The Anticipated and Observed Penetration Resistance of some Friction Piles Entirely in Clay

Résistance à la pénétration prévue et observée de quelques pieux flottants entièrement foncée dans l'argile

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Summary

Over 1,500 concrete or timber piles were used to support structures built in the Black Bush Polder area of British Guiana. They were driven into layers of soft, firm and occasionally stiff saturated clay, and their lengths were designed by applying shear strengths measured by in-situ vane tests to the conventional " static " ultimate load formula.

Eleven of the piles were load tested to failure, two of them by Whitaker's " constant rate of penetration " (C.R.P.) method as well as by the normal load-increment system.

Whereas the timber piles, which were slightly tapered, developed shaft support equivalent to the mean of the undisturbed and remoulded vane shear strengths of the adjacent clay and up to 50 per cent more than this, the straight-sided concrete piles could not be relied on to give shaft frictions as great as even the remoulded clay strengths. A range of reduction coefficients to connect strength parameters with bearing capacity was arrived at, and although this range was wide, it gave better results than any alternative. The C.R.P. method was found to be quick and convenient in difficult field conditions, and in both cases it gave exactly the same maximum load as the load increment method.

In a study of rates of settlement and strength gain with age, an empirical equation was found to represent the long term settlement of a pile loaded nearly to failure. The increase in ultimate load found in tests carried out 3 to 11 months after driving, compared with the results at 6 to 22 days, was small : in the case of the timber piles, there was no increase.

Sommaire

Plus de 1.500 pieux en bois et en béton ont été employés pour supporter des ouvrages construits dans la région du Black Bush Polder en Guyanne Britannique. Ils ont été battus dans des couches d'argile saturée, molle, ferme et quelquefois dure ; leur longueur fut calculé par la formule conventionnelle donnant la force de rupture statique en tenant compte de la résistance au cisaillement de l'argile mesurée par essais à appareil à palettes fait sur place.

Onze pieux furent soumis à un essai à rupture, deux d'entre eux par la méthode de Whitaker à taux de pénétration constant (C.R.P.) et par la méthode normale d'augmentations de la charge.

Les pieux en bois, qui étaient légèrement coniques, développèrent le long du fût une résistance équivalente à la résistance moyenne de cisaillement de l'argile adjacente non remaniée et remaniée et même jusqu'à 50 pour cent au-delà de cette valeur, tandis que les pieux de béton, qui étaient parallèles, ne pouvaient même pas assurer une résistance égale à celle de l'argile remaniée. On arriva à une série de coefficients de réduction reliant les paramètres de résistance de l'argile avec la force portante des pieux ; quoique cette série fut dans une bande assez large, elle donna de meilleurs résultats que toute autre solution. La méthode C.R.P. fut trouvée rapide et facile en dépit des conditions ardues du terrain ; dans les deux cas elle donna exactement la même force maximum que la méthode d'augmentation de charge.

Par suite d'une étude des taux de tassement et augmentation de force portante avec l'âge, une formule empirique fut obtenue donnant le tassement à longue durée d'un pieu chargé presqu'à la rupture. L'augmentation de la charge de rupture donnée par des essais faits 3 à 11 mois après battement était petite comparée aux résultats à 6 et 22 jours : avec les pieux en bois il n'y avait pas d'augmentation.

Scope and Object of the Observations

In spite of the steady accumulation of recorded experience and research on the subject, the forecasting of the bearing capacity of piles driven into soil of known properties remains an uncertain business for designers. When support is to be derived principally from friction in soft to firm clays, semi-empirical formulae relating ultimate load to soil shear strength exist, and it can be claimed for such formulae that they represent the average relationship for a large number of tested piles. The designer, however, may be more concerned with the maximum load he can confidently expect to carry on any particular pile, and for this purpose he must allow for wide and apparently uncontrollable deviations from the assumed relationships.

