

LOAD TESTING IN THE BANGKOK REGION OF PILES EMBEDDED IN CLAY

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SYNOPSIS

An analysis of the results obtained from load tests carried out in the Bangkok region on piles embedded in clay shows that it is possible to establish a relationship between the mobilized shaft friction, $a s_u$, and the *in situ* shear strength, s_u . The tests also give an indication of the ultimate point bearing capacity and suggest that only part of this is mobilized when failure starts to develop. Load tests carried out at different times after piling show that practically full bearing capacity is reached one week after piling. The tests yield enough information to help suggest the mechanism involved when a pile in clay fails.

INTRODUCTION

The Bang Pa-In to Nakhon Sawan highway, which is part of the Asian Highway, starts 52 km north of Bangkok and runs northwards for approximately 190 km parallel to the Chao Phraya River, as shown in Fig. 1. In connection with the supervision of the construction several test loadings of piles embedded in clay were carried out. Some of the tests yielded valuable information which is thought to be of a wide interest in the Bangkok area where, despite the fact that virtually all foundations are piled, very little has yet been published on the subject.

The soft clay deposits in the Bangkok area are generally all known as 'Bangkok Clay', which has led to the tendency to treat all these deposits in the same way. However, practical geotechnical work with this soft clay during recent years suggests that the strength and settlement parameters vary greatly. This variation is thought to be due mainly to different weathering conditions since deposition. There seems to be a significant difference, for instance, between the soft clays that occur in the Bang Pakong and the Bang Pa-In areas to the east and north of Bangkok respectively. It is possible, therefore, that the test results reported here only apply to the actual test location. It is hoped, however, that this contribution will promote general discussion and an exchange of experience on the problems of piled foundations in the clays of this region.

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SUBSOIL CONDITIONS ALONG THE HIGHWAY

For approximately the first 20 km of the road, from Bang Pa-In to Ayuthia, a continuous layer of soft, organic clay exists below a 0.5 to 2.0 m weathered zone of fat clay. The thickness of this soft layer tapers from 4 to 5 m at Bang Pa-In to 1 to 2 m at Ayuthia. Its shear strength has been measured to be as low as 1.0 ton/m² but it falls generally within the range of 1.5 to 3.0 ton/m²; it has a moisture content of 80 to 100%, compared to a liquid limit of 100 to 120%. Below this soft layer is found stiff, fat clay which becomes sandy with increasing depth. The top 1 to 3 m of this formation yields a somewhat lower shear strength than is generally measured in the stiff clay (see Figs. 4, 8 & 12 given later), probably due to its tendency to swell. Firm deposits of sand have been encountered below the stiff clay at some locations. At nine places on this southernmost section the road crosses what is believed to be old riverbeds where the soft clay deposits extend to depths of 9 to 22 m.

The salinity of the pore water of the soft clay varies from 1 to 5 gm/litre, indicating either a brackish water or a leached marine deposit, while the humus content is in the range of 1 to 9% with somewhat erratic distribution. For approximately 30 km north from Ayuthia soft clay is found only as isolated pockets in deposits of stiff clay and firm sand. This stretch seems to form a transition zone between the soft formation to the south and the stiff clay and firm sand deposits found along the remaining part of the highway.

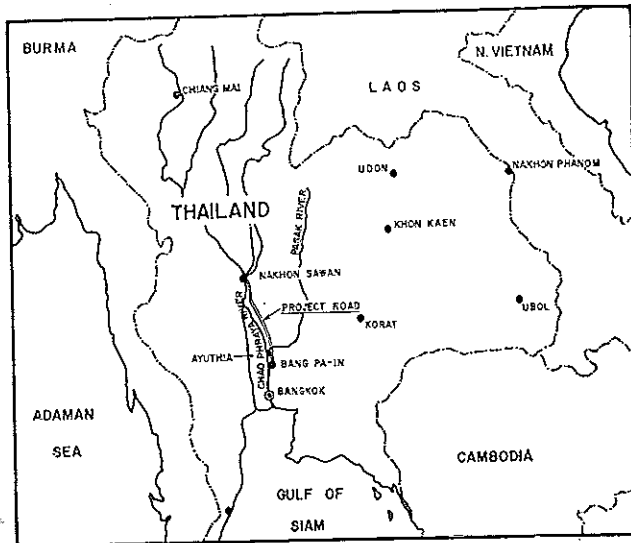


Fig. 1. Location of the Bang Pa-In to Nakhon Sawan highway.

LOAD TESTING OF PILES IN CLAY

The loading tests described in this paper were all carried out on the southernmost section of the highway where the soil formations are quite homogeneous. Undrained shear strengths were measured with the precision vane developed at the Norwegian Geotechnical Institute. Detailed description of the formation of soft clay along this section and the geotechnical problems related to the actual highway project are given by EIDE (1968).

PILE FOUNDATIONS

The road project includes 68 bridges with a total length of 3.5 km; 51 of these bridges will have pile foundations, while the remaining 16 (located on the northernmost section) will be founded on spread footings. The longest bridge spans are 63 m. Two types of reinforced concrete pile have been specified for the bridge foundations; a 35 × 35 cm square, solid pile and an octagonal, hollow pile with an inside diameter of 38 cm and a wall thickness of 10 cm. The vertical design loads are 30 to 45 ton and 55 to 110 ton for the square and octagonal piles respectively.

