BEHAVIOUR OF PILES IN ALLUVIUM

COMPORTMENT DE PIEUX DANS L'ALLUVION
РАБОТА СВАЙ В АЛЛЮВИАЛЬНОМ ГРУНТЕ

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SYNOPSIS. This paper presents the main features observed in the test loading to failure of some 60 precast concrete piles which were driven into soft alluvium.The load/settlement behaviour with respect to skin friction in pulling and in pushing and in total bearing capacity and the inverse slope as a means for the prediction of their ultimate values are presented and discussed.If the pile toe is not founded on material less compressible than that encasing the pile shaft,it is possible to assess the ultimate skin friction in compression from the load/settlement results obtained in a normal CRP test.It is found that for the alluvium there is a hyperbolic dependence of pile settlement on time and that there is no correlation between final set there is a hyperbolic dependence of pile settlement on time and that there is no correlation between final set and bearing capacity.

INTRODUCTION

The Muda River scheme includes the construction of a barrage across the Muda River,water treatment works and the laying of a twin 36-inch diameter pile line of about 12,000 feet long under the sea from the mainland near Butterworth to the island of Penang.

The sites for the main engineering structures are covered with alluvium formed from recent deposits of marine and fluvial material. The soft organic alluvium extends to a depth of more than a hundred and fifty feet at the sites. Properties of such organic clays have been given by A.J. Vail (1965), F.K. Chin (1968), J.B. Cox (1970) and W.H. Ting et al (1971). The problems presented by the low bearing capacity and the high compressibility of the soft alluvium are compounded by the wide variation in the soil strata that arises from the very localised presence of thin lenses of silt and organic matter. A comprehensive series of load tests was carried out which enabled the general behaviour of the piles to be studied. About 2,500 precast concrete piles, 12-inch and 14-inch square and from 50 to 90 feet long were driven. Of these 60 were load tested to failure.

TEST LOADING ARRANGEMENT

In the load tests, a hydraulic jack was employed for loading; the reaction was provided in the conventional way by a steel cross-beam bolted to anchor piles. The hydraulic jack was placed between the pile cap and the cross-beam for the compression tests. For the pulling tests the jack was transferred on to the top of the cross-beam and reacted against a short beam which was bolted on to the steel reinforcement of the pile under test.

Where adjacent piles were not available to provide the anchorage,kentledge was used;no pulling tests were possible under this arrangement.

SKIN FRICTION IN PULLING AND IN PUSHING

As some of the piles would also be subjected to tensile loads,a simple procedure was evolved by which the skin friction in pulling Q_{st} the skin friction in pushing Q_{sc} as well as the total bearing capacity Q could be determined.One 60-feet long pile,K200,was first subjected to a CRP compression load test until failure.After a prescribed period of time had elapsed,the pile was subjected to a pulling test with the tensile load applied at the same rate as in a standard CRP compression test until the ultimate skin friction in pulling Q_{ust} was reached. This was followed by a CRP compression test. As there was a gap at the toe of the pile created by the upward displacement of the pile in the pulling test that preceded, the results of the earlier part of this compression test enabled an assessment to be made of the skin friction in compression. The downward displacement of the pile was continued until failure so as to obtain the ultimate total bearing capacity Q_u.

Figure 1 gives the load/displacement results of one such test. To assess the thixotropic recovery, this series of tests was repeated on the same pile after a lapse of 64, 67, 74, 88 and 116 days after driving, the results of which are given in Table I.