The observations to be described were made primarily to obtain and check design assumptions, but they were somewhat extended in the hope of reducing the "factor of ignorance" improving the significance of soil and loading tests, and moving towards rational rather than empirical criteria, in the general case of piles in clay.

Nature of the Site and Soils

In the reclamation and drainage of the Black Bush Polder in British Guiana, which has been described elsewhere [1], a pumping station and 74 road bridges had to be built on the soft clay which covers much of the coastlands of this area

Designs had also to be considered for rice mills and associated structures in the same region. Piled foundations were necessary in all cases, and the general problem was to decide, as far in advance of construction as possible, the necessary lengths and numbers of piles for each proposed structure.

The Polder layout and the positions of the structures and boreholes are shown on Fig. 1. The area concerned, between the River Canje and Courantyne coast, is flat and is underlain by an average of 35 ft. of Demerara clay, a "recent" deposit consisting of saturated, soft blue, grey and brown clays, rarely silty and sometimes sandy, with some lenses of fine sand, and some plant remains. Reefs of this fine sand from former beaches run parallel to the coast, but all the structures are clear of these reefs.

Below the "Demerara", a firmer clay is found. This is the Coropina formation, a Pleistocene deposit, distinguished from its overburden by a lower voids ratio, siltier consistency, and considerably higher shear strength (firm to stiff). Layers or lenses of fine saturated sand are sometimes found at the boundary between the Demerara and Coropina strata. This boundary generally varies in depth from 30 to 37 ft. below ground level, but is much deeper in places. A typical borehole, with index properties, is illustrated in Fig. 2.

Design Data and Pre-piling Tests

Piles at the pumping station were required to carry loads varying between 30 tons per pile (canal full and pump chambers flooded) and zero (canal empty, chambers pumped dry). Round, tapered greenheart piles up to about 60 ft. long were the best economic choice here. Pile bents for the bridges (other than end bents, which were more lightly loaded) were designed for dead loads of 11·7 tons, plus live transient loads of up to 29 tons, and were to consist of either 4 or 5 piles, with a rigid capping beam. Bridge piles were to be of reinforced concrete, since, unlike those at the pumping station, they were partly exposed.

The properties of both the Demerara and the Coropina clays were found to vary, for a given depth, between as wide limits in boreholes a few feet apart as they did between the majority of the scattered sites shown on Fig. 1. This is illustrated on Figs. 3 and 4, on which the apparent cohesion determined by in-situ vane tests and by some triaxial tests on "undisturbed" samples is plotted against depth for both types of clay: the "remoulded" vane test results are also shown. It will be seen that for a given depth, the strengths deviate from a mean value as if testing error and soil variation together gave a random departure from the mean of all values. With the exception of borehole *BR*1, at the edge of the polder, and the pumping station area (where the sequence of strata is untypical, presumably because of past movements of the bed of the Canje River), the strength results from all boreholes showed no systematic departure from the general scatter pattern. The *BR*1 and pumping station strength results are distinguished from the others on the diagram: they are respectively lower and higher than the general run.

It is interesting to compare these figures with some given by Golder [2] for Demerara clay at Georgetown. The scatter in Golder's vane test observations was similar to that of all the bridge sites taken together, although his were quoted as applying to the case of a single bored pile, and the cohesion-

Fig. 1 Site plan showing locations of structures and boreholes.
Plan montrant la situation des constructions et des forages.