In addition to the piling for the bridge foundations the project also calls for relief piling to be carried out beneath the bridge approaches in areas where there is soft clay, in order that heavy differential settlements are avoided in the transitions between embankments and bridges. The piles to be used for this are 20 to 30 cm diameter wooden piles and 22 × 22 cm square, prestressed concrete piles; these wooden and concrete piles are to be used where pile lengths are to be less than 16 m and greater than 16 m respectively.

BEARING CAPACITY OF PILES

The ultimate bearing capacity of an individual pile is normally calculated as

$$Q = Q_s + Q_p \dots \dots \dots (1)$$

where Q_s is the shaft friction and Q_p is the point bearing capacity. The Danish Code of Practice for Foundation Engineering (1965) suggests that Q_s and Q_p for a pile embedded in a saturated clay are calculated thus:

$$Q_s = m \times S \times \alpha \times s_u \times A_s \dots \dots \dots (2)$$

$$Q_p = N_c \times s_u \times A_p \dots \dots \dots (3)$$

where m is an empirical material factor equal to 1.0 for concrete and wood, S is a shape factor equal to 1.0 for cylindrical and prismatic piles (equal to 1.2 for conical pressure piles with the thin end as pile toe), s_u is the undrained shear strength of the clay as measured by field vane tests, α is the adhesion factor which indicates the ratio between the shear strength of the clay along the pile at a given time after the piling and the undisturbed shear strength

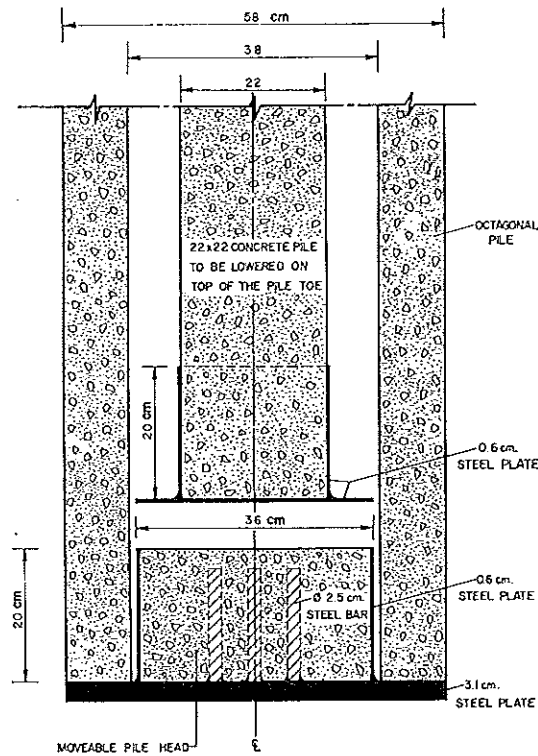


Fig. 2. Toe of octagonal pile for determining point bearing load.

(α is, therefore, also a function of time), A_s is the shaft area, A_p is the cross-sectional area of the pile toe, and N_c is an empirical bearing capacity factor. The Danish Code indicates the value of N_c to be of the order of 9 for some clays but about 18 for the Danish boulder clay.

During the design phase certain assumptions were made concerning the parameters α and N_c in order to arrive at a preliminary estimate of the pile lengths involved in the project. At an early stage in the construction period these assumptions were then adjusted or verified by means of full-scale loading tests. Besides the actual determination of N_c -values (which are related to the ultimate point bearing capacity Q_p), it was also the purpose of the tests to enable estimation of the degree of mobilization of Q_p at the commencement of pile failure.

LOADING TESTS

The sites for the initial loading tests were selected, on the basis of soil investigations made along the line of the road, in order that the pile tests would be carried out in soil conditions as homogeneous as possible. All piles were driven with a drop hammer of weight chosen according to the actual pile weight.

As mentioned earlier, hollow, octagonal piles are to be used at some bridge sites. One of these piles was fitted with a movable pile toe, as shown in Fig. 2, making it possible to measure the point bearing capacity and the shaft friction separately. The relative piling resistances for the test pile and for the two open ended octagonal piles used as reaction piles are plotted in Fig. 3. The soil conditions at the site chosen for this particular loading test are shown in Fig. 4 (note that s_u and s'_u are the undrained strengths in the undisturbed and remoulded states respectively). The values of 'sounding resistance' shown in Fig. 3 were measured as an arbitrary number measured by a rotary

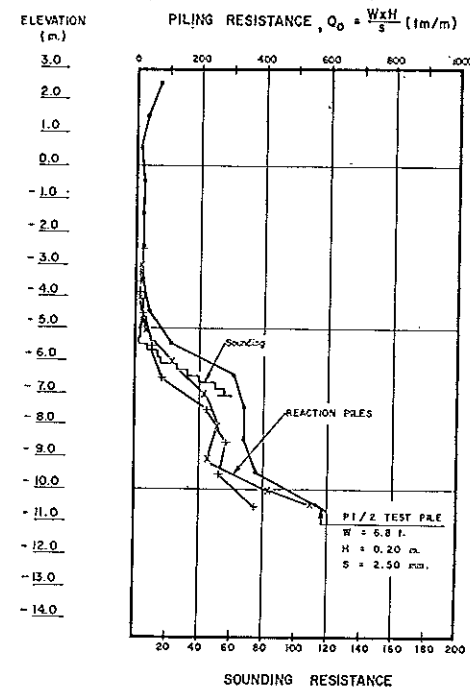


Fig. 3. Piling resistances of octagonal piles.

ELEV. (m.)