The value of the skin friction in pulling Q_{st} increased progressively as the pile was being extracted and reached its maximum value Q_{ust} at which the displacement continued to increase without further increase in load. It was thus possible to obtain the observed ultimate load, Q_{ust} from

TABLE I

Test Series No.		11	12	13	14	15	16
Date of Test		24.6.71	17.7.71	20.7.71	27.7.71	10.8.71	7.9.71
SKIN FRICTION IN PULLING	Q_{ust} - tons	-	49	42	40	39	43
	r	-	0.99956	0.99956	0.99978	0.99986	0.99903
	C	-	0.65116	0.43405	0.28314	0.19060	0.27157
	Q'_{ust} = 1/m tons	-	53.7	44.1	40.7	39.8	44.6
	Q'_{ust}/Q_{ust}	-	1.09	1.05	1.02	1.02	1.04
SKIN FRICTION IN PUSHING	r	-	0.99471	0.99715	0.99858	0.99262	0.99550
	C	-	1.92767	1.99236	1.61988	1.31559	1.51685
	Q'_{usc} = 1/m tons	-	78.2	60.8	45.4	51.8	59.6
TOTAL BEARING CAPACITY	Q_u - tons	82	80	78	82	83	80
	r	0.99708	0,99605	0.99630	0.99878	0.99832	0.99806
	C	0.99768	1.60350	3.00652	3.65087	2.33163	2.55297
	Q'_u = 1/m tons	93.5	93.7	104.6	100.3	96.5	104.7
	Q'_u/Q_u	1.15	1.17	1.34	1.22	1.16	1.31

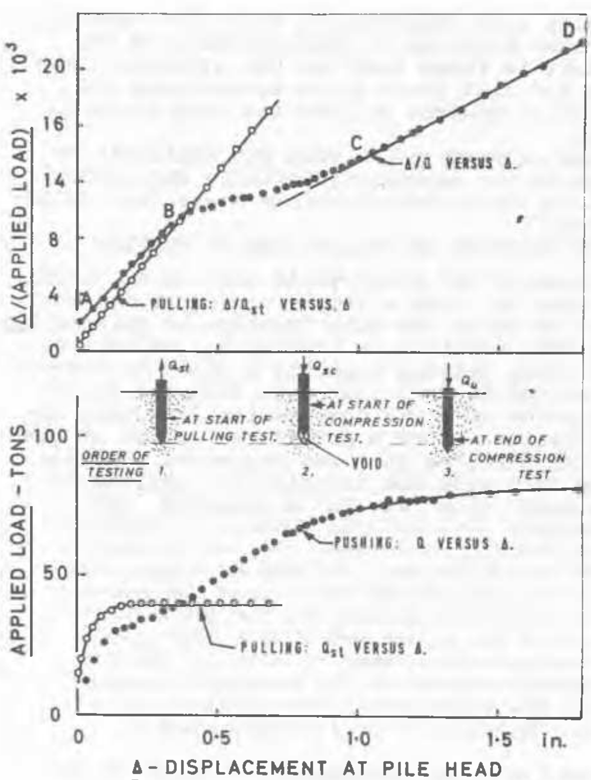

Fig. 1 Results of Test Series No. 14 - Pile K200

the pulling tests.

In the case of the pushing tests, however, it was not possible to obtain the observed ultimate value of the skin friction in pushing Q_{usc} in the same way. Though the value of Q_{sc} increased progressively, it did not reach its ultimate value before it became affected by the compression of the soil at the toe of the pile. At first it was thought that this premature interference at the toe could be eliminated by increasing the gap left by the pulling test that preceded. However, soil compression at the toe continued to manifest itself despite the provision of a much bigger gap by prejacking the pile higher before the commencement of this compression test. It was clear that the unavoidable imperfections of the pile and the non-axial application of the loading would inevitably cause the pile to move downwards in a direction that would not be concentric with that when it was extracted in the pulling test that preceded. It was, therefore, inevitable that the pile toe would impinge to a greater or smaller extent into the soil; the extent of such impingement would progressively increase with increase in its downward displacement. The ultimate value of the skin friction in compression Q'_{usc} was, however, extrapolated from the inverse slope of the plot of Δ/Q against an abscissa of Δ (F.K. Chin, 1970) using only those observed values of Q and Δ at which the soil compression at the toe was negligible, i.e. the earlier part (AB in Figure 1) of this compression test. As can be seen from Figure 1, the extent of AB which could appropriately be used for such extrapolation is clearly indicated in the Δ/Q versus Δ plot. AB is reasonably linear and beyond B there is a distinct departure from this linearity.

