SWEDISH GEOTECHNICAL INSTITUTE

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No. 25

NEGATIVE SKIN FRICTION ON LONG PILES DRIVEN IN CLAY

I. Results of a Full Scale Investigation on Instrumented Piles

II. General Views and Design Recommendations

By
Bengt H. Fellenius

STOCKHOLM 1971

Also included in IVA, Pålkommissionen, Meddelande No. 18

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PREFACE

Negative skin friction on piles has been known as a problem since a long time. However, it was not until the beginning of the 1960's that any particular interest was taken in this problem. Then, independently, research on negative skin friction started in various parts of the world.

In 1965 Mr J.C. Brodeur, Vice President of A. Johnson & Co (Canada) Ltd, requested to the parent company in Sweden, Nya Asfalt AB of the Johnson Group, to investigate negative skin friction for long precast piles. Mr S. Severinsson, Manager of Nya Asfalt AB for the western region of Sweden, supported this request. It was agreed that research on negative skin friction should be carried out by the Axel Johnson Institute for Industrial Research and by the Swedish Geotechnical Institute in cooperation.

A research group was formed which outlined a program for a full scale field test. Apart from representatives from the Swedish Geotechnical Institute and the Axel Johnson Institute also Mr Brodeur and Mr Severinsson participated in this work together with Mr L. Hellman of the Commission on Pile Research of the Royal Swedish Academy of Engineering Sciences. The project was financially supported by the National Swedish Council for Building Research.

The work initiated with a literature review and the development of a suitable pile force gauge. This part of the work has been published in the Reprint and Preliminary Report series of the Swedish Geotechnical Institute. The results of the full scale field test, which started in 1968, up to the end of 1970 is published in the present report.

The investigations have not yet been completed and the intention is to publish additional results when they become available.

Stockholm, November 1971
SWEDISH GEOTECHNICAL INSTITUTE

CONTENTS

SUMMARY

I. This part of the report describes the results of 28 months of measurements of negative skin friction on two test piles.

The soil at the test site consists of 40 m of uniform normally consolidated clay through which the test piles were driven into underlying silt and sand. The piles were instrumented with a new accurate pile-force gauge which made it possible to measure the axial loads and bending moments in the piles. Settlement gauges and piezometers were installed in the test field prior to the driving of the piles.

The test program consisted of three phases of study.

Phase 1	Influence of the driving and of the following reconsolidation of the clay.
Phase 2	Influence of a load applied on the head of the piles.
Phase 3	Influence of a surcharge fill at the test site, causing settlements in the clay around the piles.

The results which were obtained from Phase 1, after 495 days, and during the first 365 days of Phase 2 are described.

During the driving, the clay close to the piles was remoulded and displaced. The net displacement in the upper 5 m close to the piles was a heave. The ground heaved 10-15 mm close to the piles, while below 5 m depth the net displacement was a settlement. At 5-10 m distance from the piles only a small heave was observed.

The driving caused high excess pore pressures in the clay which locally exceeded the effective overburden pressure. The clay reconsolidated around the piles over a period of six months, before all excess pore pressures had dissipated. The reconsolidation and the resulting small settlements transferred load to the piles by negative skin friction. The settlements were about 2-3 mm and the drag load caused by negative skin friction was 40 tons which corresponded to approximately 25 % of the original undrained shear strength of the clay.

A small regional settlement of about two millimetres per year caused additional negative skin friction, which developed linearily with time and with depth. At the end of Phase 1, the total drag load was 55 tons.

When, in Phase 2, an axial load of 44 tons was applied on the pile heads, the negative skin friction along the upper portion of the piles was eliminated. The load in the piles at the bottom of the clay layer was only slightly affected by the applied load. However, the negative skin friction continued to develop at the same rate as in Phase 1 in addition to the previous loads in the piles.

At 860 days after the driving (one year after the loading of the piles) a total load of 80 tons was observed.

The measurements of the bending moments in the piles showed that the bending moments were not affected by the load increase due to negative skin friction. When the load increase was caused by a directly applied load on the pile heads, the bending moments in the piles increased.

The test results showed that negative skin friction can be caused by remoulding of the clay around driven piles and the subsequent reconsolidation of the soil. Also, very small settlements can cause significant negative skin friction along piles.

II. In this part of the report negative skin friction is discussed generally. It is shown that negative skin friction is a settlement problem and not a failure problem. The behaviour of a pile subjected to excessive drag loads and the differences between permanent and transient loads are discussed. Also the effects of drag loads on battered and bent piles are discussed.

General design formulas for piles considering negative skin friction are given. The formulas are intended to be used for checking that the permanent and transient working loads, which have been chosen according to ordinary design rules, are not too large when or if negative skin friction developes.

When settlements due to negative skin friction are not acceptable, the negative friction can be reduced by applying a thin coat of bitumen to the piles. References are made to investigations concerning reduction of skin friction, and practical difficulties are pointed out.

I. RESULTS OF A FULL SCALE INVESTIGATION ON INSTRUMENTED PILES

1. INTRODUCTION

Thick deposits of soft normally consolidated clays cover large areas in the middle and southwestern parts of Sweden. Buildings in these areas are generally supported on end-bearing piles which are driven through the soft clay to moraine or rock. The general lowering of the ground water table which occurs in the central parts of cities such as Stockholm, Gothenburg, Norrköping, Uppsala and Örebro causes settlements and thus an increase of the load in the end-bearing piles due to negative skin friction. Considerable uncertainty exists about the magnitude of the negative skin friction compared with, for instance, the undrained shear strength of the soil, the effective overburden pressure, the relationship between settlements and negative skin friction and the effect of pile driving on negative skin friction. An investigation was therefore initiated in 1966 to answer some of these questions.

The investigation started with a survey of the existing literature on negative skin friction (Fellenius, 1969 and 1970 b). The literature survey showed that although much had been published on negative skin friction very little was relevant to the above questions. It was therefore decided that a full scale field test had to be carried out. The immediate problem was then how to measure static forces in piles during long time periods. The conventional system to measure forces in piles with series of steel rods (tell-tales) has the disadvantage that only the changes which take place after driving can be measured. Thus the stress conditions in the pile during and immediately after driving are unknown. However, integrating the pile material into the measuring system, i.e. using the pile as a part of the force gauge, means that measurements can be obtained at relatively low cost. A study of the variations of the modulus of elasticity of concrete in driven elements of precast piles was therefore undertaken.

This study showed that not only is the value of the modulus uncertain and may vary appreciably, but the accuracy of the forces that are calculated by using the elastic modulus and the measured deformations will also, during long term measurements be affected by swelling and creep of the pile material (Fellenius & Eriksson, 1969). Thus it would not be possible to determine the forces to the required accuracy by using a tell-tale system, and it was decided that a separate pile force gauge must be used.

A survey of the market showed that there did not exist a gauge which could be placed in a pile prior to driving, which would allow subsequent measurement of forces in the pile with sufficient accuracy. The development of a suitable gauge was therefore undertaken, and by 1968 a robust and accurate pile force gauge was available which could resist the stress conditions during the driving of a pile and which immediately after driving could measure the axial forces and bending moments in the pile. The gauge is briefly described in Chapter 2.4, details are found in Fellenius & Haagen (1968 and 1969).

In June 1968, after two years of initial planning and development work two test piles were driven in a selected field in south-western Sweden and in this report the results from more than two years (860 days) of measurements are presented and discussed. Some results from the first 5 months of Phase 1 have previously been published (Fellenius & Broms, 1969).

2. GENERAL

2.1 Test program

Two long instrumented precast piles were driven through 40 m of normally consolidated clay into layers of silt and sand. The pore water pressures, soil movements and the loads and bending moments in the piles were measured.

The test program consists of three phases. The first phase concerns the effects due to the driving of the two piles. In the second phase an axial load of 80 tons is placed on each pile. The load is placed in two steps and the effect of each is studied. In the third phase a 2 m high gravel fill is to be placed over an area 40 x 50 metres around the two test piles. The fill is expected to cause considerable settlements in the clay and thus negative skin friction along the two test piles.

At present the second phase is under way. Of the 80 tons load 44 tons have been placed on top of each pile and the effect of this has been measured. The remaining 36 tons will soon be added on the piles and in 1971 the third phase will begin. It is expected that results including the third phase will be reported after about two years of measurements, i.e. 1973.

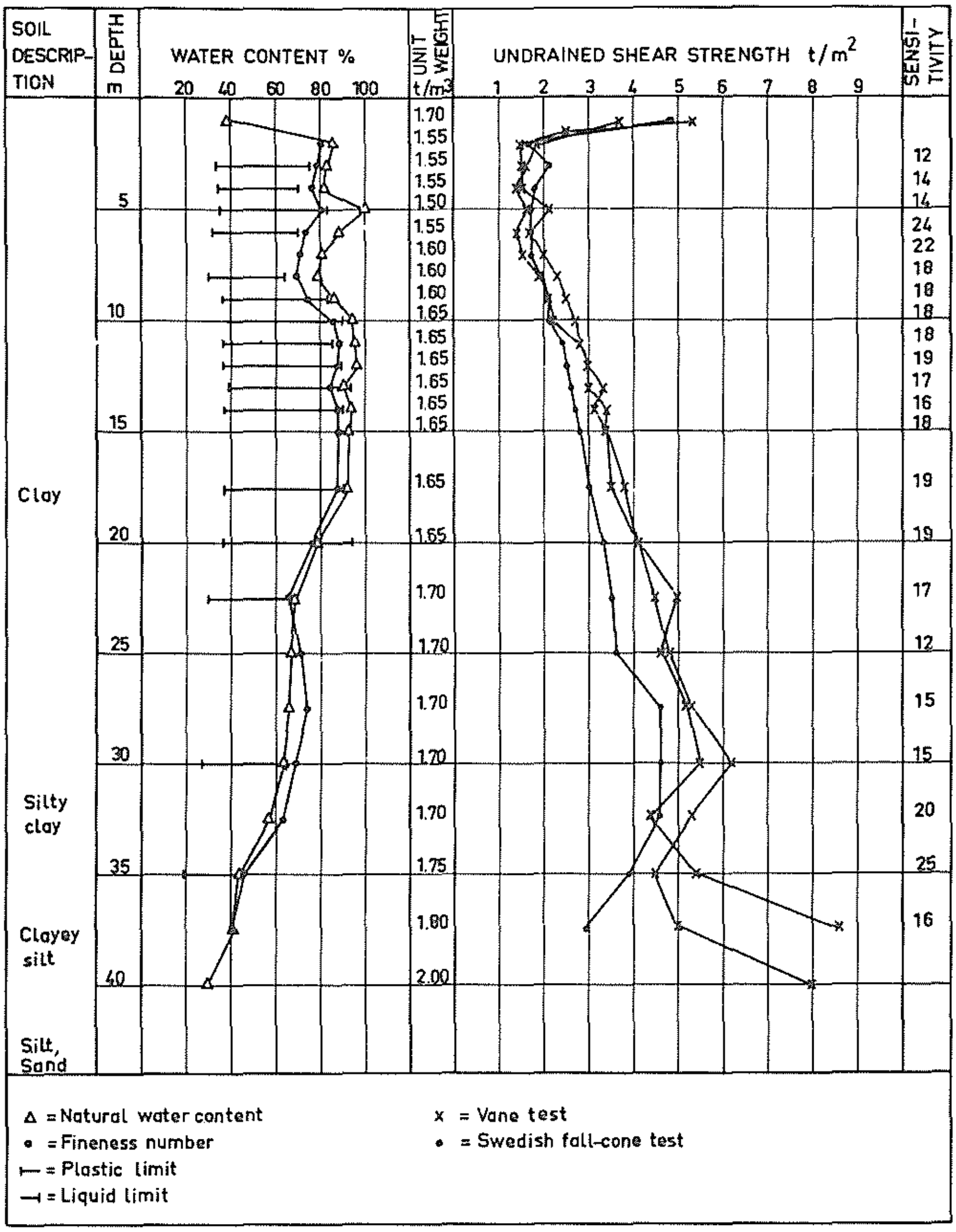


Fig. 1 Soil data

2.2 Site conditions

The test site is located in Bäckebol at the Göta River 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. Between 35 and 40 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 fineness number is determined from the Swedish fall-cone test and is normally approximately equal to the liquid limit.)

The percentage of particles smaller than 0.002 mm in the clay is about 80 % down to a depth of 20 m and decreases to about 55 % between 20 and 30 m. The sensitivity of the clay which varies between 15 and 20 is normal for Swedish clays. Oedometer tests show that the compression index (ε_2) of the clay is 10-15 % to about 30 m depth. Below this depth the compression index is about 8 %.

2.3 Pile type and driving data

Two precast hexagonal Herkules piles of reinforced concrete with a cross sectional area of 800 cm^2 (H 800) and a circumference of 105 cm are used for the investigation. Each pile is composed of 11.2 m long segments. The bottom segment is provided with a rock point of hardened steel. The piles are also provided with a center pipe, a smooth thin wall steel pipe with 42 mm inside diameter. Also special cable pipes (Ø 8 mm) were cast in the piles for the electrical cables leading from the pile-force gauges to the head of the pile.

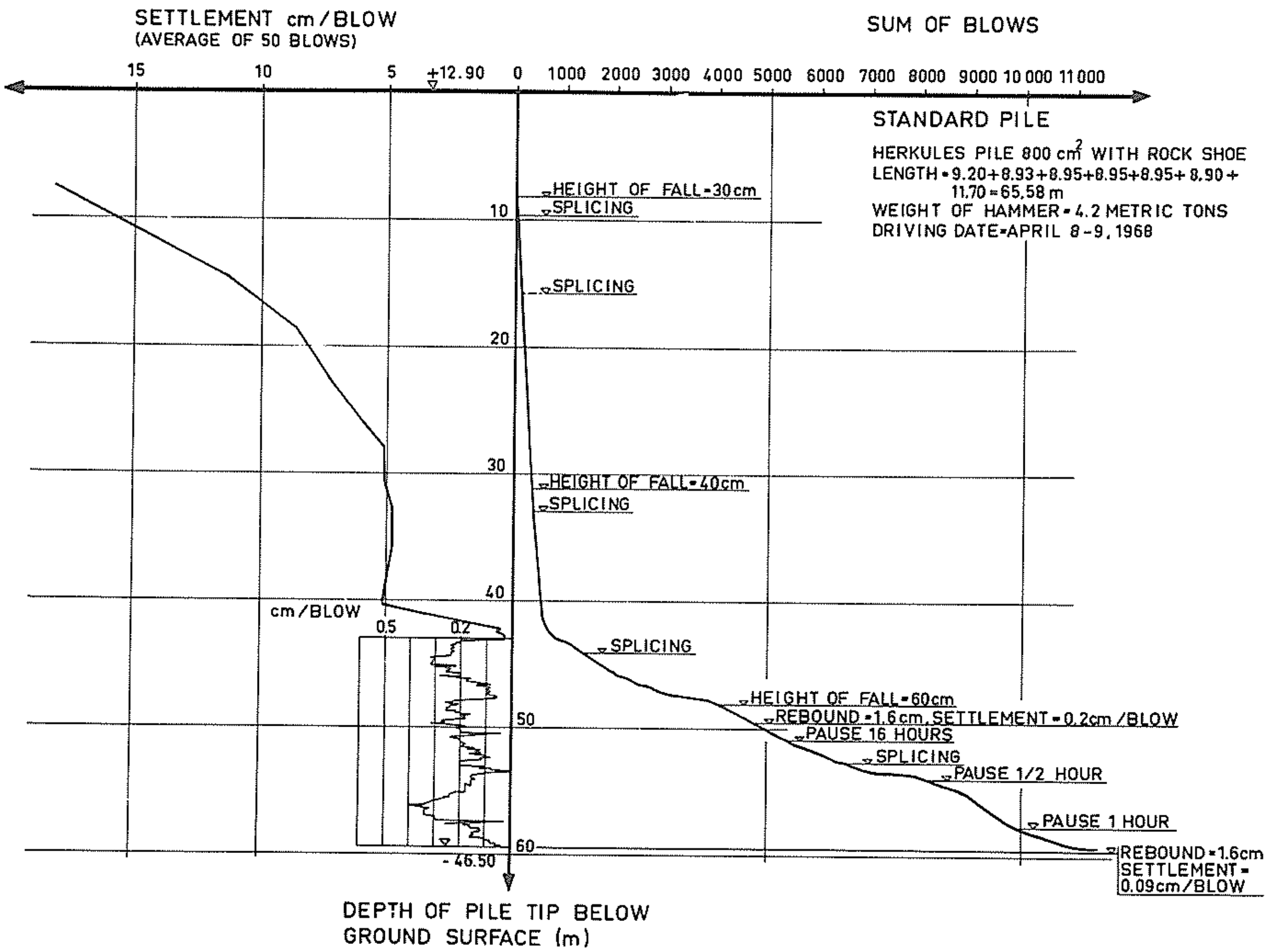


Fig. 2 Driving diagram from the standard pile

One standard Herkules pile H 800 was driven at the site two months prior to the driving of the test piles to investigate the driving conditions at the site.