1.0
0.0
-1.0
-2.0
-3.0
-4.0
-5.0
-6.0
-7.0
-8.0
-9.0
-10.0
-11.0
-12.0
-13.0
-14.0

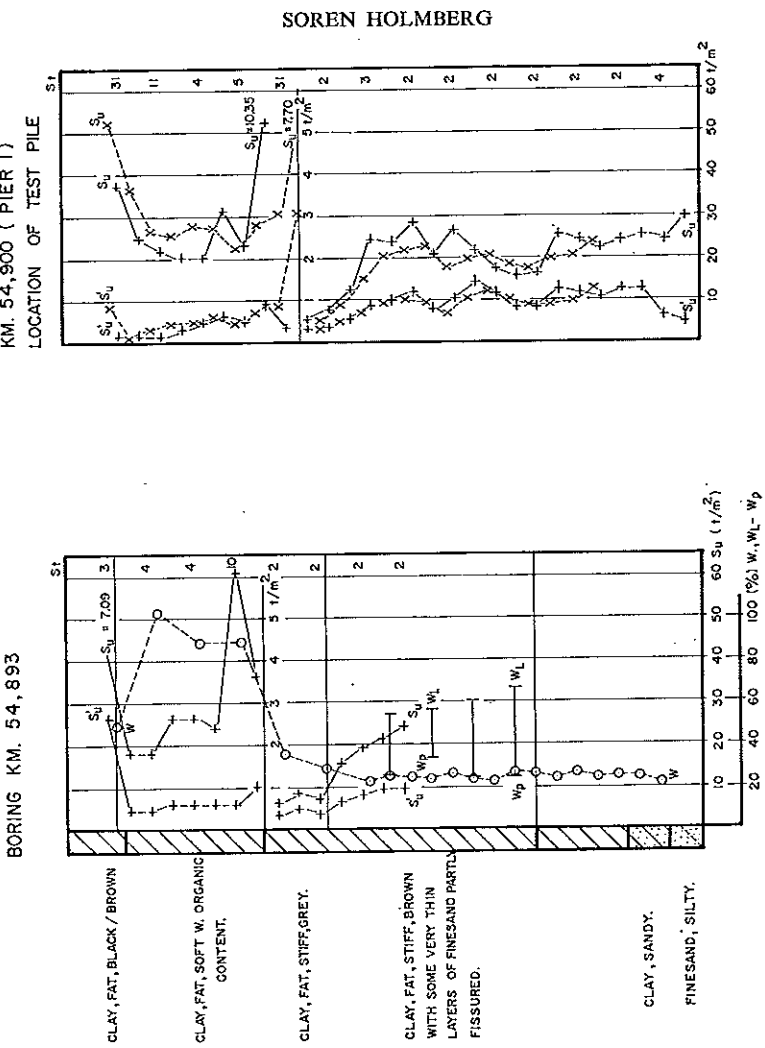


Fig. 4. Geotechnical profile at test site of octagonal pile.

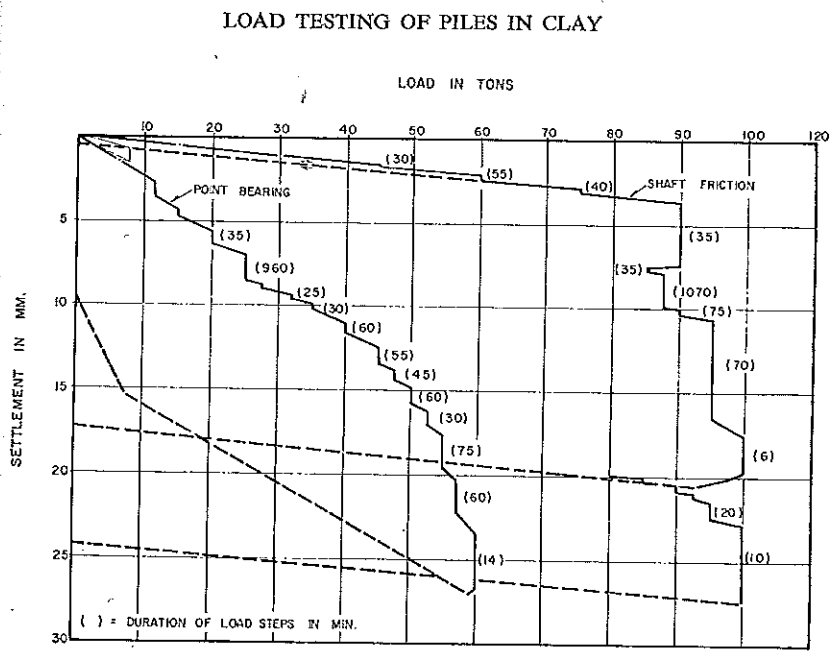


Fig. 5. Load-settlement curves for point bearing and shaft friction of octagonal pile.

sounding device⁽¹⁾. After being driven, the octagon pile was cut down to a convenient height above ground level to facilitate the placing of a 22 x 22 cm auxiliary pile for the point bearing test, which was carried out four weeks after the pile installation. The loads were transmitted to the toe via the auxiliary pile, and settlements were measured at the top of the pile with two dial gauges. The load-settlement and time-settlement curves for this test are shown in Figs. 5 and 6 respectively.

When the point bearing test was finished, the pile head was jacked 15 cm further into the ground to leave sufficient space for the hollow octagonal pile to move during the loading to determine the shaft friction. Before this was done, the top of the octagonal pile was made even by applying a layer of cement mortar; and a steel helmet was placed on the top of the pile as a base for the jack and dial gauges.

