lumb, 1982; Clemente, 1984) revealed that dragload on a pile continues to increase with increase in soil settlement well beyond 400 mm to 1000 mm. The present test results shown in Figs. 3(c) and (d) appear to support the latter finding that negative skin friction on a pile increases with increasing soil settlement until a significantly large soil settlement is reached. It is generally known that with the dissipation of excess pore pressure, there is a corresponding increase in soil effective stress and soil settlement. This would eventually lead to full mobilization of resistance at the pile/sand interface at large soil movement and after which there would be no further dragload and settlement induced on a pile.

Figure 4 also reveals that the elevation of the neutral plane does not deviate much during soil re-consolidation and is located at an elevation of about 90% depth of the clay stratum. The observation of approximately constant neutral plane elevation has also been reported by researchers carrying out field studies. In his field study on negative skin friction, Fellenius (1972) reported that the neutral plane of a pile subject to soil re-consolidation upon pile driving is approximately constant 10 days after driving. Inoue (1979) carried out a field test on steel piles and reported that the neutral plane elevation remains essentially constant 16 days after pile driving. The findings of the present centrifuge model study and early field tests suggest that the neutral plane elevation shifts from the ground surface before soil consolidation to the final neutral plane elevation within a relatively short period of time.

At the end of self-weight soil re-consolidation, a 70 kPa sand surcharge is placed on the clay in-flight causing an immediate increase of excess pore pressure in the clay, as shown in Figs. 3(a) and 3(b). This induced excess pore pressure begins to dissipate with time causing further soil settlement to develop, as shown in Fig. 3(c). Further soil settlement, as well as increase in soil strength due to further consolidation, induces larger dragload on the pile and thus causes additional pile settlement. Similarly to self-weight re-consolidation stage, the pile settlement increases with time but essentially stabilizes after 120 months (about 36.5 months after the placement of surcharge), although the soil continues to settle further (see Fig. 3(c)). The negative skin friction developed along the pile shaft also stabilizes after 120 months, as revealed by the right hand curves in Fig. 4. This observation supports the findings of Poulos (1997) that the major concerns of pile subject to negative skin friction are the induced pile settlement and the structural integrity of the pile due to significant dragload induced on the pile.

The axial force profiles along the pile shaft during the surcharge stage are similar to those under self-weight re-consolidation stage. Dragload at all elevations increases with time under surcharge loading. In addition,
the neutral plane does not move much during the entire stage. It should be noted that the pile axial force at the original ground elevation is greater than zero (Fig. 4) mainly due to the surcharge load.

The unit shaft friction along the pile shaft can be derived from the pile axial force profiles given in Fig. 4 and the results are presented in Fig. 5. The soil strength profiles for the two test stages are also plotted in Fig. 5 for comparison. It is evident that the magnitude of negative unit shaft friction increases with depth till the neutral plane. In addition, the magnitude keeps on increasing with time in both self-weight re-consolidation stage and sand surcharge stage. As mentioned earlier, with the dissipation of excess pore pressure and the corresponding increase in effective stress, the soil continues to gain strength as the clay consolidates and this directly contributes to the increase in the unit shaft friction along the pile shaft.

The total stress method is widely used in practice in which the unit negative shaft friction $f_s^e$ is given by $\alpha S_u$, where $\alpha$ is an empirical adhesion factor and $S_u$ is the undrained shear strength of the clay. At the end of self-weight soil re-consolidation stage and soil consolidation due to surcharge stage, the negative unit shaft friction values observed along the pile shaft are close to the respective soil strength profile, as shown in Fig. 5. This indicates that $\alpha$ is approximately equal to 1.

An alternative approach to estimate the negative shaft friction is the effective stress approach that relates the unit negative shaft friction mobilized along the pile shaft to the available effective overburden pressure in the soil, namely $f_s^e = \beta \sigma_v^e$, where $\beta$ is an empirical factor and $\sigma_v^e$ is the effective overburden pressure. Figure 6 shows the derived $\beta$ values obtained at the end of all the model tests in the present study. It can be seen that all the $\beta$ data consistently concentrate within a narrow band of between 0.2 and 0.3 with an average value of 0.25. This is fairly consistent with the $\beta$ values reported for clays by various researchers such as Endo et al. (1969), Burland (1973), Garlanger (1974), and Broms (1976).

**Effects of Pile Tip Condition**

To investigate the effects of pile tip condition on the development of negative skin friction on a pile, test N2 was conducted following the same test procedure as test N1 but with different soil and pile tip conditions. In test N2, the pile penetrated through 200 mm thick soft clay and rested on a stiff Perspex plate simulating an end-bearing pile condition. Figure 7 shows the comparison of pile axial load profiles between tests N1 and N2 at the end of self-weight soil re-consolidation and soil consolidation due to surcharge. It is evident that the dragload devel-
Fig. 8. The development of pile head settlement and maximum dragload for test N1 and test N2

Fig. 9. The effect of applied load on the shifting of neutral plane (Test A1)

Fig. 10. Ultimate pile capacity determined by Chin’s plot

doped at the upper portion of the pile shaft is similar for the two cases. However, the neutral plane for test N1 is located at around 90% depth of the clay stratum with a positive shaft friction below the plane. On the contrary, dragload develops along the full length of the end-bearing pile (test N2) with the neutral plane located at the pile tip. This finding is consistent with that reported by Lee et al. (2002) who carried out a numerical finite element analysis on friction and end-bearing piles subject to negative skin friction.

