NEGATIVE SKIN FRICTION FOR LONG PILES DRIVEN IN CLAY

LE FROTTEMENT NEGATIF POUR DE LONGS PIEUX ENFONCES DANS L'ARGILE

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SYNOPSIS The paper describes the load distribution in two instrumented pre-cast concrete piles which have been driven through 40 m of clay, 13 m of silt and 15 m of sand. The load is caused by negative skin friction due to reconsolidation of the remoulded clay around the piles after the driving. The soil consists at the test site of homogeneous normally consolidated clay with a water content of around 80 % and an undrained shear strength at the surface of about 2 tons/m^2 and 7 tons/m^2 at the depth of 40 m. Piezometers and settlement gauges were installed in the soil prior to the driving.
A new accurate pile-force gauge has been developed for this project which makes it possible to measure the load and the moment distribution in a pile after driving. Measurements showed that the load in the pile immediately after driving was roughly equal to the weight of the pile itself. During the five months period following the driving the load in the pile increased by about 30 tons at the bottom of the clay layer. This load increase corresponds to a negative skin friction which increases linearly from 0 at the ground surface to about 1.4 tons/m^2 at the bottom of the clay layer. The negative skin friction is equal to 17 % of the undrained shear strength or to 5 % of the average effective overburden pressure.

INTRODUCTION

Thick deposits of soft normally consolidated clays cover large areas in the middle and south western parts of Sweden. Building in these areas are generally supported by end bearing piles which are driven through a soft clay to moraine or rock. However, the general lowering of the ground water table which occurs in the central parts of the large cities causes settlements and an increase of the load in the end bearing piles due to negative skin friction. Great uncertainty exists about the magnitude of the negative skin friction with respect to the undrained shear strength of the soil or the effective overburden pressure, the relationship between settlements and negative skin friction and the effect of pile driving on the negative skin friction. An investigation was therefore initiated in 1966 in order to answer some of these questions.

A robust and accurate pile force gauge has been developed for this project. The pile force gauge was designed to resist the stress conditions during the driving of a pile. The conventional system to measure the negative skin with a series of steel rods has the disadvantage that only the changes which take place after the driving can be measured. Thus the stress conditions in the pile immediately after the driving are unknown. An additional uncertainty in the calculations is that the value of the modulus of elasticity for concrete is uncertain and may vary appreciably. These variations affect the accuracy of the results. Two instrumented reinforced concrete piles were driven in June 1968. The pore water pressures, the soil movements and the negative skin friction which develop during the driving and during a five month period after the driving are described in this paper.

TEST PROGRAM

The test program consists of three phases. The first phase concerns the distribution of negative skin friction and bending moments during and after the driving of two long unloaded piles which have been driven through a normally consolidated clay into silt and sand. In the second phase an axial load of 80 tons will be placed on the two piles and its effects on the negative skin friction of this load will be studied. In the third phase a 2 meter high gravel fill will be placed over an area 40 by 40 meters around the two test piles. This will cause settlements in the clay and thus negative skin friction along the two piles. It is anticipated that phase three of this investigation will be completed in 1973.

SITE CONDITIONS

The test site is located at the River Göta Älv approximately 20 km northeast of Gothenburg in the southwestern part of Sweden. The soil consists of 40 m of normally consolidated clay which is underlain by silt and sand. At 35 m depth the clay contains silt layers. The undrained shear strength, water content, liquid and plastic limits, fineness number and unit weight are shown in Fig. 1. The natural water content of the clay exceeded the liquid limit down to a depth of about 23 m. The undrained shear strength increased from about 1.5 tons/m^2 at the ground surface to about 5.0 tons/m^2 35 m below the ground surface. The ground water table is located at the ground surface. The percentage of clay size particles smaller than 0.002 mm in the clay is about 80 % down to a depth of 20 m. Between 20 and 30 m the percentage of clay size particles decreases to about 55 %. The sensitivity of the clay varies between 15 and 20.

