Measurement of the Compression of a Steel Pile to Rock due to Settlement of the Surrounding Clay

Mesure de la compression d'un pieu en acier appuyé sur le rocher due au tassement de l'argile environnante

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SUMMARY

The paper describes the measurements of the compression of a steel pile driven to rock. The compression is caused by the negative skin friction from the surrounding clay only. The clay is a soft to medium soft marine clay and under the weight of 10 m of fill it has settled about 1.2 m. From the shortening of the pile, it can be concluded that the stresses in the steel due to the negative skin friction are high and exceed the allowable design load near the point of the pile. An interpretation of the observed compression of the pile indicates that the developed adhesion between pile and clay is approximately distributed as the effective vertical stresses. A reasonably good agreement was obtained between observed and computed compression of the pile, when the negative skin friction was assumed equal to $\sigma'_v K \tan \phi'_a$, where σ'_v is the effective vertical stress. It was found that the ultimate value of $K \tan \phi'_a$ is of the order of magnitude 0.20.

SOMMAIRE

L'article décrit les mesures de compression causée par le frottement latéral négatif sur un pieu en acier enfoncé dans une couche argileuse jusqu'au fond rocheux. Le tassement de cette argile marine, molle à moyennement molle, sous le poids de 10 mètres de remblayage, s'élève à 1,2 mètre. On a observé un raccourcissement du pieu. On arrive à la conclusion que les contraintes dans le pieu, à cause du frottement latéral négatif, étaient élevées et ont dépassé la charge admissible à la pointe du pieu. Une interprétation de cette compression démontre, que l'adhérence de l'argile au pieu suit approximativement une distribution semblable à celle des contraintes efficaces verticales. On observe un accord raisonnable entre les valeurs calculées et les valeurs mesurées de la compression du pieu à la condition que le frottement négatif soit égal à $\sigma'_v K \tan \phi'_a$ (σ'_v = contrainte verticale efficace). La valeur ultime de $K \tan \phi'_a$ est approximativement 0,2.

SITE CONDITIONS

THE SITE FOR THE TWO TEST PILES was selected in an area where the water was about 8 m deep and which was going to be reclaimed by placement of about 10 m of fill (Fig. 1). The piles were driven in April, 1962, and the fill was placed in the following twelve months. At the site the bedrock is found at a depth of about 53 m below mean sea level. The rock is an Ordovician calcareous schist. Above the rock is found a thick deposit of late glacial marine clay. A geotechnical profile through the clay is shown in Fig. 2.

PILE TYPE AND INSTRUMENTATION

The two piles used for the measurements are hollow steel piles of the type Krupp KP 24 having a width of 47 cm (Fig. 3). The bottom of each pile was closed by a steel cap to which was fastened an Oslo point consisting of a hardened steel bolt, 10 cm in diameter (Bjerrum, 1957).

After a study of the reliability of various strain-measurement systems, it was decided to use a purely mechanical system to measure the strain in the piles. The principle of this system is to measure the shortening of the pile between the top of the pile and various points located at different depths inside the pile. This was done by having a system of guided steel rods leading from the top of the pile to the measuring points.

Within the selected cross-section it was possible to arrange 8 measurement points. In pile B, to which most emphasis will be given in this paper, a double set of measurement points were placed opposite each other at three elevations, −12.5, −24.5, and −53.5 m. At two elevations, −36.5 and −48.5 m, single measurement points were installed. In this way the effect of deflection of the pile could be eliminated at the three elevations with double measurement points.

The piles were driven in April, 1962. The driving was rather easy to an elevation of about −40 metres. The last 15-metre penetration required hard driving, however, and the number of blows per metre penetration exceeded 800. The installation in pile B remained intact during the driving. However, in pile A the lowest two steering pipes were damaged so that it proved impossible to instal the two lowest measurement rods. The measurements on pile A are therefore incomplete and will not be included in this paper.

SETTLEMENTS OF CLAY SURROUNDING THE PILES

After the driving of the piles, a number of settlement points and piezometers were installed in the clay next to the piles (Fig. 1). The settlement of the surface of the clay was measured at seven points, by settlement plates connected to the pipes through the fill. The settlements were also measured at different depths, by installation of anchors connected to the surface with steel wires. After all the installations were finished, a 4-metre-thick bottom layer of fill was placed around the test site, by dumping gravel from barges. The remaining part of the fill was then placed in the following months by end dumping from the already reclaimed area (Fig. 3a).

