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Down-drag on Piles in Clay due to Negative Skin Friction

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In *Part I* of this report the results are given from 43 months of measurements of forces and bending moments on two instrumented precast piles driven through 40 m (130 ft) of soft clay and 15 m (50 ft) into underlying silt and sand. The force in the piles increased due to negative skin friction. After the first 5 months a force of nearly 40 tons was observed at the bottom of the clay layer. During this time the reconsolidation of the clay after the driving took place. The force due to the reconsolidation effect amounted to about 30 tons, while the rest was due mainly to negative skin friction caused by a small regional settlement. The latter force increased linearly with time by about 15 tons per year. Seventeen months after the driving the pile heads were loaded with 44 tons and one year later another 36 tons were added. The load on the pile head eliminated the negative skin friction, which however started to return with the continued regional settlements.

In *Part II* of the report general design formulae for piles considering negative skin friction are given. The formulae should be used to check that the permanent and transient working loads, which have been chosen according to ordinary design rules, are not too large when negative skin friction develops.

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.

Dans la *Section I* de ce rapport on présente les résultats de 43 mois de mesure des forces et moments fléchissants sur deux pieux de béton préfabriqué instrumentés, foncés à travers 40 m d'argile molle jusqu'à 15 m dans le sable et silt sous jascent. La force dans les pieux a augmenté du fait du frottement négatif. Après les cinq premiers mois une force d'environ 40 tonnes était mesurée à la base de la couche d'argile. Pendant cette période la reconsolidation de l'argile après battage s'est développée. La force attribuable à cette reconsolidation a été de l'ordre de 30 tonnes, le reste étant dû essentiellement à du frottement négatif produit par un tassement régional limité. Cette dernière force a augmenté linéairement en fonction du temps à raison d'environ 15 tonnes par an. Dix-sept mois après le battage, les têtes de pieux ont été soumises à une charge de 44 tonnes, et un an plus tard à une charge additionnelle de 36 tonnes. L'application de la charge en tête a provoqué la disparition momentanée du frottement négatif qui a cependant recommencé à se développer par suite de la progression du tassement régional.

Dans la *Section II* on présente des formules générales de calcul des pieux tenant compte du frottement négatif. Les formules devraient servir à vérifier que les charges permanentes et transitoires, choisies selon les règles de design habituelles, sont acceptables lorsque le frottement négatif peut se développer.

Lorsque les tassements dus au frottement négatif sont inacceptables, ce frottement peut être réduit par utilisation d'un revêtement bitumineux sur le pieu. Référence est alors faite aux études concernant la réduction du frottement négatif et les difficultés pratiques qui y sont reliées.

[Traduit par le journal]

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I. Some Results of a Full Scale Test

Introduction

In June 1968 a full scale pile test started in southwestern Sweden on two instrumented precast concrete piles each about 55-m (180-ft) long. The aim of the test was to study negative skin friction on long piles in clay. This paper presents the results obtained until January 1972, 43 months (1300 days) of measurements. Preliminary information on the test has previously been published. Fellenius and Haagen (1969) describe the special, very accurate pile-force gauge being used for measurements of axial loads and bending moments in the test piles. In addition, the observations during the first 5 months of measurements are presented by Fellenius and Broms (1969), together with details on site conditions, pile types, driving data, instrumentation etc.

The program for the test consists of two phases of study.

Phase 1. Influence of the driving and the following reconsolidation of the clay.

Phase 2. Influence of load applied on the head of the piles.

The results, which were obtained from Phase 1 (495 days) and during the first 805 days of Phase 2, are described in this paper. This paper includes only the main and a few typical measurement results and previously published results and discussions are omitted. (Complete results for the first 28 months are presented in a proceeding by the Swedish Geotechnical Institute, Fellenius 1971).

General

The test site is located in southwestern Sweden. The soil consists of 40 m (130 ft) of soft normally consolidated marine clay which is underlain by silt and sand. From 35- to 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 by the Swedish fall-cone test and is normally approximately equal to the liquid limit, Karlsson 1961). The percentage of particles smaller than 0.002 mm in the clay is about 80% to a depth of 20 m (65 ft) and decreases to about 55% between 20 and 30 m. The sensitivity of the clay varies between 15 and 20 which is normal for Swedish clays. Oedometer tests show that the compression index (C_c) of the clay is 1.1 to 1.6 to about 30-m depth. Below this depth the compression index is about 0.8.

Two precast hexagonal Herkules piles of reinforced concrete were used for the investigation, each with a cross sectional area of 800 cm^2 (H 800) and a circumference of 105 cm (124 in.^{2} and 41 in., respectively). The piles are composed of 11.2-m (36.7-ft) long segments. The bottom segment is provided with a rock point of hardened steel. In between seven of the pile splices an accurate pile force gauge was placed prior to the driving. The pile force gauges measure axial forces and bending moments in the pile. Figure 2 shows a section of the piles and the soil with the location of the various gauges. Figure 3 shows the location in plan. For further information on the pile type and driving data as well as on instrumentation of piles and of soil see Fellenius and Broms (1969).

Results

The observed settlement of the ground surface and the relative movements within the clay during the 43 months measuring period are shown in Fig. 4. The movements were small, as an average only 2–3 mm (0.1 in.) per year, but for a 15 mm (0.6 in.) settlement which occurred during the fall of 1969 coinciding with a very severe drought during the summer of that same year. No movements were observed below the depth of about 20 m (65 ft). The measurement results shown in Fig. 4 are representative for the area where the test site is situated.

The settlement of the pile heads was small. In connection with the loading of the pile heads 2–3 mm settlement occurred, which corresponds to the measured axial compression of the piles.