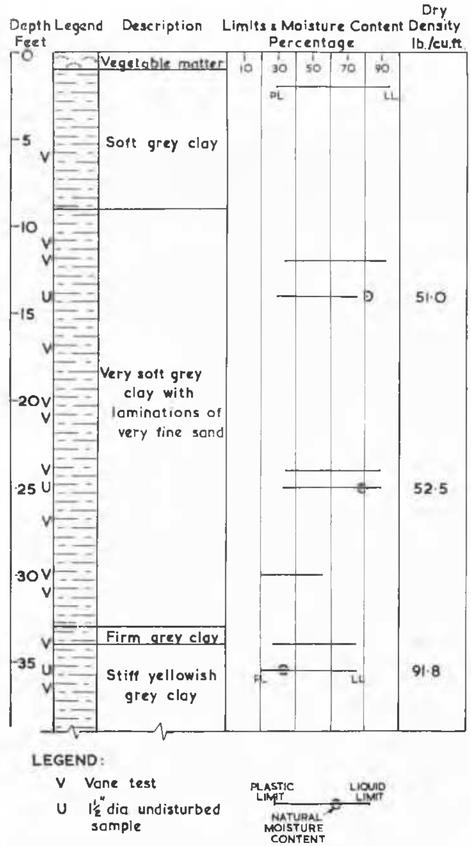

Fig. 2 Typical borehole details.
Coupe type d'un forage.

depth relationships for the undisturbed and remoulded conditions, plotted as representative of the mean strength values in the diagram in Golder's paper, have been superimposed on diagrams 3 (a) and (b). The trend of the Black Bush results towards constant strength down to depth of about 25 ft. with a linear increase at greater depths, is consistent with many recorded cases in similar clays. The triaxial tests were on unconsolidated samples and were undrained : the results were not used in calculations.

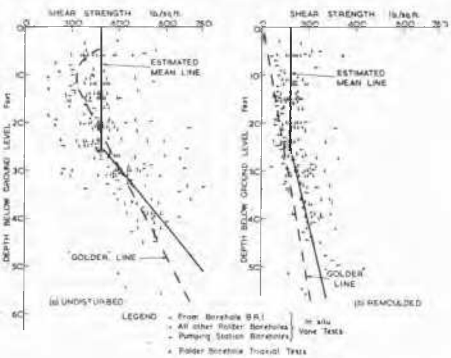

Fig. 3 Shear strength measurements in Demerara Clay.
Mesure du cisaillement dans l'argile de Demarara.

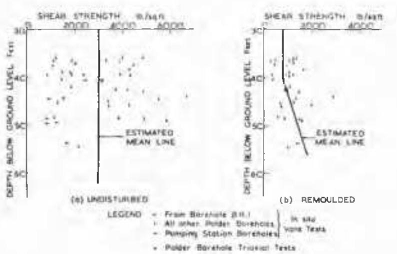

Fig. 4 Shear strength measurements in Coropina Clay.
Mesure du cisaillement dans l'argile de Coropina.

Pumping Station — Test Piles and Driving Records

Four test piles were driven near the pumping station site, on the bank of the River Canje. Details are given in Table 1. The method of test loading was to stack kentledge to produce a controlled load on the pile head, increasing the load in10-tons steps at 24-hour intervals. If movement was continuing 24 hours after the last increment, no load wad added until it appeared to stop; after the initial tests on these four piles, dial gauges were invariably used to detect and measure slow settlements. The tests on piles 1 to 4 occupied 16, 19, 7 and 5 days respectively, maximum load being reached 8, 16, 22 and 6 days after the driving of the tested pile had been completed.

As discussed in a later paragraph, there was no gain in ultimate load when piles 1, 3 and 4 were retested 3 months later.

Table I

All piles in greenheart, circular cross-section, driven by 30 cwt. hammer freely dropping 3 ft. per blow

Pile No.	*Butt girth ins.*	*Tip girth ins.*	*Length as Driven ft.*	*Penetration below ground level ft.*	*Final set (blows per ft.)*	*Maximum Load Including Pile Weight tons*	*Permanent settlement under maximum load*
1	45 ½	30	65	60	36	66	Failure
2	49 ½	31 ½	65	55	32	67·5	0·52*ins.
3	41 ½	30 ½	65	60	8	37	Failure
4	43 ½	33 ½	35	30	8	20	Failure

* Although this settlement would not normally be considered as indicating ultimate load, comparison of the initial rate of movement with records on similar piles suggests that very little additional load would have caused continuous movement. In table II Q_f for pile No. 2 has been taken as 68 tons.