In May, 1963, the fill had reached the final elevation of 2.2 metres. Unfortunately, some dredging outside the reclaimed area caused a slide, which affected the test site. This slide caused a sudden settlement of the test area of about

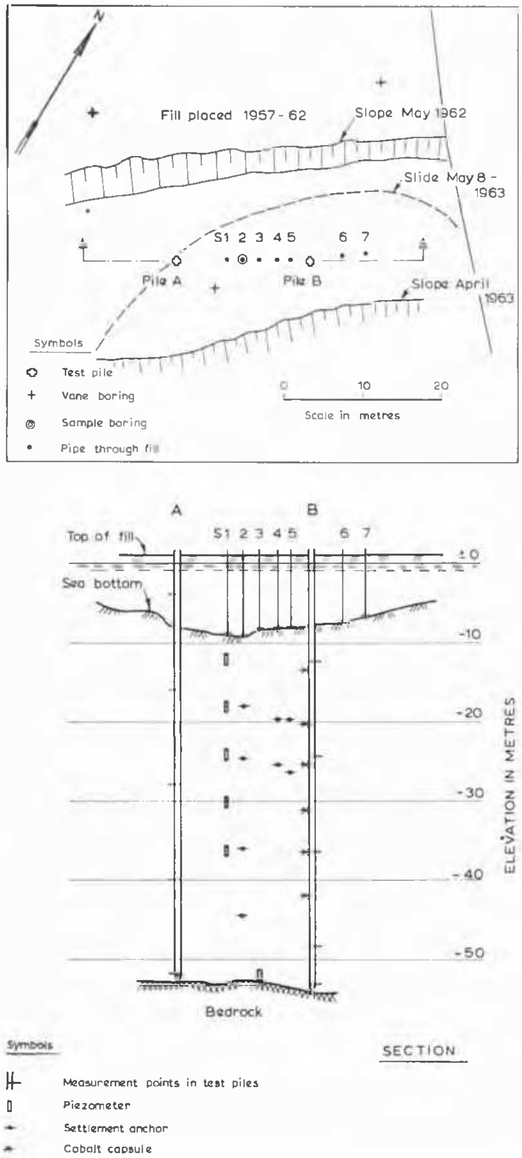

FIG. 1. Site plan and section.

0.5 m and also bending of the test piles. Inclinometer readings showed that test pile B had a bend at the elevation of about —11 metres, but remained straight above and below this elevation.

Fig. 3b shows the average settlement of the sea bottom observed during the two years' measuring period. The total settlement in April, 1964, was 1.7 m, of which about 1.2 m was due to consolidation of the clay and 0.5 m to the slide. The computed final primary consolidation settlement is 1.7 m, but it is very likely that the final settlements will exceed this value. Observations of the settlements at different depths indicate that the major part of the observed settlement is due to compression of the upper 10 m of the clay, and also that the lower clay layer above the rock consolidates rather quickly. The excess pore pressures increased as the area was loaded, their distribution being influenced by a marked drainage occurring at the bottom and at the top of the clay. Since the autumn of 1963, at which time the fill had been built up again after the slide to the final elevation, pore pressures have shown a slow reduction except in the middle part of the clay layer.

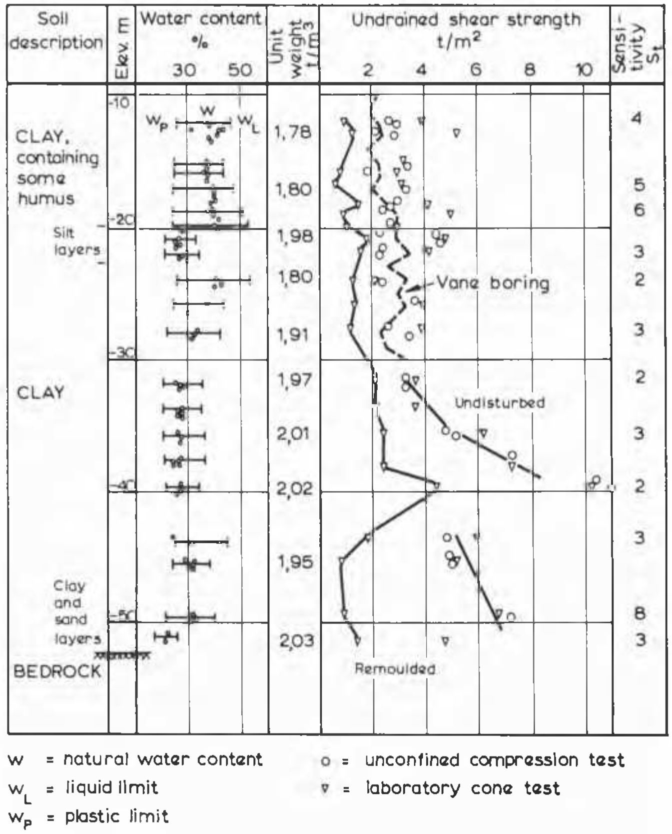

FIG. 2. Soil properties.

