Model Tests on Pile Groups in a Clay Soil with Particular Reference to the Behaviour of the Group when it is Loaded Eccentrically

Essais sur modèles réduits de groupes de pieux formés dans un sol argileux, en mettant l'accent sur le comportement du groupe de pieux sous une charge excentrique

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Summary

The paper describes the results of loading tests which have been completed on groups of model piles driven into a remoulded clay. The piles were ¼ inch diameter, the lengths varying in the tests from 12 d. to 30 d.

An indication of the variation of soil strength properties during the series of model pile tests was obtained by vane tests at points adjacent to the test area. The vane results are correlated with the values of soil strength obtained from undrained triaxial tests on the clay.

A fairly extensive testing programme has been completed on square symmetrical groups of nine pile with particular reference to the effect of non-axial vertical loading. Two systems of loading were used. A constant rate of vertical strain (i.e. settlement) was used to obtain comparative load-settlement conditions. The distribution of load amongst the piles in a group was ascertained in dead load tests. The trends of the values of load efficiency and settlement characteristics for the various pile group conditions and degrees of applied load eccentricity are discussed with reference to the test results.

Sommaire

L'exposé décrit les résultats d'un ensemble complet d'essais de chargement qui ont été faits sur des groupes de pieux en modèles réduits enfoncés dans une argile remaniée. Les pieux avaient un diamètre de 1/4 de pouce (soit environ 0,635 cm) et une longueur variable, selon les essais, entre 12 fois et 30 fois le diamètre.

Une indication de la variation des propriétés de résistance du sol au cours de la série d'essais a été obtenue par des essais avec dispositifs à pales faits sur différents points proches de l'aire d'essai. Les résultats des essais avec dispositifs à pales sont rapprochés des valeurs de résistance du sol obtenues au cours d'essais triaxiaux sur l'argile non drainée.

Un programme d'essais assez étendu a été mené à bien sur des groupes carrés symétriques de neuf pieux, programme portant principalement sur les effets d'un chargement vertical non-axial. Deux dispositifs de charge ont été utilisés. D'abord une contrainte verticale constante correspondant à un tassement constant a été utilisée pour obtenir des conditions comparables de tassement sous charge. Ensuite la distribution du chargement sur les pieux du même groupe entre eux a été vérifiée par des essais utilisant des poids. L'allure générale des coefficients de rendement touchant la résistance au chargement, et les caractéristiques des coefficients de tassement pour les différentes conditions de groupement des pieux et pour les différents degrés d'excentricité du chargement appliqué sont discutées à la lumière des résultats fournis par les essais.

Definitions

All dimensions are expressed in terms of the pile diameter (d).

Pile spacing (s) — the distance between the centres of any two adjacent piles in a symmetrical group.

Pile length (L) — the length of the shaft of a pile in contact with the soil.

Load eccentricity (e) — the offset of the point of application of the vertical load from the centre of a pile group along one main axis of the group.

Load condition (Q) — the ratio of the load on a pile group to the maximum (ultimate) load of the group.

Group efficiency (η per cent) — the ratio of the average load per pile at failure of the group to the load at failure of a comparable single pile.

Settlement ratio (r) — the ratio of the settlement of a group to that of a comparable single pile when both are at the same load condition.

Introduction

The influence of grouping of piles on the load capacity and settlement characteristics of a pile group has been appreciated for a considerable time. In a clay soil group action has been found to give less desirable conditions than the action of an equal number of individual piles. The knowledge of full scale behaviour is limited by the difficulties of insitu testing of pile groups. To overcome the main problems — excessive load requirements and the need for a controlled homogeneous soil — model tests have been used in various investigations of group action. The main existing recorded investigation of group action in clay (WHITAKER, 1957) dealt with axially loaded groups in a remoulded clay with a cohesion (c) varying from 0·6 - 1·3 lb./sq. in. The models consisted of symmetrical square groups of up to 81 piles with length varying from $12d$ to $48d$ and spacing from $1\frac{1}{2}d$ to $8d$.