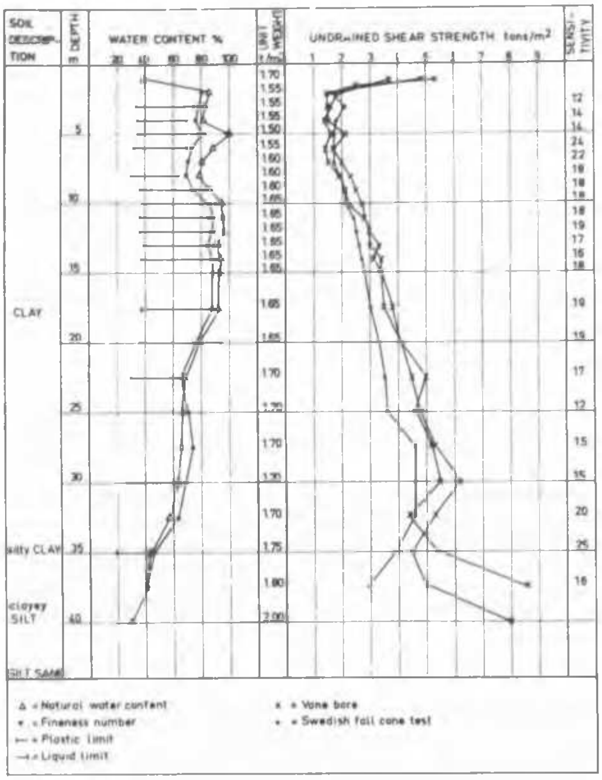

Fig. 1 Soil description

PILE TYPE AND DRIVING DATA

Two precast hexagonal Herkules piles of reinforced concrete with a cross sectional area of 800 cm^2 and a circumference of 106 cm were used for the experiments. Each pile was composed of 11.2 m long segments. The bottom segment was provided with a rock point of hardened steel. The piles were also provided with a center pipe, a thin wall steel pipe with 42 mm diameter inside diameter, in which deformation gauges were inserted after the driving. Also special cable pipes (Ø8 mm) were placed in the piles for the electrical cables leading from the pile force gauges to the head of the pile.

The nominal concrete cube strength was 500 kg/cm^2. The average measured cube strength was 607 kg/cm^2 28 days after the casting. The reinforcement consisted of six bars with 16 mm diameter and a yield strength of 60 kg/mm^2. The failure bending moment of the pile section exceeded 8.5 tonm. The segments were fastened together in the field by rigid steel joints as the driving proceeded. The strength of the joints exceeded that of the pile segment.

Test pile PI was composed of five pile segments and three pile force gauges and test pile PII of six segments and four gauges. The length of the upper segment of pile PII was 2.0 m. In this pile the lowest gauge was placed right at the pile tip as illustrated in Fig. 2. The location of the pile force gauges are shown in Fig. 3.

Fig. 2 Photo of pile-force gauge placed at the pile tip. The pile tip is provided with a rock point of hardened steel

The piles were driven with a 4.2 tons drop hammer. The height of fall was 0.3 m when the piles were driven down to a depth of 40 m. Below this depth the height of fall was increased to 0.5 m. The total number of blows required for the driving was about 5000 for pile PI and 4000 for pile PII. The driving of the first pile (PI) was terminated at a depth of 53.1 m when the penetration resistance of the pile was 8 cm per 50 blows. The second pile (PII) had to be driven to a depth of 55.1 m to reach the same final penetration resistance as pile PI (The penetration resistance at 53 m depth was low and therefore an additional 2 m long pile segment had to be added). The driving data indicated that the piles acted as combined friction and end bearing piles.

Pile PI was relatively straight after the driving. Inclinometer measurements (Kallstenius and Bergau, 1961) indicated that the pile tip deviated laterally 1.4 m from its intended position. The minimum radius of curvature of the displaced pile was 340 m. Pile PII was not as straight as pile PI. The pile tip after driving had been displaced 6.2 m away from its intended location. The minimum radius of the pile axis close to the pile point was 170 m. Laboratory tests have indicated that failure by bending will occur at a radius of 50 to 100 m.

INSTRUMENTATION OF PILES

The pile-force gauge used in this project was developed by the A. Johnson Institute for Industrial Research. The gauge is composed of three load cells which are placed between two rigid steel plates.