⁽¹⁾ This is similar to the Swedish penetrometer. The sounding resistance is the number of half turns required for a hardened auger-shaped point on a 2 cm diameter rod to penetrate 20 cm under a constant load of 100 kg.

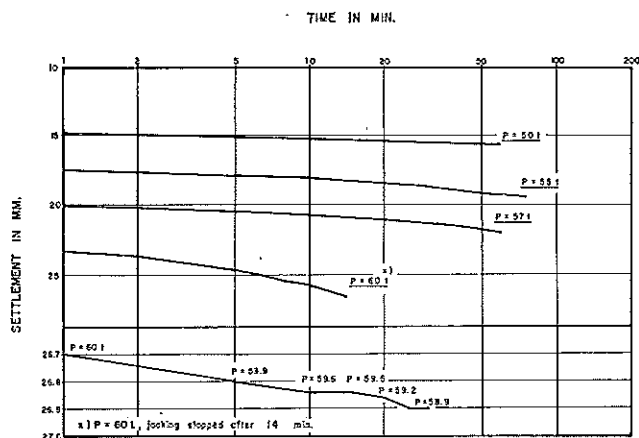


Fig. 6. Settlement-time curves for point bearing of octagonal pile.

The results of the shaft friction test are given in Figs. 5 and 7. The large settlement observed for a load of 90 ton is thought to have been caused by the start of the breakdown of the mortar layer which was placed only eight hours prior to the test (the vertical stress in the mortar corresponding to a load of 90 ton was 55 kg/cm²). The results obtained from the continuation of the test seem to confirm this suspicion.

In order to obtain some more information about the adhesion factor, α , as a function of the *in situ* shear strength and the elapsed time since piling, three 22 x 22 cm concrete piles were driven to different toe elevations on the slope of an old riverbed close to the test site of the octagonal pile. The soil conditions at this test site can be seen in Fig. 8. The three piles were driven in a triangular pattern with a spacing of 2.3 m; the piling resistances are plotted in Fig. 9. The piles were test loaded for the first time one week after being driven, and for a second time after a further interval of two months. The load-settlement and settlement-time curves obtained from these tests are shown in Figs. 10 and 11 respectively.

As mentioned previously, the project will include a large quantity of wooden piles to be used for relief piling. It was decided, therefore, to carry out some loading tests on some of these wooden piles. The testing schedule applied was similar to the one used for the three concrete piles, except that the wooden piles were only test loaded once, two weeks after being driven.

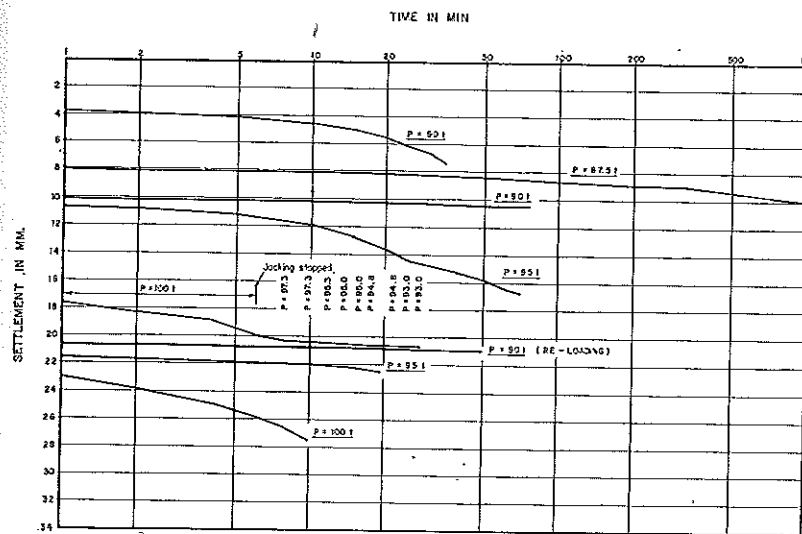


Fig. 7. Settlement-time curves for shaft friction of octagonal pile.

The geotechnical profile at the test site is given in Fig. 12, and the loading results are presented in Figs. 13 and 14.

DISCUSSION OF TEST RESULTS

A general look at all the results of the loading tests reveals that a state of failure started to develop at a deformation of approximately 5 mm in every case, regardless of the strength of the clay; the real failure deformation of the octagonal pile is also assumed to be about 5 mm. This implies that only very small settlements are required to mobilize full skin friction on a pile. In the actual project, therefore, abutment piles, for example, were designed to carry an additional load equal to the *negative* skin friction which could result from compression of the soft layers.

From the settlement-time curves it is seen that it generally takes a certain time for failure to develop. This time dependence has also been observed at nearby test embankments filled to failure.

The results of the loading tests indicate that, when a state of failure has developed, the shaft friction decreases while the point bearing increases. The ultimate point bearing capacity, Q_p , of the octagonal pile is 60 ton, which corresponds to an N_c factor of 10. If it is assumed that the shaft friction is