This paper considers an extension of such tests including the effects of non-axial vertical loading. The pile groups are located at their exposed tops by a rigid cap, thus confining the relative vertical movements to a plane and maintaining the spacing constant during deformation. A remoulded clay with cohesive strength (c) between 3·0 and 6·0 lb./sq. in. was formed in a container of four or five circular rings — diameter 12 in. — each ring three inches deep. The investigations considered two main aspects. One aspect is the overall action of the group for variations in its state, such as spacing, length of pile and size of group. The load-settlement curve providing a well-defined failure condition was obtained from a "live" application of load. The second is the determination of the action of individual piles within a group. This requires a static load period dead loading was therefore used.

Consideration of Soil Properties

The soil for the investigations was a remoulded London clay with the following properties :

Liquid Limit	Plastic Limit	Clay content	Permeability K
86 per cent	23 per cent	50 per cent	$1 \rightarrow 3 \times 10^{-7}$ cm/sec.

The moisture content during the series of tests varied from 36 per cent to 30 per cent. The relationship between the moisture content and the apparent cohesion (c) shown in Fig. 1 was obtained from undrained triaxial tests on standard 1 ½ in. diameter, 3 in. long, samples and was confirmed by laboratory vane tests (vane diameter ½ in.). Correlation between the tests is attained by assuming that the vane shear failure occurs on the perimeter of a cylinder x times the diameter of the vane. By testing samples from the same soil mass with triaxial and vane apparatus and also by testing the actual triaxial samples with the vane prior to and after failure, the value of "x" was determined as 1·2.

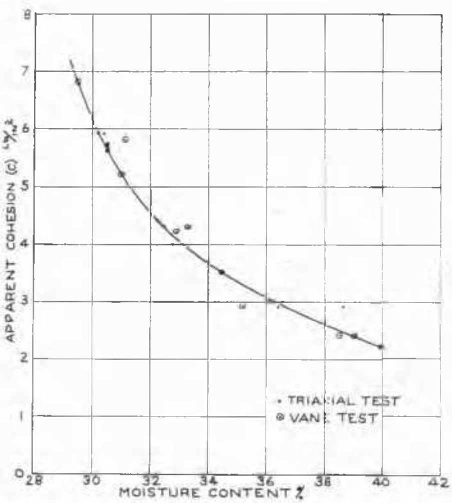

Fig. 1 Variation of apparent cohesion of soil with moisture content.

Variation de la cohésion apparente du sol avec la teneur en eau.

The laboratory vane was used throughout the model pile testing as a control of the value of shear strength in the immediate vicinity of the pile groups. In particular, tests were made on the soil immediately adjacent to the perimeter of any pile, once the pile had been pulled, by inserting the ½ in. diameter vane down the centre of the ¼ in. diameter pile hole.

As will be appreciated from fig. 1, the cohesion of the clay is very sensitive to changes in moisture content. The control strength tests showed that at any one time the variation of strength within the soil in the test containers was negligible. These control tests indicated that there is a noticeable increase in the cohesion of the soil after remoulding. An average increase in c of 12 per cent was recorded in the first two days after filling the containers and a further 2 per cent in the following two days. To give a steady test condition, therefore, a standard time lapse of four days was maintained between the remoulding of the clay and the model testing.

Single Pile Behaviour

The model piles were of ¼ in. diameter stainless steel rod with pile toe formed to a 60° cone. Four lengths of pile were used during the tests, i.e., 12 d, 18 d, 24 d and 30 d. Sufficient single pile testing was completed to enable an estimation to be made of the separate effects of the toe bearing and skin friction. Since the vane tests indicated that little change of "c" occurred with depth. The failure load of a pile can be expressed in the form

$$P_u = \frac{\pi d^2}{4} \, c N_{cq} + \pi \, d \, L \, \alpha \, c$$

where N_{cq} is the resultant bearing capacity factor (Meyerhof, 1950), and α is the degree of mobilization of cohesion along the perimeter of the pile.