In Fig. 5 an example is given of the observed pore pressures. (One of the piezometers located at 22.3-m (73-ft) depth near pile P1.) The pile driving caused large excess pore pressures which locally exceeded the effective overburden pressure. However, these pressures dissipated with time at a rate which was approximately equal for all gauges. All

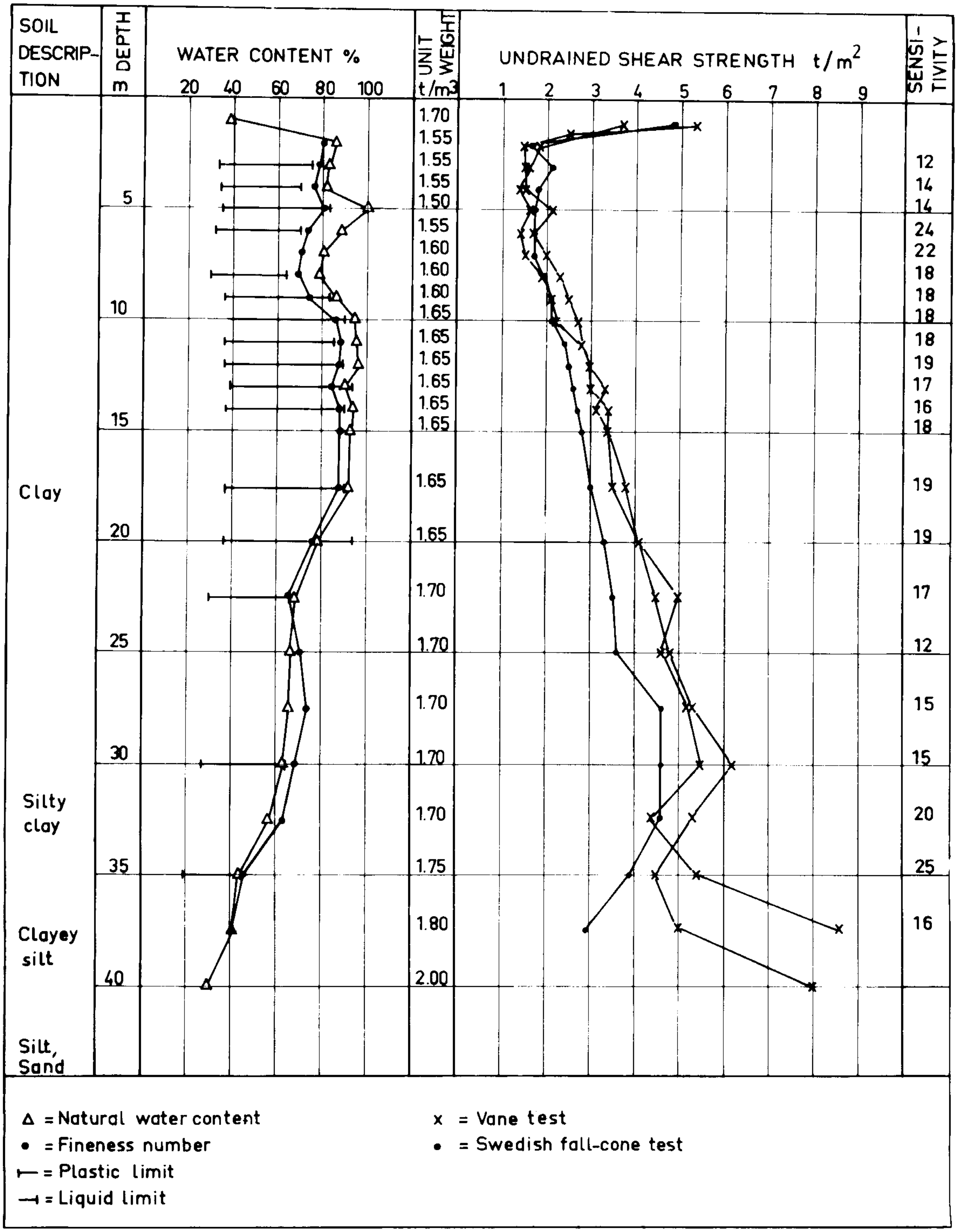

FIG. 1. Soil data.

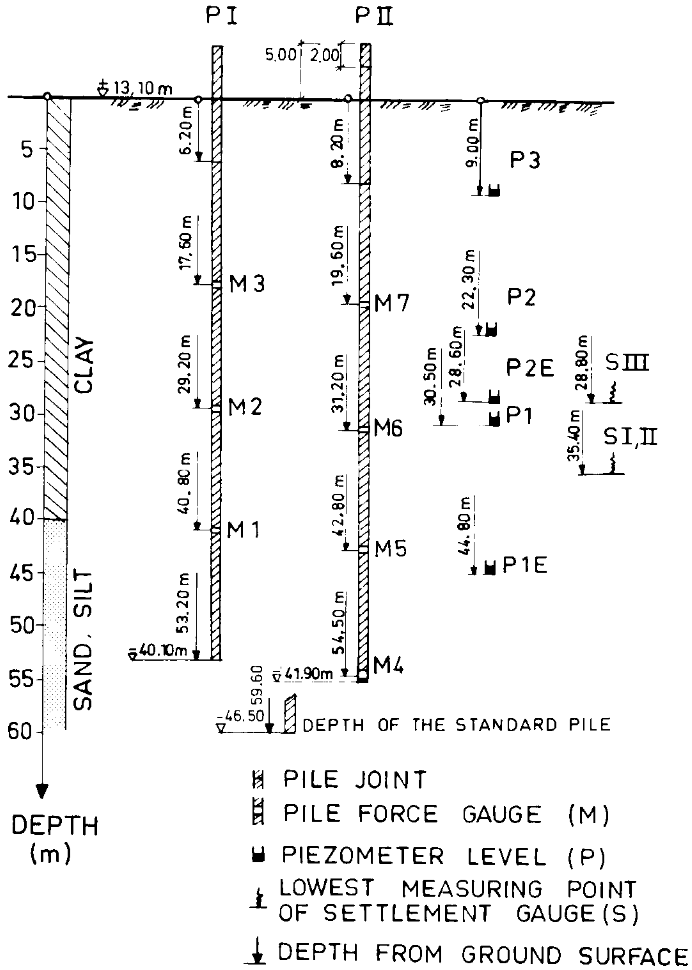

FIG. 2. Piles and instrumentation. Section.

excess pore pressures had dissipated after about 150 days or 5 months.