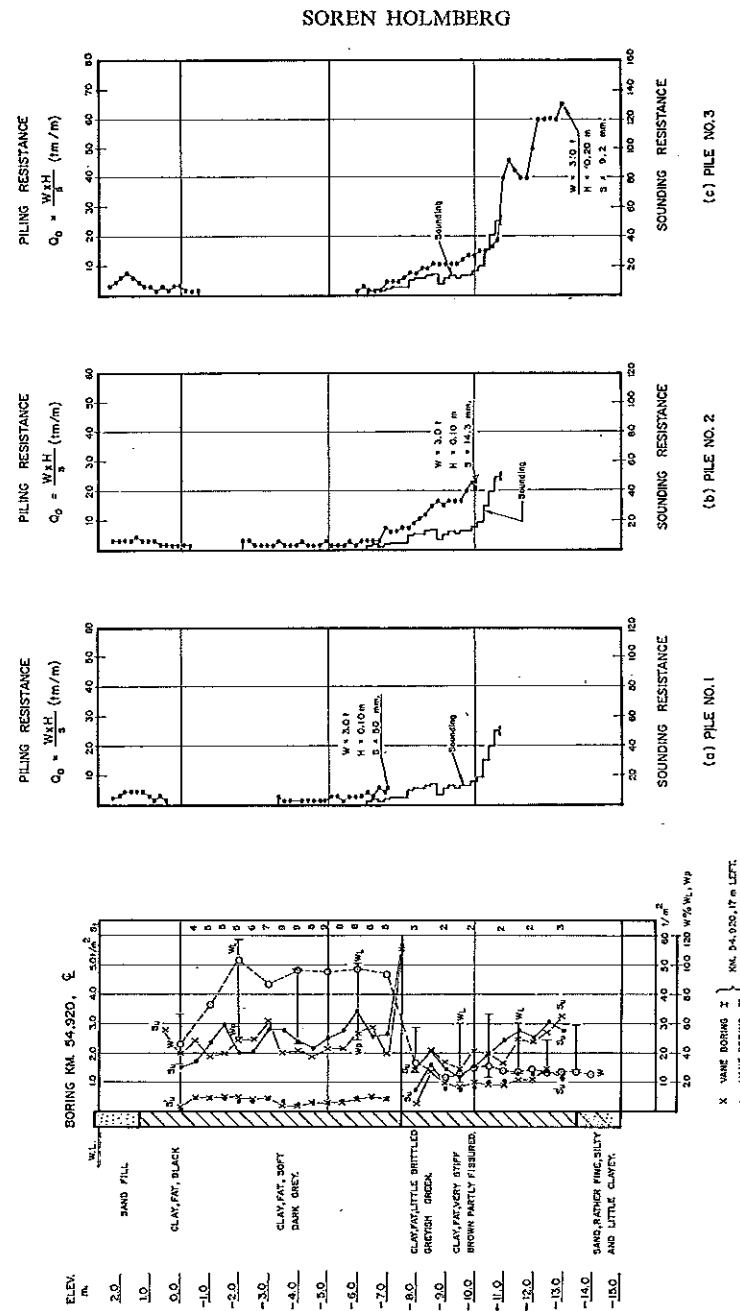


Fig. 8. Geotechnical profile at test site of square concrete piles.

LOAD TESTING OF PILES IN CLAY

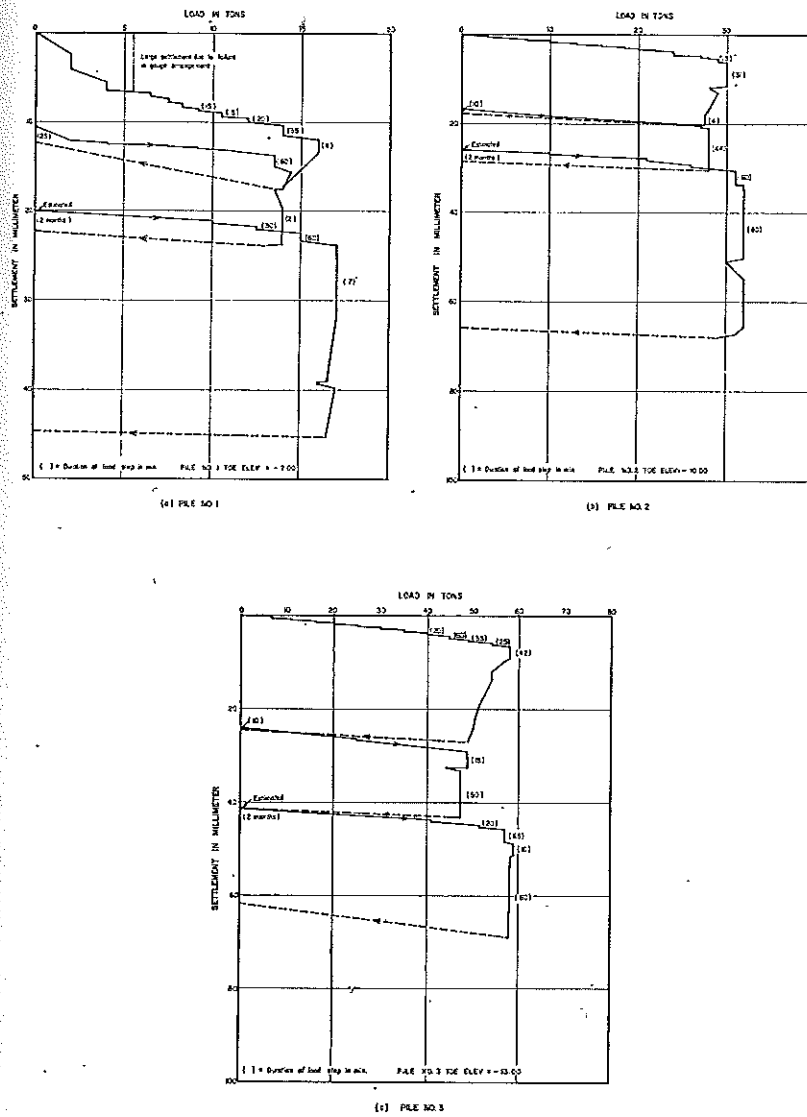


Fig. 10. Load-settlement curves for square concrete piles.

