Short and Long-term Test Loading of a Friction Pile in Clay

Essai de chargement de courte et de longue durée d'un pieu flottant dans l'argile

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Summary

At a site in Drammen, about 40 kilometres southwest of Oslo on the west side of the Oslo Fjord, a succession of loading tests has been carried out on a timber friction pile driven into a deep deposit of normally consolidated silty marine clay.

Initially the pile — a single tree trunk driven root up — was test loaded in the conventional short-term manner at different times after driving up to eight months.

The pile was then subjected to two successive long-term tests, occupying together a period of nearly two years, in which the load was built up at a rate comparable with that obtaining during construction of a pile-supported structure. On conclusion of the long-term tests a final short-time test was made.

By a comparison of these short-and long term tests the authors have tried to discover the relationship between the behaviour of a pile subjected to a conventional loading test and its behaviour in service.

Sommaire

A un chantier de construction à Drammen — ville à 40 km environ au sud-ouest d'Oslo, du côté ouest du fjord d'Oslo — on a effectué une série d'essais de chargement sur un pieu flottant enfoncé dans une argile marine limoneuse normalement consolidée.

Ce pieu (simple pieu de bois, enfoncé avec le gros bout vers le haut) a d'abord, de façon habituelle, été soumis à des charges de courte durée, appliquées à des moments différents après l'enfonçage, jusqu'à huit mois.

Après cela, le pieu a été exposé à deux essais de charge successifs, de longue durée, en tout presque deux ans ; lors de ces essais, la charge reposant sur le pieu a été peu à peu augmentée, pendant une période correspondant sensiblement à la durée de construction d'une maison fondée sur pieux.

Une fois ces essais de longue durée terminés, on a de nouveau effectué un essai court.

En rapprochant les résultats des essais de longue et de courte durée, on a essayé de trouver la relation qui existe entre le comportement d'un pieu soumis à un essai court classique, et le comportement d'un pieu normalement utilisé.

1. Introduction

At Drammen, the broad valley of Drammenselven is occupied by deep deposits of soft silty clays which form, on both sides of the river, the fairly flat terraces upon which the town stands. The river banks are prone to sliding and foundation problems in the town are difficult. The area has thus been investigated extensively. Borings made near the site of the test piles are shown in Fig. 1.

The stability of the area between this site and the river was examined in 1954, when borings 9 to 15 were made. In 1956 borings 1 to 4 were carried out and four friction piles (Nos. I-IV), of timber spliced root to root, were driven during a foundation investigation on the present site. These piles were subjected to short-term loading tests at different times after driving, which revealed that the rate of gain in bearing capacity was very slow in comparison with that normal in Norwegian clays, about nine months elapsing before 90 per cent of the final value was reached. The ultimate bearing capacity was, on the average, approximately 30 per cent higher than had been calculated from the soil properties.

To explore further the somewhat unusual behaviour of piles in this clay it was decided, a year after the driving of the four spliced piles, to drive a further pile (No. V) and subject it to long-term loadings. For largely practical reasons a single timber pile was chosen, and driven root up.

During the period of the long-term loading, vane borings 5, 6 and 7 were made, a series of samples taken from borehole 8 and piezometers put down well to each side of the test pile, as shown on Fig. 1.

On conclusion of the testing programme a sample series was taken close in to the test pile on the side adjoining vane bore 6.

2. Ground conditions

The results of the ground investigations performed before the driving of test pile V revealed that the area consisted of relatively homogeneous, somewhat silty, clay deposits to a considerable depth — no rock being encountered. The shear strength properties were also found to be fairly uniform and are summarized in the section on Fig. 1.

In an evaluation of the loading test results from pile V it is naturally the nearby borings 6, 7 and 8 that are of most interest. The results of vane borings 6 and 7 are given in Fig. 2, while the profile at borehole 8 is shown on Fig. 3.

The results of borings 7 and 8 agree closely with the general pattern established by the earlier investigations. The upper metre or so consists of fill, below which the dry crust extends to a depth of about 3·5 metres beneath ground surface. Beneath the crust lies a deep deposit of silty, marine clay. Grain size distribution curves for the material are shown on Fig. 4 and show its silt content to vary between about 50 and 70 per cent. The deposit is fairly homogeneous down to a depth of about 10 metres below ground level, but from 10 to 17 metres depth thin layers of clayey silt are common. Between 17 metres depth and the deepest boring depth (25 metres) the clay appears to become more homogeneous again.