The effect of pile tip condition is further examined by plotting the development of maximum induced dragload on the pile and pile head settlement with time as shown in Fig. 8. As the pile tip of the end-bearing pile in test N2 is rested on a hard stratum, the pile settles considerably less as compared with the friction pile in test N1. However, the induced dragload is far more significant for test N2 as the elevation of its neutral plane is at the pile base and significantly lower than the neutral plane elevation of test N1.

PILE UNDER SIMULTANEOUS APPLIED LOAD AND NEGATIVE SKIN FRICTION

Piles are installed to support superstructure loads in the field and they are normally subject to axial loads and negative skin friction concurrently. The mechanism of negative skin friction on an axially loaded single pile is examined in this section. In test A1, the soil condition and the test procedure is the same as test N1 except that axial load is applied after the self-weight soil re-consolidation but prior to the application of surcharge. The test results reveal that the development of ground settlement and excess pore pressure during the self-weight soil re-consolidation is essentially the same as those observed in test N1. The leftmost curve in Fig. 9 shows that at the end of self-weight re-consolidation in test A1, the dragload is about 780 kN. This value is very close to the 788 kN recorded for test N1 shown in Fig. 4. After the clay has completely re-consolidated, an axial load of 4750 kN has gradually been applied to the pile.

To determine the ultimate pile capacity, the observed pile load-settlement data are re-plotted using Chin’s (1970) hyperbolic method and the results are shown in Fig. 10. The pile capacity hence estimated is 9600 kN and the applied load of 4750 kN is thus about 50% of the ultimate pile capacity. Figure 9 reveals that as the axial load increases, the ‘locked-in’ dragload of 780 kN has gradually been overcome and the neutral plane gradually shifts upwards along the pile shaft. When the axial load reaches 2400 kN, which is about 3 times that of the dragload developed on the pile prior to the application of axial load, the ‘locked-in’ dragload is completely overcome. Upon further loading, the pile shaft moves further downwards relative to the surrounding soil and the shaft resistance acts upwards relative to the pile and thus the
load transfer at the pile-soil interface becomes positive. This observation suggests that as long as the soil consolidation settlement has been completed, it may not be necessary to consider negative skin friction in pile design if the applied load is considerably larger than the dragload. Similar postulation has been made by Kog (1990).

After the relatively small magnitude of excess pore pressure generated during axial loading has fully dissipated, sand surcharge is placed in-flight to induce consolidation settlement of the clay. The right hand curves in Fig. 9 show that upon application of surcharge, dragload again develops along the upper pile shaft and the dragload increases with time as the clay consolidates. The neutral plane is observed to shift gradually downwards along the pile and finally stabilizes at about 12 m below the original ground surface. This observation is contrary to that observed for the case of pile subject to negative skin friction only in which the elevation of the neutral plane rapidly reaches its final position of around 90% depth of clay layer upon a relatively small magnitude of soil consolidation settlement.

The neutral plane is located at an elevation where pile settlement equals to soil settlement. The shifting of the neutral plane reflects the relative movement between the pile and the surrounding soil. The soil settlement at the surface is monitored regularly by two displacement transducers throughout the surcharge stage. As the soil settlement within the clay stratum could not be measured directly in the present experimental setup, Terzaghi’s (1943) one-dimensional consolidation theory is employed to estimate the soil settlement profile in the clay stratum. The clay stratum is divided into 8 layers each of 2 m thick and the settlement of each layer is calculated using a compression index of 0.553 obtained for the soft clay. The estimated soil settlement thus calculated for each clay layer is plotted as a percentage of soil surface settlement as shown in Fig. 11. A polynomial equation that fits the settlement profile is hence derived by regression and given in Fig. 11.

The concave shape of the settlement profile is consistent to the observed settlement profiles under an embankment loading reported in the field studies by Clemente (1984) and Indraratna et al. (1992). Indraratna et al. further showed that the concave shape of the settlement profiles remain essentially unchanged throughout the soil consolidation process. Based on the field observations, it is assumed that the ratio of soil settlement at any depth over total surface settlement remains approximately the same throughout the soil consolidation process. The soil settlement profile in the clay stratum at any time can hence be estimated based on the measured ground surface settlement.

Prior to the application of surcharge, the pile settles by 87 mm under the applied axial load. After the placement of surcharge, the clay settles as the soil consolidates and drags the pile down. The pile settlement profile along the pile shaft can be deduced by subtracting the pile elastic compression from the measured pile head settlement.