A linear relationship again emerges after the point C, beyond which the bearing capacity of the pile is contributed by both skin friction in compression as well as by toe resistance. The inverse slope of this line CD thus provides an estimate of the ultimate total bearing capacity Q_u of the pile.

As is seen from Table I, the average values of skin friction in pulling is about two-thirds of that in pushing. Q_{ust} must clearly be less than Q_{usc} because in pushing a pile into the alluvium, the shear

deformation of the soil encasing the pile shaft is resisted by the lower layers of soil thereby increasing their skin friction whereas in pulling no such resistance can be developed because of the free surface of the ground

Since the load/settlement relationship is hyperbolic, the ultimate load is an asymptote of the hyperbola and the settlement at which the ultimate load is reached is theoretically very large and is, therefore, not distinctly defined. It would, therefore, be necessary in a study of settlement to consider the settlement at some fraction - say at 90% - of the ultimate load. On this basis Table II is produced giving the deformations at which 90% of the skin friction in pulling and in pushing are mobilised. The magnitudes of these deformations initially decrease with each subsequent series of tests and seem to stabilise after the fourth series. The deformations at 90% of Q_{usc} are much greater than the corresponding deformations at 90% of Q_{ust}; the ratios between them increase from 3.50 to stabilise at about 4.85 after the fourth test series.

TABLE II

Test Series	Deformation when 90% of Skin Friction is mobilised		Δ_c/Δ_t
	In Pulling Δ_t(in.)	In Pushing Δ_c(in.)	
12	0.140	0.491	3.50
13	0.109	0.420	3.86
14	0.087	0.403	4.64
15	0.068	0.330	4.85
16	0.069	0.334	4.85

It is clear from the results given in Table I, that except for the first pulling test (in Test Series No. 12), there is no significant change in the values of ultimate skin friction in pulling and in total ultimate bearing capacity for the full range of the increasing periods of rest between one test and the next. This finding does not seem to be consistent with the expectation of an increase in bearing capacity with time that is given by the sensitivity of the soil as determined by insitu vane tests, and by the significant increase in driving resistance of the piles when driving was resumed after a short interruption. The average sensitivity of the alluvium as determined by insitu vane tests at the Treatment Works, the Barrage site and at the Syphon Intake are 2.05, 2.75 and 2.74 respectively. Though these vane tests were extended to depths of up to 20 feet only, their values are typical for the alluvium at greater depths (W.H. Ting et al, 1971).

THE INVERSE SLOPE AS A PREDICTION OF Q_{ust}, Q_{usc} AND Q_u

Since the pile load tests were extended until failure, this test programme provided the opportunity for the examination of the inverse slope as a means for the prediction of ultimate load. From a comparison of the observed failure loads with the corresponding values as computed from the inverse slopes, the following points are noticed:-

(a) All the test results show that the inverse slope gives a higher value of ultimate load than the observed value, e.g. those for the tests given in Table I, and

(b) The inverse slope, however, gives a closer prediction of Q_{ust} than of Q_{usc}.

These two points can be explained by the following analysis:-

It has been shown (F.K. Chin, 1970) that the stress strain or the load/deformation relationship of a soil is hyperbolic, i.e.

$$\rho/Q = m\rho + C \qquad (1)$$

where ρ is the soil deformation under a load Q. A plot of ρ/Q against an abscissa of ρ would therefore be linear and the inverse slope 1/m, of this straight line would give the ultimate value of Q.