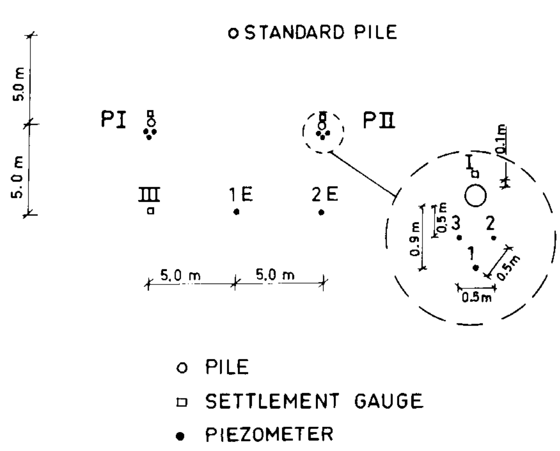

FIG. 3. Location of piles and instrumentation. Plan.

The observed development of axial forces in the piles is presented in Fig. 6. The forces in the piles immediately after the driving corresponded approximately to the weight of the pile in air. The forces then increased, rapidly at first and after about 5–7 months the rate of load increase was approximately linear. The average rate of force increase for gauges M1 and M5 located at the bottom of the clay layer in piles PI and PII is shown as a straight line in the figure.

The slow but constant load increase in the piles after the first 180 days is due to the small amount of regional settlement. After 495 days the total drag load was about 55 tons and the corresponding negative friction was equal to about 30% of the undrained shear strength, or to about 10% of the effective overburden pressure.

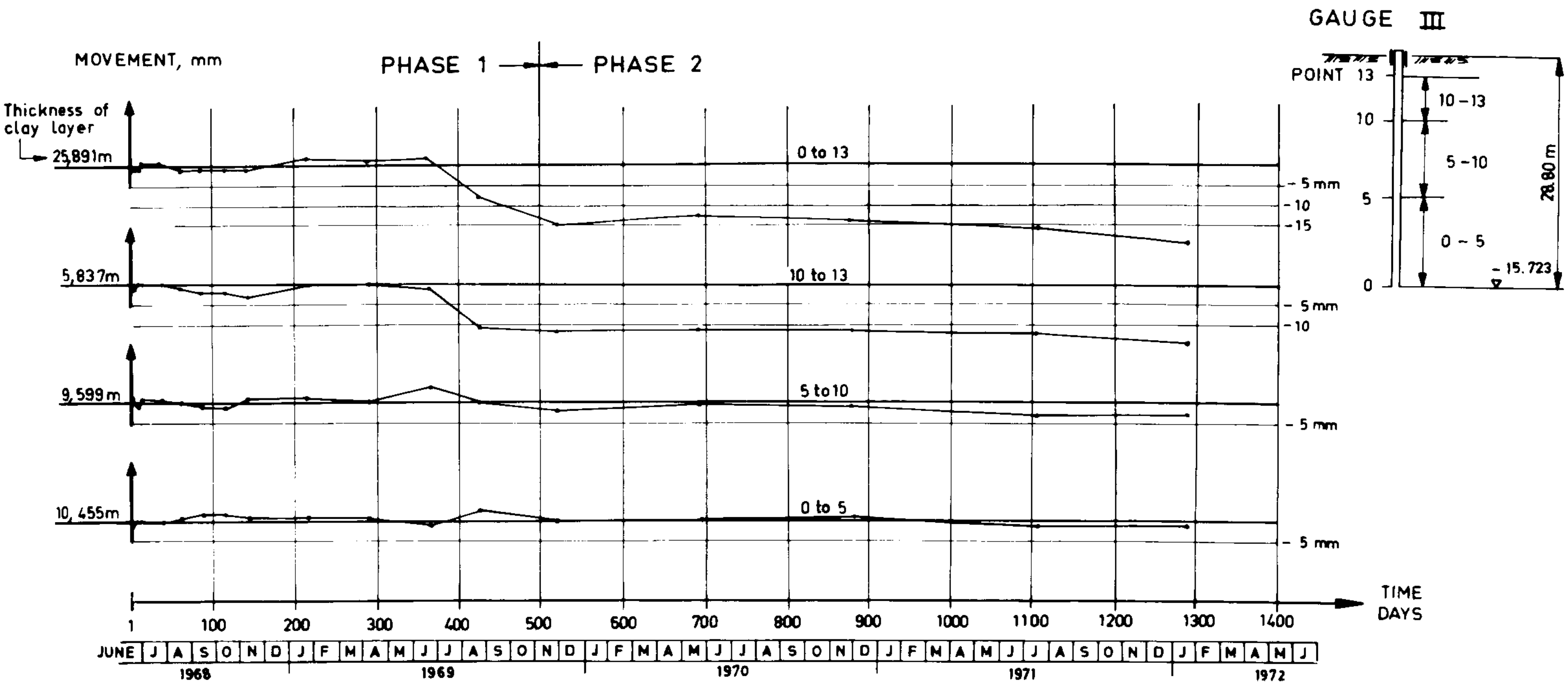

FIG. 4. Settlement of the ground surface and relative movements within the clay.