![Fig. 11. Calculated soil settlement profile based on Terzaghi’s 1-D consolidation theory](image)

![Fig. 12. Comparison of soil settlement profiles and pile settlement profiles after surcharge loading](image)

Figure 12 shows the soil and pile settlement profiles at various times after the placement of surcharge. The results clearly show that the relative movement between the pile and the soil, and the neutral plane is observed to shift downward and essentially stabilized at about 12 m below the top of the clay. This observation is consistent with the neutral plane elevation revealed in Fig. 9.

**Effects of Magnitude of Axial Load**

Piles may be subjected to various magnitudes of axial loads in the field and the effects of load magnitude on the development of negative skin friction along a pile are
examined here. Tests A2 and A3 were conducted following the same test procedure and soil condition as test A1, except that the axial loads on the pile in tests A2 and A3 were 3350 kN (28% of pile ultimate capacity) and 8000 kN (67% of pile ultimate pile capacity), respectively.

The net dragload developed along the pile shaft due to clay consolidation under surcharge loading can be determined as the difference in pile axial forces before and after surcharge loading. The results thus obtained are shown in Fig. 13 which reveals that the distribution and magnitude of the net dragload at the end of the 3 tests are fairly close. This is not unexpected as the large clay settlement under surcharge loading shown in Fig. 12 would definitely exceed the relatively small pile settlement and thus drags the pile down again. The location of the maximum dragload was also observed to be similar for all the three cases.

The above results lead to the following practical implications on the design of piles subject to simultaneous axial load and negative skin friction. If a pile is also subjected to concurrent soil settlement around its shaft, Fig. 9 reveals that dragload would develop along the pile and needs to be taken into consideration in pile design. Figure 13 suggests that the process of load transfer along the pile shaft due to applied load is independent to the development of dragload due to soil settlement. The former process depends on the magnitude of applied load and the latter depends on the magnitude of relative pile/soil movement. Thus the magnitude of maximum load on a pile under such condition would be the sum of applied load plus dragload at the neutral plane elevation (as those shown in Fig. 13) minus the load transfer due to applied load between the ground surface to the neutral plane elevation, as indicated in Fig. 9.

The development of additional soil surface and pile settlements due to surcharge loading with time for the 3 tests is shown in Fig. 14. Although the development of soil surface settlement with time for all the three tests is similar, the magnitudes of additional pile settlement in the tests are different, ranging from 27 mm in test A3 with the least applied load of 3350 kN to 68 mm in test A2 with the largest applied load of 8000 kN.

**Effects of Socket Length**

The earlier reported tests A1, A2 and A3 were all carried out on piles socketed 2.5 m into the underlying dense sand layer. To investigate the effect of socket length on the development of negative skin friction on a pile, test A4 was conducted on a pile with a longer socket length of 6 m into the sand layer. The test procedure, soil condition and the applied load magnitude for tests A1 and A4 were essentially identical. Figure 15 shows the development of dragload with time for the tests after surcharge placement. The maximum load experienced by the pile in test A4 is 5977 kN that is larger than the 5850 kN measured in test A1. The shift of neutral plane in the tests is also indicated in Fig. 15. The final elevation of the neutral plane at the end of test A4 is observed to be lower than that in test A1.

Under the maximum applied load of 4750 kN, the measured pile head settlement in test A4 was 45 mm which is smaller than the measured pile head settlement in test A1. This is attributed to the longer socket length in the dense sand layer in test A4 that provides stronger resistance against external loading. Figure 16 shows the development of additional soil and pile settlements in both tests. It can be seen that the soil settlement in both tests is essentially the same. The final magnitude of the additional pile settlement in test A4 is only 35 mm, as
sufficiently large soil settlement, the pile does not settle further and the dragload also reaches a maximum value. The magnitude of negative shaft friction developed is established to be close to the soil strength. For a pile subject to simultaneous negative skin friction and axial load, it is established that when there are no concurrent soil settlements around the pile, the 'lock-in' dragload is completely overcome when the applied load exceeds 3 times the dragload and negative skin friction need not be considered in the pile design. However, when there are concurrent soil settlements around the pile, the pile would be dragged down and dragload needs to be included in the pile design. It is established that the development of dragload and the load transfer due to applied load from the ground surface to the neutral plane elevation is essentially independent of each other. Thus the magnitude of maximum load on a pile under such condition would be the sum of applied load plus dragload at the neutral plane elevation minus the load transfer due to applied load from the ground surface to the neutral plane elevation. The pile socket length in the underlying dense sand also has an important effect on the pile behavior. Although a shorter pile socket length may result in a higher elevation of neutral plane thus a lower dragload developed on the pile, the induced pile settlement can be more severe as compared to a pile with a longer socket length.

ACKNOWLEDGEMENTS

This project is funded by NUS research grant R-264-000-021-112. The Authors would like to express their gratitude to the university for the funding and also to the laboratory officers in the Geotechnical Centrifuge Laboratory of the National University of Singapore for their valuable assistance in conducting the centrifuge tests for the present study.

REFERENCES


