BEHAVIOUR OF SINGLE PILE IN EXPANSIVE CLAY

by

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ABSTRACT This paper describes the results of a series of laboratory experiments carried out to study the behaviour of piles in expansive clay.

Measurements are presented for the progress of swelling with time, at various points in the clay after it is subjected to a controlled change in moisture content. Measurements of pile head movement and tensile force in the pile are then presented with particular attention being paid to their relationship with soil surface movement.

The influence of a number of variables is then examined including:

i) compressive axial load on the pile,
ii) an applied overburden pressure on the soil surface,
iii) the presence of a stable sand layer below the expansive clay layer.

Finally, comparisons are made between the observed behaviour and that predicted from a theoretical boundary element analysis. The general trends of behaviour are adequately reproduced by the theory, but the detailed variations of pile movement and pile force with time are not always predicted accurately by the theory employed. Suggestions are made for refinements to the theory to reproduce more closely the real behaviour.

INTRODUCTION

Piles are frequently used to support structures on expansive soils, as they can substantially reduce the amount of movement which the structure may undergo if the soil shrinks or swells. Despite the widespread use of piles under these circumstances, data on the behaviour of piles in expansive soil are sparse.

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Model pile test data have been presented by Chen (1965), Sorochan (1965) and Shalaby (1989). Collins (1958) and Donaldson (1967) have presented field data on movements and forces in single piles in swelling soils, while O'Neill and Poormoayed (1980) have reported measurements of load distribution in a loaded belled pier with soil uplift being inititated by ponding water over the test site. More recently, Blight (1984) has reported tensile forces developed in three piles within a group, while BRE (1986) have presented results of tests on uniform-diameter and under-reamed piles at a site in India.

Relatively simple methods of design of piles in expansive soils have been proposed by Collins (1953), O'Neill and Poormoayed (1980), Garcia-Iturbe et al (1980), and Chen (1975), but the reliability of these methods has not been well-established. A theoretical approach for the behaviour of piles in expansive soil has been presented by Poulos and Davis (1980), using a simplified boundary element analysis, and design charts have been presented relating the pile movement and the force induced in the pile to the "free-field" soil movement and the soil properties. Some further solutions have been presented by Poulos (1989).

Despite the existence of theoretical relationships between the soil movements and pile behaviour, there is relatively little published experimental data which can be used to check these theoretical relationships. Consequently, a programme of model pile testing has been undertaken by the authors with the following aims:

i) to measure the relationship between the movement of a swelling soil and the movement of a pile in the soil, for different conditions of embedment of the pile.,

ii) to measure the evolution of force with time at various locations along the pile,

iii) to compare the measured behaviour with that predicted by a theoretical approach based on the boundary element method.

This paper describes the test procedures employed and presents experimental results for the swelling movements within the clay, the movements of the pile head and the force distribution along the pile. Finally, comparisons between theory and experiment are presented, and some of the limitations of the theory are identified.

CHARACTERISTICS OF SOIL TESTED

The clay used in the test programme was a black clay from Karan Downs in Queensland, Australia and its physical properties are indicated in Table 1. The swelling potential of the clay was investigated by mixing samples at the liquid limit and then allowing them to air dry to different moisture contents. The samples were then placed in an oedometer ring and allowed to saturate under an applied pressure of 15 kPa. The progress of swelling movement with time was measured for each sample. Figure 1 plots the maximum swelling movement (as a percentage of initial sample height) against initial moisture content, and it can be seen that the swelling movement increases as initial moisture content decreases. At a moisture content of about 30% (i.e. near the optimum moisture content for standard compaction), the swelling was of the order of 8% and this was considered to be suitable for the model pile tests.