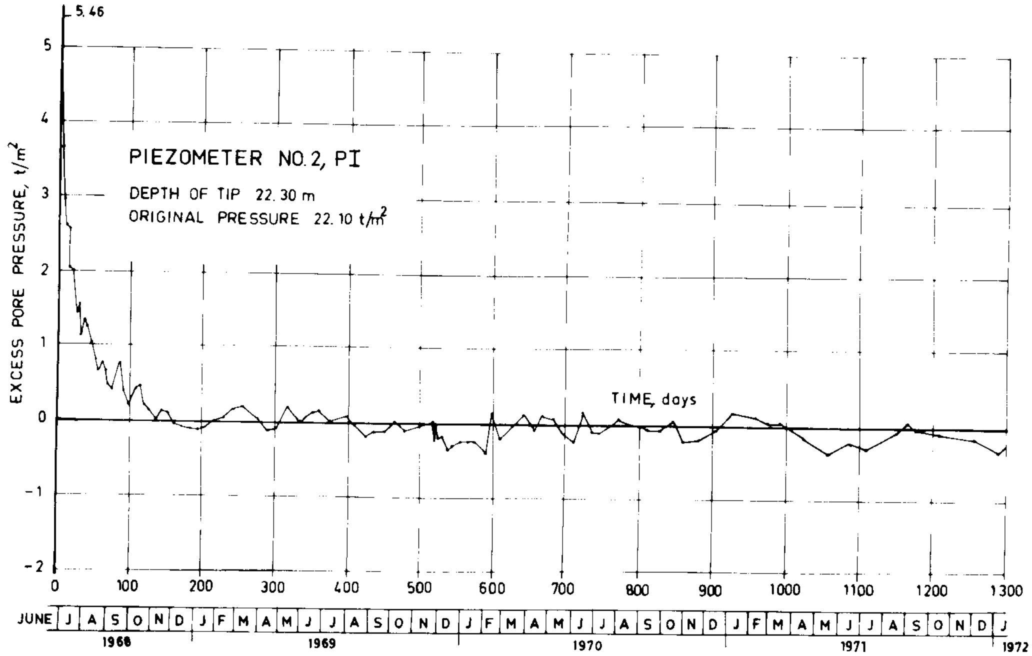

FIG. 5. Pore water pressure in the soil related to the original pressure.

During the first 180 days after the driving, the clay reconsolidated with dissipation of excess pore pressures and consequently additional settlements were obtained near the piles. The relatively rapid load increase during this period was caused mainly by negative skin friction which developed during the reconsolidation of the clay. The total drag load at 180 days was about 40 tons.[2] The settlements which caused this drag load were small, of the order of 2 to 3 mm (0.1 in.). By prolonging the line of average rate of force increase to intersection with the vertical axis an estimate can be made of the drag load caused by the driving and subsequent reconsolidation of the clay alone. The intersection occurs at 35 tons load. Reducing this value with the net weight of the pile, about 5 tons, the rest (30 tons) is the drag load caused by the pile driving.

The average negative skin friction calculated over the area of pile between the force gauges is shown in Fig. 7 for measurements taken 120, 300 and 480 days after the driving. The results indicate that the negative skin friction (τ_{neg}) increased linearly with depth. Also, in this figure the undrained shear strength (τ_{fu}) is shown for comparison.

At 495 days the piles were loaded by a 44 ton concrete slab cast on the pile heads. After another year of measurements the piles were loaded to a total load of 80 tons by 36 tons of concrete blocks which were added on the slab. The three photos in Fig. 8 show the loading arrangement. Figures 9*a* and 9*b* show the vertical load distribution in the two test piles at various times after the driving. The applied loads of 44 and 36 tons, respectively, were transmitted almost straight down the piles. For the rapid loading to 80 tons a small portion of positive skin friction was obtained. In both cases the negative skin friction in the upper two thirds of the piles was eliminated. The load at the bottom of the clay layer was only slightly affected. By comparing the approximately parallel lines of average rate of force increase in Fig. 6 it can be seen that the force at this depth increased by 5–8 tons. The force at the pile tip was not affected as obviously the increasing force in the piles was resisted by positive skin friction in the silt and sand.

The measurements of bending moments in the piles show a distinct difference in pile

[2]Metric tons.