In the pile load tests, however, it is the settlement Δ at the pile head and not the displacement ρ of the supporting soil that is being measured. The displacement at the pile head not only reflects the displacement of the soil in which the pile is embedded, but also the deformation Δ_e resulting from the elastic compression of the pile and Δ_m the deformation of the pile due to any eccentricity in the application of the test load with respect to the longitudinal axis of the pile. This eccentric loading will result in the pile being also subject to a bending moment M. The effect of M, whether the pile is being pulled or compressed, is to decrease the distance between the two ends of the pile. In the case of a compression test, the effect of M will be to increase the downward movement of the pile that is due to the soil deformation by an amount Δ_m, whereas in a pulling test it will be to reduce the upward deformation of the soil in which the pile is encased. The elastic deformation of the material of the pile will, in both pushing and pulling, be in the same direction of the corresponding soil deformation.

If Δ_1 is the observed displacement measured at the pile head under a compressive load Q_1, then

$$\Delta_1 = \rho_1 + \Delta_e + \Delta_m \qquad (2)$$

where ρ_1 is the deformation of the soil in which the pile is embedded, Δ_e is the elastic change in the length of the pile due to a load Q_1 and Δ_m is the deformation due to eccentricity in the application of the test load Q_1 with respect to the longitudinal axis of the pile.

$$\text{Therefore } \rho_1 = \Delta_1 - \delta \qquad (3)$$

$$\text{where } \delta = \Delta_e + \Delta_m \qquad (4)$$

Let Δ_2 be the observed settlement at the pile head when the compressive load is increased to nQ_1 where $n > 1$. Then the settlement due to the deformation of the soil under the new load nQ_1 is $(\Delta_2 - n\delta)$, since δ is directly proportional to applied load.

The ultimate load Q'_u as computed from the inverse slope of the plot of Δ/Q against an abscissa of Δ

is given by:

$$Q'_u = (\Delta_2 - \Delta_1)/(\Delta_2/nQ_1 - \Delta_1/Q_1)$$

$$= nQ_1(\Delta_2 - \Delta_1)/(\Delta_2 - n\Delta_1) \qquad (5)$$

The actual ultimate load Q_u, which is the inverse slope of the plot of ρ/Q against an abscissa of ρ is given by

$$Q_u = \{(\Delta_2 - n\delta) - (\Delta_1 - \delta)\}/\{(\Delta_2 - n\delta)/nQ_1 - (\Delta_1 - \delta)/Q_1\}$$

$$= nQ_1\{\Delta_2 - \Delta_1 - (n-1)\delta\}/(\Delta_2 - n\Delta_1) \qquad (6)$$

Therefore,

$$Q'_u/Q_u = (\Delta_2 - \Delta_1)/\{\Delta_2 - \Delta_1 - (n-1)\delta\} \qquad (7)$$

$$> 1 \qquad (8)$$

Pulling Test

If Q_1 is a tensile instead of a compressive force,

then $\Delta_1 = \rho_1 + \Delta_e - \Delta_m$ (9)

Let $\delta_t = \Delta_e - \Delta_m$

then $\delta_t < \delta$ (10)

It can be shown that,

$$Q'_{ut}/Q_{ut} = (\Delta_2 - \Delta_1)/\{\Delta_2 - \Delta_1 - (n-1)\delta_t\} \qquad (11)$$

(12)

where Q'_{ut} is the ultimate skin friction in pulling computed from the inverse slope of the plot of Δ/Q against an abscissa of Δ, and Q_{ut} is the ultimate skin friction in pulling which is the inverse slope of the plot of ρ/Q against an abscissa of ρ.

The results given in (8) and (12) show that the inverse slope obtained from the plot of Δ/Q against an abscissa of Δ always gives a value of ultimate load higher than the observed value.

Since $\delta_t < \delta$, it is clear that Q'_{ut} is a closer estimate of Q_{ut} than Q'_u is of Q_u.

The intercept C on the ordinate appears to be an indicator of how close the inverse slope is to the observed failure load. As is seen from Table I, the smaller the value of C, the closer is the inverse slope to the observed failure load.