The load in each cell is measured separately by a system of vibrating wire gauges. This makes it possible to determine the total axial force and bending moment in the test pile at the level of the force gauge. The 0.4 m long force gauges were connected to a pile in the same manner as the pile segments. The pile force gauge was designed to resist the stress conditions which develop during the driving. Laboratory and field tests have indicated that the maximum error in the recorded forces is less than 2 % of the design load. The gauges were designed to resist a tension load of 50 tons and a compression load of 150 tons. These values can, however, be exceeded three times without impairing the function of the gauges. (Fellenius and Haagen, 1969.)

INSTRUMENTATION OF SOIL

Piezometers and settlement gauges were installed two months prior to the driving of the piles. All piezometers but one were type SGI which are provided with a closed oil system. Pore water pressures are read directly on a manometer (Kallstenius and Wallgren, 1956). To measure the pore pressure in the permeable bottom layers an open pipe with a filter tip was used. Each settlement gauge consisted of a number of 2 m sections of flexible steel spring reinforced rubber hoses with 32 mm inside diameter. The steel spring reinforcement allowed the hose to change its length axially but prevented the hose from collapsing when subjected to lateral earth pressure. The hose sections were connected by brass rings. The settlement gauges were placed vertically in pre-drilled holes in the soil. The flexible settlement gauges and the brass rings followed the movements of the soil. It was possible to determine the settlements of the soil from the location of the brass rings with respect to a reference point at the ground surface by lowering a plumb bob inside the hose. When the plumb bob came in contact with the brass rings an electrical circuit was closed which could be observed at the ground surface. With this method it was possible to determine the settlements every 2.0 m with an accuracy of ± 2 mm (Wager, 1969).

Two settlement gauges were installed next to each pile. An additional gauge was installed at some distance away from the pile. The gauges were brought down to a depth of 36 m. The location of the gauges are shown in Fig. 3. The various gauges could not be installed absolutely vertically. However, the deviations were small down to a depth of 10 m.

Three piezometers were installed next to the predetermined pile locations at the depths 9.0, 22.3 and 30.5 m below the ground surface. One additional piezometer was installed at a depth of 28.6 m some distance away from the two pile. At about the same distance from the piles an open pipe with a filter tip was installed at a depth of 45.0 m.