The undrained shear strength of the clay falls from about 1·5 tons sq. m at 4 metres to 1·1 tons sq. m at 7 metres depth and then increases steadily with depth at a c/p ratio of 0·14. The clay is sensitive, the ratio of the undrained undisturbed and remoulded shear strengths lying generally in the range 4 to 8. The moisture content decreases fairly linearly with the depth from 40 per cent at 4 metres to 35 per cent at

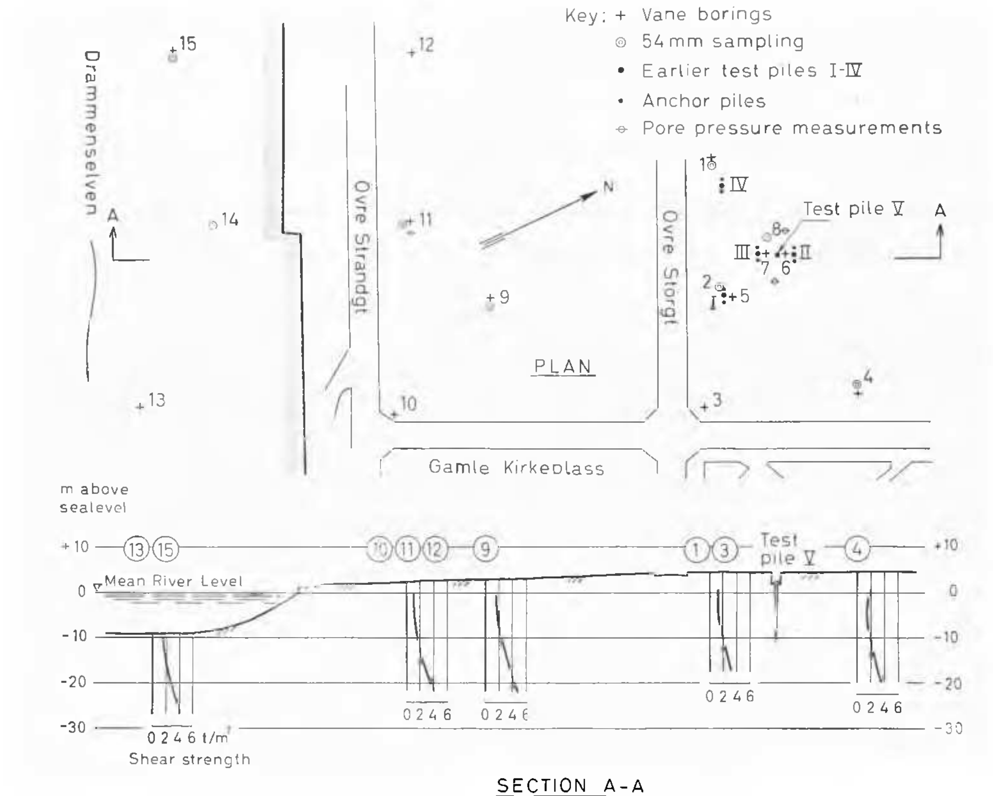


Fig. 1 Plan and section of site.
Plan d'ensemble et coupe.

12 metres and then further from 32 per cent at 13 metres to 27 per cent at 25 metres depth. The liquid limit lies between 38 and 40 per cent from a depth of 3 to 12 metres, below which it drops to an average of 32 per cent. The plastic limit is fairly constant at about 20 per cent. The salt content of the upper 12 metres increases with depth from about 3 to 21 gr NaCl per litre. Below this depth the salt content is fairly constant at about 21 gr NaCl per litre. The content of humus is small, varying between 0·1 to 0·6 per cent of the dry weight.

The consolidation characteristics of the clay have been determined by standard œdometer tests, which give values of the compression index, C_c, and the coefficient of consolidation, c_v, ranging between 0·30-0·35 and 1·0-4·0 sq. m/sec respectively. These tests, in conjunction with the piezometer measurements (made well outside the range of influence of the piles) indicate the clay to be normally consolidated under its present overburden load.

The borings were taken down to a maximum depth of 25 metres, but no rock was encountered.

The results of vane boring 6 show a marked divergence from the general shear strength pattern described above, indicating a rather firmer layer from 11 to 16 metres depth.

3. Test pile

The test pile (No. V) is a single timber pile driven root up at the location shown on Fig. 1. It is of spruce from which the bark had been removed, and was in an air-dry condition at the time of driving. The diameter of the pile is 35 cm at the head and 15 cm at the toe. It has an embedded length of 13·1 metres. A sketch of the pile in-situ is shown with the details of borehole 8 on Fig. 3.

At the test site, the dry crust was removed to a depth of 2·4 metres before the pile was driven. The driving was done with a hammer of 800 kg the height of drop varying from 1 to 2 metres. The driving diagram for the pile is shown on Fig. 2.

4. Test loading arrangements

In the short-term tests, a hydraulic jack is employed for loading, the reaction being provided in the conventional manner by a steel cross-beam bolted to anchor piles which straddled the test pile.