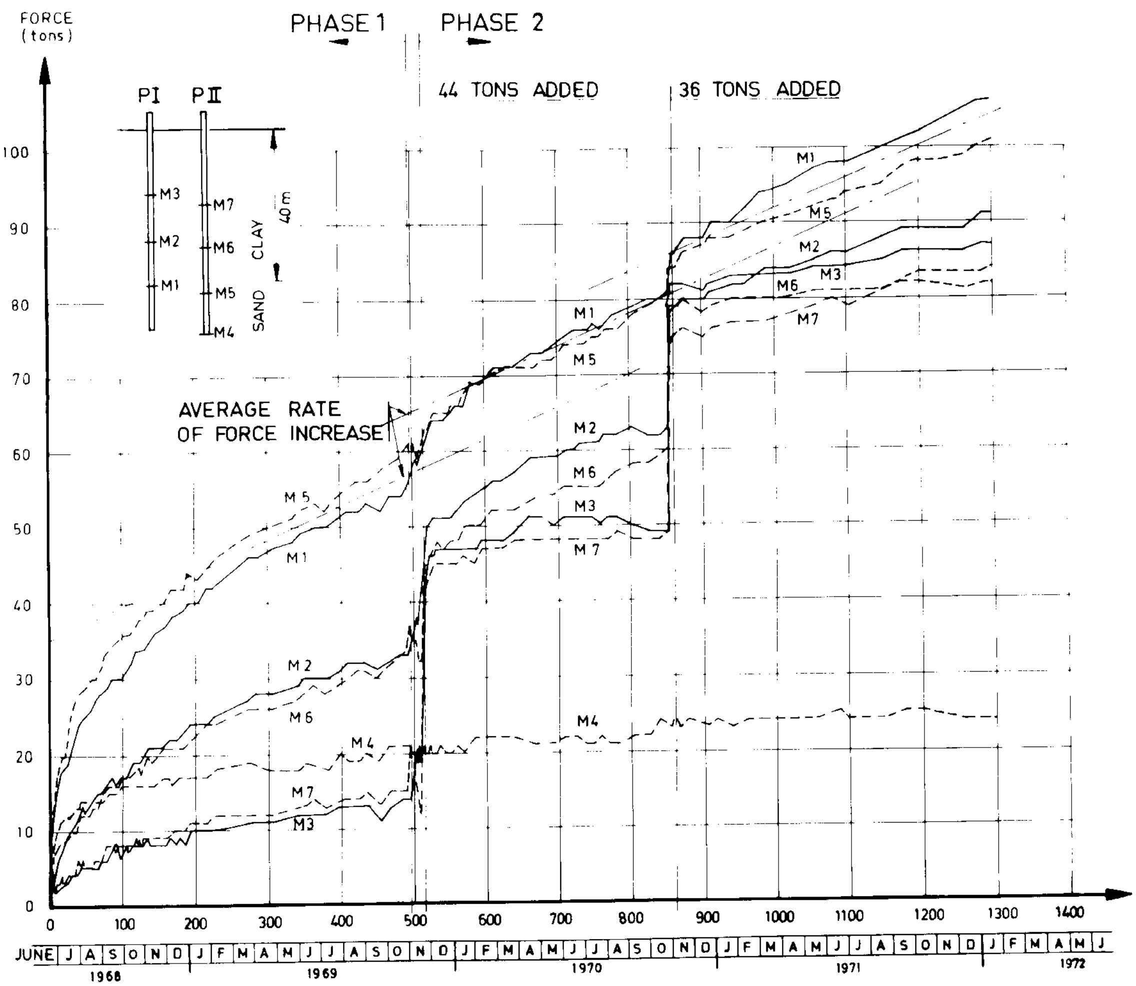

FIG. 6. Development of axial forces in the piles *versus* days after driving.

behavior 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. Despite the increasing drag loads the bending moments even decreased a little at some gauges. The measurements of bending moments during Phase 1, 150–495 days, during Phase 2, 520–859 days and during 859–1300 days after the driving of the piles are presented in the following table.

Pile PI was driven straight and pile PII was bent. The bending radius of the pile segments varied from 600 to 2700 m and 230 to 360 m for PI and PII, respectively.

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 gradual reconsolidation of the clay and dissipation of the excess pore pressures which follow cause small additional settlements of the clay and consequently load is transferred to the piles due to negative skin friction.

In this case the reconsolidation time was about 5 months after which time a drag load of 40 tons was observed. About 30 tons of this load is estimated to be caused by the reconsolidation alone and the rest is caused by the small regional settlement.

When friction piles in clay are test loaded one often finds that two piles which have

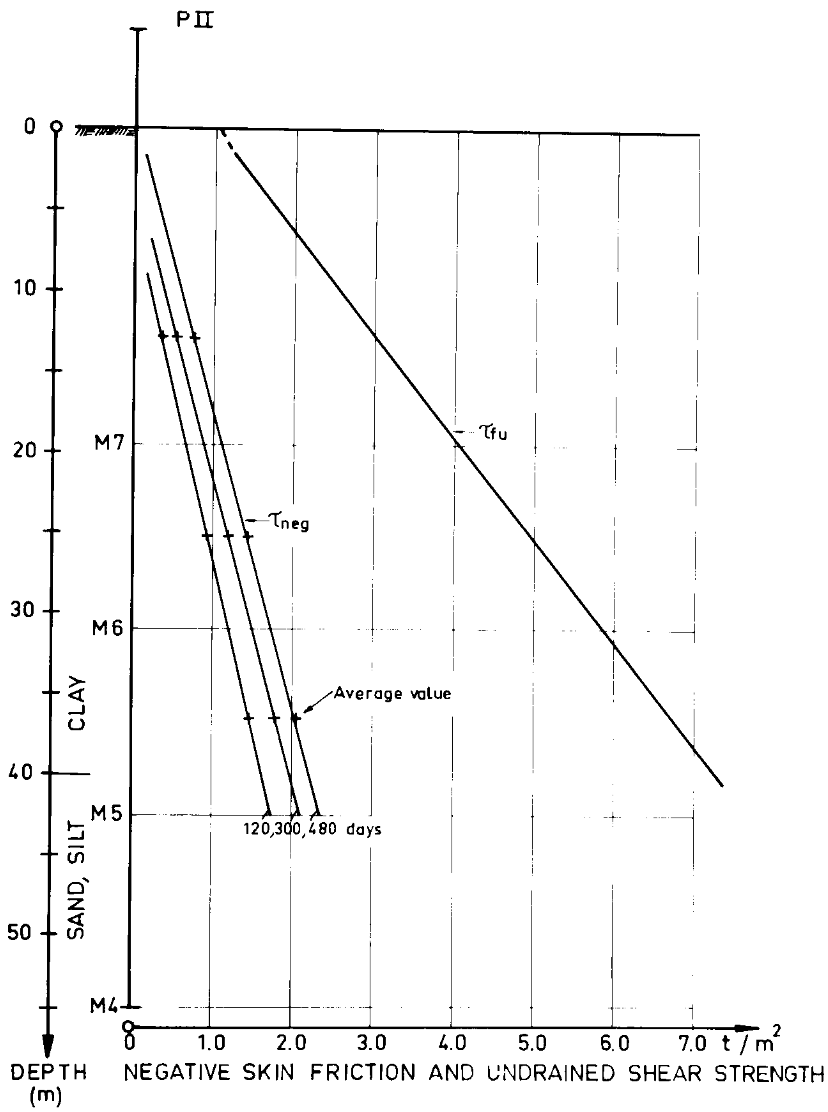

FIG. 7. Negative skin friction (τ_{neg}) of pile P II at 120, 300 and 480 days after the driving compared to the original undrained shear strength (τ_{fu}) of the clay.

approximately the same bearing capacity or ultimate resistance, may show considerable difference in load deformation characteristics before reaching the ultimate load. The results of this investigation suggest an explanation to this behavior; 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.