Using a value of 9·5 for N_{cq}, a value of α was obtained for each of the four pile lengths

Pile length	12 d	18 d	24 d	30 d
α	0·51	0·51	0·56	0·52
Percentage of total load carried by toe bearing	27·5 %	20 %	15·0 %	13·5 %

For all lengths of pile the contribution of toe bearing is small compared with total load and consequently, toe bearing would be expected to have a minor influence on load efficiency and possibly on settlement. Group action will therefore arise primarily from the interaction of the vertical deformations of the soil masses surrounding adjacent piles resulting from the mobilization of the skin friction as the piles settle.

Fig. 2 (a) indicates the relationship between the ultimate load and the settlement characteristics of an individual pile as expressed by the inverse of the initial tangent modulus of the load settlement diagram (c_p). Within the limits of the sensitivity of measurements the separate effects of pile length and soil properties could not be distinguished. The initial settlement per increment of unit load is greater for the lower ultimate load cases, i.e. for short piles and weaker soil. The relationship between load and settlement of single piles at failure condition (Fig. 2 (b)) appears to be independent of pile length.

The Testing Rig

The rig was designed to accommodate both live and dead load conditions of test. The arrangement for dead load tests is shown in Fig. 3. In such a test, the static load applied provided its own record of the total load on the pile cap; the distri-

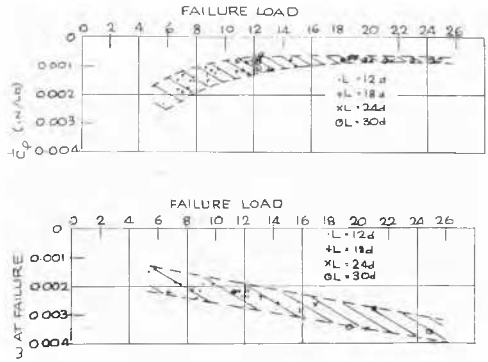

Fig. 2 The settlement characteristics of single model piles.
Les caractéristiques du tassement des pieux en modèle réduit isolés.

bution of load amongst the piles was registered by small proof rings (1 in. diameter) placed between the tops of the individual piles and the pile cap and was recorded by electrical resistance, strain gauges as described by WHITAKER (1957). Load was applied to the cap in either 10 lb. or 20 lb. increments up to failure. To enable the loads on the individual piles to be recorded after each load increment, a minimum

Fig. 3 The testing rig showing arrangements for dead load tests.
Le bâti de chargement montrant la disposition des essais de charge.

period of 5 minutes was required between successive application of load.

In the live load test a constant rate of vertical strain was applied to the top of a proof ring. The load recorded on the proof ring was transmitted from the base of the ring to the required load position on the pile cap by means of a ball bearing plunger held in a guide. The proof ring had a range of 0 - 200 lb. and a sensitivity of 1·28 lb. per 0.001 in. deformation. During the test the applied strain was absorbed in two ways :

(a) Deformation of the proof ring providing the measure of load,

(b) Settlement of the pile group.

In the initial stages of the test, the rate of settlement of the piles was negligible compared with the rate of deformation of the proof ring. During this stage, therefore, the piles were loaded at constant rate. Immediately prior to failure the change in load (and hence in proof ring deformation) was small ; the piles then settle at a constant rate. Such a condition coincides with the failure criteria that the ultimate load of a pile or a pile group is the load necessary to produce settlement at some arbitrarily chosen rate.

The following rates of applied vertical strain were selected :

(a) Groups : 0.030 in./min. — the average time for a 3 × 3 group test was 4 min.

(b) Single piles : 0.0075 in./min. — the average time of test being 1·5 min.

Tests on single piles at different selected rates indicate that the derived settlement ratios for groups will not be greatly affected by such variations. Since the ultimate load of single piles was increased by 4 per cent when tested at the rate chosen for group tests, values of efficiency are related to the rate of testing. The general trends of the results should not be altered by these effects.