For the long-term tests, the device illustrated on Fig. 5 is used. It consists of two steel lever-beams, each hinged at one end to a horizontal steel beam which is in turn bolted

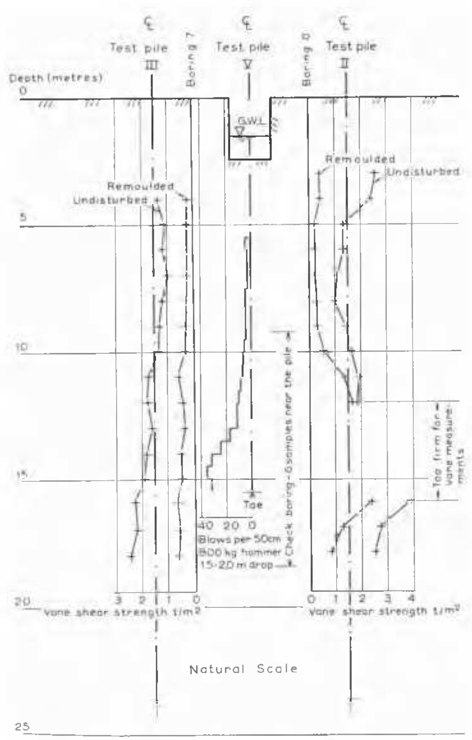


Fig. 2 Section through the test pile and the two nearby vane borings 6 and 7.

Coupe du pieu d'essai et des deux forages voisins 6 et 7 (forages effectués à l'aide d'un appareil à palettes).

to the anchor piles. The lever-beams bear upon the head of a steel dolly, axial with the test pile, at about 0·9 metre from their hinged ends, and have each a total length of nearly eight metres. At the free end of each lever-beam hangs a steel loading bucket. The load applied to the pile is derived partly from the weight of the dolly, levers and buckets and partly from kentledge placed in the buckets. This method of loading ensures an almost constant load on the test pile over long, unattended periods.

From the head of the test pile, two unstressed steel rods are carried up to above ground level (see Fig. 5). Relative movement between these rods and an independently supported measuring bridge was measured by twin dial gauges and taken, after correction for temperature, as the settlement of the pile. The trussed measuring bridge was founded on four steel tubes, carried down through protective casing tubes to frost-free depth and situated at about two metres from the test pile. In both types of test it is thus the relative movement between the test pile and the nearby ground that is recorded.

5. Loading tests

The several loading tests made on pile V are shown in time relation to the date of driving on Fig. 6. In both short and long-term tests the failure load is taken as the least load which causes a slow continuing settlement of the pile.

Short-term loading tests—Conventional short-term tests, in each case occupying only a few hours, were carried out on the pile at 3, 31 and 71 days after driving and again, after completion of the long-term tests, at 799 days after driving. Details of these tests are shown on Fig. 7, the curves to the left showing the variation of load and settlement with time and those to the right, the relationship between load and settlement. The final short-term test reveals only a small further increase in bearing capacity.

The bearing capacity as determined by these tests is seen from Figs. 8 and 9 to increase, as would be expected, at a more rapid rate than was the case for the spliced piles, reaching 90 per cent of its full value at three months after driving. The final value of the bearing capacity is very much greater than that determined by calculation.

Long-term loading tests—Two long-term loadings of pile V are carried out. The principle of these tests is to load the pile at such a slow rate that failure occurs under fully drained conditions, such as probably obtain in practice. The load increments are initially large (as is unavoidable with the loading rig used), but are reduced as failure is approached. The cessation of "primary consolidation" settlement is taken as an indication that excess pore pressures are dissipated, and anew load increment is not added until this stage has been reached. Consolidation curves for selected increments of load are given on Fig. 10. The settlement observations are corrected for the effect of temperature and for the varying deviation of the lever-beams from the horizontal.

The first test commenced 125 days after driving, the loading being built up in the increments, and at the times, shown in Fig. 11. Failure is reached after 198 days, i.e. 323 days after driving.

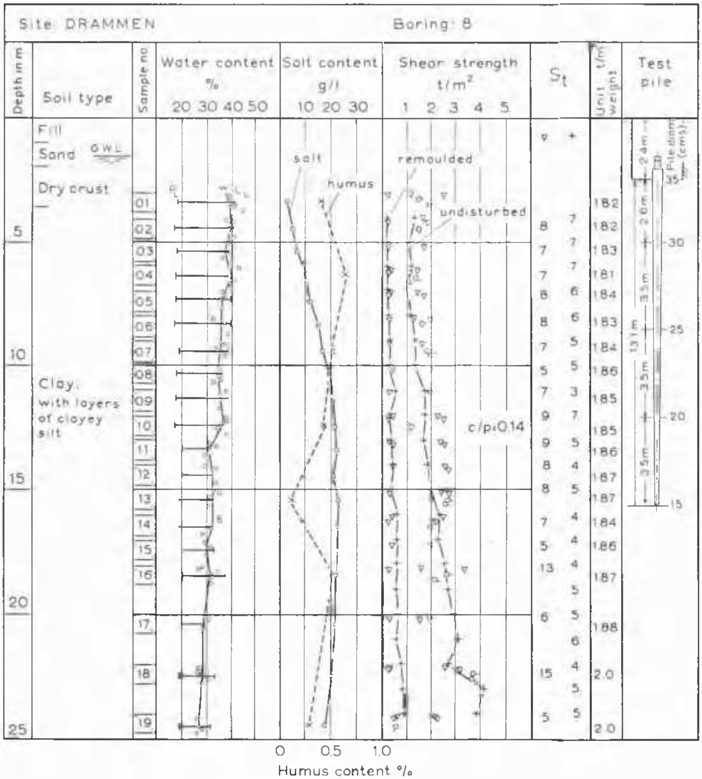
After unloading the pile, the second long-term test loading was begun 343 days after driving, the size and timing of the load increments again being apparent from Fig. 11. This loading occupied 438 days. Failure was not quite reached.