Negative skin friction can be caused by extremely small settlements. In this case the force in the piles at 40-m (130-ft) depth increased by about 15 tons per year. The corresponding settlement of the ground surface was 2–3 mm (0.1 in.) per year. Below 20-m (65-ft) depth no settlement was observed within the measuring range (1 mm) of the settlement gauges. When a load is applied on the head of the piles, only the part of the load exceeding the drag load will increase the load in the lower part of the pile. However, the negative skin friction which is obtained later will, in full, be added to the previous load in the pile.

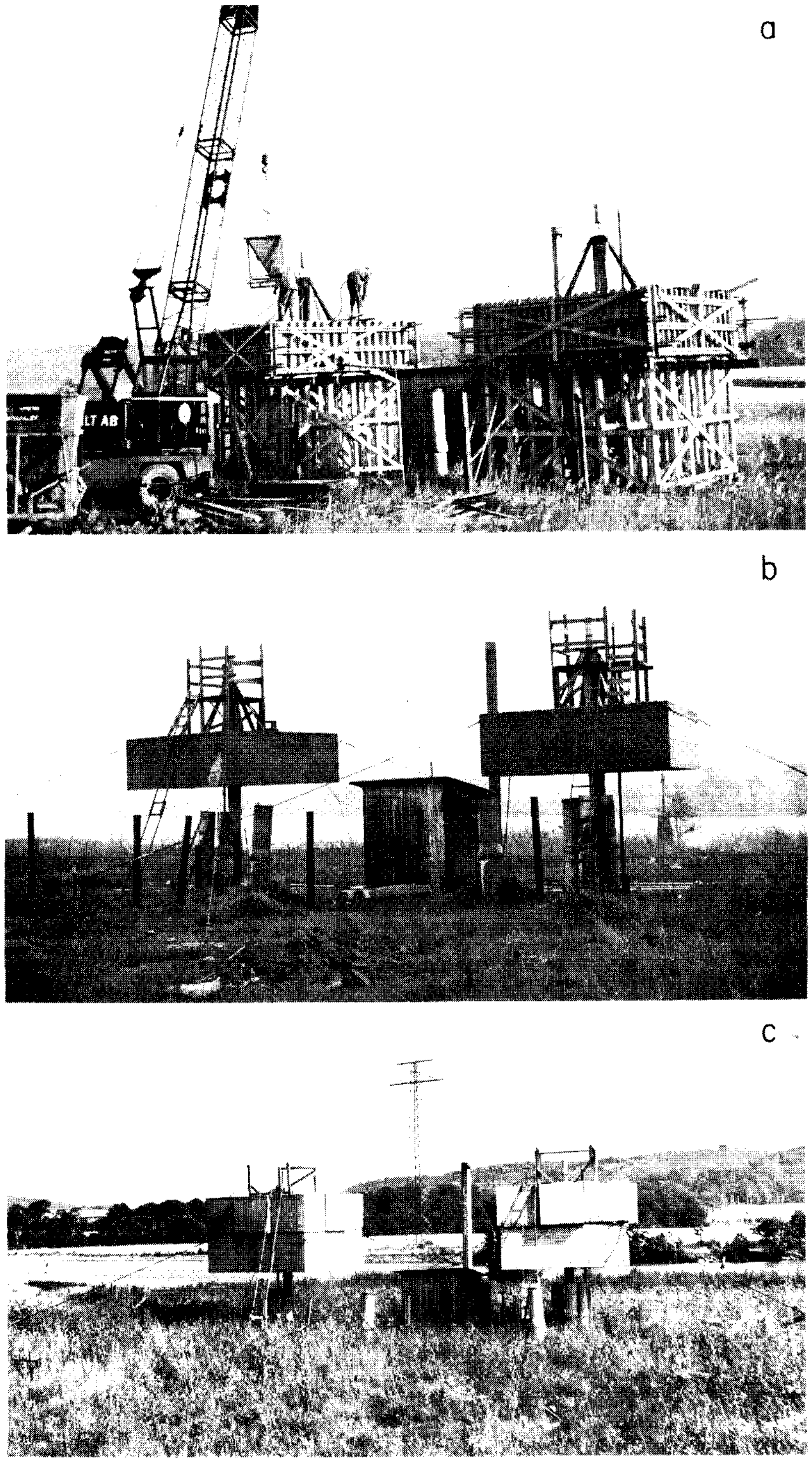

FIG. 8. Arrangement for loading of Phase 2: 8*a*. Casting of concrete at 495 days after driving, 8*b*. After removal of the bracings at 516 days after driving, and 8*c*. After adding the concrete blocks at 859 days after the driving.