Efficiency and Settlement Ratio

The efficiencies and settlement ratios of pile groups were determined from the results of live load tests. As can be seen from the typical load-settlement diagrams in Fig. 4, the maximum loads are well defined and are followed by a period of settlement under constant or slightly decreased load. A fairly

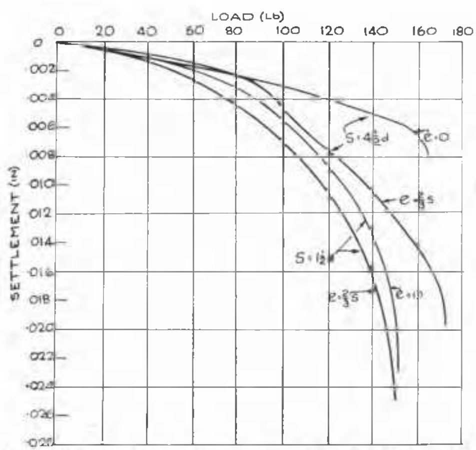

Fig. 4 Typical load-settlement diagrams for groups of 3 × 3 piles.
Diagrammes charge-tassement types pour des groupes de 3 × 3 pieux.

extensive programme of testing has been completed on groups of 3 × 3 piles under both axially and eccentrically applied loads. Two eccentricities of load (1/3 s and 2/3 s) were used throughout the testing. Three pile lengths were considered, i.e. 12 d, 18 d and 30 d, and the pile spacing varied from $1\frac{1}{2}d$ - 5 d; further tests are proceeding with larger size groups and groups with asymmetrical spacing.

The typical trends in efficiency and settlement ratio for a group of 3 × 3 piles are indicated in Figs. 5 to 8.

Load Efficiency

(Fig. 5) – when the load carried by the toe is appreciable as for short piles (28 per cent when $L = 12d$), the efficiency is related to the pile length. The curves of experimental observations for both $L = 18\ d$ and $L = 30\ d$ cannot be distinguished, the corresponding values for $L = 12\ d$ are, however, about 6 per cent higher. Consequently, for most practical purposes (length $L > 18\ d$ spacings 3 d to 6 d.) efficiencies in the range 80 to 90 per cent are indicated. Larger groups will tend to have lower efficiencies.

The efficiency is not noticeably affected by eccentric loading in the range of eccentricities up to 2/3 spacing. Average points for $e = 2/3\ s$ have been included in Fig. 5.

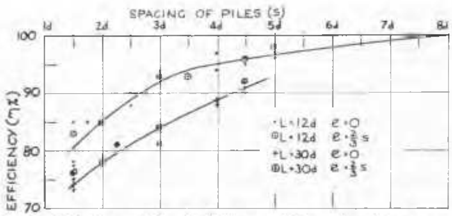

Fig. 5 Variations of load efficiency with spacing for groups of 3 × 3 piles under axial ($e = 0$) and excentric ($e = \frac{2}{3} s$) loading.

Variation de l'effet de la charge avec l'écartement des pieux pour des groupes de 3 × 3 pieux chargés en leur centre ($e = 0$) et excentriquement ($e = \frac{2}{3} s$).

Settlement Ratio

The variations of settlement at failure with pile length for any given spacing, Fig. 6, are within the experimental limits of estimation of the settlements of control piles. Clearly very large spacing of piles would be necessary to eliminate all group action effect on settlement.

A comparison of the results of eccentrically and axially loaded groups is shown in Fig. 7. For the larger spaced groups eccentric loads tend to produce higher mean settlements at failure than axial loads. Such marked increases in failure settlement possibly arise from a partial plastic behaviour of the soil structure about one line of piles prior to failure of the group.

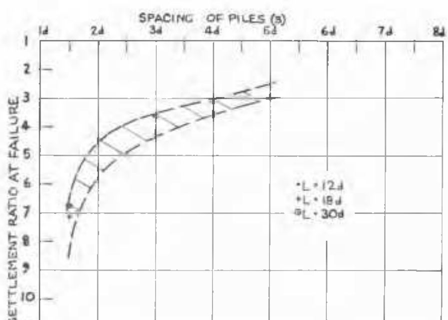

Fig. 6 Variation of settlement ratio at failure with spacing for groups of 3 × 3 piles under axial loading.