In addition to the clay, a non-swelling silica sand was used in some of the tests to provide a stable founding layer. This material was a relatively uniformly graded Sydney beach sand with the D_{60} value of 0.32 mm, and a uniformity coefficient of 1.45. Maximum and minimum unit weights values were 15.9 and 14.1 kN/m³ respectively.
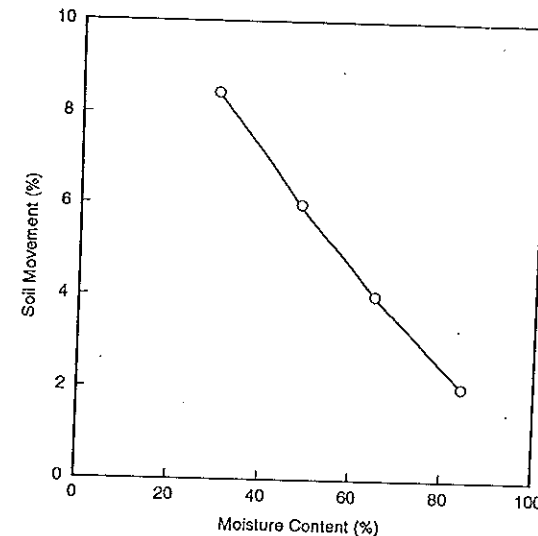

Fig. 1 Soil movement vs initial moisture content

Table 1 Soil Properties

| PROPERTY	VALUE
percentage passing No. 200 sieve	62.8
liquid limit	91%
plasticity index	49%
shrinkage limit	8%
optimum moisture content (standard compaction)	34%
specific gravity	2.54

TEST APPARATUS AND PROCEDURES

Testing Vessel

All pile tests were carried out in a cylindrical steel vessel of internal diameter 302 mm and depth 382 mm. The vessel is shown diagrammatically in Figure 2. The vessel basically consisted of a central body section, a base plate and a top lid. The base and the top of the vessel were grooved allowing rubber O-rings to be fitted, thus ensuring water-tight seals. The cell pressure was applied inside the lid of the apparatus using pressurised water from an air/water exchange pot. A rubber member of 1.5 mm thickness was used at the top of the vessel to eliminate direct contact of the pressurised water and the clay and pile. Back pressure was applied to the specimen, with two water inlets being provided on the top of the vessel (item N_1) and in the centre of the base plate (item N_2) as shown in Figure 2. These inlets were provided with fine wire mesh and filter paper to allow free flow of water. An average of three weeks of swelling was allowed for each test. Swelling movements at various levels in the vessel were recorded and printed using a 3052A Hewlett-Packard Data Acquisition System.
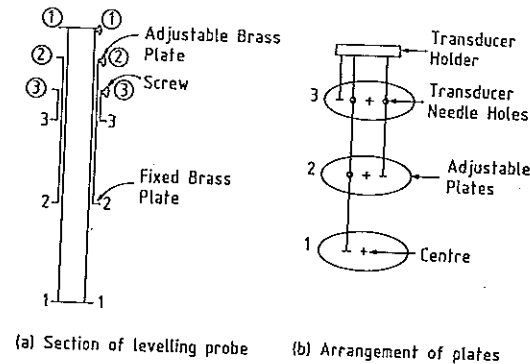
Levelling Probe

A levelling probe was designed to determine the movements of the clay at various levels in the closed vessel, and its location in the vessel is shown in Figure 2. The probe is shown diagrammatically in Figure 3 and consisted of three concentric hollow tubes and a rod which could slide one over the other. Two brass plates were connected to the end of each tube and the rod. The plate was fixed at the bottom of each tube or rod and was adjustable at the top. The
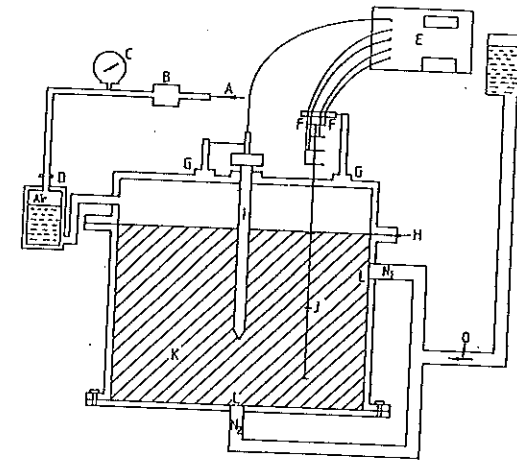
A Compressed Air
B Pressure Reducer Valve
C Pressure Gauge
D Air/Water Exchange Pot
E Data Acquisition System
F Displacement Transducer
G Magnetic Clamps
H Rubber Membrane
I Pile
J Levelling Probe
K Clay
L Filter Paper and Wire Mesh for Drainage
N_1, N_2 Top and Bottom Inlet
O Back Pressure

Fig. 2 Apparatus Set-up for Instrumented Pile Tests in Expansive Clay.