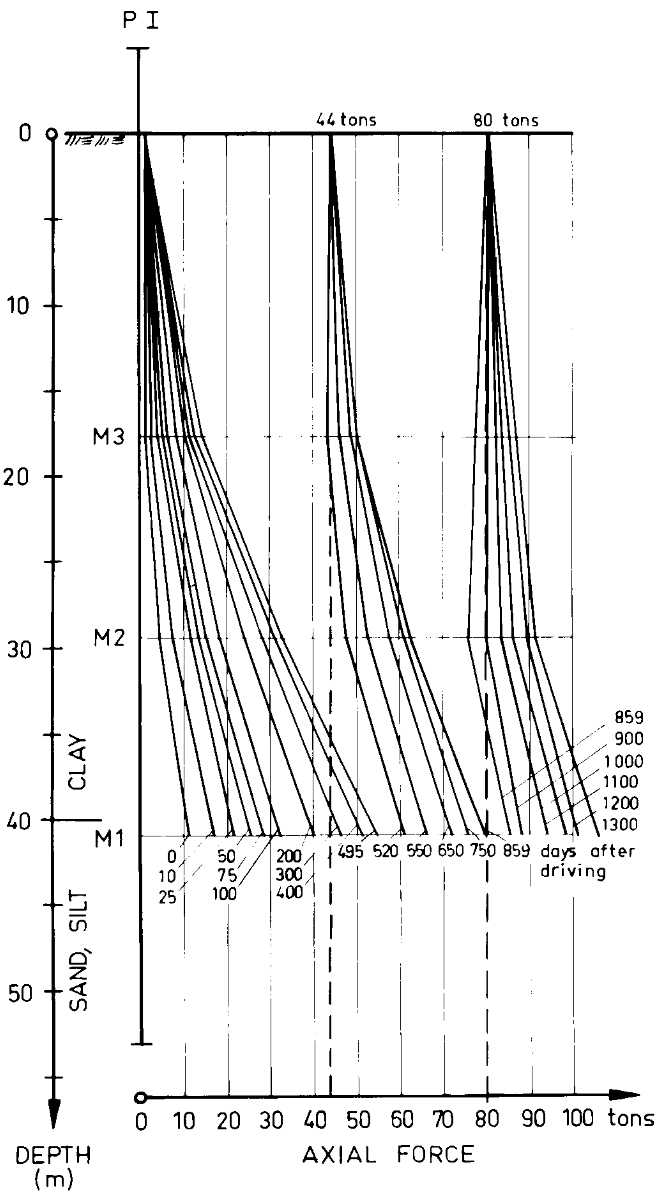

FIG. 9*a*. Vertical load distribution in pile P I at various times after the driving.

II. Design Considerations

The following is an abbreviation of general views and design considerations presented in the previously mentioned complete report (Fellenius 1971). Views on pile groups, battered and bent piles have been omitted and the discussion is concentrated on the design of single vertical piles.

Allowable Load 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

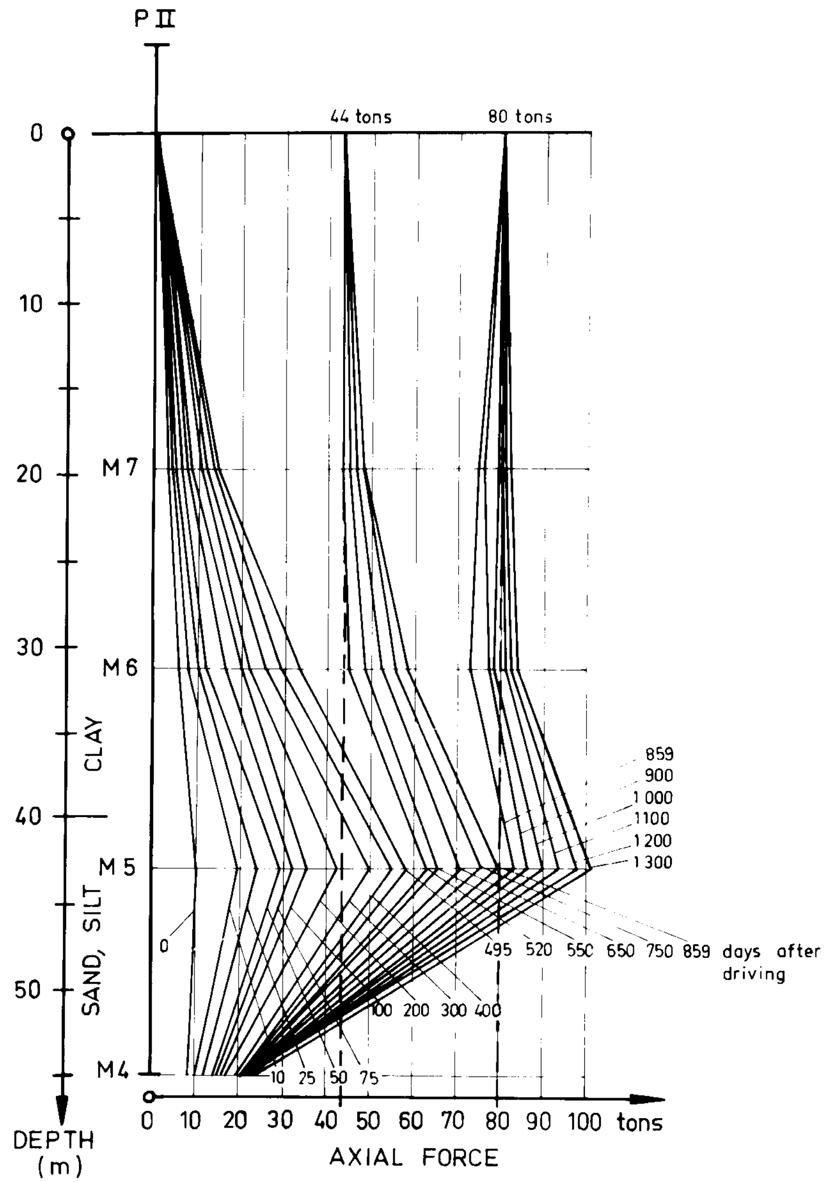

FIG. 9*b*. Vertical load distribution in pile P II at various times after the driving.

been arrived at by ordinary methods, is not too large when negative skin friction is considered. Such methods are the requirements of the Building Code, settlement considerations, the quality of the piles that are to be used etc. The main feature of the recommended approach 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 this approach should be performed in cooperation with an experienced soils engineer.

First, the ultimate bearing capacity, Q_u, of the pile is estimated (See Fig. 10). This estimate can be obtained by a calculation from known soil data or from a load test. However, no positive skin resistance ($Q^{\mathrm{skin}}_{u,b}$) from the upper soil layers settling around the pile should be included in the estimate. Thus, the bearing capacity will consist of the tip resistance, Q^{tip}_u, and the positive skin

TABLE 1. Forces (tons) and bending moments (ton m) during Phase I and Phase 2

Time	Load at head	Gauge M3		Gauge M7		Gauge M2		Gauge M6	
		Force	Mom	Force	Mom	Force	Mom	Force	Mom
Phase I 150 days	0	8	0.4	9	2.3	21	1.7	21	1.5
Phase I 495 days	0	14	0.4	15	2.3	33	1.7	34	1.5
Phase II 520 days	44	44	0.7	44	3.3	48	2.0	46	1.6
Phase II 859 days	44	49	0.7	48	3.2	62	2.0	59	1.6
Phase II 859 days	80	79	0.8	75	3.7	78	2.1	73	1.7
Phase II 1300 days	80	87	0.8	82	3.5	91	2.0	83	1.7