The nominal concrete cube strength is 500 kg/cm^2. The average measured cube strength 28 days after casting was 607 kg/cm^2. The reinforcement consisted of six bars of 16 mm diameter with a yield strength of 60 kg/mm^2. The failure bending moment of the pile section exceeded 8.5 tonm. The pile segments were spliced in the field by rigid steel joints (Herkules system). (The strength of the joints exceeds that of the pile segment.)

Test pile P I is composed of five pile segments and three pile force gauges and test pile P II of six segments and four gauges.

The piles were driven by a 4.2 ton drop hammer. (See driving diagrams in Figs. 2, 3 and 4.) The height of fall was reduced to 0.3 m when the piles were driven throught the clay down to a depth of 40 m. The reduced height of fall was used in order to eliminate the risk for dynamic tensile forces in the pile during driving in accordance with the requirements of the Swedish Building Code of Pile Foundations (Statens Planverk, 1968). 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 P I and 4000 for pile P II. The driving of the first pile (P I) was terminated at a depth of 53.1 m when the penetration resistance of the pile was 8 cm per 50 blows. The driving of the second pile (P II) was terminated at a depth of 55.1 m at about the same final penetration resistance as pile P I (The penetration resistance at 53 m depth was con-

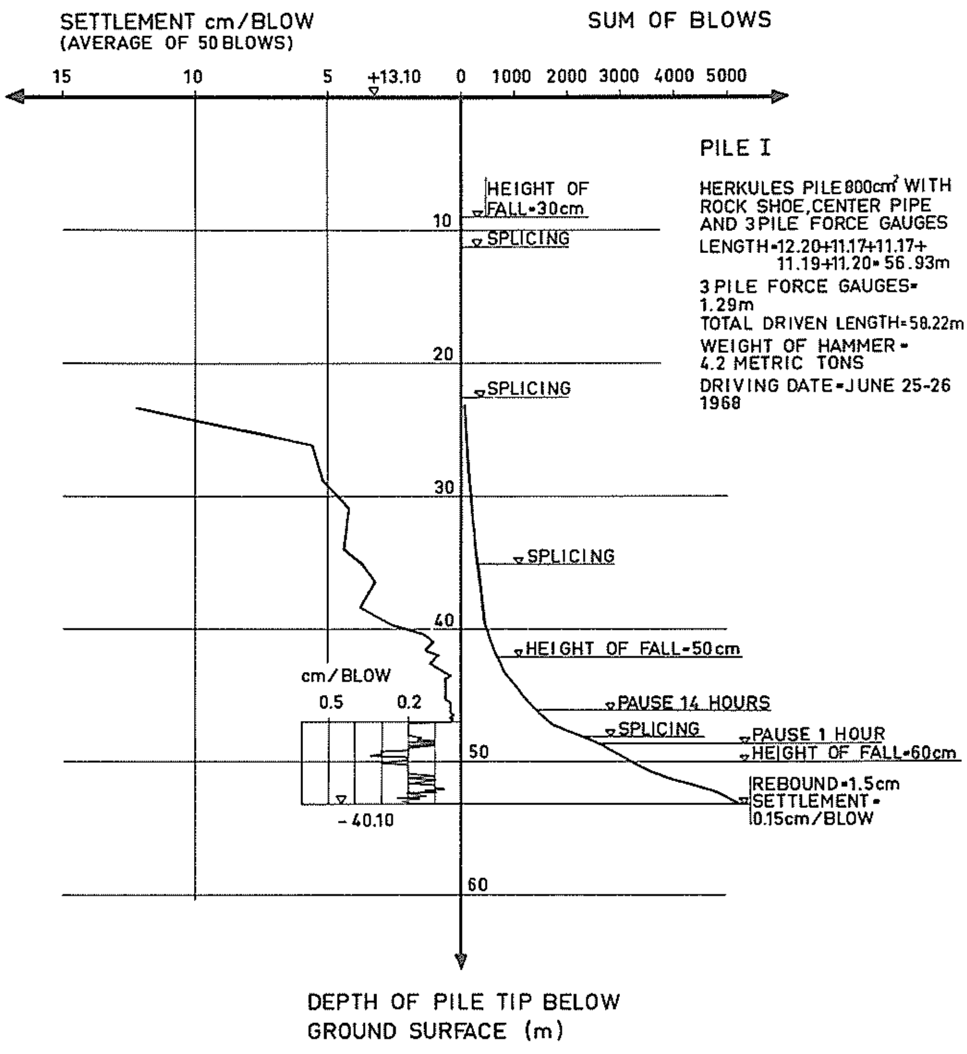


Fig. 3 Driving diagram from pile P I

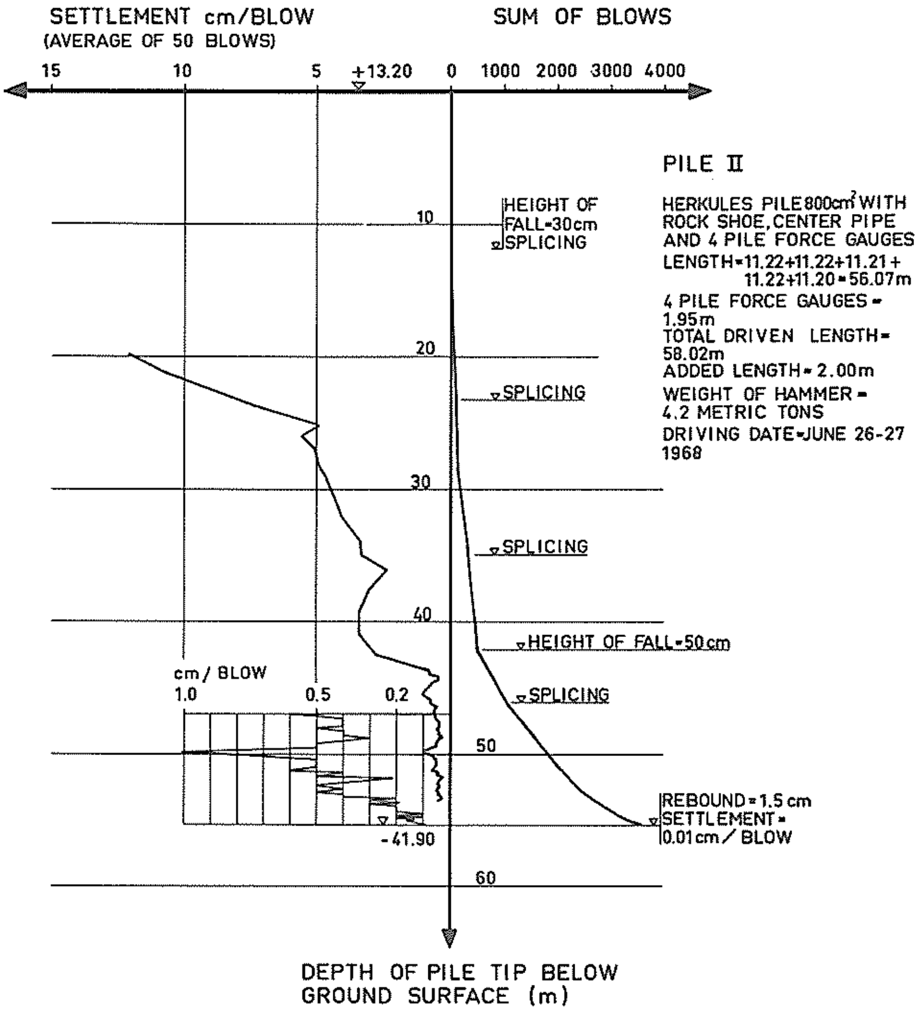


Fig. 4 Driving diagram from pile P II

sidered too low and therefore an additional 2 m long pile segment was added). The driving data indicate that the piles act as combined friction and end bearing piles.

Inclinometer measurements in the center pipes (Kallstenius & Bergau, 1961; Fellenius, 1971) after the driving indicated that pile P I is relatively straight (Figs. 5 and 6). The pile tip has deviated laterally 1.5 m from its intended position. Pile P II on the other hand is not as straight as pile P I. The pile tip is displaced 6.3 m away from its intended location. The minimum radius of the lowest pile segment is 170 m. (Laboratory tests have indicated that failure by bending will occur at a radius of about 50 m.)

Pile P I is only slightly more bent than is normal for piles in this area. The larger bending of pile P II is considered caused by the pile force gauge at the tip of the pile (cf. Fig. 9) and is not representative for piles that are not provided with pile force gauges. This has been confirmed in another investigation where the pile force gauge has been employed. An additional conclusion of this is that an investigation of the bending and the bearing capacity of a driven standard pile provided with pile force gauges should not be carried out on the same pile, unless the aim is to investigate the bearing capacity of bent piles. However, the bending of the two test piles would not normally cause rejection of the piles, i.e. as standard contract piles. (Fellenius, 1971).

2.4 Pile instrumentation

The pile-force gauge used in this project was developed at the Axel Johnson Institute for Industrial Research in Sweden. 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 axial forces, the bending moments and the direction of these moments in the test pile at the level of the force gauge. The 0.4 m long force gauges are connected to a pile in the same manner as the pile segments. In fact, they act as short pile segments.

The pile force gauge has been designed to resist the stress conditions which develop during the driving. Laboratory and field tests indicate that the maximum error in the recorded forces is less than 2 % of the design load. The gauges in the two test piles were designed to measure a tension load of 50 tons and a

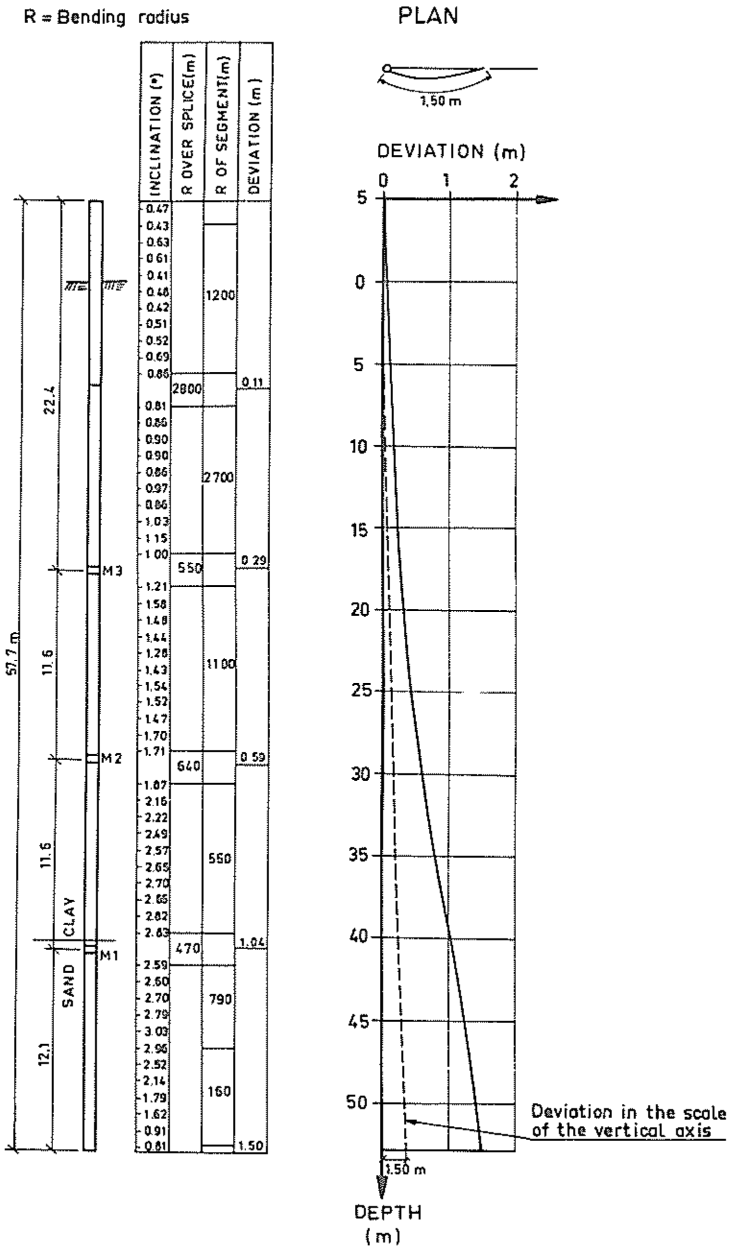


Fig. 5 Results of inclinometer measurements on pile P I

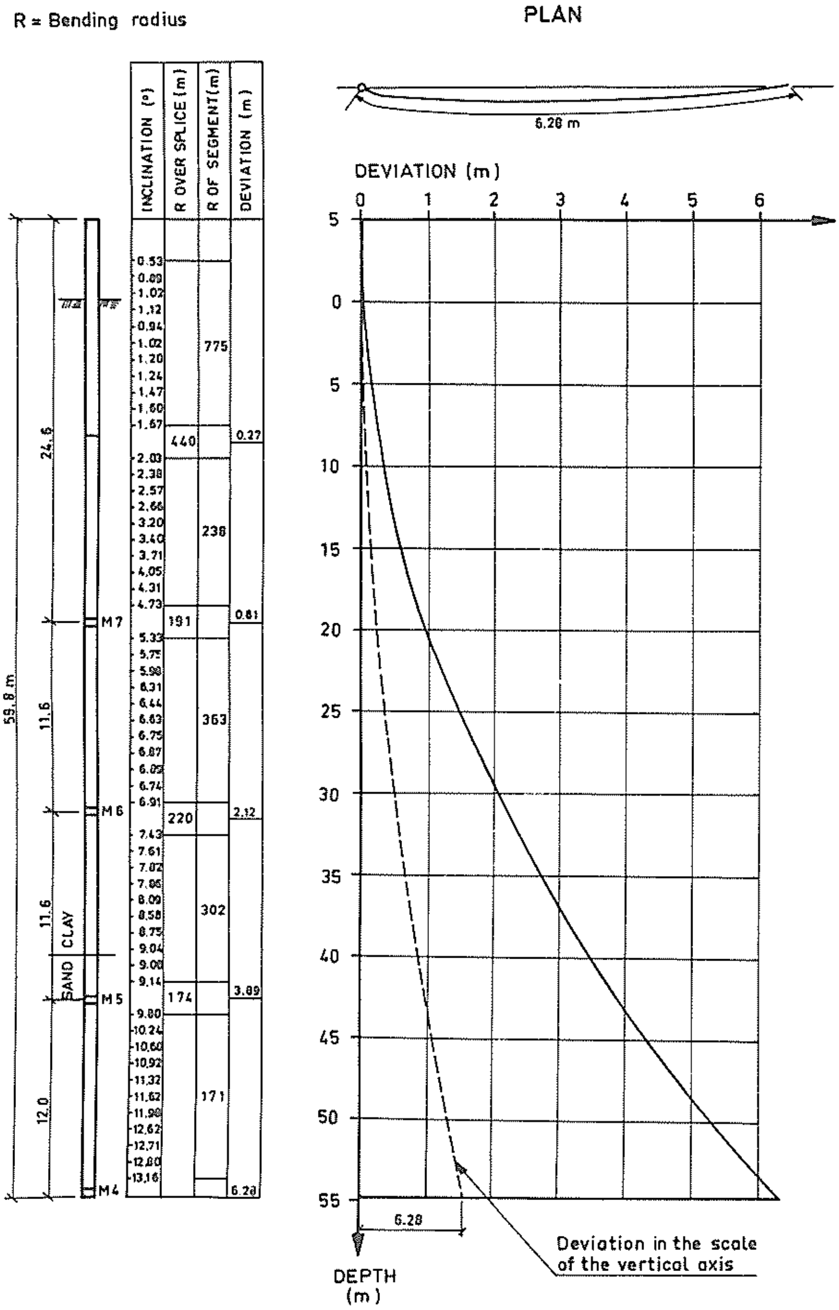


Fig. 6 Results of inclinometer measurements on pile P II

compression load of 150 tons. These values can, however, be exceeded by a factor of three without impairing the function of the gauges (Fellenius & Haagen, 1968 and 1969).

The location of the pile force gauges in the piles is shown in Figs. 7 and 8. The length of the upper segment of pile P II is 2.0 m. In this pile the lowest gauge is placed right at the pile points as illustrated in Fig. 9.

The center pipe was used for inclinometer measurements and for inserting two tell-tale rods into the pile for measurements of the deformations of the pile. The tell-tales consist of a steel rod which is placed at the pile tip and a steel pipe provided with a special expanding unit which is locked to the inside of the center pipe at 40 m depth (Broms & Hellman, 1968). The deformations of the full length of the piles and the deformation of the length of the piles located in the clay and in the sand are measured.