It should be noted that the rather large differences between Q'_u and Q_u for Test Series 13, 14 and 16 are exceptional and are no doubt the result of bad centering of the jack. It was difficult to centre the jack due to the presence of the steel reinforcement of the pile which was used in the pulling tests. Where better centering was achieved, Q'_u is much closer in value to Q_u, as e.g. in the case of the load test of Pile U51 details of which are given in Figure 2. In this test, the ratio of the inverse slope to the observed failure load is 1.02.

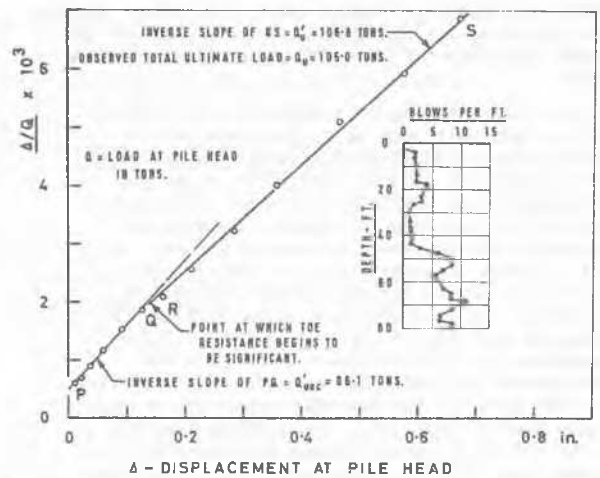

Fig. 2 Determination of the Ultimate Skin Friction in Pushing From a Normal CRP Test.

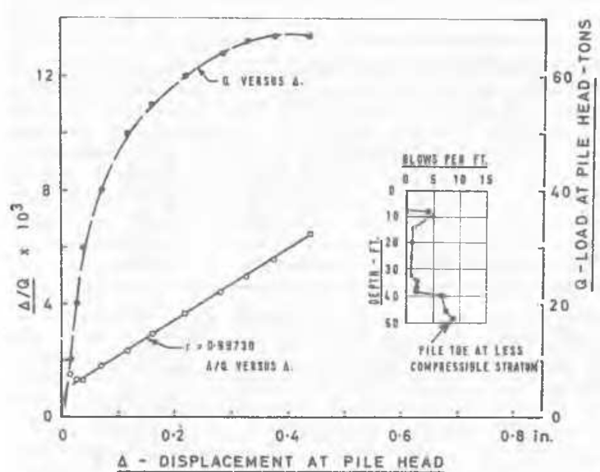

Fig. 3 Load/Settlement Behaviour of Pile D6; Toe of Pile Resting on Less Compressible Stratum Than Soil Encasing Pile Shaft.

DETERMINATION OF Q_{usc} FROM A LOAD TEST

In the case of long piles embedded in soft soils for which toe resistance requires relatively large displacements to mobilise as compared to that necessary for the full mobilisation of skin friction in compression, the initial load acting on the pile will almost entirely be borne by skin friction. This skin friction in compression may substantially be mobilised before the toe resistance begins to make appreciable contribution to bearing capacity. Such behaviour would, in the Δ/Q versus Δ plot, manifest itself as shown in Figure 2, in which the initial part of the plot consists of a straight line PQ, the inverse slope of which gives the ultimate value of the skin friction in compression Q'_{usc}. This is followed immediately by another straight line RS which has a higher value of inverse slope which gives the prediction of the ultimate total bearing capacity Q'_u. For piles responding to such behaviour, it is then possible to obtain from the results of a standard load test, the value of the ultimate skin friction in pushing as well as the ultimate total bearing capacity of the test pile concerned. Such behaviour has manifested in those test piles the lower portions of which were embedded in the soft alluvium. Figure 2 gives the results of one such pile; the inverse slope of PQ and of RS giving an estimated ultimate skin friction in pushing and an ultimate total bearing capacity of 86.1 tons and 106.8 tons respectively. For those test piles in which the toes were driven on to a layer of sandy material less compressible than the alluvium in which the pile shafts are embedded, the inverse slope plot of the observed load/settlement results does not produce two straight lines. A single line is produced instead (Figure 3) indicating that toe resistance provides a significant part of bearing capacity as soon as such a pile is loaded.