Variation du tassement en fonction de l'espacement des pieux à la rupture pour des groupes de 3 × 3 pieux avec chargement axial.

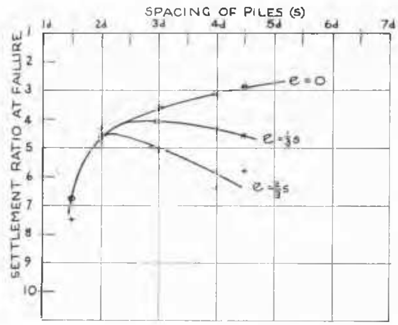

Fig. 7 The effect of the excentricity of applied load on the settlement ratio at failure of groups of 3 × 3 piles $L = 30\ d$.

Effet de l'excentricité de la charge sur le rapport du tassement à l'écartement dans le cas d'un groupe de 3 × 3 pieux de longueur $L = 30\ d$.

The variations of the settlement ratio with spacing at $Q = 50$ per cent, Fig. 8 (typical working load) are far less marked than at $Q = 100$ per cent. A gradual decrease in settlement ratio occurs as the spacing is increased and there is only a very slight difference between the axial load and maximum eccentric load cases for all spacings. This indicates that the differences between settlement ratios for $Q = 100$ per cent obtained from axially and eccentrically loaded groups are due to differences in behaviour of the groups after the load on the groups has exceeded the normal working range (i.e. $Q = 50$ per cent).

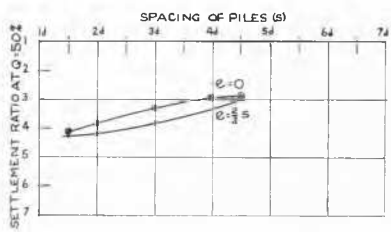

Fig. 8 The variation of settlement ratio at 50 per cent load with spacing for groups of 3 × 3 piles.

Variation du tassement en fonction de l'écartement de groupes de 3 × 3 pieux pour des charges égales à la moitié de la charge de rupture.

Distribution of Load Amongst Piles in Group

The typical behaviour under dead load of the total reaction of each row of piles normal to the axis of load symmetry is indicated by the graphs (a) and (c) in Fig. 9 for a group of 3 × 3 piles, $s = 4{\cdot}5\ d$ and $L = 30\ d$. The total load-settlement diagrams obtained by live load tests for a comparable group are shown in Fig. 9 (b).

Three positions of applied load were investigated.

(1) *Axial loading* $e = 0$—The distribution of load was symmetrical and no individual pile in the group failed until the total load on the piles was greater than 90 per cent of the failure load of the group. Since the distribution of load was almost uniform, no detail of the observations is recorded.

(2) *Eccentric loading* $e = 1/3\ s$ (Fig. 9a)—Under this condition of loading the two rows of piles nearer the load application point (rows A and B) both failed when 80 per cent of the failure load of the group had been applied to the pile cap. The failure of these two rows can be associated with the sudden change of slope in the total live load-settlement graph

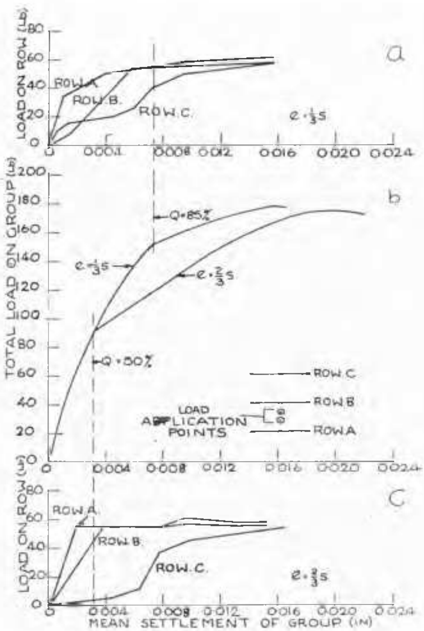

Fig. 9 Load distribution to rows of piles normal to axis of load symmetry for a group of 3 × 3 piles

$S = 4\ 1/2\ d \quad L = 30\ d$

a) $e = \frac{1}{3} S$

b) comparable results from live load tests

c) $e = \frac{2}{3} S$

Répartition de la charge à des lignes de pieux normales à l'axe de symétrie de la charge pour un groupe de 3 × 3 pieux

$S = 4{\cdot}5\ d \quad L = 30\ d$

a) $e = \frac{1}{3} S$

b) résultats comparables avec charges mobiles

c) $e = \frac{2}{3} S$

(9b) at $Q = 85$ per cent. Failure of the row of piles furthest from the load application point (row C) occurs at the failure of the group.