As might be anticipated, the tell-tale measurements were influenced by initial swelling of the pile material and temperature variations (cf. Chapter 3.4). It was also observed that during the winter ice formed in the center pipes, which bound the tell-tales together and with the center pipe. Due to these effects the long term measurements of deformation of the piles were very inaccurate and they have therefore not been included in this report, although references to a few of the observations will be made.

2.5 Soil instrumentation

Piezometers and settlement gauges were installed two months prior to the driving of the piles.

All piezometers but one are of type SGI which is provided with a closed oil system (Kallstenius & Wallgren, 1956). With the SGI gauge the pore water pressures are read directly on a manometer. To measure the pore pressure in the permeable bottom layers, an open pipe with a filter tip is used.

Prior to the pile driving, 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 at some distance away from the two piles. At about the same distance from the piles the open pipe was installed with the tip at a depth of 44.8 m (Figs. 7 and 8).

Each settlement gauge consists of a number of 2 m segments of flexible steel-spring reinforced rubber hoses with a 32 mm inside diameter. The steel spring reinforcement allows the hose to change its length axially but prevents the hose from collapsing when subjected to lateral earth pressure. The hose segments are connected by brass rings. The settlement gauges are placed vertically in pre-drilled holes in the soil and the flexible hose segments and the brass rings follow the movements of the soil. It is then 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 in combination with a measuring tape inside the hose. When the plumb bob comes in contact with a brass ring an electrical circuit is closed which can be observed at the ground surface. With this method it is possible to determine the settlements at every 2.0 m with an accuracy of ± 2 mm (Wager, 1971).

Two settlement gauges (S I and S II) were installed at a distance of 0.1 m away from each pile at the ground surface. An additional gauge (S III) was installed at some distance away from the piles. The gauges were

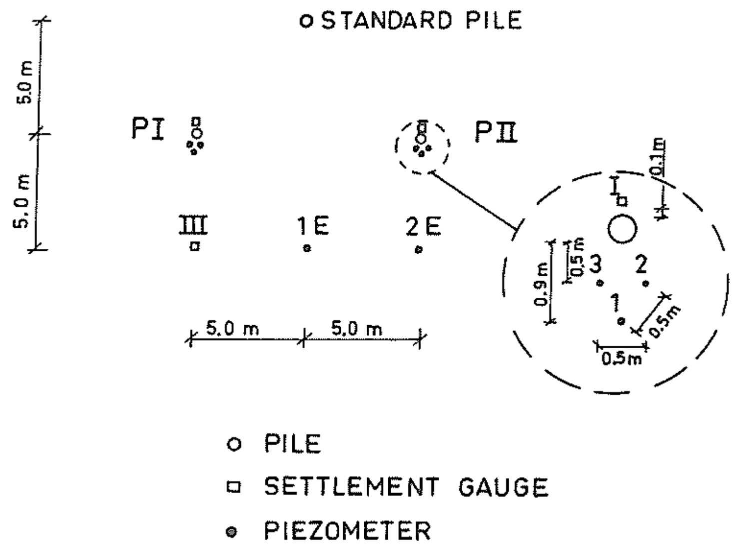


Fig. 7 Location of piles and instrumentation. Plan

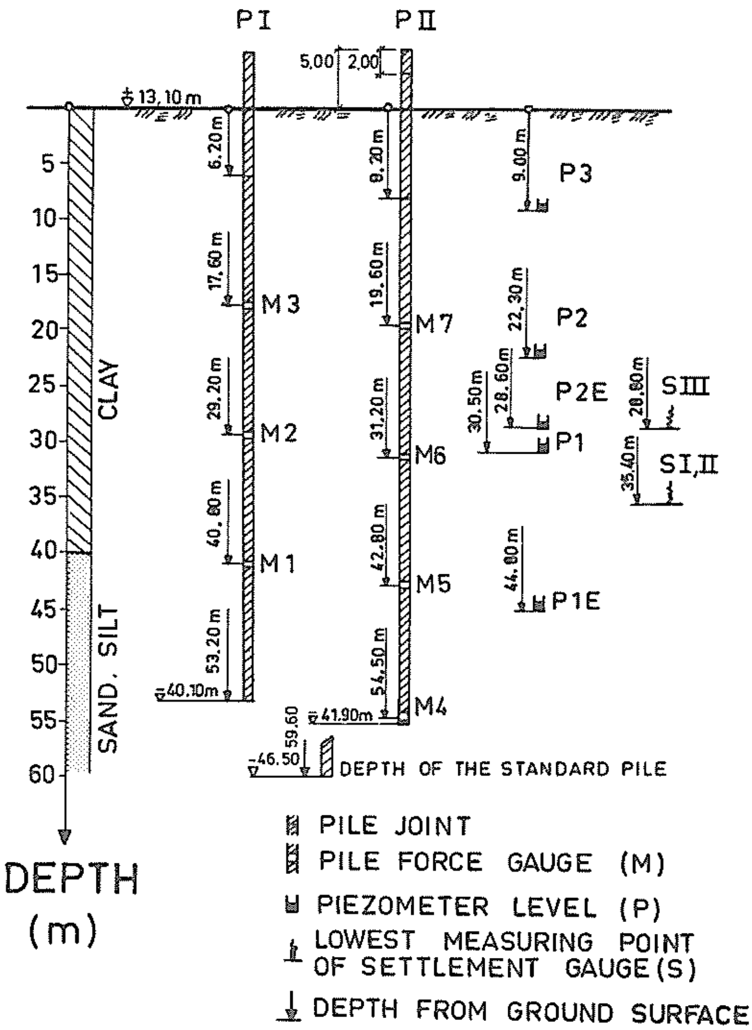


Fig. 8 Piles and instrumentation. Section

Fig. 9 Pile force gauge placed at the pile tip, provided with a rock point of hardened steel (rock shoe)

brought down to a depth of 35 m and 29 m, respectively. The location of the gauges are shown in Figs. 7 and 8. The various gauges could not be installed absolutely vertically. However, the deviations are small down to a depth of 10 m.

3. BEHAVIOUR DURING AND IMMEDIATELY AFTER DRIVING

3.1 General

The various gauges were read during the driving of the test piles in order to investigate the soil movements and the induced excess pore pressures in the clay due to the driving.

Immediately after driving, each pile was measured by a pile inclinometer and then the tell-tale system was installed in the piles.

3.2 Soil movements

The driving caused the ground surface to heave 20 mm close to the pile, as shown in Fig. 10. The heave decreased, however, with increasing depth and settlements were measured from a depth of 4 to 6 m below the ground surface. The maximum settlement (50 mm) was measured close to pile P II at a depth of 11 m. The observed heave was caused by upward displacement of the soil above the pile tip and the observed settlements by downwards displacement at and below the pile tip.

The measurements at Gauge III located 5 and 11 m away from the test piles shows that at this distance from the piles the soil heaved due to the driving. The heave was 5 mm at the ground surface and decrease with depth. The soil movements, which are shown in Fig. 10, were 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 measuring point of Gauge I had settled 9 mm. The corresponding settlements of Gauge II and III were 7 mm and 4 mm, respectively. These settlements were primarily caused by compaction of the silt and sand layers below the clay since the two test piles were driven 13 and 15 m respectively into these layers.

3.3 Pore water pressures

Prior to the driving of the two test piles, the pore water pressures measured by all piezometers corresponded to a ground water table at the ground surface. The driving caused an increase of the pore water pressure in the clay (cf. Fig. 12 a - g). Large excess pore pressures were measured. Particularly at the gauges located at a depth of 22.3 m below the ground surface. All piezometers have afterwards been checked and are functioning properly. The total pore pressure was 40 tons/m^2 at the level of the piezometers at 22.3 m depth (cf. Fig. 12 e). The corresponding total vertical overburden pressure is 32.9 tons/m^2 at the same depth. Thus the measured pore pressure induced by the pile driving locally exceeded the total overburden pressure by more than 20 %.

The pore pressure measured by Gauge 2 E located 5 and 11 m, respectively, away from the piles, showed no increase of pore pressure, i.e. no influence of the pile driving.

3.4 Forces and bending moments

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

The measurements showed that the force in the two piles immediately after the driving was roughly equal to or slightly less than the weight of the pile (in air) above the gauge. The weight of the pile is about 200 kg/m, which gives a total weight in air of about 10 tons. Thus the driving does not cause any significant axial forces to be "locked" into the piles (cf. Fig. 13).

As mentioned in Chapter 2.3 the bending moments in the straight pile P I are small (cf. Fig. 14). The bending moments varied between 0.4 and 1.3 tonm. Larger values were measured in the bent pile P II. Gauge M 5 located 12 m above the pile tip at the boundary between the clay and the underlying 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 is 174 m over the length of the gauge (Fig. 9). Gauges M 6 and M 7 indicated a bending moment of 0.9 and 2.4 tonm, respectively. The corresponding radii are 220 and 190 m.

3.5 Pile compression

As mentioned in Chapter 2.4 the deformations of the piles were measured by means of a tell-tale system. The deformations were measured over the full length of the piles, i.e. the full length of the center pipes and over the length of the piles located in the clay.

The tell-tales were installed immediately after the inclinometer measurements were taken, i.e. the day after the driving.

Initially, the measurements showed that the piles were extended longitudinally. The major portion of this strain took place during the first three days and no further change in strain was observed after about two weeks. The strain of Pile P I is 0.8 mm and of pile P II 1.6 mm and occurred mainly in the lower portion of the piles, i.e. in the permeable silt and sand layers.

The strain may be explained by suction of water into the piles and a following swelling of the pile material. This has been found in other tests, both in the laboratory (Fellenius & Eriksson, 1969) and the field (Fellenius, 1970 a). With free access to water the swelling is of the order of $1 \cdot 10^{-2}$ mm/m, which is obtained after a few days. In the present case the swelling was about 0.8 to 1.6 mm over 13 to 15 m length of pile or about $1 \cdot 10^{-2}$ mm/m, which indicates that the observed swelling of the lower portion of the pile is approximately the total amount that will be obtained. However, the swelling of the portion of the piles located in the clay is delayed due to the low permeability of the clay. Therefore, the deformations which are measured along the upper length of the piles include the effect of swelling even after the first two weeks.

The swelling of the pile is resisted by the soil, which results in a compressive force in the pile. This force can be estimated by using the elastic modulus of the pile, which is around 350.000 kp/cm^2.

$$P_{swell.} = E \cdot A \cdot \varepsilon = 350.000 \cdot 800 \cdot 10^{-5} \cdot 10^{-3} \approx 3 \text{ tons}$$

This load of 3 tons is the estimated maximum force due to swelling that can influence the readings of the pile force gauges and is considered negligible.

4. BEHAVIOUR AFTER DRIVING. PHASE 1. FIRST 495 DAYS

4.1 General

The driving disturbed the clay around the piles. It was anticipated that reconsolidation of the clay would cause settlements of the soil and subsequently drag forces on the piles. To study this phenomenon the various instruments were read regularly during the sixteen month period (495 days)following the driving.

4.2 Settlements

Fig. 10 shows the vertical distribution of the settlement within the clay layer during the driving and the settlement that occurred during the first 5 months after the driving. The settlement of the ground surface during the complete measuring period of 28 months (860 days) is shown in Fig. 11. The settlement of the ground surface is taken as the settlement of a point 2.0 m below the actual ground surface to eliminate the influence of frost action during the winter periods. Also shown in Fig. 11 is the relative movement within the clay. The measuring points Nos. 9 and 3 in Figs. 11 a and 11 b and Nos. 5 and 0 in Fig. 11 c correspond to the depths of the pile force gauges in pile P I.

The measurements indicated that during the first 400

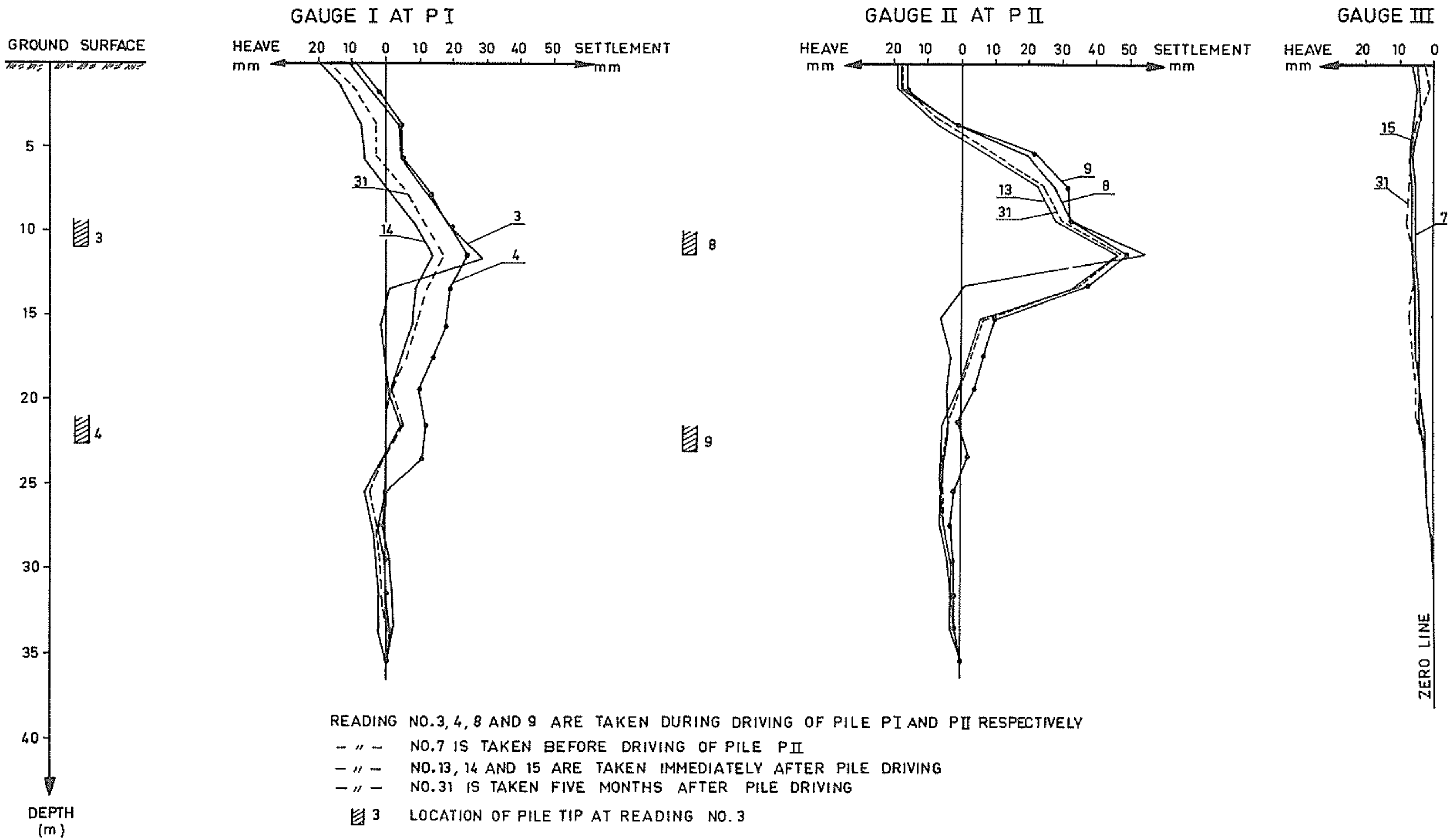
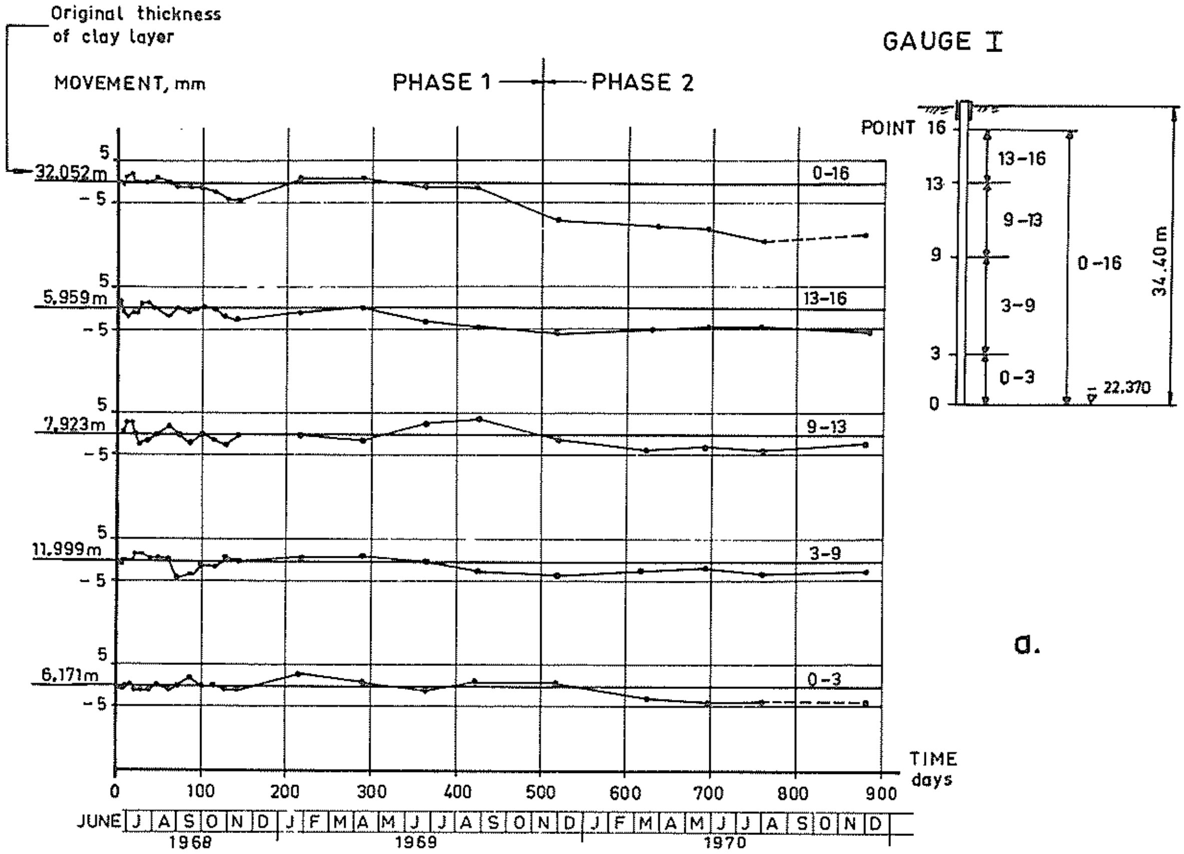
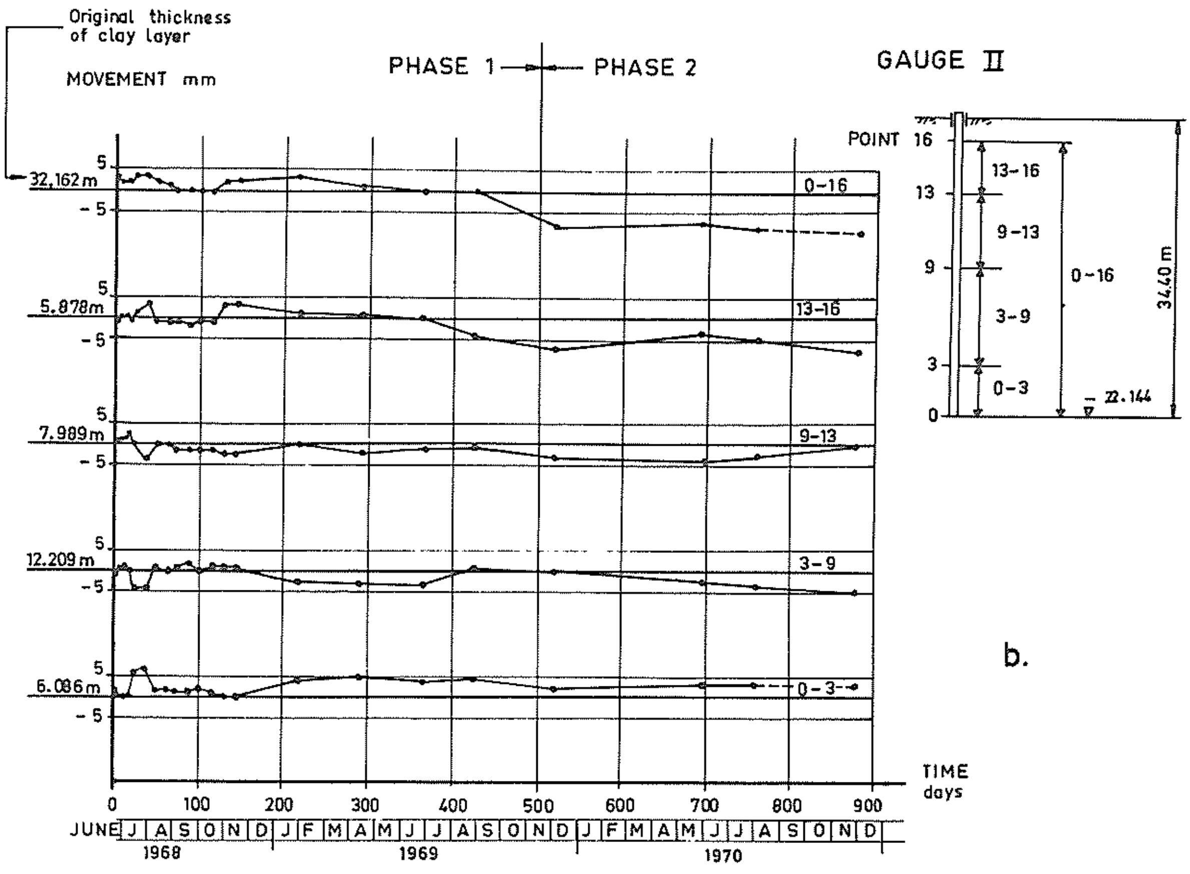