DRIVING RESISTANCE

Driving resistance measured in terms of the number of blows per foot of penetration, was recorded for all the piles. The driving resistances when matched against the corresponding observed failure loads reveal that for the alluvium no reliance can be placed on impact formulae of the Hiley type in which the ultimate driving resistance is based on final set of penetration per blow. This is clear from Table III which relates to Piles J10 and L2. While the final set of J10 is nearly 2.7 times that of L2, the ultimate bearing capacity of the former is only 1.25 times that of the latter.

TABLE III

Clear Water Tanks Site					
Pile	Drop Hammer			No. of Blows per foot at end of Driving	Observed Ultimate load obtained by CRP test (tons)
	Length (ft)	Weight (tons)	Drop (ft)		
J10	50	4	4	8	109
L2	50	4	4	3	87

A comparison of the driving resistances (Table IV) for Piles A124 and K200 gives the anomalous result that a smaller final set yields a smaller ultimate load. This contradicts the Hiley formula which states that driving resistance is inversely proportional to final set.

It was also observed that impact resistance increased abruptly when driving was resumed after an interruption. It was found that while the set did not increase with increase in length of embedment, a substantial increase in bearing capacity was nevertheless registered.

TABLE IV

Barrage Site				
Pile	Length (ft)	Hammer used	No.of Blows per foot at end of Driving	Observed Ultimate load obtained by CRP test (tons)
A124	60	Delmag D/12 diesel hammer	19	120
K200	60		36	81

HYPERBOLIC DEPENDENCE OF SETTLEMENT ON TIME

In conducting a Maintained Load test on piles embedded in plastic soils, the important question arises as to what period of time must be allowed to elapse before a fresh loading is applied. This time interval is normally determined by taking a sufficient number of time/settlement readings until the settlement under each load increment has substantially ceased. Theoretically the soil deformation, though its rate decreases rapidly with time, takes place over a duration which is too long to comply with in a load test for a working pile.

The two curve fitting methods commonly used to determine the coefficient of consolidation from oedometer tests imply that the relationship between compression and time is parabolic or logarithmic.

Suklje (1969) in his oedometer tests with dry powder of a lacustral clay found that some minutes after the application of a fresh load, the strain curve had the form.

$$\varepsilon = a + b \log_{10} t \qquad (13)$$

This logarithmic dependence of strain ε on time t continued after 120 hours of observation.

On the basis of Terzaghi's theory of one-dimensional consolidation, it has been shown (F.K. Chin, 1971) that under a constant load, the relation between compression ρ of a soil and time t is approximately hyperbolic, viz:-

$$\rho/t = At + B \qquad (14)$$

where ρ is the compression corresponding to a lapse of time t, and A and B are constants. It is clear that a plot of ρ/t against an abscissa of t will be a straight line the inverse slope of which is equal to the final settlement under the load concerned.

The programme for the load tests offered the opportunity for a study of the nature of the dependence of settlement on time in a normal Maintained Load test on a pile founded in alluvium. Table V gives a typical set of results. These results were obtained with a pile 60 feet long.

To determine whether the logarithmic or the hyperbolic dependence on time holds, the linearity of the plot of settlement against the logarithm of time was evaluated for comparison with that of the plot of t/Δ against an abscissa of t. To determine which of these two plots has the better linearity, a computer was used to evaluate the product moment correlation coefficient r from the settlement/time observations, using the method of least squares. If all the points representing the observed results lie on a straight line, that is, if there were no deviation from regression, it can be shown that $|r| = 1$. The closer r is to 1, the higher is the degree of linear association existing between the variates.

The test results show, as e.g. those quoted in Table V, that the hyperbolic dependence of settlement on time has higher values of r than the logarithmic relationship.