SHORTENING OF THE PILE

The directly observed shortenings between the pile top and the various measurement points are shown in Fig. 3c as a function of time. A comparison of the compressions observed on two measuring rods installed opposite each other at the same depth shows clearly that the pile has not remained straight, but that in addition to shortening it has undergone bending. As mentioned above, the bend is located at an elevation of —11 m and was caused by the slide. The shortening of the pile above the depths at which two measuring points were installed is equal to the average value of the two readings. At those depths where only one measuring point was installed, the determination of the shortening requires some assumptions concerning the bending. To get a reliable basis for making such assumptions the shape of the pile has been established by inclinometer measurements, permitting determination of the neutral axis of the bend.

In spite of the bend caused by the slide, it was possible to obtain what is believed to be an approximately correct picture of the compression of the pile and its distribution with depth. The results of the interpretation of the measurements are shown in Fig. 4d giving the accumulated shortening of the pile measured from bottom of the pile. As seen from this figure, the pile has undergone a gradual increase in compression amounting in April 1964 to a total shortening of 14.3 mm. It is also seen that the slide, which occurred between April and May, 1963, caused a temporary relief of the compression in the upper part of the pile.

When the curves in Fig. 4d showing the shortening of the pile at various times are considered in detail, it is observed that the compression of the lowest part of the pile just above rock varies irregularly, possibly due to bearing capacity

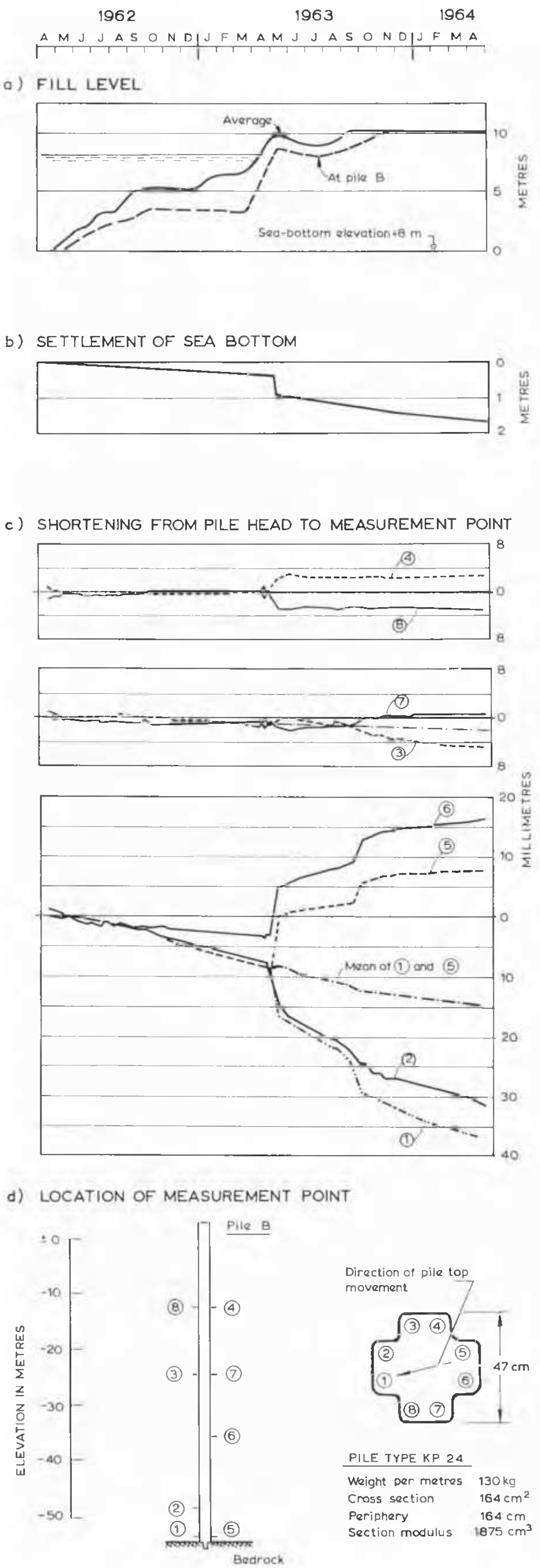

FIG. 3. Results of observations of settlements and compression of pile.

failure of the rock at the pile point. When the load on the pile point is built up to a certain value, it exceeds the bearing capacity of the rock, and the subsequent settlement of the pile point relative to rock causes a temporary release of stresses in the pile followed by an elongation of the lower part of the pile. As the pile point is pressed into the rock, its bearing capacity increases and the load on the point is built up again. This interpretation of the compression of the lower part of the pile appears to be confirmed by measurements of settlement of the pile top, which indicate that the pile point has settled about 6 cm.