Fig. 10 Vertical movements in the clay during and after driving



a.



b.

Fig. 11a – b Settlements of the ground surface and relative movements within the clay

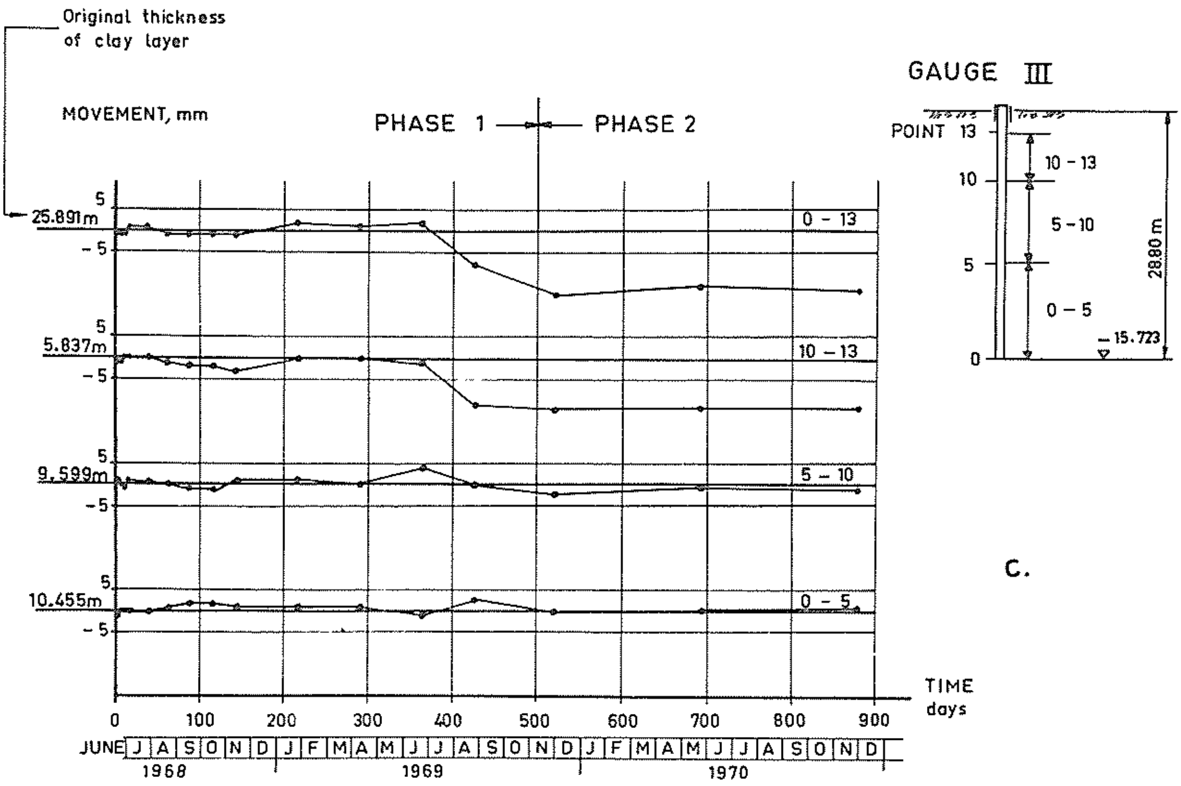


Fig. 11c Settlements of the ground surface and relative movements within the clay

days the movements have been small. During the first 3 to 4 months settlements of the order of 2 to 3 mm occurred. Then a small heave of about the same magnitude was measured. This heave coincides in time with a small increase of pore water pressure (cf. Chapter 4.3). Settlements were again observed after about 400 days, i.e. July 1969, terminating at about 500 days. This is most likely due to the extremely dry summer and autumn of that year.

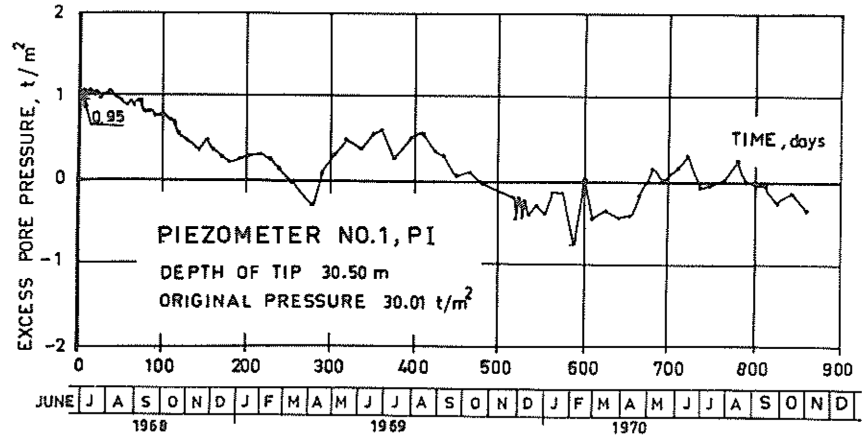
The settlement of the ground surface measured between July and November (400 and 500 days) close to the two piles was 8 mm while at Gauge III remote from the piles the settlement during this period was 16 mm. This indicates that the piles resisted the settlements of the clay around them. The settlements occurred in the upper 8 - 10 m of the clay layer and below this depth no appreciable settlement was observed.

4.3 Pore water pressures

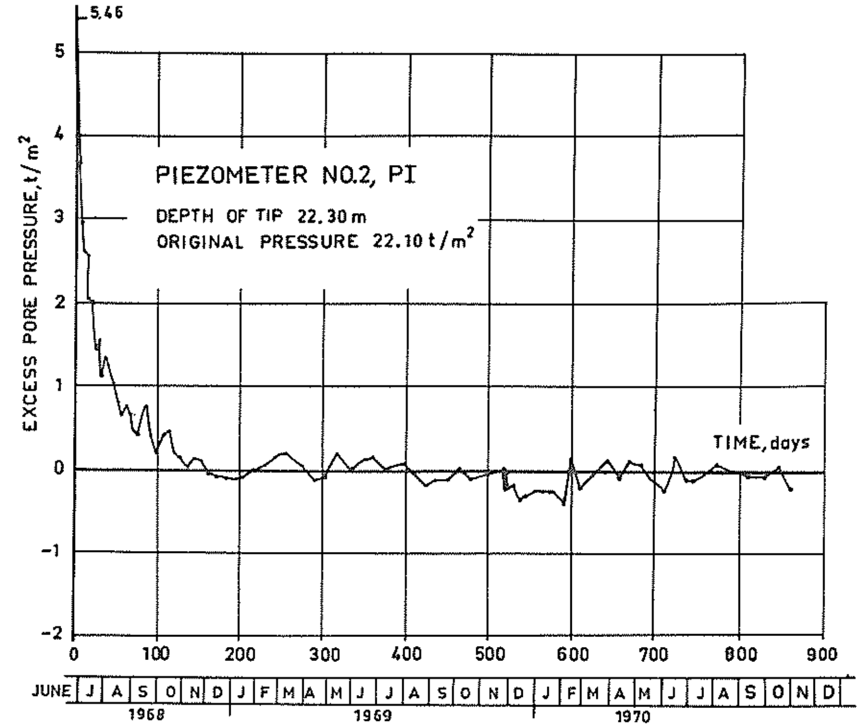
The pore pressures observed by means of the 8 piezometers are shown in Figs. 12 a - 12 g. The diagrams show the changes in pore pressure relative to the original pore pressure as measured prior to the driving of the piles.

The excess pore pressures that developed around the piles dissipated during the first 5 - 6 months. Then the pore pressures in the clay again increased by about 0.2 to 0.3 tons/m^2. As mentioned, this increase coincided with a small heave of the clay. However, the measured pore pressure changes and soil movements were too small to allow any definite interpretation of the readings apart from the observation that the tendencies of the two measurements agree with each other. It may be noted, as mentioned in Chapter 4.4, that these observations have no observable effect on the forces in the piles.

Obviously, the severe drought during the summer and autumn of 1969 caused a reduction of pore pressures in the clay, which coincided with settlements. This reduction was measured in the piezometers close to the piles, but not in the one (Gauge 2 E) remote from the piles. It is possible that the concrete piles are acting a.



b.



c.

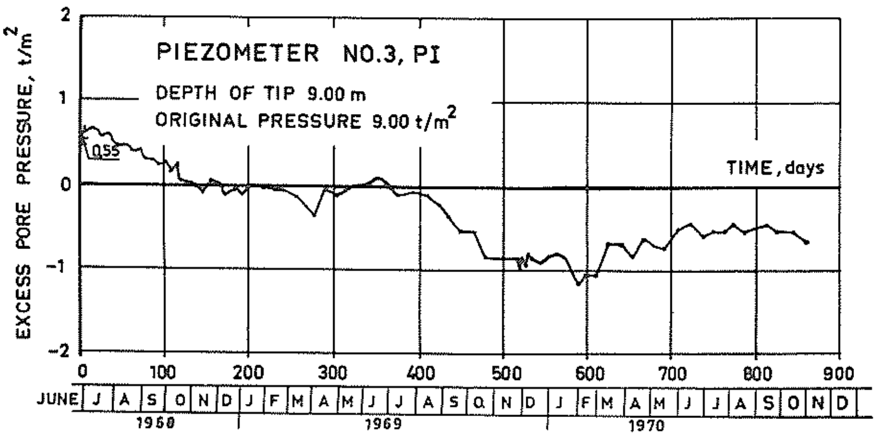
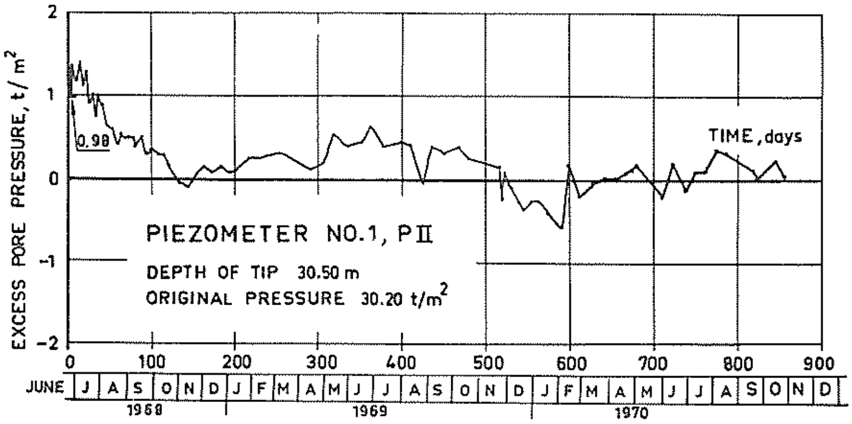
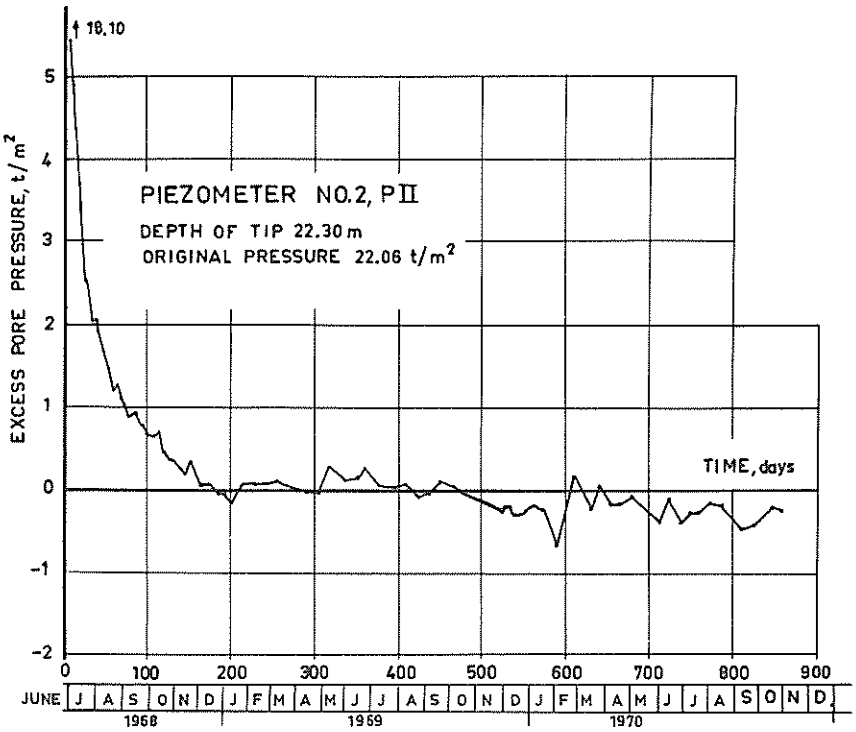


Fig. 12a – c Pore water pressures in the soil related to the original pressures d.



e.



f.