In the formula

$$Q_f = N_b A_b C_b + N_s \Sigma A_s C_s \qquad .... \quad (1)$$

(in which Q_f = ultimate load on pile, A_b and A_s are the tip and shaft surface areas, and C_b and C_s are the relevant soil shear strengths), there is little doubt that the parameter N_b is about 9·5 when the tip is fully buried. (See Meigh [3] and others). N_s is considered to be a variable, depending on the extent of disturbance and compression experienced by clay adjacent to the pile shaft during driving, and on subsequent adjustments in particle structure and pore water pressures, while the cohesion values might be taken as either undisturbed or remoulded. Table II shows the values of N_s computed from formula 1, for piles 1 to 4 by substituting observed maximum loads for Q_f, and shear strengths determined by in-situ vane tests at the corresponding depths in boreholes alongside the various piles. N_b has been taken as 9·5 .

Table II

Pile No.	$N_s = \dfrac{Q_f - 9{\cdot}5\, A_b C_b}{\Sigma\, A_s C_s}$		
	(a) *Substituting Undisturbed Strengths*	(b) *Substituting Remoulded Strengths*	(c) *Substituting Mean Strengths*
1	1·02	2·45	1·45
2	1·05	2·60	1·50
3	0·68	2·15	1·05
4	0·79	2·80	1·25
Estimated Minimum value	0·65	2·00	1·00

The lengths of the 39 greenheart piles actually used in the pumping station foundation were such that Q_f estimated on the minimum values of Table 2 would be (a) 54 tons, (b) 76 tons and (c) 60 tons, compared with a maximum working load of 30 tons per pile. The structure was virtually completed by June, 1959 and no measurable movement has so far been recorded. In arriving at a target load factor for these piles, it was necessary to consider that the foundation is subject to vibration, and also to changes in total load and load distribution according to water levels in the pump wells. Significant permanent movement under load fluctuations between zero and 30 tons per pile might have caused unacceptable tilt in the structure, but the indications are that at half the ultimate load, settlement is almost entirely recoverable.

Driving resistance, measured in terms of set per standard blow, was recorded for all piles on this site, but it was confirmed that no reliance could be placed on impact formulae of the Hiley type. The driving records were of value in indicating the level at which the pile tips entered the firmer clay, but since impact resistance increased abruptly on resuming driving after an interruption, care had to be taken to compare only continuous records.

The Bridge Sites

The dead loads imposed on the pile bents by the timber beams and decks of the bridge (see Fig. 5) are small compared with the vehicle loads to be allowed for. Moreover, while many thousands of repetitions of vehicle loads up to about 5 tons per bent are foreseen, the bridges will seldom if ever have to carry anything equivalent to the nominal design criterion, viz the M.O.T. load train, which would impose 29 tons on the centre bents. The meaning of the term "factor of safety" as applied to maximum working load in this case contrasts with conditions at the pumping station. At the bridges, anything approaching maximum load would last for only a few minutes a year at most : settlement at the rate of a few thousandths of an inch per minute at full load would not have serious results, and to provide with certainty against such deflections would be wasteful, in view of the impossibility of accurate forecasting.

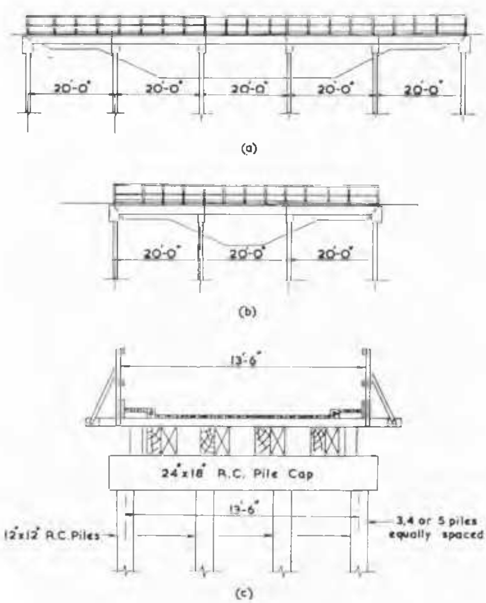

Fig. 5 Bridge designs.
Projet de pont.