The ultimate bearing capacity reached in the first long-term test and the near-ultimate value given by the second are plotted on Figs. 8 and 9 for comparison with the values obtained from the short-term tests. The value from the first long-term test is somewhat lower than the equivalent short-term value. The value indicated by the second long-term test is higher however, and more or less identical with the final short-term value.

6. Sampling close to test pile

As mentioned in section II, vane voring 6, 2·3 metres north of the test pile, indicated a rather firmer layer to exist there, extending from a depth of about 11 metres to about 16 metres. On conclusion of the loading tests a series of 54 mm samples was taken up close to the test pile, chiefly in order to establish whether any such layer existed at the test pile itself. These samples also lay for the most part within the zone of reconsolidation around the pile and thus gave some information as to its properties and extent.

By chance, the uppermost sample (depth 9·2 to 10·0 metres) skimmed the pile, a thin sliver of the pile timber being found on one side of the sample. The entire remainder of the sample exhibited marked vertical layers of clay and clayey silt which in cross-section showed themselves to be concentric to the pile. These layers must have originally been horizontal, being forced into their vertical orientation by pile driving. The zone of vertical layering was seen to extend to at least 6 cm from the surface of the pile shaft; the total thickness of the disturbed zone there must be several times greater than this.



Key: ▽ cone test; + vane test; unconfined compression test (pointer indicates failure strain)

Fig. 3 Soil profile, boring 8.
Protocole de sondage, trou 8.

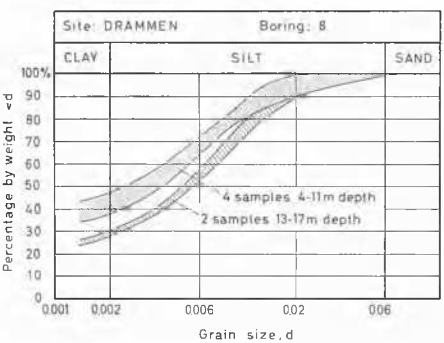


Fig. 4 Grain-size distribution curves, boring 8.
Courbe de granulométrie, trou 8.

The average moisture content, plastic and liquid limits of this reconsolidated zone are about 24, 20 and 36 per cent respectively. The undisturbed, undrained shear strength is about 4 tons/sq. m, the remoulded value being only slightly less, giving a sensitivity of less than 1·5. The unit weight is high, about 2·0 tons/cu. m, and the siltier material appeared to be dilatant. Corresponding values for the same depth in borehole 8 are 35, 18 and 36 per cent; 1·3 to 1·8 tons/sq. m, sensitivity 5 and unit weight 1·85 tons/cu. m.

The deeper samples were extracted beyond the immediate neighbourhood of the pile. They show the zone of clayey silt layers to extend, though less markedly, to about 15·5 m depth, that is almost exactly to pile toe level, where there is a fairly sharp transition to soft, slightly sensitive clay. Grain size analyses of the siltier material between 10 and 15 metres depth give results rather similar to those shown on Fig. 4.

From the evidence of these samples it may be concluded that the lower 5 or 6 metres of the pile stand in clay that is reinforced by frequent thin layers of clayey silt, especially in the upper part of this region. This material does not however extend beneath the pile toe. It was probably these

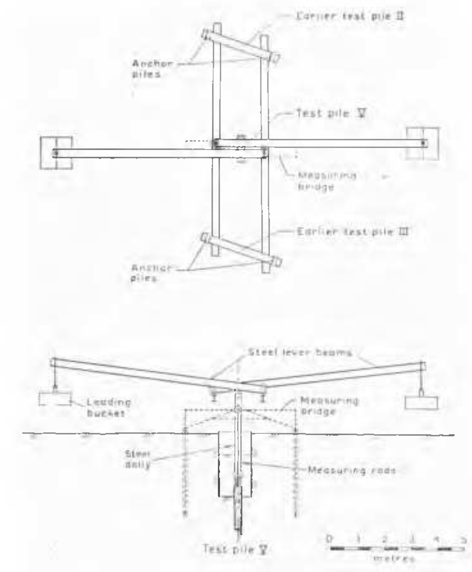


Fig. 5 Long-term loading arrangement.
Dispositif de l'essai de longue durée.

somewhat dilatant silt layers which hindered the pressing out of the vane in boring 6.

The driving of the pile has caused, in the upper part of the silt-layered region, violent distortion of the original structure in a zone probably of the same order of thickness as the pile itself. A corresponding disturbance was not observed in the lower samples, probably because the borehole diverged from the pile.

7. Theoretical bearing capacity

The bearing capacity of a friction pile, which is made up of the toe resistance and the shaft adhesion, can be calculated for clays in terms of either the undrained or the drained shear strength of the surrounding soil. These two approaches are described separately below.