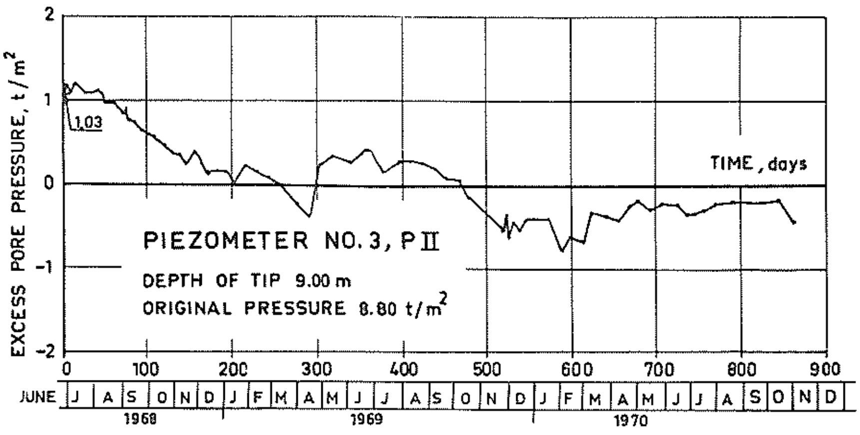


Fig. 12d – f Pore water pressures in the soil related to the original pressures g.

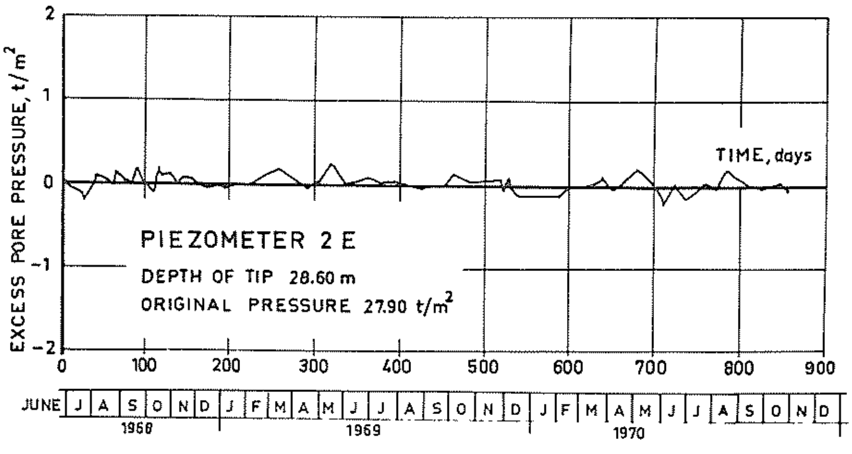


Fig. 12g Pore water pressures in the soil related to the original pressures

as drains in the clay permitting pore pressure changes to take place. In any case, the drainage possibility at Gauge 2 E is small.

The open pipe gauge, No. 1 E, showed that the pressure head in the permeable sand layers was constant throughout the measuring period.

4.4 Forces and bending moments

The axial force development with time for both piles is shown in Fig. 13, and it is evident that the two piles behaved similarly. At first the axial load in the two test piles increased rapidly at the different measuring levels. Two or three weeks after driving, the rate of the load increase slowed down. After about six months (180 days) the rate was constant. Then the measurements at Gauges M 1 and M 5 indicated a rate of load increase of about 1.2 tons per month. This average rate of load increase of Gauges M 1 and M 5 is represented by the dotted line in Fig. 13. The observed rate of load increase was smaller at the other gauge positions located higher up in the piles.

Fig. 14 shows the measured bending moments, i.e. the bending moments in the piles at the level of the pile force gauges. As mentioned in Chapter 3.3, the bending moments in the straight pile, pile P I, were smaller than in pile P II. The bending moments stabilized quickly after the driving and since July 1968 no significant variations were observed.

Fig. 15 shows the measured vertical distribution of load in the two piles at different times after driving. The load distributions immediately and 1, 5, 15, 30 and 60 days after the driving and then the distributions measured every 60 days are shown in the figure. Pile P I is not equipped with a pile force gauge at the pile point. Thus, the load at the pile point can only be obtained from pile P II.

During the first 180 days after the driving, the clay reconsolidated with dissipation of excess pore pressures and consequently a small reduction of volume, i.e. settlements, occurred. The relatively rapid load increase during this period was probably caused mainly by negative skin friction which developed during the reconsolidation of the clay. The total drag load due to this effect is estimated to be about 40 tons. The settlements which caused this drag load were very small, the order of magnitude of 2 - 3 mm, and hardly measurable by the settlement gauges. The 40 ton drag load corresponded approximately to 25 % of the maximum drag load as calculated by the original undrained shear strength of the clay times the surface area of the pile. The load corresponds also to about 8 % of the effective overburden pressure in the clay.

The slow but constant load increase in the piles after the first 180 days is thought to be due to a small regional settlement in the area. After 495 days the total drag load was 55 tons and the negative friction corresponded to 33 % of the undrained shear strength

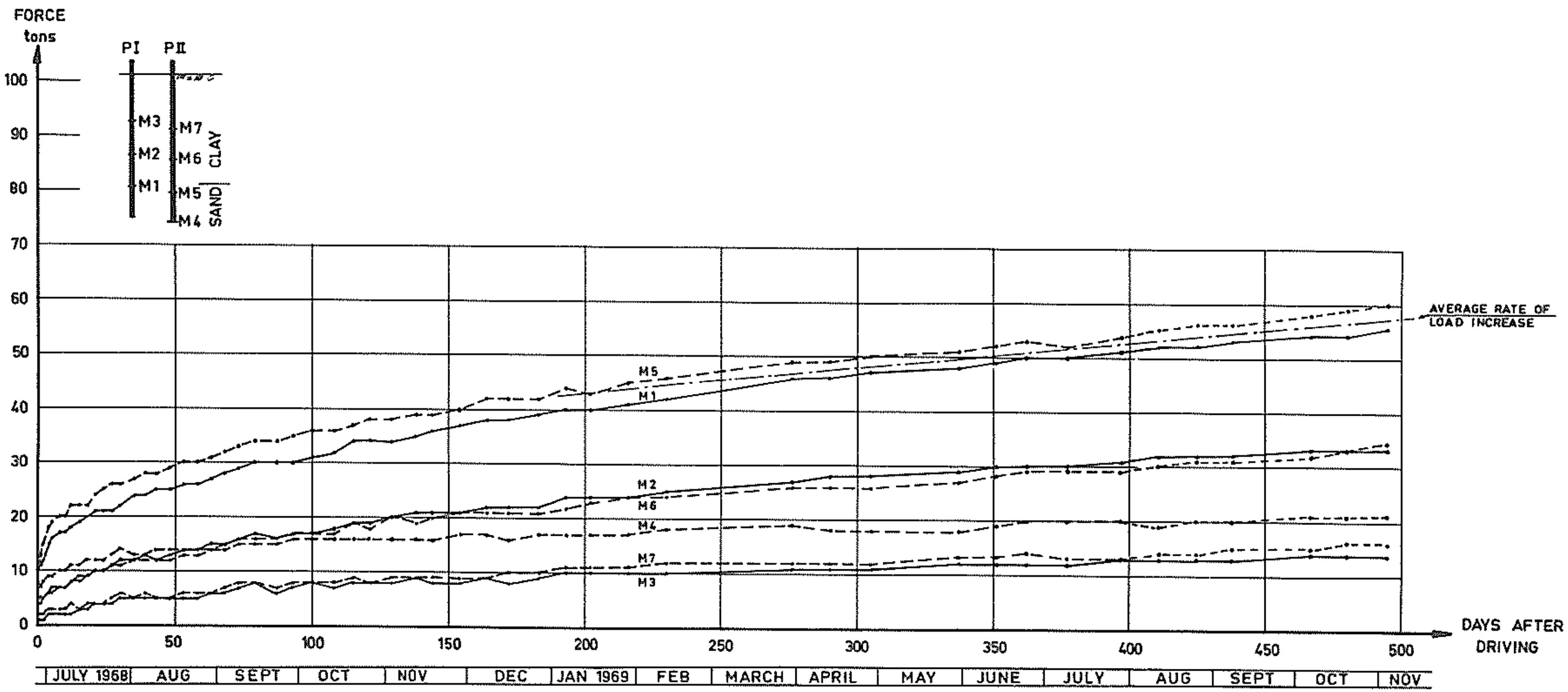


Fig. 13 Development of axial forces in the piles versus days after driving. Phase 1 0 - 500 days

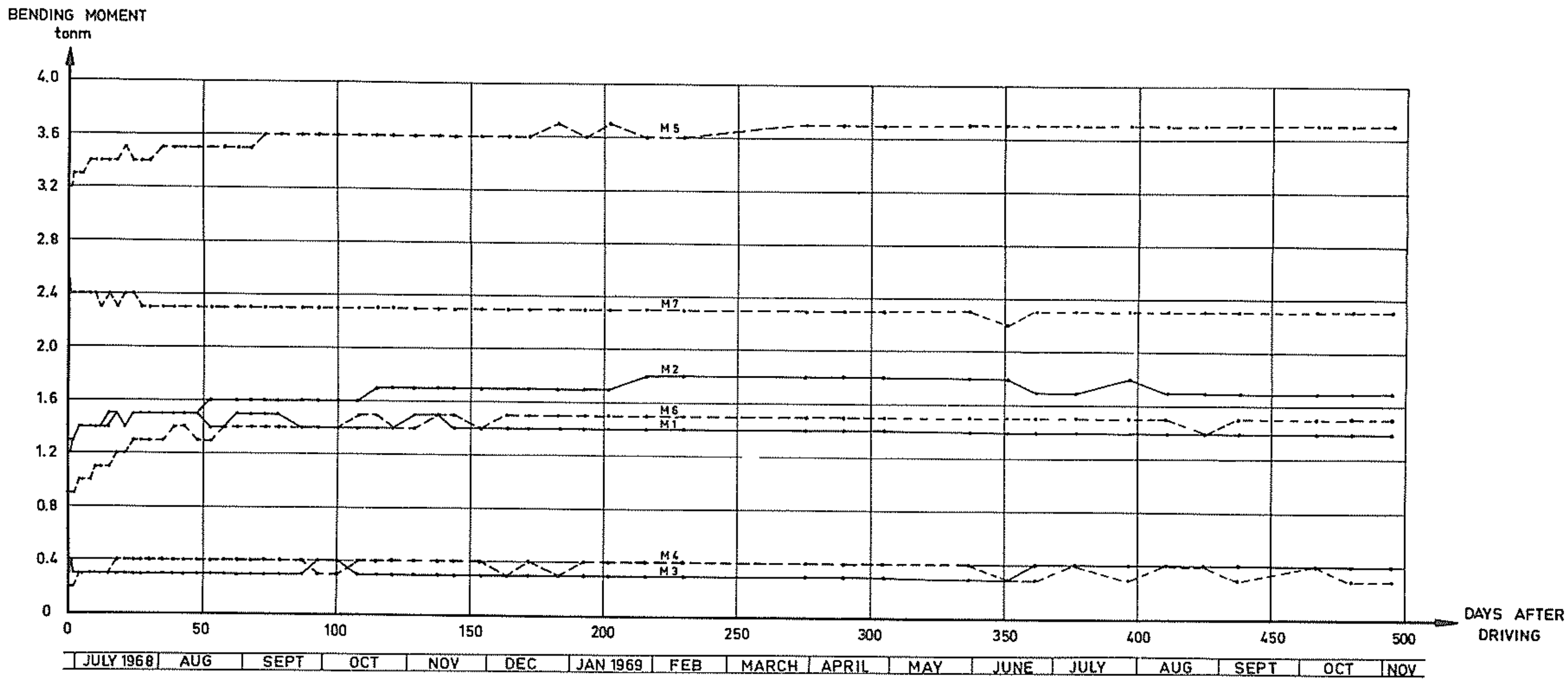


Fig. 14 Development of bending moments in the piles versus days after driving. Phase 1

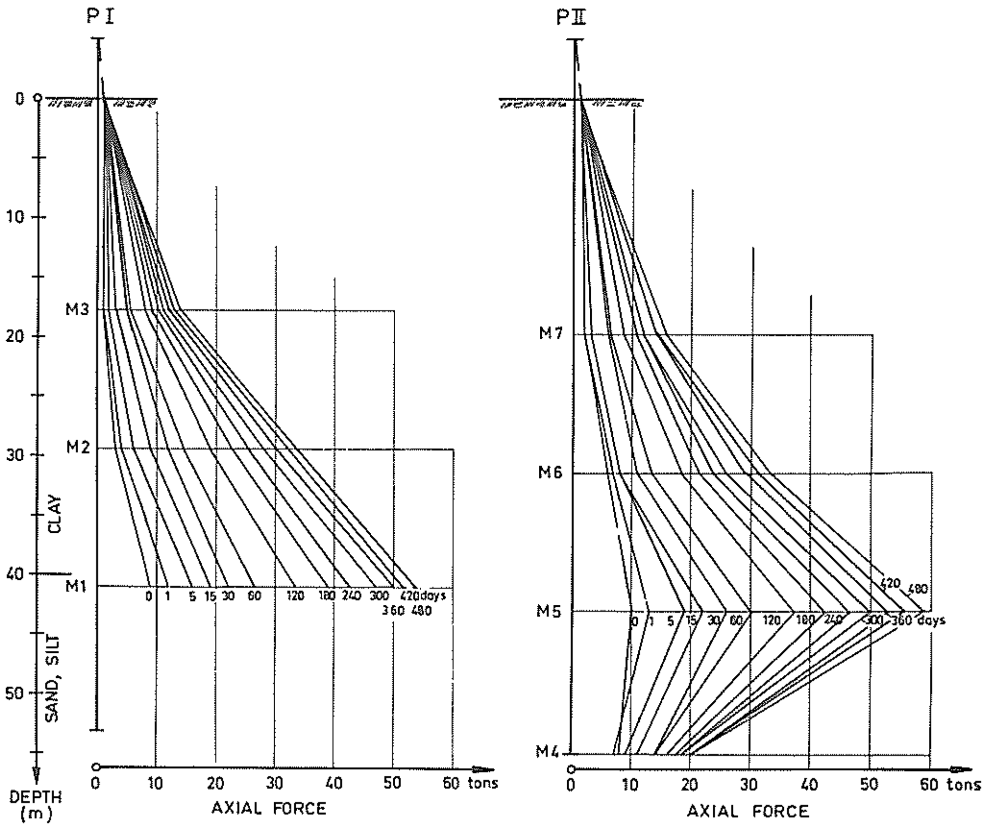


Fig. 15 Vertical distribution of load in the piles at certain intervals after driving. Phase 1

or to 11 % of the effective overburden pressure.

The average negative skin friction can be estimated from the difference in load between the pile force gauges acting over the area of pile between the gauges. Fig. 16 shows the results of this calculation for measurements taken 120, 300 and 480 days after the driving and it indicates that the negative skin friction (τ_{neg}) increased linearily with depth. Also, in this figure the undrained shear strength (τ_{fu}) is shown for comparison. Only the results from pile P II are shown as pile P I gives a similar pattern.