All the bridge piles were of precast reinforced concrete 12" square in cross section. By varying the number of piles per bent between 4 and 5 for the centre bents, and between 3 and 4 for the end bents, the total maximum load to be carried per pile could be made either 10·9 tons or 8·2 tons. If the criteria adopted for the tapered greenheart piles at the pumping station had been applicable to these straight-sided concrete piles there would have been no need to worry about the level of the Coropina clay, as a pile driven to a depth of 35 ft. in Demerara clay with the mean strengths shown on Fig. 3 would have supported (a) 16 tons, (b) 18·7 tons or (c) 16·8 tons, according to the "estimated minimum N_s value" assumptions of Table II.

However, it was soon found that such assumptions were

Table III

All bridge piles in reinforced concrete, 12" square cross section, driven by 3 ft. free drop of 30 cwt. hammer

Pile No.	*Final set (blows per* ft.)	*Length* (ft.) *as driven*	*Length in Demerara Clay* (ft.)	*Length in Coropina Clay* (ft.)	*Time between driving and Testing* (days)	*Observed Ultimate Load in* tons *(incl. wt of pile)*
5	40	40	36·5	3	12	19·7
					305	20·7
6	11	35	34·5	nil	20	9·3
					130	10·5
7	15	40	34·9	2	60	22·7
8	5	40	32·9	nil	30	11·4
					300	14·4
9	5	40	40·5	nil	20	9·8
10	17	42	39·5	Tip only	325	18·8
11	8	42	39·5	Tip only	325	18·9

Table IV

Pile No.	$N_s = \dfrac{Q_t - 9\cdot5\,A_bC_b}{\Sigma A_sC_s}$ (using highest values for Q_t)		
	(a) *Undisturbed*	(b) *Remoulded*	(c) *Mean*
5	0·21	1·37	0·49
6	0·41	1·27	0·64
7	0·31	1·79	0·68
8	0.64	1·88	0,97
9	0·28	0·92	0.45
10	0·55 *	1·73 *	0·64 *
11	0·55 *	1·73 *	0·64 *

* For these values the coefficient of the term A_bC_b was assumed to be 7·0, as the tip only just penetrates the firmer clay.

optimistic in the case of the bridge piles. Load tests indicated lower and more variable value of N_s, with little increase with time. Details of load tests on seven bridge piles are given in Tables III and IV, for which a form similar to Tables I and II has been used.

At most of the sites it was possible for the piles to extend into the Coropina clay, thus providing for estimated failure loads of 18 tons or so (on criteria derived from Table IV) without exceeding a length of 42 feet. At 12 sites the Coropina was too deep, and the additional length of pile required in soft clay to give the same load capacity would have been excessive. It was decided to accept a lower margin of estimated penetration resistance over maximum load in these cases for the reasons given under "Design Data" above, following a study of the rates of settlement under loads near the ultimate. Several of these bridges have already carried a fair amount of heavy construction traffic : none has given trouble, but one appears to have settled evenly by approximately 1 inch.

Constant Rate of Penetration Tests

At the time of the tests described, Mr. T. Whitaker of the Building Research Station, Garston, England, was working on a new method of pile testing, in which the thrust necessary to cause a given constant rate of penetration was recorded. Although the stage of general publication of the work had not then been reached, the writer was permitted to see full descriptions of it, including detailed results from the first few field tests. It was decided to try out the method on some of the bridge piles, for the following reasons :

(a) It was desirable to know more about the relation between rates of settlement and loads near the ultimate in assessing factors of safety.