Undrained conditions—An approximation to these conditions exists in a conventional short-term loading test. The toe resistance, Q_{pu}, is then given, for a parallel-sided pile by the formula :

$$Q_{pu} + W = A_p(N \cdot c_p + \gamma H)$$

where W and H are respectively the weight and embedded length of the pile, A_p the area of its toe, c_p the undrained shear strength of the clay at toe level, N a dimensionless bearing capacity factor and γ the average total unit weight of the soil along the length of the pile.

Taking $N = 9$, and making an approximation to allow for the taper of the pile, Q_{pu} is found for test pile V to be about 1·4 metric tons.

Assuming that the unit shaft adhesion at a depth h within the embedded length of the pile, H, is equal to c (the undrained, undisturbed shear strength of the surrounding clay at the same depth), the total shaft adhesion, Q_{su}, is given by the general expression :

$$Q_{su} = \pi \sum_{0}^{H} cd \cdot \Delta h$$

where Δh is the length of an element of the pile of diameter d at depth h.

In the present instance there is some difficulty in choosing a representative value of c for insertion in this expression, because of the heterogeneity in shear pattern revealed by vane boring 6. Using the normal shear profile for the site, however (closely agreeing with vane boring 7) Q_{su} is calculated to be 15·0 metric tons.

The calculated ultimate bearing capacity of the test pile under undrained conditions is therefore 16·4 metric tons. This value is compared on Figs. 8 and 9 with that actually measured.

Drained conditions—For a friction pile in clay under fully drained conditions, such as exist in long-term loading tests and for foundation piles in service (unless a significant part of their load is "live"), the bearing capacities of the toe and shaft of the pile may be expressed in terms of the effective shear parameters of the surrounding soil (JANBU, BJERRUM and KJAERNSLI, 1956).

For a parallel-sided pile the toe resistance may then be written :

$$Q_{pd} = A_p \left(\frac{1}{2} N_\gamma \gamma' d_p + p'_p N_q \right)$$

where γ' is the average submerged unit weight of the soil along the length of the pile, d_p the diameter of the pile toe, p'_p the effective vertical pressure at pile toe level and N_γ and N_q are dimensionless bearing capacity factors.

Thus for test pile V, if the taper be neglected, Q_{pd} is calculated to be 6·5 metric tons.

Again for a parallel-sided pile, the shaft bearing capacity under drained conditions, Q_{sd}, is given for a normally consolidated clay by the expression :

$$Q_{sd} = \pi \sum_{0}^{H} kp'd \tan \delta \cdot \Delta h$$

where at a depth h within the total embedded length H, of the pile, k is the coefficient of earth pressure, p' the effective vertical pressure and δ the angle of friction between the clay and the pile.

Allowance for the pile taper may be made by inserting α, the angle between the pile surface and the vertical, as follows :

$$Q_{sd} = \pi \sum_{0}^{H} kp'd \cdot \tan(\delta + \alpha) \cdot \Delta h$$

As neither k nor δ are known it is not possible to calculate, with any reliability, the drained bearing capacity of the shaft. If δ is assumed equal to φ' for the undisturbed clay and the latter is taken to be 28° (SIMONS, 1960), a value for k may be calculated from the measured ultimate bearing capacity of 29·6 tons, as follows :

$$Q_{sd} = 29{\cdot}6 - Q_{pd} = 29{\cdot}6 - 6{\cdot}5 = 23{\cdot}1 \text{ metric tons.}$$

Then from the above expression for Q_{sd} (with allowance for taper) k may be calculated to be about 0·5.

8. Discussion

The rate of growth of bearing capacity of test pile V is slow in comparison with the average for normally consolidated Norwegian marine clays, and the ultimate value attained is considerably higher than that usually found. In both these respects the behaviour of the single pile accords with the character of the clay indicated by the earlier tests on this site with spliced piles. In exhibiting a rather faster rate of growth and higher ultimate value of bearing capacity

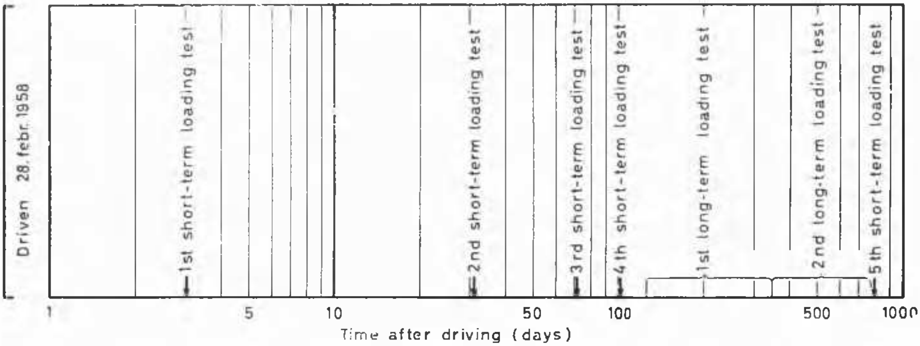


Fig. 6 Timing of loading tests on the test pile. — Programme de l'essai.

than the spliced piles, the single pile reflects typically the relative superiority of its form.