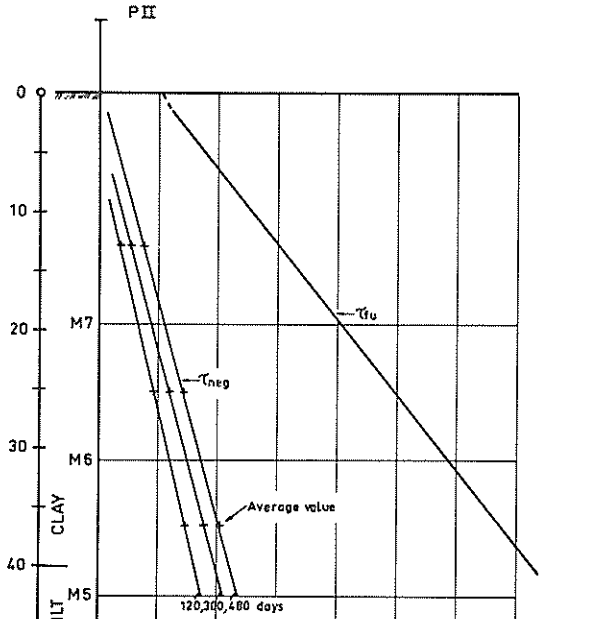


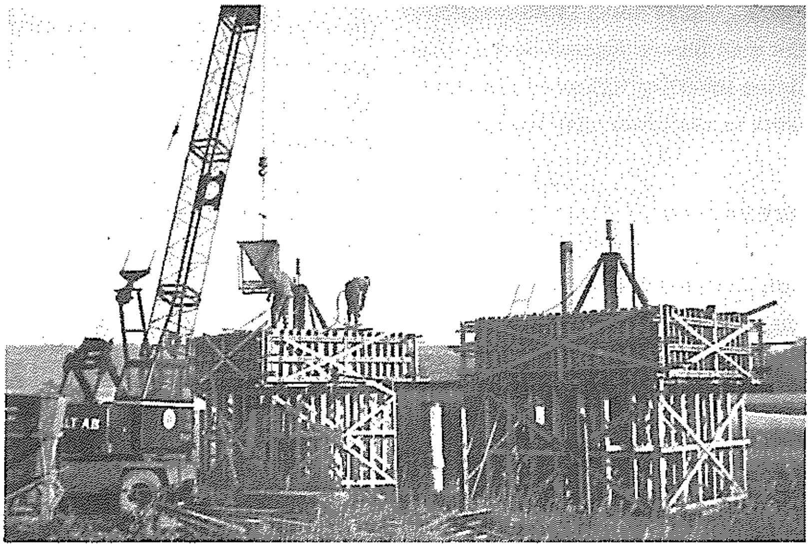
Fig. 16 Negative skin friction (τ_{neg}) of pile P II at 120, 300 and 480 days after the driving compared to the original undrained shear strength (τ_{fu}) of the clay

5. BEHAVIOUR DURING PHASE 2. LOADING OF THE PILES

5.1 General

As mentioned in Chapter 2.1, Phase 2 consists of loading of the piles in two stages. In the first stage a concrete slab weighing 44 tons was cast on each pile (November 1969, 495 days after the driving). In the second stage yet to come concrete blocks with a total weight of 36 tons will be added on the slab.

The concrete slab was cast in a form supported by bracings. When the concrete was cured after three weeks, the bracings were removed. Fig. 17 is illustrating the arrangement for casting and the appearance of the piles after removal of the bracings.



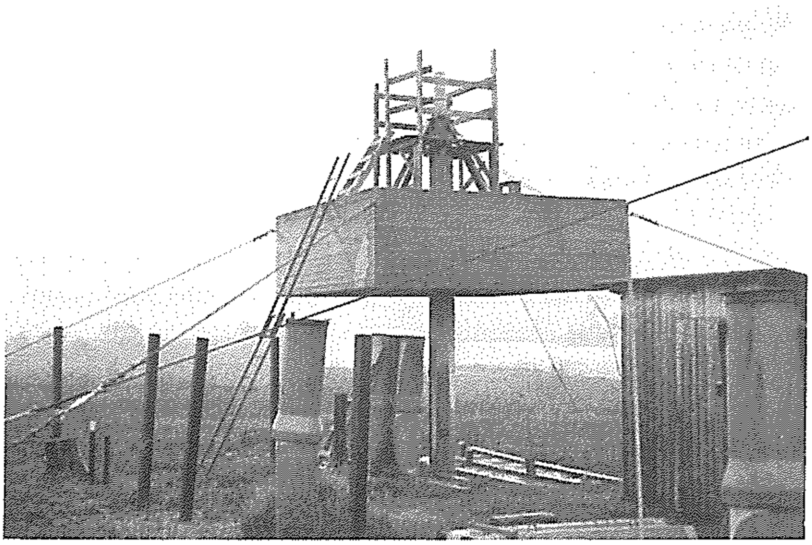


Fig. 17 Arrangement for loading of Phase 2.
Upper figure: Casting of concrete.
Lower figure: After removal of the bracings.
The wires serve as a precautional support in case of large wind loads on the concrete slab would be obtained

5.2 Settlements

The settlements of the ground surface, which increased during the dry summer of 1968, were again small at the end of this year, cf. Fig. 11. The additional settlement of the ground surface was only about 2 mm from the beginning of the second phase in November 1969 until November 1970. The measurements seem to indicate that a small, presumably regional, settlement of approximately 2 mm occurs each year. However, this settlement is too small and the measuring period too short to allow any detailed study of its actual magnitude and distribution with depth. There seem to be extremely small movements in the clay layers located deeper than about 10 m. The precision levelling indicated also that there were no movements in the lower clay layers.

5.3 Pore water pressures

The pore water pressure increased again after 1968 (Fig. 12). However, the measurements did not show any significant pressure changes.

5.4 Forces and bending moments

The forces measured by the pile force gauges are plotted against time in Fig. 18 which is a continuation of Fig. 13. To simplify comparison with the development during Phase 1, the measurement results between 300 and 500 days are plotted on both diagrams. The measured bending moments are shown in Fig. 19 as a continuation of Fig. 14. As before, the results of the measurements between 300 and 500 days are shown in both figures.

Initially, the applied load was resting on the ground through the bracings and the load was then gradually transferred to the piles during the curing of the concrete to an unknown extent. Finally, after three weeks, when the bracings were removed, the whole load was transferred to the piles.

When the load was applied on the pile head, it is evident that considerable changes of axial load occurred in the upper portion of the piles, while only small changes took place in the lower portion of the piles. This will later be discussed in detail. Also considerable increase of the bending moments was obtained in the upper portion of the piles. Gauge M 7 in the bent pile P II showed an increase of the bending moment from 2.3 tonm to 3.3 tonm, when the load increased from 15 tons to 44 tons. The "twin gauge" in pile P I (Gauge M 3) indicated an increase of the bending moment from 0.4 to 0.7 tonm and the corresponding increase of the axial load from 14 to 44 tons. Thus the increase of the bending moment in the bent pile was considerable, 1.0 tonm at this level, whereas the increase was small, 0.3 tonm, in the straight pile for the same increase of the axial load. However, the relative increase of the bending moments was about the same in both piles. The change of bending moments occurred immediately after the removal of the bracings and no further change was observed despite of the continued increase of axial load. Apparently, the bending moments in the piles reacted more to a load applied on the pile head than to the slow loading caused by negative skin friction.

The dotted line in Fig. 13 representing the average rate of load increase for Gauges M 1 and M 5 is continued in Fig. 18 beyond the end of Phase 1 at 495 days. A similar line for the average rate during Phase 2 is also drawn. As can be seen, the two lines are parallel and the load difference between them is 5 tons. Thus, it is concluded that the applied load on the piles did not appreciably change the development of negative skin friction on the piles. In other words, if Phase 1 had been continued, the load at Gauges M 1 and M 5 would have been about the same as the load that now was observed. That is, the total drag load would have been 75 to 80 tons, or approximately 50 % of the maximum load calculated by using the undrained shear strength of the clay. Furthermore, it is believed that if the applied load had been placed on the piles immediately after the driving, the negative skin friction would then have been added to the applied load, and the total load after the 850 days would have been of the order of 120 tons.

However, the rate of the load increase is small at the upper Gauges M 3 and M 7. In fact, during the last five months (150 days) no appreciable load increase was observed for these gauges. It is noted that the settlement rate in the upper clay layers has been small during this time (Fig. 11).

The vertical load distribution in the two piles is shown

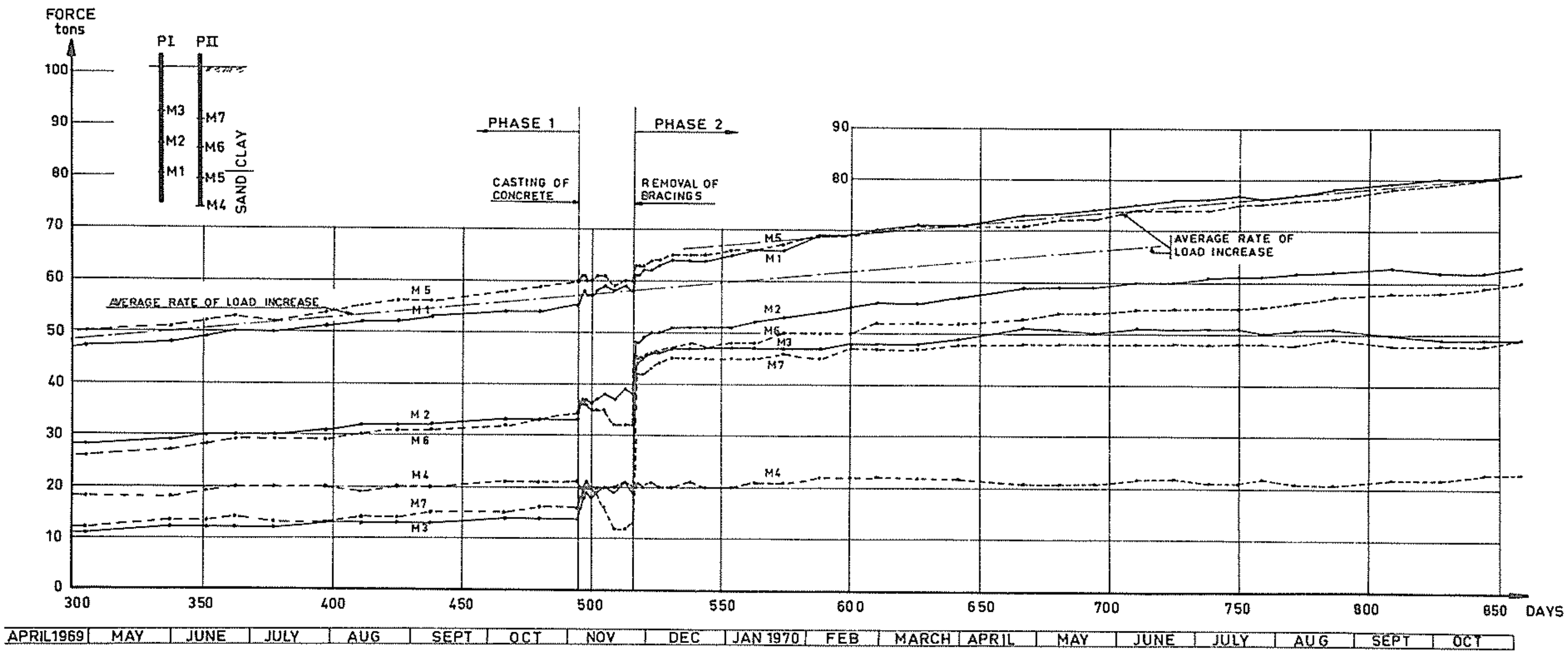


Fig. 18 Development of axial forces in the piles versus days after driving, 300 - 860 days

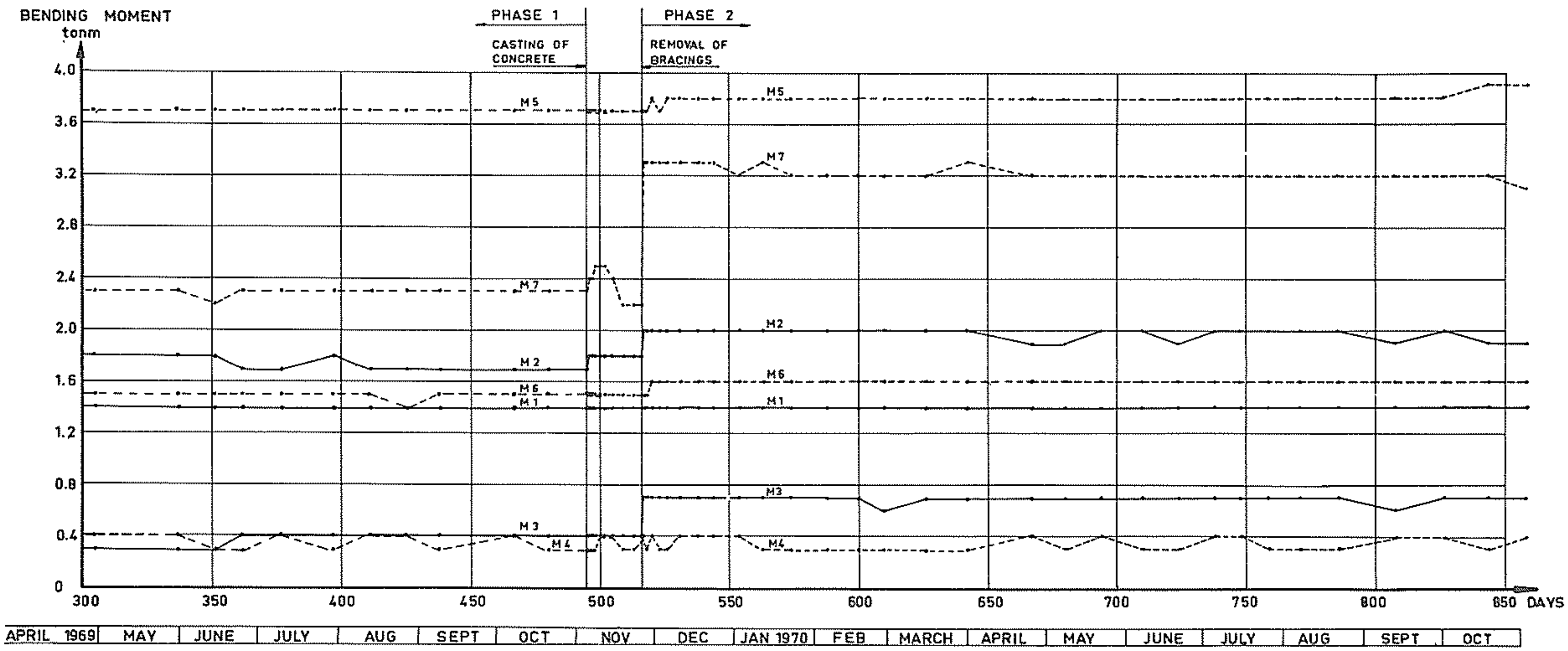


Fig. 19 Development of bending moments in the piles versus days after driving, 300 - 860 days

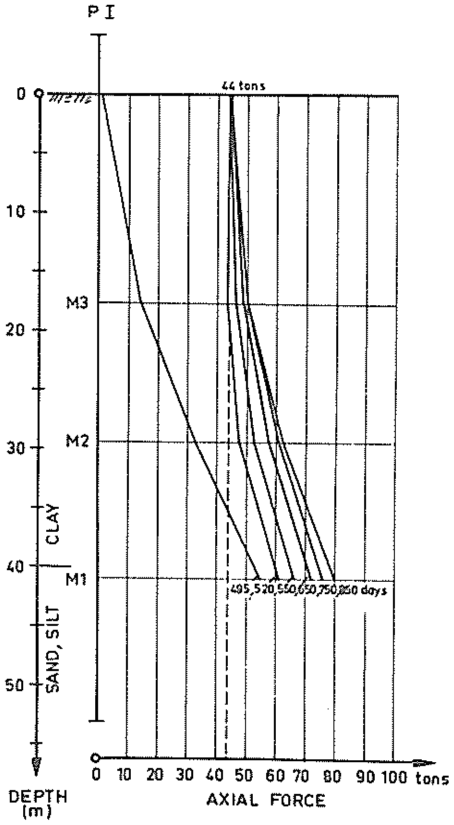


Fig. 20 Vertical distribution of axial force in pile P I, phase 2 (495 = day before casting of concrete, 520 = day after removal of bracings)

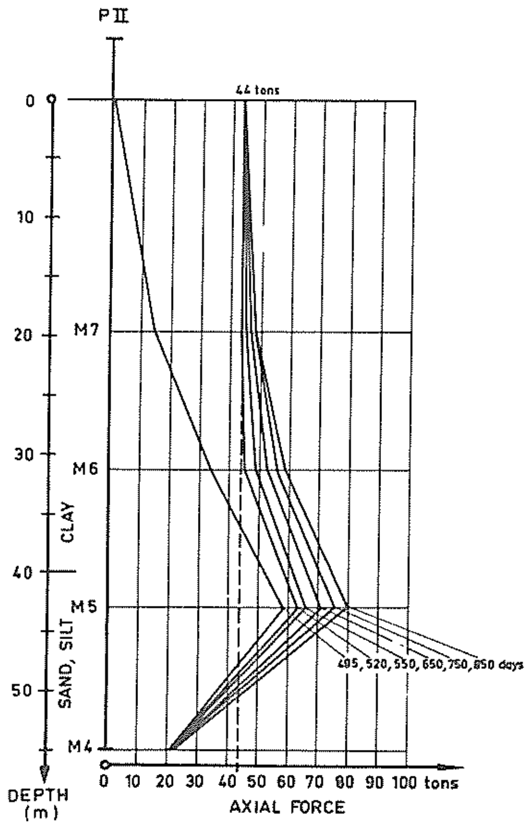


Fig. 21 Vertical distribution of axial force in pile P II, phase 2 (495 = day before casting of concrete, 520 = day after removal of bracings)

in Figs. 20 and 21, which are continuations of Fig. 15. It should be noted that the force scale is not the same as in Fig. 15. The line for 495 days shows the measured loads immediately before the casting of concrete and the line for 520 days the loads after the removal of bracings. The other lines show the loads at 550, 650, 750 and 850 days respectively.