(a) Section of levelling probe   (b) Arrangement of plates

Fig. 3 Details of Soil Levelling Probe.

fixed plates were placed at pre-determined levels at the time of packing the clay. The upper adjustable plates had 3 mm diameter circular holes for the transducer needles to pass through, and displacement transducers were used to measure the movements of the soil at the various levels in the vessel.

Sample Preparation

The clay was thoroughly mixed to bring the moisture content close to its liquid limit. The sample was then broken into small pieces and to attain uniform drying, very thin layers of soil were placed in an oven. The sample was then dried at 105° C to about its optimum moisture content (standard compaction) of 34%.

At the beginning of the test fine wire mesh and wet filter paper were placed on the base plate above the bottom water inlet; similar arrangements were repeated for the top inlet. Clay was then compacted in several layers, 35 mm thick layers being compacted to about 25 mm using 25 blows with a hammer weighing 2.5 kg and dropping 400 mm. In order to measure swelling, the displacement probes were placed at various levels midway between the pile and the vessel wall during the packing of the clay. The outermost probe was fitted to the membrane which rested on the surface of the clay. After placing the membrance on the clay surface, the top lid was placed over the membrane, the apparatus assembled, and the pile was installed.

Pile Installation and Testing

Tests were conducted on an alumimium instrumented single pile, of 25 mm diameter and 230 mm length, which was instrumented at four locations with strain gauges. The pile was jacked into the compacted clay sample and level measurements and strain gauge readings in the pile were recorded during the jacking. Movements of the probes at various levels in the clay were measured and found to be very small. To prevent the release of overburden pressure around the pile, a sealing arrangement was used, of a type similar to previous researchers (Lee 1987, Allman 1988).

After jacking of the pile was completed, the overburden pressure was increased to the value to be used for the test. Wetting of the sample was carried out by application of back pressure. Both the overburden pressure and the back pressure were constant throughout the swelling test. Loads in the pile and movements of the soil were measured continuously during the experiment.

After swelling of the clay at the specified overburden pressure, the pile was connected to a loading machine and loaded at failure at a rate of about 0.1 mm/min. This test was done to determine the axial load capacity of the pile at the end of the swelling test and also to enable the Young's Modulus (E_s) of the soil to be backfigured from the load-settlement behaviour.

TEST PROGRAMME AND RESULTS

The model test programme involved a total of seven tests in three categories:

a) three swelling tests on the clay without a pile installed,

b) two tests on a pile embedded entirely in the clay, one with, and one without, an axial load,

c) two tests on a pile embedded in clay with sand underneath.

Details of these tests are given in Table 2.

The test results will be presented in the following order:

i) swelling movements in the clay as a function of tme,
ii) pile behaviour at installation,
iii) pile load capacity after soil swelling,
iv) pile head movements during soil swelling,
v) forces induced in pile during soil swelling.

Swelling Movements in Clay

Figure 4 plots the measured surface movement versus time for the three clay swelling tests. The amount of swelling for test T01 (carried out with an overburden pressure of 30 kPa) is significantly less than for the tests T02 and T03 (carried out at 15 kPa overburden pressure). It was therefore decided to use an overburden pressure of 15 kPa in the pile tests to enable larger soil movements to be developed. Test T02 and T03 were nominally replicate tests, but showed considerable difference in the amount of swelling developed at early times. However, after about 15 days, the amount of surface swelling was similar in both tests, about 15 mm. It is abvious that the swelling process had not been completed after a period of about 20,000 minutes (15 days), but because of time constraints, it was not possible to carry on either the clay swelling tests or the pile tests for periods longer than about 20 days.