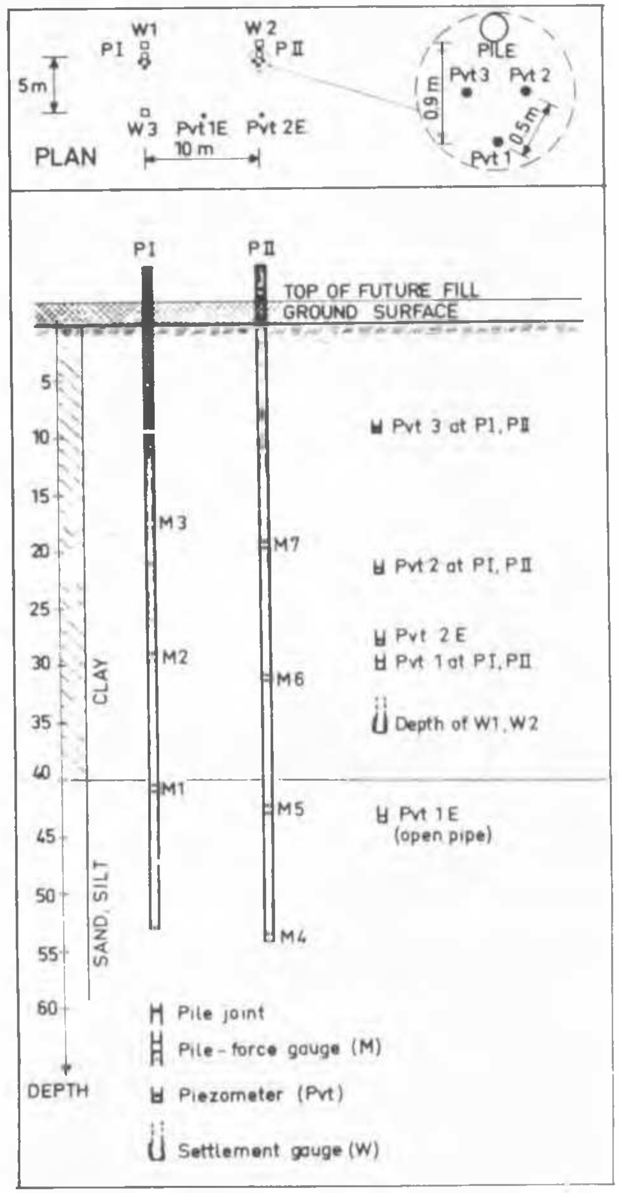

Fig. 3 Location of piles and instrumentation

BEHAVIOUR DURING AND IMMEDIATELY AFTER DRIVING

The driving of the two test piles caused movements in the soil. These were measured close to the two piles and at some distance away from the piles. Also high excess pore pressures were measured.

Settlements. The driving caused the ground surface to heave 20 mm close to the pile as shown in Fig. 4. The heave decreased, however, with increasing depth. Settlements were measured below a depth of 5 to 6 m. The maximum settlement (50 mm) was measured close to pile PII at a depth of 11 m. The observed heave was caused by upward displacements of the soil above the pile tip and the observed settlements probably by downwards displacements at and below the pile tip.

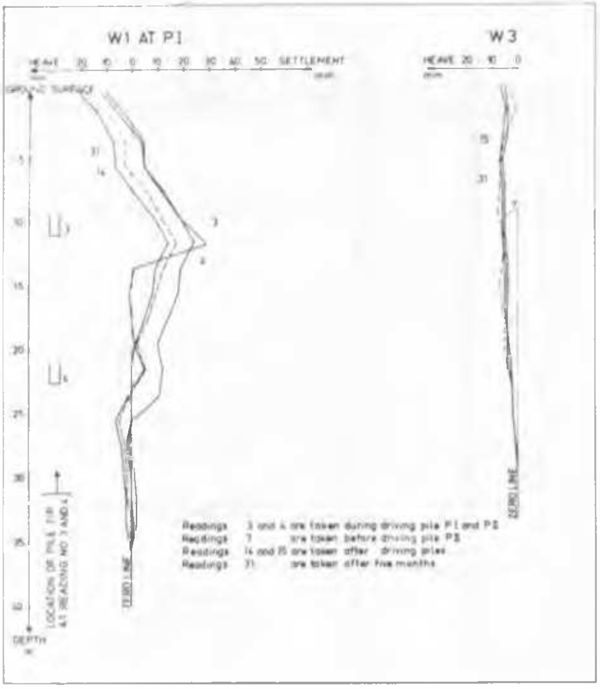

Fig. 4 Vertical movements in the clay during and after driving

The continued driving caused the soil to heave about 10 to 15 mm. At gauge WIII located 5 and 11 m away from the test piles the driving caused a heave of 5 mm at the ground surface. The heave decreased, however, with depth. The displacements of the soil shown in Fig. 4 have been evaluated from the assumption that the soil at the lowest measuring point did not move. The reported values thus represent relative movements within the clay layer. A precision levelling before and after the driving indicated, however, that the lowest measured point of gauge WI had settled 9 mm. The corresponding settlements of gauges WII and WIII were 7 mm and 5 mm, respectively. These settlements are primarily caused by compaction of the silt and sand layers below the clay. The two test piles were driven 13 and 15 m into the bottom layers.

Pore Water Pressure. The pore water pressures measured by all piezometers corresponded prior to the driving of the two test piles to a ground water table at the ground. The driving caused, however, a large increase of the pore water pressure at the gauges located at a depth of 20.3 m below the ground surface. The pore pressure increase at the gauges located at 30.5 m depth was small since these gauges were located several meters away from the two piles. Also the pore pressure increase observed at a depth of 9.0 m was small. All piezometers have afterwards been checked and are functioning properly. The excess pore pressure caused by the pile driving and the dissipation of the excess pore pressures with time are shown in Fig. 5 for the two gauges located close to piles PI and PII at a depth of 20.3 m. The maximum total pore pressure was 40 tons/m^2 at this level. The corresponding total vertical overburden pressure is 32.9 tons/m^2 at the same level. Thus the measured pore pressure exceeded locally the total overburden pressure by 20 %.