The applied load of 44 tons is at first transmitted straight down the two piles to a depth of about 25 metres below which depth the load in the pile was greater than 44 tons. It is evident that the negative skin friction was eliminated along the upper portion of the piles when the 44 ton load was applied to the pile head. As the applied load was smaller than the maximum drag load, the negative skin friction still existed in the lower portion of the piles. One might have expected some positive friction in the upper clay layers, i.e. Gauges M 3 and M 7 would then have shown a load smaller than 44 tons. However, the movements required to change the skin friction from negative to positive are presumably larger than the obtained movements (see below), whereas only very small movements are apparently needed to eliminate negative skin friction.

Fig. 21 shows that the load at the tip of pile P II is not increasing, i.e. the load in the piles was resisted by positive skin friction in the silt and sand layers. The loading of the piles caused the pile heads to settle approximately 1 mm. The tell-tales in the piles indicated that this settlement was due to compression of the upper part of the piles. No tip settlement was observed during the entire measuring period.

6. DISCUSSION AND CONCLUSIONS

When piles are driven into soft normally consolidated clays, the clay close to the piles is remoulded and displaced and large excess pore pressures develop. The following gradual reconsolidation of the clay and dissipation of the excess pore pressures cause small additional settlements of the clay and consequently load is transferred to the piles due to negative skin friction.

In the present case the reconsolidation time was about six months and the drag load caused by negative skin friction was 40 tons which corresponded to about 25 % of the skin friction resistance calculated from the original undrained shear strength of the clay times the circumferential area of the pile.

When friction piles in clay are test loaded one often finds that two piles which have approximately the same bearing capacity or ultimate shaft resistance, yet may show considerable difference in load deformation characteristics before reaching the ultimate load. The results of this investigation suggest an explanation to this behaviour; the piles are subjected to initial drag loads caused by the reconsolidation of the soil and this preloading may not be the same for the two piles.

Only very small settlements are required to cause negative skin friction. The observed settlement of the ground surface close to the piles was only about two millimetres. Due to a continued small regional settlement of about 2 - 3 millimetres per year, the negative skin friction increased by about 15 tons per year. It is most probable that the final total drag load will be considerable.

When a load is applied on the head of the piles, only the part of the load exceeding the drag load will be added. However, the negative skin friction which is obtained later will be added to the previous load in the pile in full.

The measurements show a distinct difference of pile behaviour for loads applied on the pile heads and the slow loading due to negative skin friction. When a load was applied on the pile head, the gauges indicated an increase of the bending moments in contrast to the loads which were caused by negative skin friction. The measurements of bending moments during Phase 1, 150 - 495 days and during Phase 2, 520 - 850 days after the driving of the piles are presented in the following table.

Table 1. Bending moments during Phase 1 and 2

Gauge	PHASE 1				PHASE 2			
	150 days		495 days		520 days		850 days	
	tons	tonm	tons	tonm	tons	tonm	tons	tonm
M 3	8	0.4	14	0.4	44	0.7	49	0.7
M 7	9	2.3	15	2.3	44	3.3	48	3.2
M 2	21	1.7	33	1.7	48	2.0	62	2.0
M 6	21	1.5	34	1.5	46	1.6	59	1.6

Only the initial results of the test are reported in this paper and it is too early to draw full final conclusions. However, in the following chapter some general views on negative skin friction on vertical piles will be discussed from the basis of this test.

II. GENERAL VIEWS AND DESIGN RECOMMENDATIONS

1. INTRODUCTION

As found from the literature survey (Fellenius, 1969) that preceeded the field study reported in Part I of this report, there exist today only a few methods of calculating negative skin friction. However, the relevance of these methods to Swedish soil conditions still remains to be shown. Furthermore, only one of these methods has been derived from actual field tests, i. e. the one proposed by Johannessen & Bjerrum (1965) and Bjerrum et al. (1969). In this method the negative skin friction (τ_n) is calculated by the following simple formula

$$\tau_n = K \tan\varphi \cdot \bar{p} = 0.3\,\bar{p} \qquad (1)$$

where $\bar{p}$ is the effective overburden pressure in the soil. Field measurements have shown values of the factor $K \tan\varphi$ ranging from 0.20 to 0.28.

The final drag load would according to this formula be about 150 tons for the piles P I and P II, which is approximately equal to the drag load of 170 tons as estimated from the original undrained shear strength of the clay. However, whether the final drag load will be of this order or not remains to be seen. In fact, one basic aim of the investigation is to obtain a method for predicting the drag load on piles.

Until further field results are obtained, one has to assume that the negative skin friction for single end-bearing vertical piles can finally reach a magnitude corresponding to the shear strength of the surrounding soil. When estimating drag loads for piles in a pile group, it is suggested that the method proposed by Terzaghi & Peck (1948) should be applied, i. e. the maximum drag load on a pile in a pile group corresponds to the weight of the soil within the group and that this weight is shared between the piles.

Furthermore, it has been shown that already very small settlements of the surrounding soil will cause negative skin friction. Such settlements will occur in old or new building areas due to unavoidable general lowering of the ground water table and to the placing of fills on the surface during construction of streets and other utilities.

In fact, it is most probable that old pile foundations in many areas are already subjected to drag loads of an unknown magnitude. Normally, until quite recently, no allowance for loads due to negative skin friction is made. However, neither has any damage on buildings supported by long high quality piles been observed, as mentioned by Severinsson (1965). This supports the opinion that the effects of negative skin friction are exaggerated and can normally be neglected. However, it is the Author's opinion that the load due to negative skin friction, even if it cannot be added directly to the working load on the piles when designing a pile foundation, should be taken into account in the design, as discussed in the following sections.

2. BEHAVIOUR OF PILES SUBJECTED TO LARGE DRAG LOADS

Consider a single vertical end-bearing pile having a working load P and being subjected to negative skin friction. The largest load in the pile will then with time be obtained at or near the pile tip. In Fig. 22 a is shown the load distribution of a pile some time after the driving and the errection of a building, when the drag load at the pile tip has increased to P_n.

If the pile material is stronger than the soil at the pile tip, the sum $P + P_n$ cannot exceed the bearing capacity of the pile tip, Q_u. When $P + P_n = Q_u$, the pile tip will settle more or less suddenly and consequently the load P_n is reduced or eliminated. This is demonstrated in Fig. 22 b. The negative skin friction has been reduced along the lower portion of the pile, due to the downwards movement of the pile shaft caused by the settlement of the pile tip. This settlement will be very small, only a few millimeters, as the settling will cease as soon as the load P_n is reduced, which occurs with very small deformations (Fig. 23).

The reduction of load in the lower portion of the pile will cause a corresponding lengthening of the pile and no movement will be observed at the pile head. With time the negative skin friction will build up again and the lengthening will be reclaimed. Thus, the settlement at the pile tip will slowly be transferred to the pile head, as shown in Fig. 22 c. Apart from the settlement, the situation in Fig. 22 c is the same as in Fig. 22 a, which indicates that the procedure is cyclic. Eventually, large settlements of the pile can occur, depending on the individual circumstances.

It is important to note that even if the ultimate bearing capacity of the pile tip is reached, the pile does not fail since only a very small movement will reduce the load P_n. This case can be compared with the case when the working load P exceeds the bearing capacity of the pile and rapid excessive settlements could occur. Negative skin friction is therefore not a failure problem but it can, as shown, involve a substancial settlement problem even for end-bearing piles.

The example in Fig. 22 concerns a pile which is stronger than the soil at the pile tip. The case of a pile which has been driven to hard bedrock can be treated similarly. The largest load in the pile is obtained near the pile tip or at the lower part of the settling soil layer. The sum of $P + P_n$ is in this case limited by the compressive strength of the pile section and if this limiting stress has been reached, the pile material will deform and settlements will be obtained at the pile head in due course. In the case of a concrete pile, the deformation will increase rapidly as this limiting stress is approached due to an increased creep in the pile material. However,

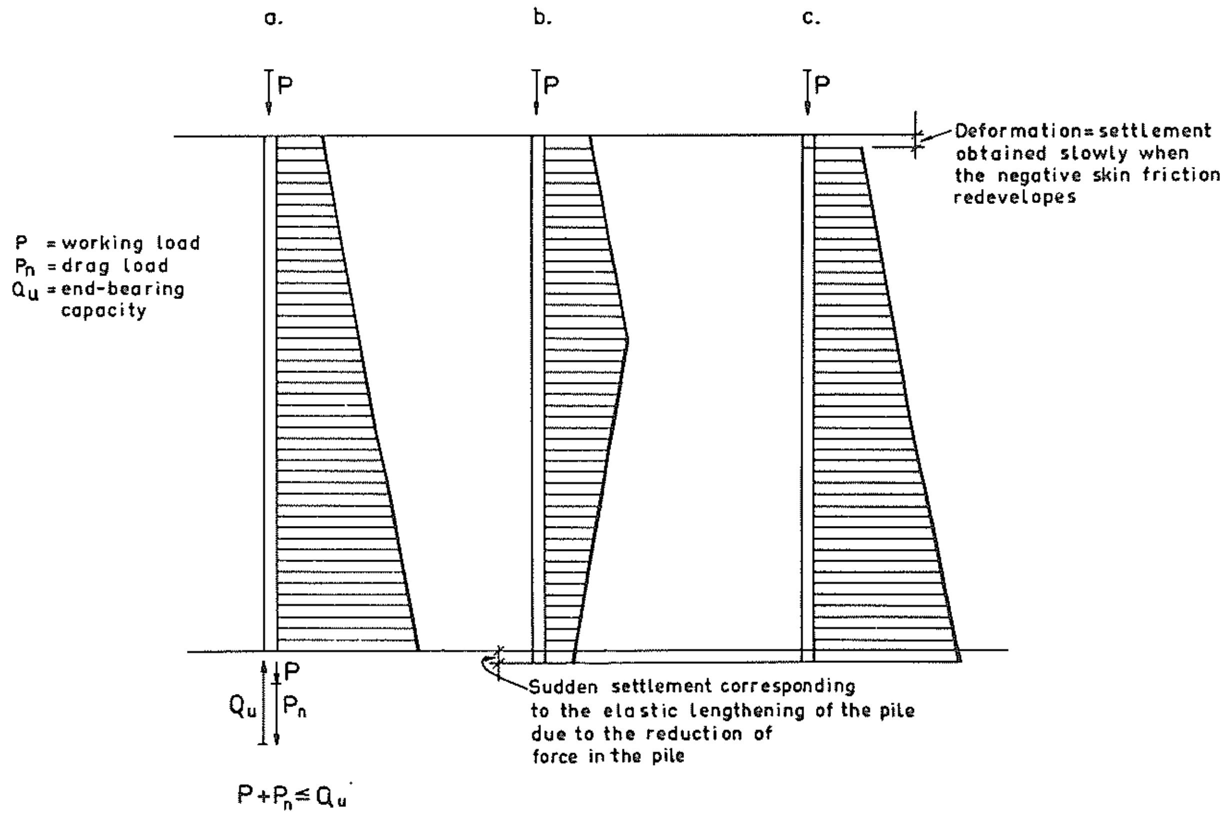


Fig. 22 Load distribution in a pile subjected to large drag loads and the cyclic development of settlements when the end-bearing capacity of the pile is exceeded

the deformation is probably not located to a definite point in the pile, but along some length of the pile. If, however, the compressive strength of the pile section is exceeded the end-bearing capacity of the pile is reduced. Additional movements will be concentrated to this section and as a result the pile settlement will increase rapidly.

The situation described in Fig. 22 demonstrates the cyclic consequences when the drag load exceeds the end-bearing capacity of a pile or exceeds the strength of the pile material. It should be observed that even if the drag load does not reach this magnitude, it will cause deformations in the pile, i.e. settlements at the pile head.

The discussions above deal with the problem of a single pile. The problem of piles in a group is naturally more complicated. The ideal case, when all the piles in a foundation are of equal length, carry the same working loads and have the same bearing capacity, or rather have the same deformation characteristics under load, does not exist in reality.

The differences in acutally applied working loads, lengths etc. cause differential settlements of the pile foundation. However, these differential settlements will generally be small and besides, they occur mainly during the construction period. Thus, they are normally negligible. When negative skin friction developes, further settlements of the piles will occur. These settlements will occur after the construction and can be large. Furthermore, they will be uneven, depending upon differences of pile lengths and of deformation characteristics.

If one pile settles more than the neighbouring piles due to difference in magnitude of drag load, pile length etc., the working load on this pile will be reduced and transferred to the neighbouring piles. Consequently, the settlement of the neighbouring piles will increase and the differential settlement will be equalized to some extent. However, this process will induce additional stresses in the foundation. One therefore has to choose between a stiff foundation, capable of accepting the induced stresses, or a foundation which can accept differential settlements. Usually, it is more practical to take measures against the differential settlements by avoiding piles of considerably different lengths in the same foundation or allowing larger loads on short piles as compared to longer piles and using a larger number of slender piles with a closer spacing instead of more heavily loaded piles with a wider spacing. Of course, friction-bearing piles should not be used in combination with end-bearing piles.

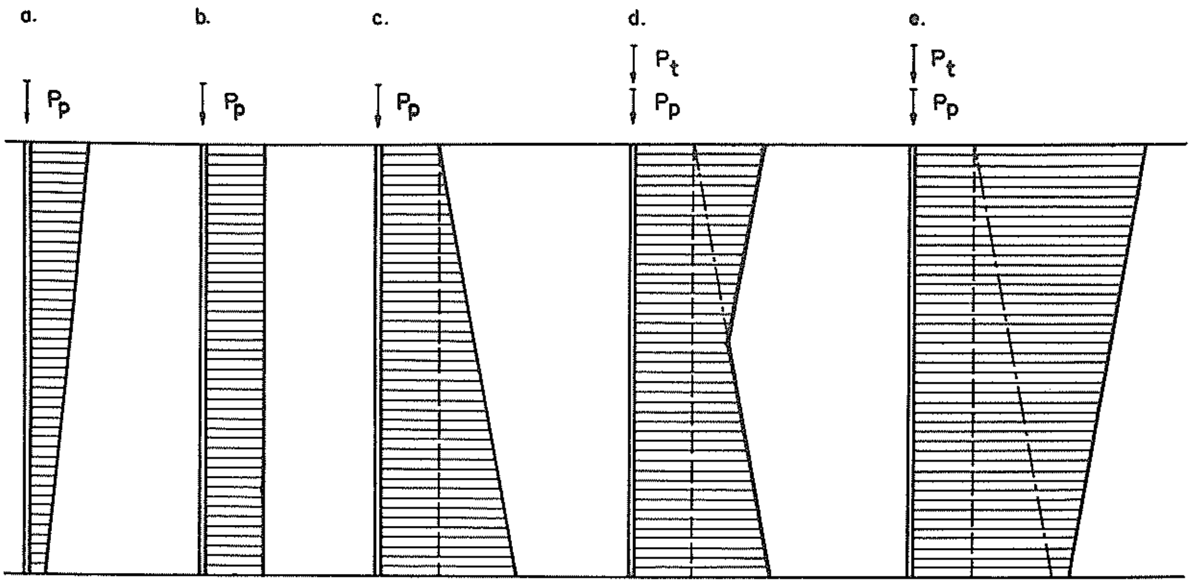
A pile foundation is employed to eliminate or reduce settlements and especially differential settlements. It must be pointed out that even if negative friction can, as discussed above, cause differential settlements, these settlements will be smaller than if no piles were used.

3. PERMANENT AND TRANSIENT LOADS ASSOCIATED WITH NEGATIVE SKIN FRICTION

When the piles in the investigation described in Part I were loaded, the load was not added to the drag load that already was acting on the pile (cf. Chapter 5.4). This is believed to be because the movement due to the elastic compression of the pile is sufficient to reduce the negative skin friction. When a load larger than the drag load is applied on the head of the pile the pile is compressed and consequently the direction of the movement between the pile shaft and the soil is reversed and positive skin friction will develop.

On the other hand, if a load on a pile is carried by positive skin friction, only very small settlements in the clay would reduce the positive friction and the full load will then go further down the pile to the pile tip or to the lower part of the settling layer. Then, if further settlements occur, the load will increase because of negative skin friction.