(3) *Eccentric loading* $e = 2/3\ s$ (Fig. 9c)—With this largest eccentricity three phases were observed:

(a) $Q = 0$ per cent - 40 per cent — load on the row of piles nearest application point (row A) increased to failure of the row, load on centre row (B) increased to approximately 50 per cent of its failure load, and the load on the row of piles furthest from the load (C) was negligibly small.

(b) $Q = 40$ per cent - 60 per cent — row A continued to carry its failure load, whilst the load on row B increased to failure of this row. Effectively there was still no load carried by the piles in row C.

(c) $Q = 60$ per cent - 100 per cent — all additional load applied during this phase was carried by row C.

The sudden change of slope in the total live load-settlement diagram for $e = 2/3\ s$ at $Q = 50$ per cent is associated with the failure of the row of piles nearest the load application point at $Q = 40$ per cent, and is magnified by the failure of the centre row of piles at $Q = 60$ per cent.

The dead load test showed that the high failure settlements obtained for groups of spacing above 3 d under eccentric loading are due to failure of individual piles in the groups prior to complete failure of the groups, as in nearly all cases no pile has failed before $Q = 50$ per cent, it would be expected that eccentricity of load would have little effect on the settlement ratio at working loads ($Q = 50$ per cent) for the small groups considered. This fact has been shown by the results of live load tests (Fig. 8).

Conclusions

This paper has considered the results of loading tests on 3 × 3 groups of $\frac{1}{4}$ diameter model piles driven into a remoulded clay.

Ultimate load

(1) For all practical spacings and lengths of pile the efficiency of the group is within the range 80 per cent - 90 per cent.

(2) The efficiency of the group is influenced to a slight extent by the rate of testing. The general trend of results should be unaffected.

(3) For short piles ($L < 18\ d$) the efficiency for any given spacing is related to pile length. For pile lengths greater than 18 d where the toe-bearing contributes less than 20 per cent of the total load carried by the pile, the efficiency for any given spacing appears to be independent of pile length.

(4) Eccentric loading does not noticeably affect the ultimate load capacity of the group.

Settlement

(5) Under all conditions of loading the settlement ratio of the 3 × 3 group is independent of the length of pile.

(6) The settlement is increased by at least threefold ($r = 3$) for the normal range of spacing. Very large spacing would be necessary to eliminate such effect. Symmetrical square groups of more than 9 piles are expected to have an even more marked trend and the effect of length may then become significant.

(7) The effect of eccentric loading or close spacing on settlement for $Q = 100$ per cent is very marked. The settlement to failure under eccentric loading ($e = 2/3\ s$) can be over twice the settlement to failure under axial loading for comparable groups. However, eccentric loading does not seriously the working load settlements.

(8) The behaviour of the individual piles in an eccentrically loaded group is related to the settlement characteristics of the group. An examination of the distribution of load amongst the piles has shown that the large settlement to failure of an eccentrically loaded group is due to a partial plastic behaviour of the soil structure about individual piles prior to failure of the group.

The interaction of piles within a group has a considerable effect on the behaviour of the group. The increase in settlement is particularly marked, both the position of load application and the geometry of the group affect the value of settlement at failure. Further model investigation of these effects are proceeding.

References

[1] MEYERHOF, G. G. (1950). The ultimate bearing capacity of foundations, *Geotechnique*, vol. 2, p. 301.

[2] WHITAKER, T. (1957). Experiments with model piles in groups, *Geotechnique*, vol. 7, pp. 147-167.