Vane borings 6 and 7, and particularly the sample series close to the pile, provide the most likely explanation of these deviations from the normal. These borings indicate that, at about 12 metres depth, the strength of the ground increases, probably as a result of the occurence there of frequent clayey silt layers. This seems to be a fairly local effect, being most marked in vane bore 6, and absent in vane bore 7, while the slight discontinuity in water content and liquid limit at 12 to 13 metres in borehole 8 is perhaps a faint echo of the same heterogeneity. Close to the test pile itself sampling indicated a rather thinner, strongly layered zone between about 9 and 12 metres depth and a more homogeneous and soft silty clay from 12 metres to the toe of the pile, which stood in soft clay.

This latter fact rules out the possibility of the high ultimate bearing capacity resulting from the build-up of a high toe resistance : the theoretical (undrained) value for this of 1·4 tons is probably not far from the truth. The high ultimate bearing capacity shown by the short-term tests must therefore be the result of high shaft adhesion.

In general, the ultimate shaft bearing capacity of a friction pile in clay is built up above the calculated value (pile shaft area multiplied by average undisturbed shear strength of the clay for the depth of pile penetration) by the reconsolidation around the pile of the zone of clay disturbed by the driving. For timber piles in our normally consolidated clays, particularly when the latter are strongly leached, the undisturbed shear strength of the clay is exceeded by both the shear strength of the reconsolidated clay and the adhesion of the latter to the pile surface. Thus, failure of a mature pile will take place at some distance from the surface, this distance depending on the characteristics of the reconsolidated zone.

Preliminary investigations (Bjerrum and Hutchinson, 1960) indicate that this amount by which the calculated ultimate bearing capacity of a pile may be exceeded, is related to the liquidity index of the clay in which it is driven. The greatest ratios of measured to calculated ultimate bearing capacity also appear to occur for piles driven in clays of liquidity index between about 0·5 and 1·0. The present clay, therefore, with a liquidity index of 0·8, may be expected to yield measured ultimate bearing capacities well over the calculated values.

Test pile V has an average diameter of 25 cm and if, for example, failure occurred at a distance of 5 cm from the pile surface, an ultimate shaft bearing capacity 40 per cent in excess of the calculated value would result. The uppermost of the samples taken close to the pile showed the thickness of the reconsolidated zone there to be at least 6 cm and probably considerably more. This seemed to apply only for the silt-reinforced region however, and the average zone thickness for the whole pile may well be smaller.

It is likely that some contribution to the high ratio between measured and calculated ultimate bearing capacity for test pile V is made by the clayey silt layers which reinforce the clay in the lower strata penetrated by the pile. Thus it is probable that the c value used in the calculations has been somewhat underestimated, based as it is on the shear strength profile of boring 7. On the other hand, the large values of ultimate bearing capacity measured for the earlier spliced piles on the same site, at locations remote from this local heterogeneity, suggest that its influence on the results for test pile V has been moderate.

The approximation involved in inserting the toe area of the pile in the expression for toe resistance, rather than some greater area to allow for the pile taper, has also tended to increase the ratio between measured and calculated bearing capacity. This effect is very small in the present case however.

The slow rate of growth of bearing capacity is a further indication that the pile is in a predominantly cohesive relation with the surrounding ground and that the clayey silt layers have not contributed any considerable frictional component to the shaft bearing capacity. From the boring close to the pile however, it seems that these layers have had the effect of extending the compass of the disturbed zone and, through this, of retarding the growth of bearing capacity.

From the Institute's general experience with the successive testing of friction piles in clay, it seems that neither the rate of growth of bearing capacity nor its final value are significantly affected by conventional short-term loading tests.

The most striking fact revealed by the comparison of the short and long-term tests on this pile is that the ultimate bearing capacity given by both types of tests is almost identical.

From studies of the stability of slopes it is known that, for normally consolidated clays, a slope will stand considerably greater stresses if these are built up slowly under drained conditions than if they are applied rapidly under undrained conditions. For heavily overconsolidated clays, the reverse is true, while between these two extremes are

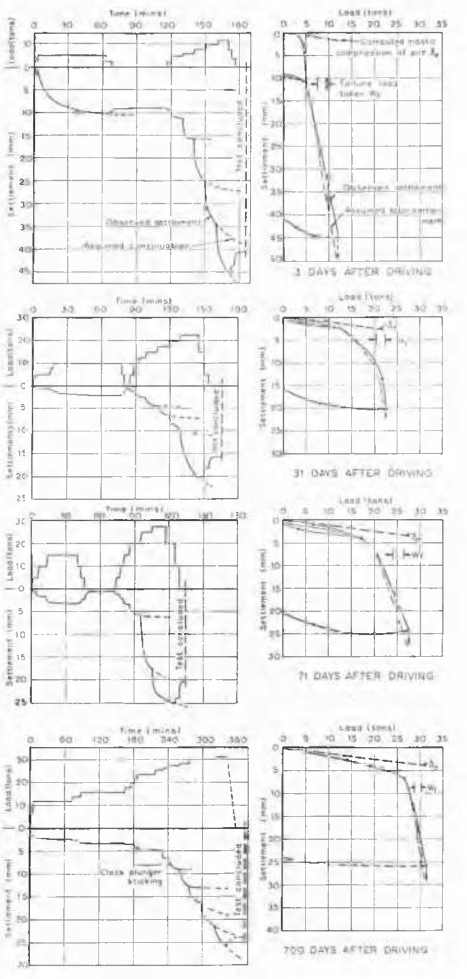


Fig. 7 Short-term loading tests; curves showing the relationship between load, settlement and time for the various tests.