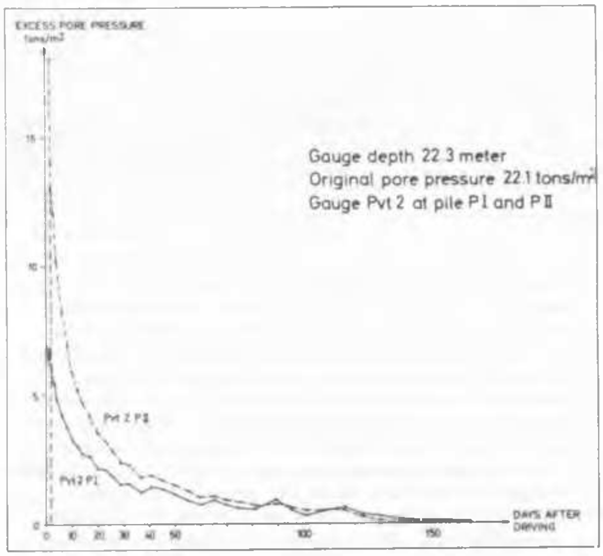

Fig. 5 Excess pore pressure caused by pile driving and its dissipation with time

Forces and bending moments. The force gauges in the piles were read each time a new pile segment was added.

Measurements indicate that the force in the two piles immediately after the driving was roughly equal to or slightly less than the weight of the pile above the gauge. Thus the driving did not cause any axial forces to be "locked" into the piles.

The bending moments in the straight pile PI were small. These varied between 0.4 and 1.3 tonm. Larger values were measured in the bent pile PII. Gauge M5 located 12 m from the pile tip at the boundary between the clay and the underlaying silt and sand indicated a bending moment of 3.2 tonm. This bending moment corresponds to about 35 % of the failure value. The corresponding radius of curvature was 170 m over the length of the gauge.

Gauges M6 and M7 indicated a bending moment of 0.9 and 2.4 tonm, respectively. The corresponding radii were 220 and 190 meters.

BEHAVIOUR AFTER DRIVING

The driving disturbed the clay around the piles. It was anticipated that reconsolidation of the clay would cause settlements of the soil and drag forces in the piles. To study this phenomenon the various instruments were read regularly during the five months period which followed the driving.

Settlements. The settlement gauges indicated that the movements of the soil were small. The recorded settlements varied between one and three millimeters.

Pore water pressure. High excess pore water pressures developed around the driven pile in the clay as indicated by the gauges located at a depth of 20.3 m. The excess pore water pressures dissipated with time. No excess pore water pressures remained 150 days after the driving as can be seen in Fig. 5.

Forces and bending moments. The axial force distribution with time is shown in Fig. 6. At first the axial load in the two test piles increased rapidly at the different measuring levels. Two or three weeks after the driving the rate of the load increase slowed down. After about eight weeks the load increase was very small at the upper levels.