STRESSES IN THE PILE

From the observed compression of the pile, the average stress in the pile between the individual measuring points can be computed. Such a computation is made from the readings taken in May, 1963, and April, 1964, and the points are plotted in Fig. 4c. Errors in measurement might have a considerable influence on the results, however. From Fig. 4c it can be seen that the average stresses observed in the steel increase with depth and reach an average value of about 900 kg/sq.cm. for the lower half of the pile in April, 1964. From the general shape of the compression curve, it can be concluded that the stresses in the pile, just above the pile point, are considerably greater than this value and might be of the order of 2,000 kg/sq.cm. The load at the pile point caused by the drag from the clay might very likely be of a magnitude of 250 tons, a value which easily can explain why the pile point has penetrated into the rock.

ADHESION BETWEEN CLAY AND PILE

To estimate the adhesion between clay and pile from the observed compression, a double derivation of the readings is required, and the accuracy of such a computation is therefore rather limited. It was thus preferred for the preparation of this paper to estimate the adhesion by making a reasonable assumption concerning the distribution of the adhesion along the pile, and to compare the compression, resulting from this assumption, with the direct measurements.

The assumption made concerning the distribution of the adhesion was that, in cases where the displacement between pile and clay is relatively large, the adhesion will be governed by the effective horizontal stresses on the pile. The effective horizontal stresses are assumed to be proportional to the effective vertical stresses, which are known. According to this assumption the adhesion τ_a is considered to vary along the pile according to the expression

$$\tau_a = \sigma'_h \tan \phi'_a = \sigma'_v K \tan \phi'_a$$

where $K \tan \phi'_a$ is assumed to be constant along the pile. This assumption was chosen after several unsuccessful attempts with a number of possible distributions. It proved to give a considerably better agreement with the observations than for instance an assumption which used a distribution of the adhesion corresponding to the variation of the undrained shear strength of the clay.

Fig. 4d shows the result of an interpretation of the observations, based on the assumption that the adhesion shows a distribution at depths equal to the distribution of the effective stresses. The comparison between computed and observed compressions of the pile has been made for two typical sets of readings taken in May, 1963, and April, 1964. In both cases, the effective stresses are computed from the observed piezometer observations. The method of computation has been first to calculate the load in the pile and the stresses in the steel, assuming that the adhesion is proportional to the