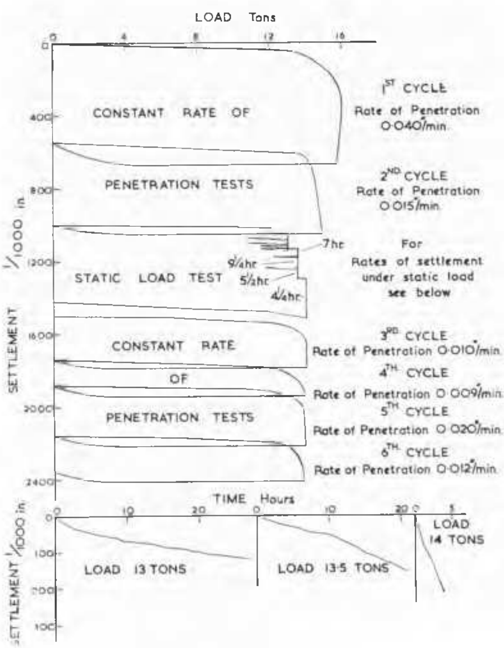

Fig. 6 Load tests on pile No. 10.
Essais de charge sur le pieu n° 10.

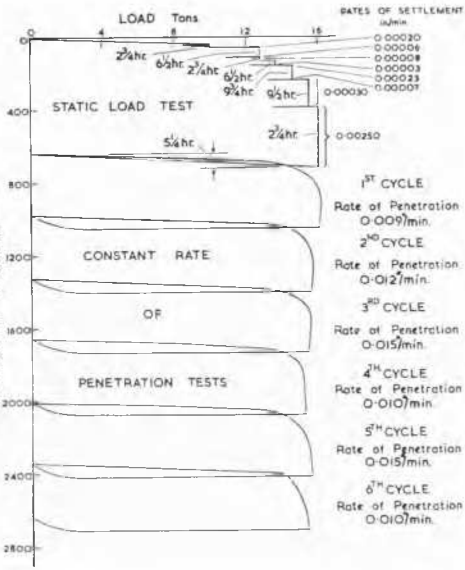

Fig. 7 Load tests on pile No. 11.
Essais de charge sur le pieu n° 11.

(b) The load-increment method can be very slow in soft clays, if settlement under each addition of load is to be complete.

(c) There is a prospect that the new method will be more definite in establishing an ultimate load, as well as being quicker.

(d) Piles in comparable strata were not available elsewhere for trials of the new method.

(e) It was thought useful to establish whetner Whitakers's method could be successfully applied with the relatively crude apparatus, difficult field conditions and limited staff resources of a remote site overseas.

In fact, two bridge piles were successfully subjected to the constant rate of penetration test, and the same piles were at the same time tested by incremental loading.

The results are plotted on Figs. 6 and 7. It will be seen that definite load envelopes, independent of the rate of penetration within the range 0·009 to 0·040 inches per minute, were obtained, and that a maximum resistance is reached on first thrust. In both cases, the C.R.P. test gave exactly the same result as a static load test performed immediately before the C.R.P. readings : where the static test was carried out first (giving 16 tons ultimate) the C.R.P. envelope gradually fell from 16 to 15·2 tons, whereas when the static test was sandwiched between C.R.P. tests, the C.R.P. envelope fell from its initial maximum of 16 tons to 15·2 tons, the static test gave 14 tons, and subsequent C.R.P. tests remained constant at 14 tons.

In these tests, as in the other bridge pile loading tests, loads were applied by jacking off the capping beam. A reference beam was supported on separate posts and relative movement was checked by cantilevers from the adjacent pile bent.