Essai de courte durée. Relation, pour les différents essais, entre charge, enfoncement et temps.

clays which display equal strength for either manner of loading.

Although the failure of a driven friction pile differs in many respects from the failure of a natural slope, it may be that the general trend evidenced by the latter, and summarized above, is reflected to some degree in the behaviour of friction piles in clays.

The coincidence of the ultimate bearing capacity under undrained and drained conditions in the present instance is therefore best considered as a result particular to this loading test. In different clays, or possibly with piles of

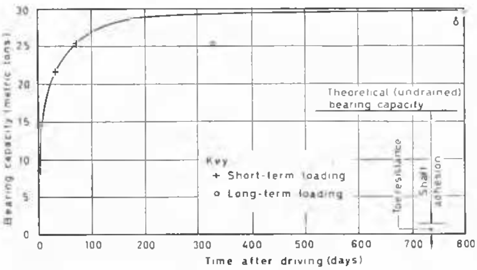


Fig. 8 Diagram showing growth of bearing capacity with time (natural plot).

Augmentation, dans le temps, de la capacité portante (abscisse : temps naturel).

different length in the same clay, the drained and undrained bearing capacities may well diverge.

The much greater settlement, for a given load, in the long-term as opposed to the short-term tests is also very marked for loads on the pile of more than about 12 tons.

The absolute downward movement of the head of the pile can conveniently be considered as being split into three components; that due to deformations in the pile material itself; that due to shear deformations within the zone of disturbance surrounding the pile; and that due to deformation in the mass of ground lying around and beneath the pile and embraced by its bulb of pressure. The last two of these components may be further subdivided into elastic, primary consolidation and secondary consolidation settlements.

The arrangement of the measuring bridge, founded in the upper layers of soil and about 2 metres to either side of the test pile, has the result that the bulb deformations referred to above will affect the pile and the measuring bridge about equally. Thus only the first and second components of deformation referred to above are recorded in the present tests.

Clearly, the elastic deformation of the pile itself and of the relatively limited zone between the pile and the surrounding ground are common to both short and long-term tests. Reference to the unloading curves for both types of test shows that in the sort-term tests, these elastic settlements generally constitute the major parts of the total settlements measured — at least until just before failure occurs — whereas in the long-term case, for loads in excess of about 12 tons, the elastic rebound forms but a small fraction of the total settlement.

The source of the considerable extra settlement for the higher loads in the long-term case is best seen from the consolidation curves for this test, shown in selection on Fig. 10. The curves, on a semi-logarithmic plot, show clearly the characteristic form of a consolidation curve for clay, with its primary and secondary components. For the load step from 21·0 to 21·4 tons, for instance, the primary consolidation amounts to about 11 mm and occupies 30 days. The start of the secondary stage of consolidation has been taken as a sign of the dissipation of excess pore pressures and therefore as the signal for adding the next increment of load.

Comparison of this consolidation curve with the corresponding load steps in the quick tests, held for a maximum time of only 30 minutes, reveals that in a conventional short-term test there is not time enough for more than a small fraction of the latent primary consolidation within the disturbed zone around the pile to occur.

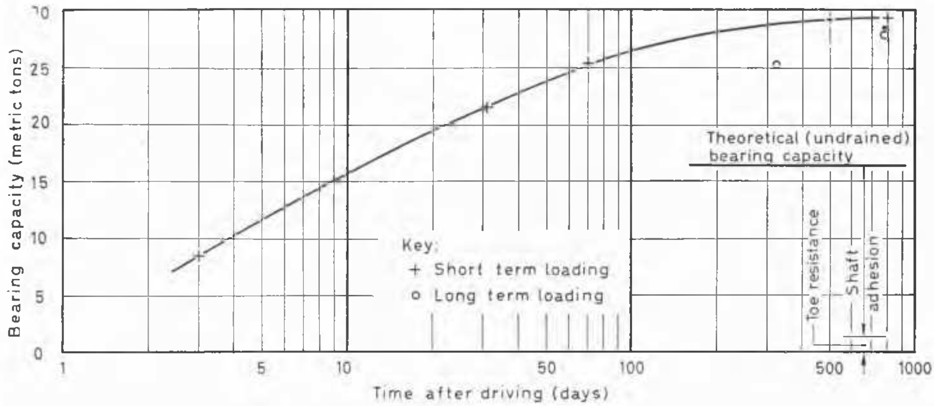


Fig. 9 Diagram showing growth of bearing capacity with time (semilogarithmic plot).
Augmentation, dans le temps, de la capacité portante (abscisse : temps semi-logarithmique).