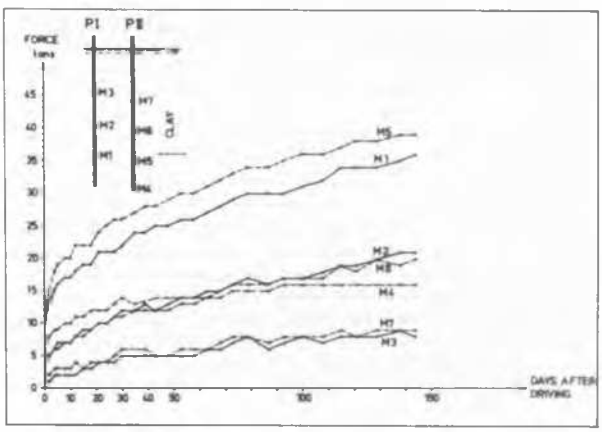

Fig. 6 Recorded forces in piles PI and PII

In Fig. 7 the measured loads have been plotted at different time intervals after the driving. The dotted line in this figure represents the weight of the pile. The line marked 0 represents the load distribution immediately after the driving. It can be seen from Fig. 7 that the axial load in both piles was less than the weight of the pile immediately after the driving down to a depth of nearly 40 m. The load in the piles increased with time and 144 days after the driving, the pile load exceeded the weight of the pile by 25 to 30 tons at the level of the interface between the clay layers and the underlaying silt and clay layer. This load is still increasing five months after the driving. Gauge M 4 in pile PII indicates that most of the load due to negative skin friction along the pile is resisted as positive skin friction in the underlaying silt and sand layers. The average skin friction can be calculated from the load distributions. It is proportional to the load difference between adjacent levels. In Fig. 8 is shown the average skin friction resistance for the two piles. The points which on the same occasion represent the average skin friction have been connected. The measurements indicate that the skin friction resistance increases approximately linearly from zero at the ground surface to 1.4 t/m^2 at a depth of 40 m. The skin friction resistance below this depth was positive. The negative skin friction corresponds to 17 % of the undrained shear strength of the clay or to 5 % of the effective overburden pressure of the soil.

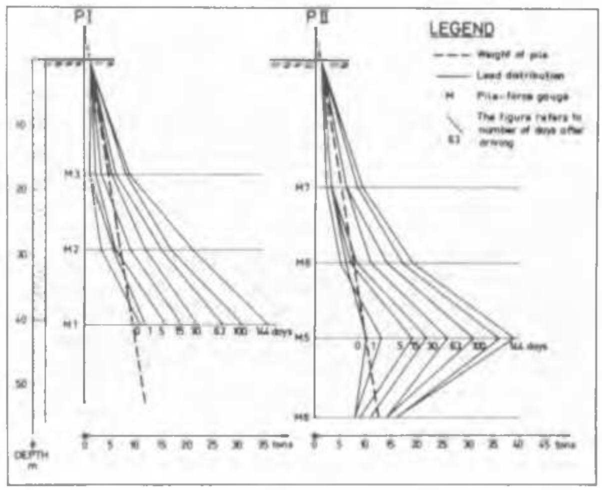

Fig 7 Vertical distribution of load in the piles at different times after driving

The recorded bending moments increased after the driving. However the increase was small. The bending moment at gauge M5 which immediately after the driving recorded 3.2 tonm increased by 12 % to 3.6 tonm five months after the driving. This relative increase was the largest recorded by any of the gauges.

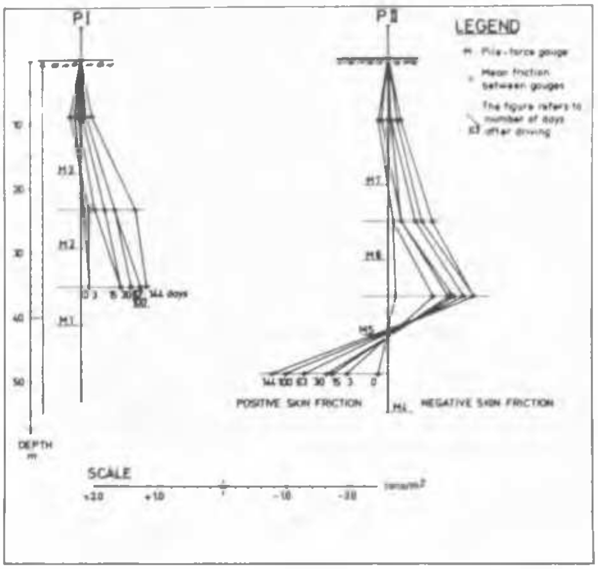

Fig. 8 Calculated average skin friction at different times after driving

CONCLUSIONS

The test results indicate that negative skin friction can be caused by the remoulding of the clay around driven piles and the subsequent reconsolidation of the soil even if the settlements of the soil are very small. The measured negative skin friction after a period of five months corresponded to 17 % of the average undrained shear strength of the clay or to 5 % of the average effective overburden pressure. The resulting axial forces in the piles were resisted by positive friction in the silt and sand layers at the lower parts of the piles. Considerably higher values of the skin friction resistance will undoubtly develop when a fill is placed over the area.

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