Now consider a pile subjected to both a permanent load on the head and a load caused by negative skin friction. If a transient load due to traffic, wind, snow etc. then is added to the pile, this load will consequently not increase the load in the lower portion of the pile, pro-



a. Permanent load, only positive skin friction
b. —"— —"—, small settlements around the pile
c. —"— —"—, negative skin friction
d. Small transient load added to case "c"
e. Large —"— —"— —"— —"— —"— "
P_t Transient load
P_p Permanent load

Fig. 23 Load distribution in a pile during different loading conditions

vided the transient load is smaller than about twice the drag load. Half the transient load will be used for eliminating the negative skin friction and the rest will be resisted by positive skin friction. Thus, as will be outlined in Chapter II:5, transient loads should be treated separately from permanent loads and drag loads in the design of a pile foundation.

The discussion in this section is summarized in Fig. 23, which shows the load distribution in a pile during different conditions as follows. For reasons of simplicity it has been assumed that the skin friction along the pile is constant.

Case "a" shows the load distribution when only a permanent load is acting on the pile. The load is resisted by positive skin friction and no negative friction exists along the pile.

Case "b" shows the effect of very small settlements in the soil. The settlements correspond to the elastic compression of the pile, i.e. no relative movement between the pile and the surrounding soil. As discussed above, the load on the pile head is, due to the settlements, transferred further down the pile. This would in fact be the long term situation for an end-bearing pile through soft clay, due not necessarily only to actual consolidation settlements in the clay, but to rheological effects in the clay.

Case "c" shows the effect on pile "a" when settlements larger than in case "b" are obtained. In this case the load in the pile would increase with depth due to negative skin friction along the pile.

In case "d" a small transient load is added to the pile "c". To a certain depth the transient load is larger than the previous drag load in the pile and below this depth the pile is unaffected by the increased load on the pile head.

In case "e" the transient load is larger than the sum of permanent load and drag load at the pile end and the case is similar to that of case "a", i.e. positive skin friction is acting along the entire length of the pile.

The intention has been to show the differences of pile behaviour between permanent and transient loads on piles with and without negative skin friction. The indicated load distributions do not necessarily show the correct proportion between skin resistance and tip resistance. It is probable that a repeated large transient loading in conjunction with negative skin friction will give a further increase of load in a pile as compared to the case of no negative skin friction.

4. NEGATIVE SKIN FRICTION ON BATTERED AND BENT PILES

The drag loads, which are caused by negative skin friction, in a vertical pile act in the direction of the pile. However, when settlements occur around a battered pile, the pile will also be subjected to lateral load. The stress increase which is induced by this lateral load depends on the rate of the settlements, the length of pile involved in the settlements, the bending resistance of the pile etc. In particular, the rate of settlement will be important as a slow rate of settlement will have a less important effect than would a higher rate of settlements. For example, it is well known that a concrete pile segment, which during storage in the factory is supported only at the ends, will with time bend due to its own weight.

The effects of settlements on a battered pile depend also on the magnitude of the settlements, i. e. the enforced lateral movement. However, the lateral movement for a given magnitude of settlement depends on the batter of the pile, i. e. deviation from the vertical.

It is recommended that when large settlements are expected, battered piles should be avoided. However, it is believed that the effect on piles with a moderate batter of about 8:1 and less can be neglected and thus these piles can be treated as vertical piles, as far as negative skin friction is concerned. For instance, an investigation in Japan measuring and comparing the effect of drag load on vertical and battered (7:1) piles showed no difference in behaviour related to the batter. (Endo et al., 1969).

The question of bent piles is very similar to that of battered piles. The investigation described in Part I indicated that a slowly increasing drag load does not increase the bending moments in a pile (See Table I). On the other hand, when the load increase was due to a quickly applied load, the bending moments increased. However, the load increase which has been observed or applied in the investigation is quite small.

As mentioned in Chapter 1 (Part II), drag loads can become very large. Large drag loads on a bent pile may give a smaller increase of bending moments in the pile than would a suddenly applied load of the same magnitude. However, the bending stresses in the pile will be superimposed on the stresses which are caused by the drag load. Then, if the compressive strength of the pile material is exceeded further settlements of the pile head will be obtained, as is pointed out in Chapter 1 (Part II). Thus, the pile settlements will definitely increase if the pile is bent during the installation. The allowable bending of piles is to some extent dependent on the expected working load on the piles (Fellenius, 1971). It is recommended that the risk of bending of piles should be considered even at low working loads, when settlements causing negative skin friction are expected.

5. DESIGN CONSIDERATIONS FOR ALLOWABLE LOADS ON PILES TAKING NEGATIVE SKIN FRICTION INTO ACCOUNT

When determining the allowable working load on piles, there are many factors which have to be considered in the design of which the negative skin friction is one. The approach that is outlined in the following is essentially a check that the working load, which has been chosen according to the normal factors, is not too large when or if negative skin friction comes into the picture. Such normal factors are the requirements of the Building Code,

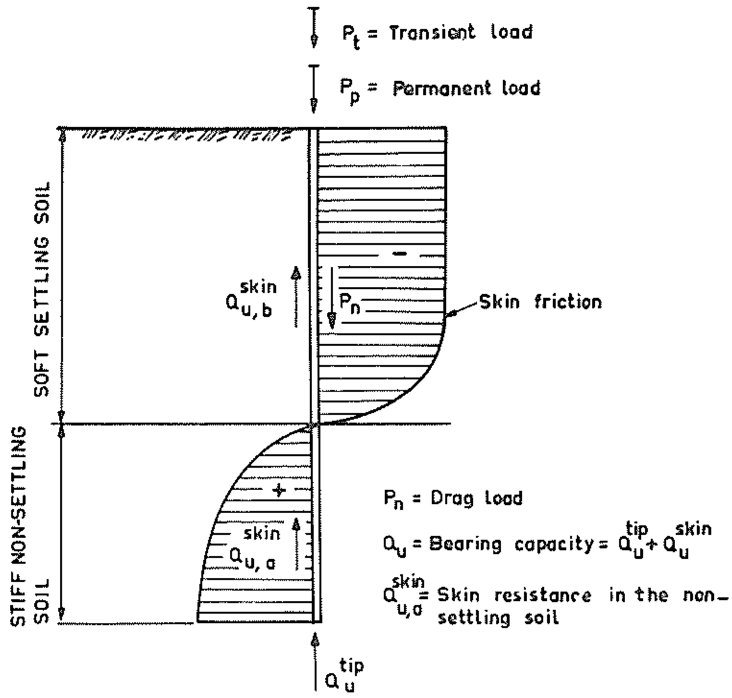


Fig. 24 Skin friction distribution along a pile in an upper layer of soft settling soil and a lower layer of stiff non settling soil

settlement considerations, the quality of the piles that are to be used etc. The main feature of the recommended approach in this section is that the permanent and transient working loads should be treated separately in connection with negative skin friction. Design of a pile foundation according to the recommended approach should be performed in cooperation with an experienced soils engineer.

First, the ultimate bearing capacity, Q_u, of the pile is estimated. This estimate can be obtained by a calculation from known soil data or from a load test. However, positive skin resistance from soil layers settling around the pile should not be included in the estimate (See Fig. 24). Thus, the bearing capacity will consist of the tip resistance, Q_u^{tip}, and the positive skin resistance, $Q_{u,a}^{skin}$ from the part of the pile that is in the non settling soil. A friction (floating) pile which has been driven into a normally consolidated clay, which settles for instance due to surcharge on the ground surface, will to some extent settle with the soil. Somewhere along the pile the settlement of the pile will be smaller than the settlement of the surrounding soil and at this (the so called neutral point) the skin friction will change from negative to positive. However, the location of the neutral point is difficult to determine as it depends on a variety of factors, such as the length of the pile, the load at the pile head, the distribution of settlements with depth, etc. The following recommendation is therefore mainly meant to be applied on end-bearing piles driven through soft settling soil to good end-bearing resistance in firm soil or on friction piles driven through settling soil layers into firm, non settling layers, where the neutral points is well defined.

When the bearing capacity is determined, the drag load due to negative skin friction, P_n, in the settling soil is estimated. As mentioned in Chapter 1 (Part II), this drag load cannot be accurately calculated by means of any formulas applicable to Swedish soil conditions. However, it is a safe assumption that the maximum drag load is equal to the shaft resistance of the settling soil as calculated from the known strength properties of the soil without applying the usual reduction of the shear strength.

If a transient load, P_t, on the pile head is smaller than twice the drag load, $P_t < 2P_n$, the transient load will not be added to the load in the lower portion of the pile (cf. Chapter 3, Part II). Thus, only the permanent load, P_p, on the pile head has to be considered. The following equation applies

$$P_p + P_n \leq Q_u^{tip} + Q_{u,a}^{skin} \qquad (2\ a)$$

or

$$P_p \leq Q_u^{tip} + Q_{u,a}^{skin} - P_n \qquad (2\ b)$$

where

P_p = The permanent load on the pile head

P_n = The drag load due to negative skin friction

Q_u^{tip} = The tip resistance

$Q_{u,a}^{skin}$ = The positive skin resistance

However, a design must always lie within certain margins of safety. Due to the different nature of the factors in the above equation the method of partial factors of safety will be applied as follows.

Assume

$f_{P,p}$ = Partical factor of safety on the permanent working load, P_p

$f_{P,t}$ = Partial factor of safety on the transient working load, P_t

$f_{P,n}$ = Partial factor of safety on the drag load, P_n

f_Q = Partial factor of safety on the ultimate bearing capacity of the pile, Q_u

When these partial factors of safety are applied the Eq. (2 b) becomes

$$f_{P,p}\ P_p \leq \frac{1}{f_Q}\ (Q_u^{tip} + Q_{u,a}^{skin}) - f_{P,n}\ P_n \qquad (3\ a)$$

which, as mentioned, is valid if

$$f_{P,t}\ P_t \leqq 2\ P_n \qquad (3\ b)$$

In Eq. (2 b) it is assumed that the positive skin friction in the soft settling soil, $Q_{u,a}^{skin}$, is equal to the negative skin friction, P_n, in the same soil. (See Fig. 24). This is, however, not quite correct, but this simplifying assumption is believed to be justified for practical design purposes.

As the drag load is determined by a calculation on the safe side, its partial factor of safety should on most occasions not exceed 1.00. The other partial factors of safety cannot be generally stated but must be chosen according to the requirements in each case.

When the transient load is larger than twice the drag load, the bearing capacity of the pile must also be checked for the total working load which is acting on the pile. However, positive skin friction will then develop along the entire length of the pile and the Eq. (3 a) becomes

$$f_{p,t}\ P_t + f_{P,p}\ P_p \leq \frac{1}{f_Q}\ Q_u \qquad (4)$$

When designing piles of moderate lengths the recommended approach is simple and applicable on both friction piles and end-bearing piles. However, when the piles are longer or rather, when the settling layer is thicker than about 40 metres, the estimated drag load, P_n, can be much greater than the permanent working load, P_p. Furthermore, the end-resistance of the pile becomes more difficult to estimate. Thus, the calculated difference between the bearing capacity and the drag load may become small, of the same order as the errors that are involved in the estimations. Consequently, the design equation will not be practical. Instead, the safety factor with respect to negative skin friction can be checked by the condition that the permanent and transient loads should be smaller or equal to the bearing capacity of the full length of the pile shaft, i.e.

$$f_{P,p}\ P_p + f_{P,t}\ P_t \leq \frac{1}{f'_Q}\ Q_u^{skin} \qquad (5)$$

The safety factor f'_Q can normally be smaller than the factor f_Q which would be applied in the previous equations.

This approach is especially recommended for long piles driven through soft clay to end-bearing on bedrock or in firm soil strata. By the approach the safety against a possible collapse of the pile is studied, should the end-bearing capacity of the pile be destroyed by the effect of negative skin friction. However, large settlements will occur if the end-bearing capacity is reduced. Therefore, it is important that the piles are driven to a high end-bearing capacity. Consequently, much emphasis will have to be put on the inspection of the driving. Also, it may be desirable to provide the piles with a rock-shoe to ensure a sound tip penetration into the bearing soil layers. Often a more slender pile is to be preferred to a pile of larger diameter as it is normally easier to drive the slender piles to a high end-bearing capacity. Of course, the calculated maximum load in the piles must not exceed the strength of the pile section.

6. REDUCTION OF NEGATIVE SKIN FRICTION ON PILES

When considering negative skin friction in a design there will be cases when the recommendations in Chapter 5 (Part II) cannot be applied. For instance, when the drag load is judged to be very large, or when only very small settlements can be accepted, the negative skin friction must be reduced.

This problem can be approached by changing the foundation system, for example, by using caissons instead of piles. Then the drag load will be small relative to the strength and bearing capacity of the "pile". (The negative skin friction increases linearily with the pile radius, but the pile area increases by the square of the radius). However , the method may often be uneconomical.

The negative skin friction on driven slender piles can be reduced by two methods. The skin friction of steel piles can be reduced by electro-osmosis. However, this method calls for continuous inspection of the piles. Furthermore, the method cannot be used for precast concrete piles unless provision is made for conducting the electrical current down through the skin of the pile.

Bjerrum et al. (1969) have employed electro-osmosis on steel piles and compared the reduction effect to the method of coating the pile with bitumen. The latter method was proved to be more efficient than electro-osmosis. In the investigation an approximately 1 mm thick coat of bitumen with penetration 80/100 was applied to one pile. When comparing the results obtained from this pile with the result from an unprotected pile, it was found that the drag load was reduced by more than 90 %.

The bitumen coat acts approximately like a viscous fluid, i.e. the magnitude of shearing stress that can be transferred from the settling soil to the pile through the bitumen coat is dependent on the settlement rate. A laboratory investigation shearing soft clay along a concrete surface coated with 1.0 mm bitumen with penetration 120 has shown that even this relatively hard bitumen will effectively reduce the shearing force (Fellenius, 1970c).

The bitumen can be choosen among a wide range of penetration values. The important factor is really to ensure that after the installation of a coated pile, the coat must be intact. It must be hard enough to resist the scraping effect of the soil during the driving and soft enough not to peel of during the driving due to the induced dynamic shocks. In a hot summer day, there is a risk that the bitumen flows off the pile and in a cold winter day the bitumen can become brittle and may crack or peel during driving. If the pile surface is wet, the bitumen will not adhere properly unless a primer is used.

Protection against negative skin friction with a bitumen coat is mainly a practical problem. Furthermore, it can involve large extra costs and it is recommended that this method should not to be employed unless proven to be necessary by a detailed study.

7. CONCLUSIONS

The maximum drag load can be estimated approximately by assuming that the negative skin friction is equal to the shear strength of the surrounding soil. When designing single piles with respect to negative skin friction, the approach discussed in Chapter 5 (Part II) is recommended of this approach. The main feature is that the difference between pile behaviour for loads due to permanent and transient loads on the pile head and drag loads is considered.

When applying the design approach to a group of piles, it is recommended that piles of different lengths in the same pile foundation should be avoided and friction-bearing piles should not be used in combination with end-bearing piles.

The design of floating piles and of end-bearing piles of moderate lengths (about 40 m) according to Eqs. (3 a, 3 b) and (4) is relatively simple. Designing longer end-bearing piles is more complicated and then Eq. (5) provides a more practical approach. The conditions for Eq. (5) are that the piles must be driven to a high end-bearing capacity and that the deviations of the piles are within acceptable bending tolerances.

When it is evident that the drag load due to negative skin friction cannot be accepted, the negative friction can be reduced effectively by applying a thin coat of bitumen on the surface of the piles.

8. SUGGESTION FOR FURTHER RESEARCH

The investigation which is reported in Part I of this report, has shed light upon the effect of negative skin friction on the behavior of long single vertical piles in soft clay. The continued investigation will provide further valuable information. However, the investigation will provide no information regarding negative skin friction on piles in stiff clays, on battered piles, etc. In particular the effect of negative skin friction on piles in a group considering the interaction between the foundation and the piles will have to be carefully studied before a full understanding of the problem of piled foundations in settling soil can be reached. Furthermore, it would be very valuable if a thorough field investigation concerning the end-bearing capacity of long piles could be performed as the correct bearing capacity of the pile tip, the factor Q_u^{tip}, is of great importance for the design formulas in Chapter 5 (Part II).

ACKNOWLEDGEMENTS

This project was carried out in co-operation with the Axel Johnson Institute for Industrial Research and was supported financially by the Swedish Council for Building Research. Also the Pile Commission of the Royal Swedish Academy of Engineering Sciences participated in the project.

The Author wishes to acknowledge the help of Mr. Thomas Haagen and Mr. Rune Adolfsson of the Axel Johnson Institute who were responsible for the successful development of the pile force gauge without which this project would not have been possible.
Mr Haagen is thanked especially for his continued personal interest and assistance in the planning and organizing of the test. Also the Author wishes to thank Mr. Anders Janzon of the Municipal Office for Public Works, Gothenburg who carried out the readings and the necessary field inspection.

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