Rate of Settlement Observations

After the initial tests at the pumping station, the settlement of all tested piles was measured continuously with dial gauges

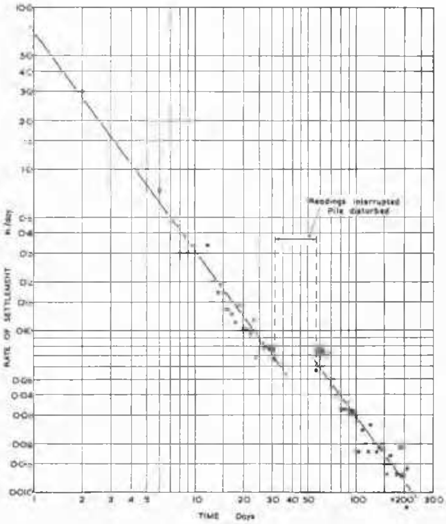

fig. 9 *a*

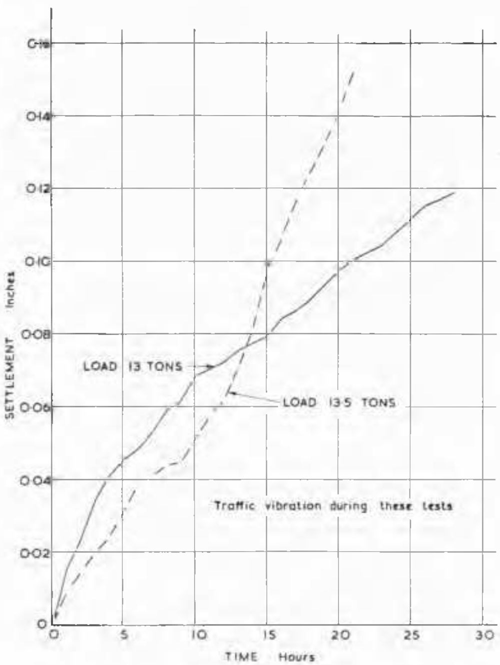

fig. 9 *b*

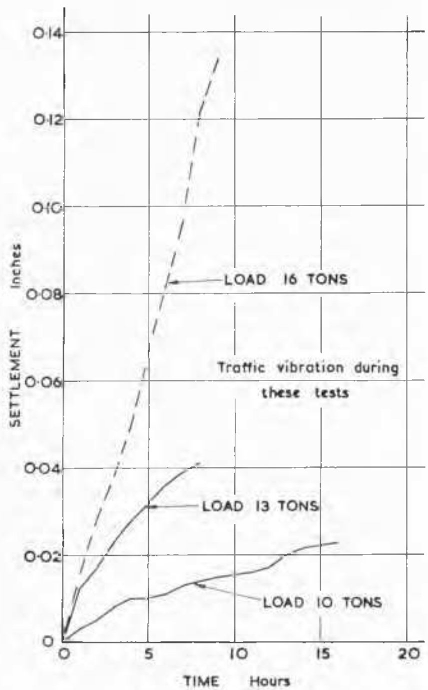

fig. 9 *c*

Fig. 9 Rate of settlement observations.
Allure des tassements.

reading in thousandths of an inch, and time-settlement curves were plotted for the various constant loads applied.

Results from three piles are presented as Fig. 9. Pile No. 4 which had failed at an applied load of 19·2 tons (the pile weight was 0·8 tons) 6 days after driving, was loaded with 18·6 tons of kentledge 3 months later, and this load was left in position for the following 209 days. Readings were unintentionally interrupted between 31 and 56 days and during this period the pile was knocked during adjustment of kentledge. When readings were resumed, the rate of settlement, which had been decreasing as shown on Fig. 9 (a) (where log rate of settlement is plotted against log time) continued to decrease according to the same relation as before, but from a higher starting value. The relation continued to hold to the end of the test, and can be expressed as

$$X = 19{\cdot}5[1-(t+1)^{-0{\cdot}36}] \qquad \ldots. \quad (2)$$

Where X = settlement in inches, and t = time in days, both measured from moment of application of load. Observed settlement started at a rate of 5 $\frac{3}{4}$ inches per day, and had dropped to 0·010 inches per day by the 209th day. If equation 2 had remained valid indefinitely, settlement would have increased asymptotically to about 19·5 inches, having reached 16·3 inches at 150 days.