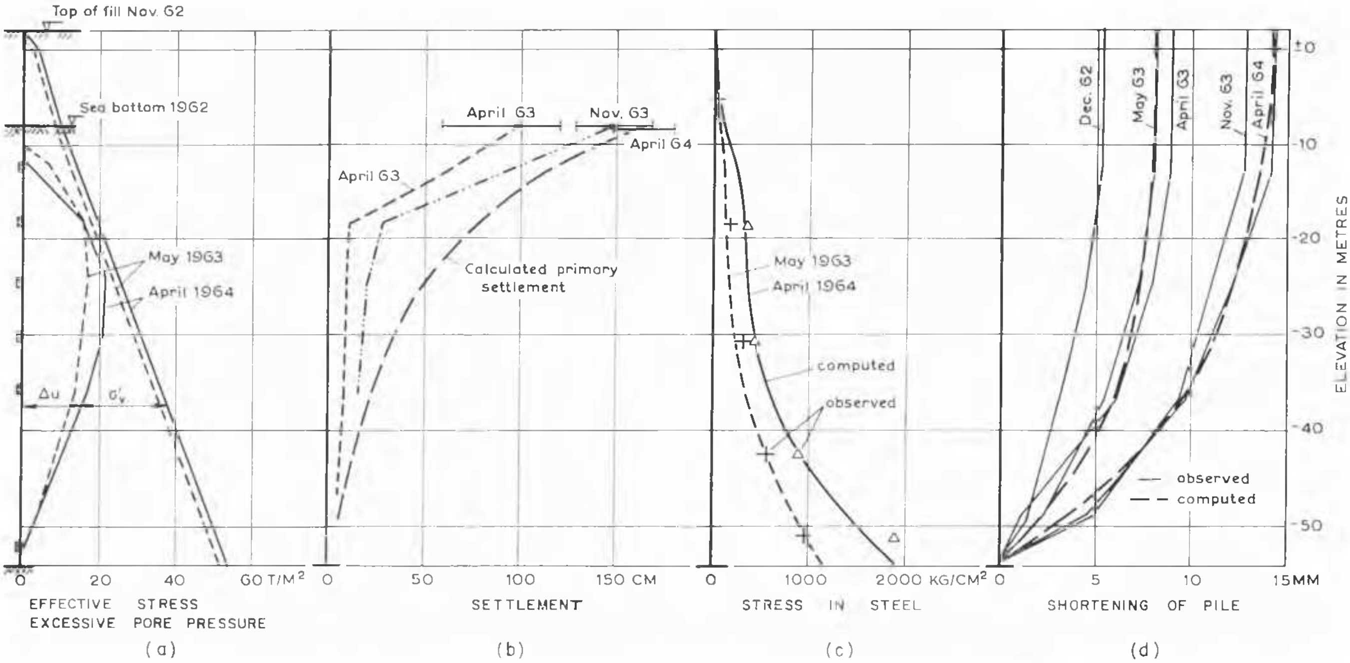

FIG. 4. (a) excess pore pressure and effective stresses in ton/sq.m; (b) settlement in cm; (c) computed and observed steel stresses in kg/sq.cm.; (d) computed and observed shortening of pile in mm.

vertical effective stresses in the soil. The resulting compression of the pile is then compared with the measured value and the ratio of the observed to the computed total shortening of the pile is equal to the value of the factor $K \tan \phi'_a$ with which the vertical effective stresses should be multiplied to obtain the "negative skin friction." Next, the stresses and strains in the pile are computed at different depths for the adhesion obtained in this way. The resulting curves can now be compared with the observations.

The result of such a computation is shown in Fig. 4d for the two sets of observations, dated May, 1963, and April, 1964. As seen from the comparison with the direct observations, there is a reasonably good agreement between the computed and observed shortening of the pile and stresses in the steel. This indicates that the assumption made concerning the distribution of the adhesion is approximately correct.

In this computation no attempt has been made to separate the contributions to the drag from the fill and from the clay. This decision is based on the observations that the friction from the fill is relatively small, as indicated by the low average stresses measured in the upper 12.5 m of the pile.

A detailed study of the curves in Fig. 4c shows that the observed stresses in the pile over the distance from elevation −6 to −18 m increase more abruptly than predicted from the computations. This observation might be explained as a result of the bend which was caused by the slide and which was found at elevation −10 to −12 m. The upper straight part of the pile has thus, very likely, picked up a downward directed drag, which is not included in the computation assuming a constant value of $K \tan \phi'_a$.

The values of $K \tan \phi'_a$ resulting from the above computations are given in Table I. This table includes an estimate of the final value of $K \tan \phi'_a$, corresponding to the conditions when all excess pore pressures have dissipated. To estimate this value the total final shortening of the pile has been estimated from the time-compression curves to be 25 mm. It should be mentioned, however, that to compute the value of $K \tan \phi'_a$ for the final condition, the assumption was made that a relative movement between pile and clay would occur which was sufficient to mobilize the full value of friction over the total length of the pile. This assumption will probably not hold good for the last part of the consolidation process, and therefore the computed value of $K \tan \phi'_a$ more likely represents a maximum value.

TABLE I. VALUES OF $K \tan \phi_a'$ FOR DIFFERENT TIMES

	Total shortening of pile (mm)	Value of $K \tan \phi_a'$
May, 1963	8.2	0.12
April, 1964	14.3	0.20
Final condition	25.0	0.20

CONCLUSIONS

From the field measurements described above it is possible to draw the following conclusions.

1. In cases where a very long pile is driven to rock through soft clay and where the settlement of the surrounding clay is large, the drag from the clay might induce additional loads in the pile and cause the allowable design load to be exceeded.

2. This drag will, in such cases, result in additional settlements, which are due partly to the compression of the pile and partly to penetration of the pile point into the bearing stratum.

3. The distribution of the adhesion between clay and pile seems at any time to be governed by the distribution of the effective stresses in the surrounding clay. The ultimate value of the adhesion might, for design purposes, be taken as $\sigma'_v K \tan \phi'_a$, where σ'_v is the effective vertical stress in the soil. The ultimate value of $K \tan \phi'_a$ is about 0.20 in the case considered where the clay is soft to medium soft marine clay.

REFERENCE

BJERRUM, L. (1957). Norwegian experiences with steel piles to rock. *Géotechnique*, Vol. 7, pp. 73–96, 146. (Norges Geotekniske Institutt, *Publ.*, 23)