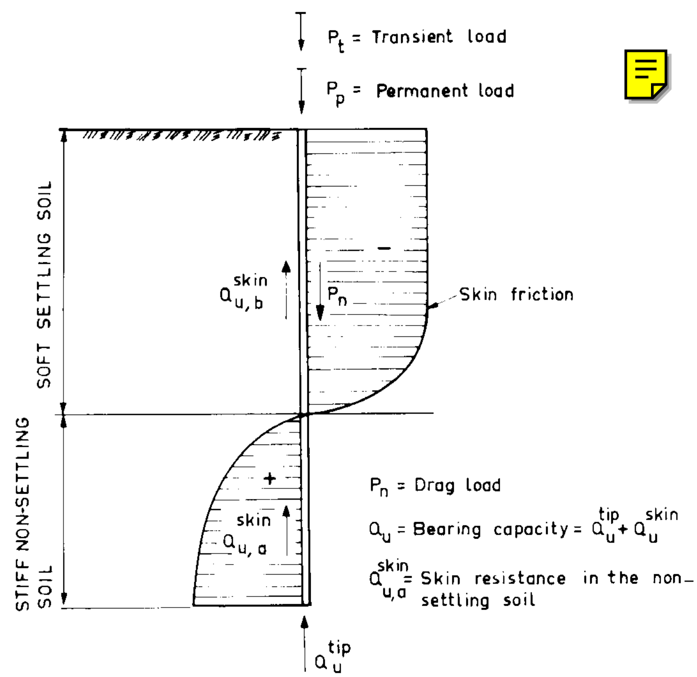

FIG. 10. Unit skin friction distribution along a pile in an upper layer of soft settling soil and a lower layer of non-settling soil.

resistance, $Q_{u,a}^{\text{skin}}$, from the part of the pile that is in the nonsettling soil.

When the bearing capacity, $Q_u^{\text{tip}} + Q_{u,a}^{\text{skin}}$, is determined, the drag load due to negative skin friction, P_n, in the settling soil is also estimated. This drag load cannot be accurately calculated. 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. Thus, only the permanent load, P_p, on the pile head has to be considered. The following equation applies

$$[1a] \qquad P_p + P_n \leq Q_u^{\text{tip}} + Q_{u,a}^{\text{skin}}$$

or

$$[1b] \qquad P_p \leq Q_u^{\text{tip}} + Q_{u,a}^{\text{skin}} - P_n$$

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, and $Q_{u,a}^{\text{skin}}$ = the positive skin resistance.

However, a design must always lie within certain margins of safety. Clark *et al.* (1966) have proposed an overall safety factor, F, to be applied.

$$[1c] \qquad P \leq \frac{1}{F}\left(Q_u^{\text{tip}} + Q_{u,a}^{\text{skin}} - P_n\right)$$

By this safety factor the drag load is reduced together with the bearing capacity. The method of an overall safety factor can therefore not be applied.

Due to the different nature of the factors in the above equation the method of partial factors of safety will have to be applied as follows.

Assume: $f_{P,p}$ = partial 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; and f_Q = partial factor of safety on the ultimate bearing capacity of the pile, Q_u.

When these partial factors of safety are applied Eq. [1*b*] becomes

$$[2a] \qquad f_{P,p}\, P_p \leq \frac{1}{f_Q}\left(Q_u^{\text{tip}} + Q_{u,a}^{\text{skin}}\right) - f_{P,n}\, P_n$$

which, as mentioned, is valid if

$$[2b] \qquad f'_{P,t}\, P_t < 2P_n$$

In the equations it is assumed that the positive skin friction in the soft settling soil, $Q_{u,b}^{\text{skin}}$, is equal to the negative skin friction, P_n, in the same soil (See Fig. 10). This is, however, not quite correct, but this simplifying assumption is believed to be justified for practical design purposes. Especially if the transient load is cyclically applied on the pile some portion of it may reach the pile end. Therefore it is recommended that the safety factor, $f'_{P,t}$, in Eq. [2*b*] be larger than the values which are normally chosen for partial factors of safety. Until further research results are available the recommended value is 2.00.

As the drag load is normally calculated 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 load acting on the pile. However, positive skin friction will then develop along the entire length of the pile and Eq. [2*a*] becomes

$$[3] \qquad f_{P,t}\, P_t + f_{P,p}\, P_p \leq \frac{1}{f_Q}\, Q_u$$

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 m, 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 than or equal to the bearing capacity of the full length of the pile shaft, *i.e.*

$$[4] \qquad f_{P,t}\, P_t + f_{P,p}\, P_p \leq \frac{1}{f'_Q}\, Q_u^{\text{skin}}$$

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

This approach, Eq. [4], is recommended especially 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 straight and to a high end-bearing capacity. Consequently, much emphasis will have to be placed on the inspection of the driving. Also, it may be desirable to provide the piles with a rock shoe to ensure a sound 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.

Reduction of Negative Skin Friction on Piles

When considering negative skin friction in a design there will be cases when the approach in the previous section indicates that the drag load is too large to be accepted or cases when only very small settlements can be accepted. Then 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 linearly 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 electroosmosis. 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 electroosmosis 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 electroosmosis. 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 1970). The bitumen can be chosen among a wide range of penetration values. The important factor is really to ensure that the coat is intact after the installation of a coated pile. 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 be employed unless proven to be necessary by a detailed study.

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 discussed approach is recommended. The main feature is that the difference between pile behavior for loads due to permanent and transient loads on the pile head and drag loads is considered.

The design of floating piles and of end-bearing piles of moderate lengths (less than about 40 m) according to Eqs. [2a], [2b] and [3] is relatively simple. Designing longer end-bearing piles is more complicated and then Eq. [4] provides a more practical approach. The conditions for Eq. [4] 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.

Acknowledgments

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