Settlement-time curves for periods of constant load on piles Nos. 10 and 11 are shown on Figs. 9b and 9c respectively.

Discussion

It has been established by many workers, notably by Seed & Reese [4], that even in the most uniform soft clays, the "frictional" support given to the pile shaft by the soil is not distributed in proportion to the local shear strength at ultimate load, or indeed at any other load. Also, it seems to the writer that during driving in soft clays, soil "flow" from the compressed region near the tip, and the subsequent movement of the shaft through this disturbed soil must cause particles to be redistributed near the pile in turbulence patterns of variable void ratio. Especially with vertical sided piles, the slightest deviations from straight-line driving can clearly make important differences to the effective pressure and particle arrangement in the contact layer. Subsequent evening-out of intergranular pressures and dissipation of excess pore water pressure is very unlikely ever to restore a uniform particle structure to the stressed zones. Hence there does not seem any reason why the penetration resistance of any particular pile should be limited by the shear strength determined in any fashion before driving, even if such shear measurements can give probable average values for a large number of piles. This conclusion has often been reached (e.g. by Morgan & Haswell [5] but is worth repeating, as summaries such as those of the U.S. Highways Research Board (1958) [6], and refinements of calculation such as were proposed by Seed & Reese, may lead to unjustified reliance being placed on the static formula.

The results summarized in this paper, though limited and unrefined, tend to support the following hypotheses :

(1) That tapered piles develop a higher and less variable proportion of local soil shear strength as shaft friction than do straight-sided piles.

(2) That the fully remoulded vane shear strength does not indicate a lower limit to penetration resistance in the case of vertical sided piles.

(3) That for the conditions described in this paper, the least unreliable values of measured cohesion for use in the static formula are obtained by averaging the undisturbed and remoulded vane shear strengths, and that by substituting these values as C_b and C_s in the equation

$Q_f = 9{\cdot}5A_bC_b + \Sigma A_sC_s$ for tapered timber piles

or

$Q_f = 9{\cdot}5A_bC_b + 0{\cdot}4\Sigma A_sC_s$ for straight sided concrete piles

reasonable forecasts of minimum expected ultimate load can be made.

(4) That for piles driven into soft sensitive clay, some gain in bearing capacity sometimes occurs as months elapse after driving, but that this cannot be relied on, especially if the piles are subject to load variation and vibration.

(5) That when ultimate load is approached by a series of increments, it is possible for rates of settlement at constant load to decrease so that virtual stability is reached after a long period, to remain constant and fairly small, so that failure for practical purposes would only be reached some time after load application, or to increase from a slow start. Definitions of failure according to an arbitrary limit of total settlement, or as "continued movement under constant load", are imprecise for these conditions.

(6) That both for the reasons given above, and for practical convenience in testing, specifications for load-bearing capacity of piles in clay could advantageously be based on the thrusts necessary to cause stated rates of penetration.

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References

[1] Scott, P. A., Camacho, R. F., Sharman, F. A. (1960). The Reclamation and Irrigation of the Balck Bush Polder in British Guiana. Institution of Civil Engineers, *Conference on Civil Engineering Problems Overseas* 1960. Paper No. 4.

[2] Golder H. Q. (1957). A Note on Piles in Sensitive Clay, *Geotechnique*, vol. VII, No. 4 December 1957.

[3] Meigh, A. C. (1950). Model Footing Tests on Clay *Degree Thesis*, Imperial College, London.

[4] Seed, H. D. and Reese, L. C. (1955). The Action of Soft Clay along Friction Piles. *Proc. Amer. Soc. Civ. Eng. 81*, p. 842.

[5] Morgan H. D. and Haswell, C. K. (1953). The Driving and Testing of Piles. *Proc. Inst. Civ. Eng.* 2, 1. Jan. 1953.

[6] Peck, R. B. (1958). A Study of the Comparative Behavior of Friction Piles. U.S. Highway Research Board, *Special Report 36*.