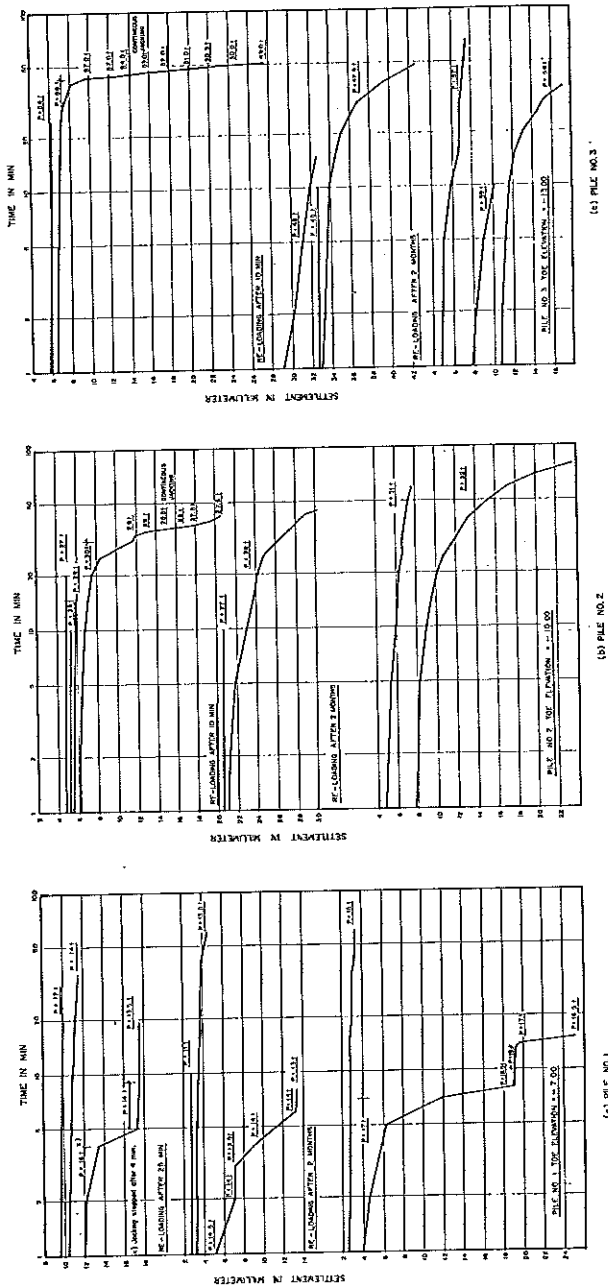


Fig. 11. Settlement-time curves for square concrete piles.

LOAD TESTING OF PILES IN CLAY

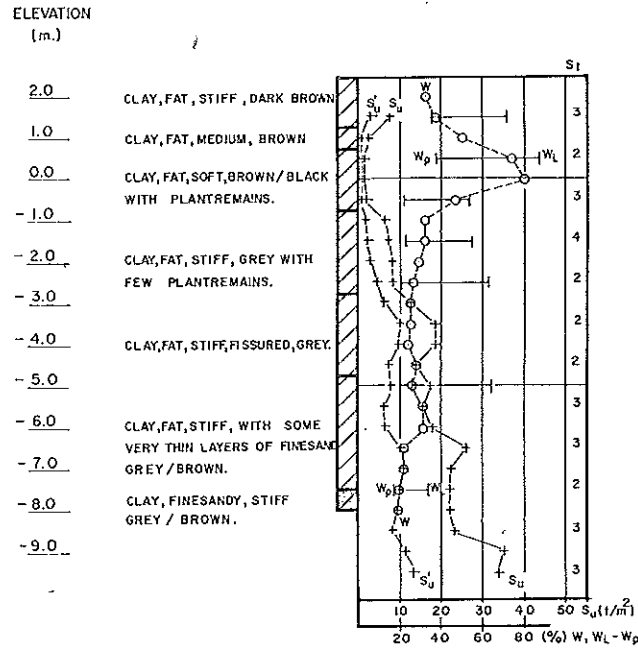


Fig. 12. Geotechnical profile at test site of wooden piles.

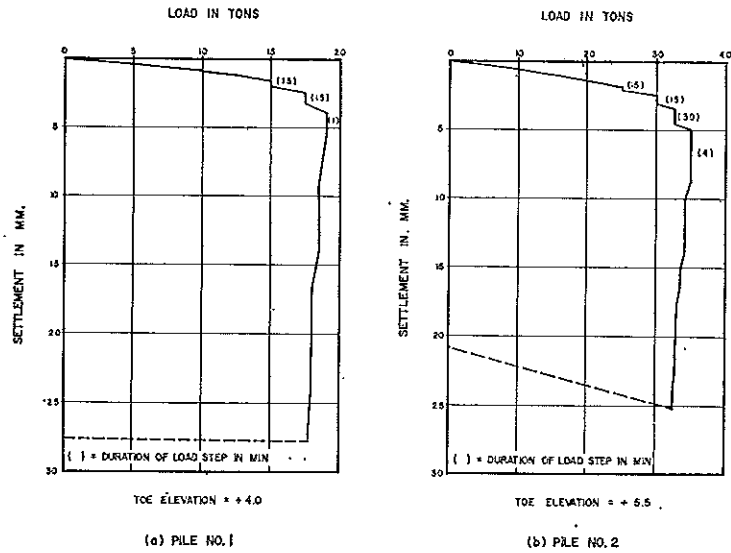
fully mobilized for a deformation of about 5 mm, and the elastic compression of the 22 x 22 cm pile is also taken into account, the mobilized point bearing capacity at failure seems to be 35-40% of Q_p . Since it is the shaft friction which governs the failure, this means that the ultimate bearing capacity of this pile should be expressed as

$$Q \approx Q_s + 0.37 Q_p \dots \dots \dots (4)$$

The order of magnitude of the mobilized point bearing will, of course, depend upon the shear strength of the clay at the pile tip and probably, to a certain extent, on the pile diameter. Unfortunately, only one test loading of a pile toe was carried out, so that it is not possible to establish any relationship between the parameters involved.