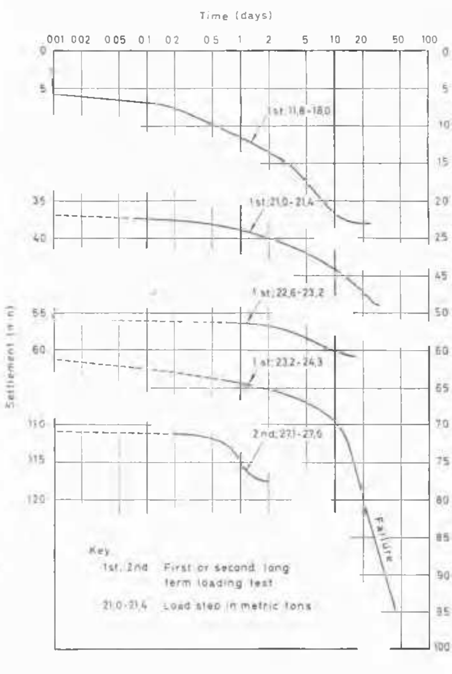


Fig. 10 Long-term loading tests ; consolidation curves for selected load steps.
Essai de longue durée. Courbes de consolidation pour certains échelons des charges appliquées.

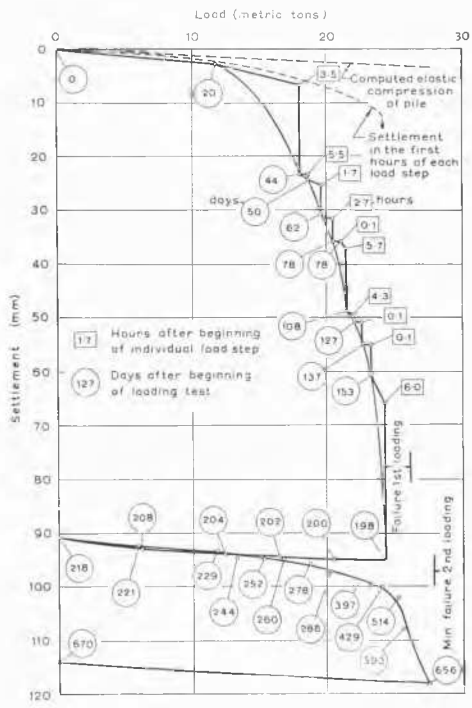


Fig. 11 Long-term loading tests ; curves showing the relationship between load and settlement.
Essai de longue durée. Relation entre charge et enfoncement.

Thus, in the present investigations, the settlement measured in the short-term tests arise, except near failure, chiefly from the elastic deformations of the pile and the disturbed, zone around it. In the long-term tests, on the other hand, these elastic deformations, while of the same order of magnitude, are completely outweighed by the primary consolidation settlement of the reconsolidated zone between the pile and the surrounding ground. The long-term tests were not run slowly enough to allow significant secondary consolidation settlements of this zone to be distinguished.

An interesting feature of the load-settlement curve for the first long-term test is the break-point at a pile load of about 12 tons. At lesser loads, the long-term settlements approximate to the short-time ones and are probably thus predominantly elastic; at greater loads this long-term test shows increasingly large, plastic settlements in comparison with the short-term tests. (As a further comparison, the curve obtained by plotting the settlement occurring in only the first few hours of each load step of the first long-term test is shown with a broken line on Fig. 11.)

The reason for this threshold value of 12 tons has not been found.

9. Conclusion

The ultimate bearing capacity indicated by the short-term tests is 29·5 tons. The corresponding calculated value is 16·4 tons, that is 45 per cent too low. Both neglect of the effect of the silt layers local to the pile and, to a lesser extent, the incomplete allowance for the pile taper contribute to an underestimation of the calculated figure. Even after account is taken of these effects however, it is probable that a discrepancy of between about 25 and 35 per cent remains. This is a rather greater divergence between measured and calculated ultimate bearing capacity than is usual with such piles in Norwegian marine clays. The slow rate of growth of bearing capacity and the evidence of the sampling close to the pile both suggest that the reason for this large divergence is the build-up of a relatively thick layer of reconsolidated clay around the pile.

The ultimate bearing capacity indicated by the long-term tests coincides in the present case with that measured by short-term testing. It is not yet possible to predict the ultimate drained bearing capacity of a friction pile in clay, but a computation based on the measured value shows that the theoretical expression quoted above is satisfied by δ and k values of a reasonable order. This gives hope that further investigations of this nature may lead to a practical method of calculation.

The chief difference between the results of the short and long-term tests is the magnitude of the settlements involved. Up to a pile load of about 12 tons, that is about 40 per cent of the ultimate load, the settlements are chiefly elastic and are similar for both types of test. Beyond this load however, the settlements in the long-term test greatly exceed those produced by similar loads in a short-term test. These large settlements are plastic moreover, and the threshold value at about 12 tons load will thus be of decisive importance in the determination of a safety factor for a permanent loading of the pile.

References.

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