TABLE V

Time elapsed t-hrs.	Settlement Δ-10 in.	t/Δ x 10^{-3}	$\log_{10} t$	Product moment correlation coefficient	
				$/\Delta$ vs t	Δ vs $\log_{10} t$
Maintained load: 75 tons					
1.05	12.50	0.084	0.0212		
2.05	13.25	0.155	0.3118		
5.05	14.00	0.361	0.7033		
17.05	16.75	1.018	1.2316		
21.05	17.00	1.238	1.3232		
31.05	18.00	1.725	1.4921		
41.55	17.75	2.341	1.6185		
43.05	18.25	2.359	1.6340	.999428	.988302
Maintained load: 50 tons					
2.05	6.50	0.315	0.3118		
6.05	7.25	0.834	0.7818		
8.55	7.25	1.179	0.9320		
12.55	7.50	1.673	1.0986		
17.55	8.00	2.194	1.2442		
22.55	8.75	2.577	1.3532		
29.05	8.00	3.631	1.4631		
38.55	8.25	4.673	1.5861		
42.55	8.75	4.863	1.6289	.997666	.922144

From the final values of the settlements evaluated from the inverse slopes, it would appear that for the alluvium, there is no significant increase in settlement after a load has been maintained for about 40 hours. As a whole, about 25% of the final settlement is reached after one minute. For Pile No. b3, which is 60 feet long, the estimated final settlements under a maintained load of 50 tons and 75 tons are 0.008 and 0.017 in. respectively. It should be emphasied that these settlements are based on observations extending over durations which are normally associated with a Maintained Load test and that they do not adequately reflect the effects of creep which occur over much longer periods of time.

CONCLUSIONS

From this study of the pile driving records and the load test results, the following conclusions may be drawn in respect of the behaviour of precast concrete piles in the soft alluvium:-

(1) The ultimate skin friction in pulling and the pile displacement at which it is mobilized are less than the corresponding values for skin friction in pushing.

(2) Repetitive loading though producing little change in the values of Q_{ust}, Q_{usc} and Q_u, does significantly reduce pile displacements.

(3) The inverse slope normally overestimates the values of Q_{ust}, Q_{usc} and Q_u. The degree of this overestimation depends on the imperfections of the pile and on the eccentricity of loading. The magnitude of the intercept C is an indication of the degree of accuracy of the inverse slope as a prediction of ultimate bearing capacity.

(4) If the pile toe is not founded on material comparatively less compressible than the soft alluvium encasing the pile shaft, it is possible to assess the value of Q'_{usc} from the load/settlement results of a normal CRP test.

(5) For the duration normally associated with a Maintained Load test, there is a hyperbolic dependence of settlement on time.

(6) No reliance can be placed on final set as an estimation of bearing capacity.

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REFERENCES

CHIN, F.K.(1968),"Some soil characteristics and foundation problems", J.Institution of Engineers, Malaysia, Vol.IV, No.1, pp. 1-9.

CHIN,F.K.(1970),"Estimation of the ultimate load of piles from tests not carried to failure" Proc. 2nd Southeast Asian Conf.Soil Eng., Singapore pp.81-90.

CHIN, F.K.(1971),"Estimating the final total settlement - A theoretical analysis", Proc. Fourth Asian Regional Conf. Soil Mech. & Found. Engg., Bangkok, July, 1971, Vol.2.

COX, J.B.(1970), "Strength of recent marine clays in Southeast Asia", J. Southeast Asian Society Soil Eng., Vol.I, No.1, pp. 1-26.

SUKLJE, L.(1969), Rheological Aspects of Soil Mechanics, John Wiley, London, p. 106.

TING, W.H. and CHAN S.F.(1971), "Bearing capacity of Bakau timber piles in the coastal alluvium of West Malaysia", Proc. Fourth Asian Regional Conf. Soil Mech. & Found. Engg., Bangkok, July, 1971, Vol. 1, pp. 217-322.

VAIL, A.J. (1965), "Some deep pile foundations at Sabak Bernam", J.Institution of Engineers, Malaysia, Vol. II, No.1, pp. 3-14.