The test loadings of the 22 x 22 cm piles showed that, during the two months interval between the tests, the bearing capacity increased only slightly in the soft and medium clay, while there was no increase at all in the stiff clay; this indicates that load tests can be carried out within about two weeks of

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(a) PILE NO. 1

(b) PILE NO. 2

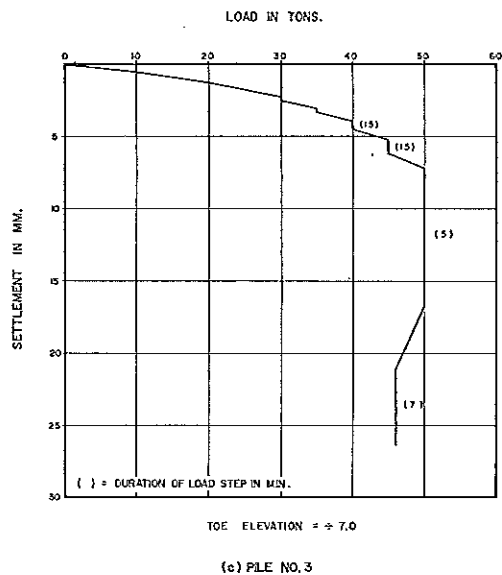
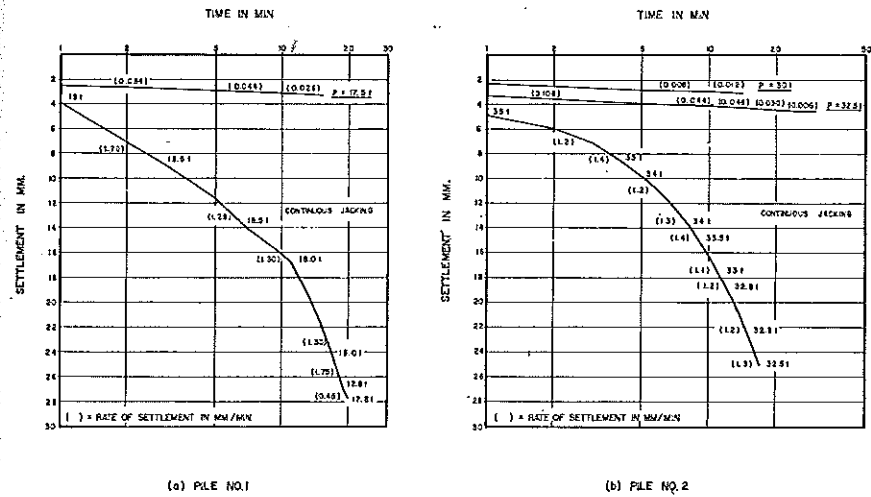


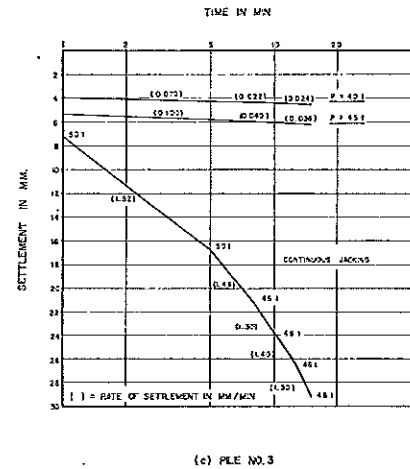
Fig. 13. Load-settlement curves for wooden piles.

LOAD TESTING OF PILES IN CLAY



(a) PILE NO. 1

(b) PILE NO. 2



(c) PILE NO. 3

Fig. 14. Settlement-time curves for wooden piles.

driving test piles. The mobilized point bearing loads are assumed to be negligible for the pile with a toe elevation of -7 m, and are taken as $0.3 Q_p$ and $0.4 Q_p$ for the other two piles with toe elevations of -10 m and -13 m respectively. By combining the test results from the three square piles and

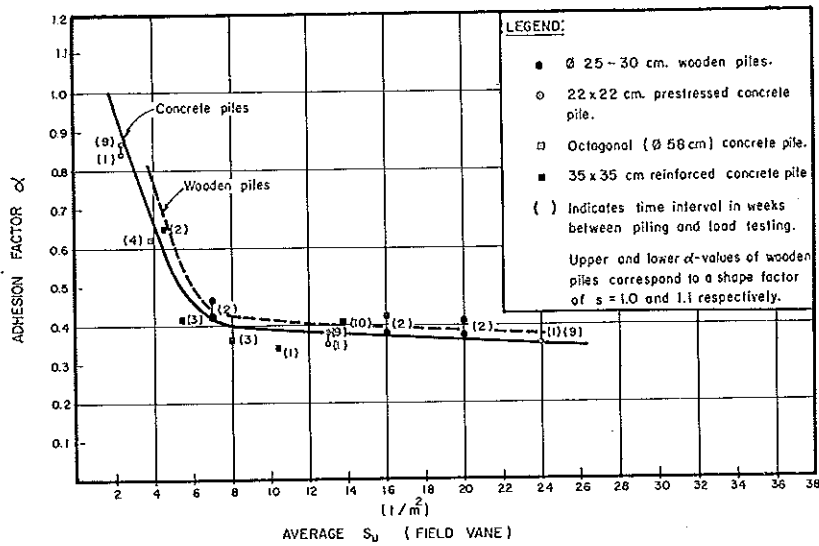


Fig. 15. Relationship between adhesion factor and shear strength of the clay.

octagonal pile, it is possible to calculate the adhesion factor as a function of the average shear strength of the different layers. The results of these calculations are plotted in Fig. 15. It is possible that all the calculated α -values for these large piles would have been slightly higher if the influence of the pile diameter on the mobilized point bearing could have been taken into account.

On the basis of all the mentioned test loadings a relationship between adhesion factor, α , and undrained shear strength, s_u , was estimated as shown by the curve in Fig. 15. The data obtained from the initial tests were then used in the calculation of some subsequent test loadings of concrete piles (35 x 35 cm) embedded in clay deposits very similar to the ones described above. Corresponding values of α and s_u from these tests are also plotted in Fig. 15.

In Fig. 16 the mobilized skin friction, αs_u (with α from the full-line curve in Fig. 15), is compared to the *in situ* shear strengths, s_u and s'_u . The comparison shows that the remoulded shear strength, s_u , of the stiff clay found by vane tests is not the true remoulded value. The indicated values of the sensitivity, $S_t = s_u/s'_u$, of the stiff clay on the boring profiles are, therefore, somewhat too low.

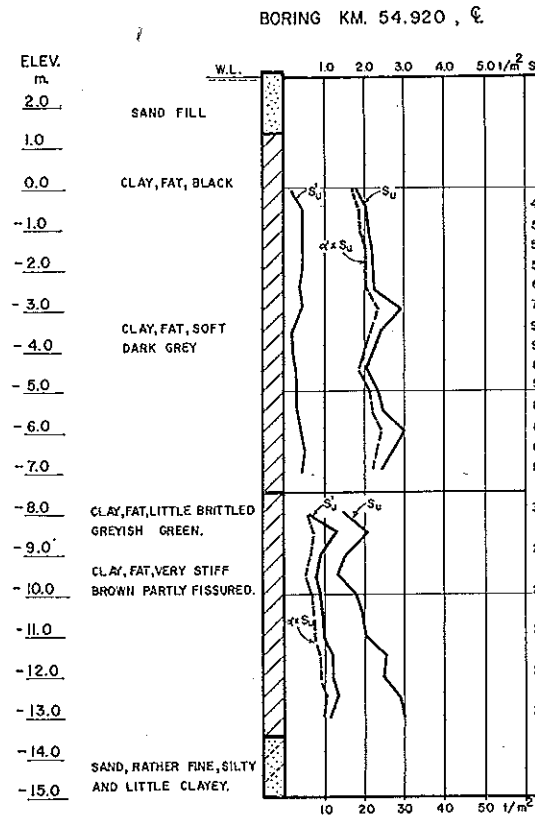


Fig. 16. Comparison of the mobilized skin friction and vane strengths of the clay.

The test results from the wooden piles have been calculated in the same way as described above for the concrete piles. The three wooden piles tested were almost cylindrical, so it is doubtful whether a shape factor should be applied at all; α -values for $S=1.0$ and 1.1 are plotted on Fig. 15. As can be seen, the adhesion factor for the wooden piles is slightly higher than for the concrete piles; this is thought to be due to better drainage along the wooden piles. The relationship between α and s_u is suggested by the dotted curve in Fig. 15.

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SUMMARY AND CONCLUSIONS

The test loadings described yielded sufficient information to obtain an indication of the failure mechanism involved when a pile embedded in saturated clay fails. The failure, which is time dependent, starts to develop when the ultimate shaft friction is reached, which happens at a very small deformation (about 5 mm). For continued deformation the shaft friction decreases while the point bearing increases, but at a slower rate. The failure, therefore, is governed by the ultimate shaft friction.

The ultimate point bearing capacity, Q_p , seems to correspond to $N_c = 10$. One test suggested that the proportion of point bearing capacity mobilized when failure starts to develop is 35-40% of Q_p . However, more testing will be necessary before a general concept of the mobilized point bearing as a function of shear strength and pile diameter can be established.

On the basis of the test results a tentative relationship between the adhesion factor, α , and the *in situ* shear strength, s_u , is suggested. This relationship will probably require minor adjustment when more details about the nature of the mobilized point bearing are available.

It is also suggested that test loadings of piles can be carried out one to two weeks after the piling has taken place.

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REFERENCES

- EIDE, O. (1968). Geotechnical Problems with Soft Bangkok Clay on the Nakhon Sawan Highway Project, *Norwegian Geotech. Inst. Pub. No. 78*.
- LUNDGREN, H. and HANSEN, J.B. (1965). *Geoteknik*, pp. 238-249. Teknisk Forlag, Copenhagen, Denmark.
- NORMER FOR BYGNINGSKONSTRUKTIONER. (1965). 6 *Fundering*, DS 415, *Code of Practice for Foundation Engineering*. Teknisk Forlag, Copenhagen, Denmark.