Table 2 Detailed Description of Tests

Description	Test No.	Location of Levelling Plate From Bottom (mm)		Sample & Thickness (mm)	Overburden Pressure (kpa)	Initial Moisture Content (%)
Clay Swelling Testing						
Clay Swelling Test	T01	Top Intermediate Bottom	= 382 = 230 = 115	Clay = 382	30	30 – 35
Clay Swelling Test	T02	Top Intermediate Bottom	= 382 = 280 = 115	Clay = 382	15	31 – 35
Clay Swelling Test	T03	Top Intermediate Bottom	= 382 = 282 = 115	Clay = 382	15	30 – 36
Pile in Clay						
Clay Test	T04	Top Intermediate Bottom	= 382 = 230 = 115	Clay = 382	15	28 – 33
Clay Test with Axial Load	T05	Top Intermediate Bottom	= 382 = 230 = 115	Clay = 382	15	29 – 35
Pile in Clay and Sand						
Clay-Sand Test	T06	Top Intermediate Bottom	= 382 = 260 = 120	Clay = 230 Sand* = 152	15	30 – 38
Clay-Sand Test	T07	Top Intermediate Bottom	= 382 = 300 = 230	Clay = 150 Sand* = 232	15	31 – 37

* sand layer at bottom overlain by clay layer
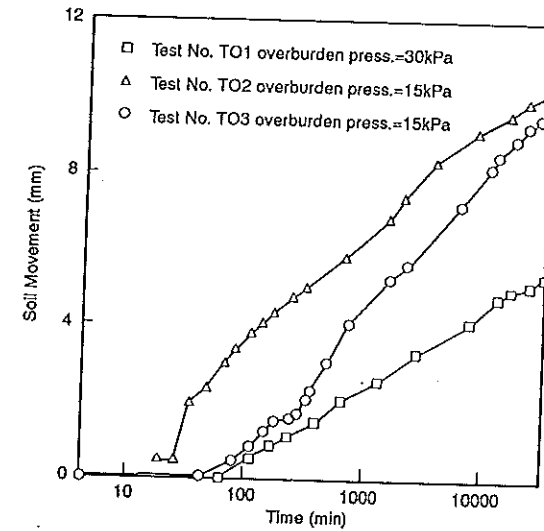
Fig. 4 Measured Soil Surface movements vs Time.
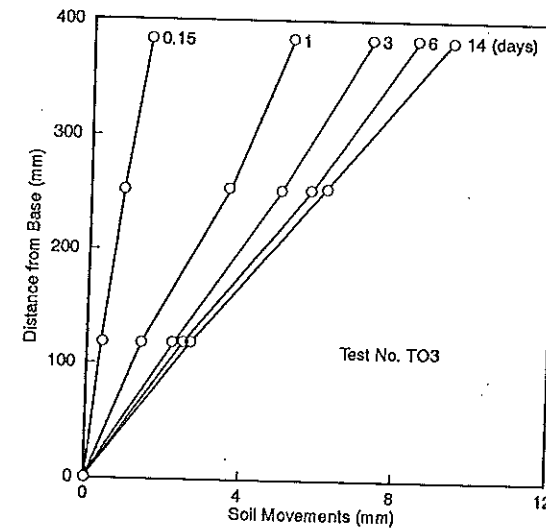
Fig. 5 Soil Movement Profile at Various Time Intervals.

Figure 5 shows for Test T03, the variations of measured soil swelling movement with depth at various times after commencement of swelling. It is interesting to note that the swelling movement decreases almost linearly with depth from the surface, even at relatively early times. Similar results were obtained for tests T01 and T02.

Pile Behaviour at Installation

During jacking of the piles, readings were taken of pile head load versus penetration and axial load along the pile. Figure 6 summarizes the total load, base load and shaft load readings at final penetration (230 mm), just at the completion of installation of the pile. In the two "clay only" tests, T04 and T05, the measured loads agreed closely. Because of the low overburden pressure used, the final penetration resistance decreased for the tests in which sand underlaid the clay, with the minimum resistance being experienced in test T07 in which a 232 mm sand layer underlay a 150 mm clay layer. In Tests T06 and T07, the peak penetration resistance occurred at a penetration of between 120 and 150 mm.

Pile Capacity After Soil Swelling

At the completion of the soil swelling period, each pile was loaded to failure and the load-settlement behaviour and axial load distribution were monitored. The measured total, base and shaft load capacities are indicated on Figure 6. As would be expected, the final load capacities were generally
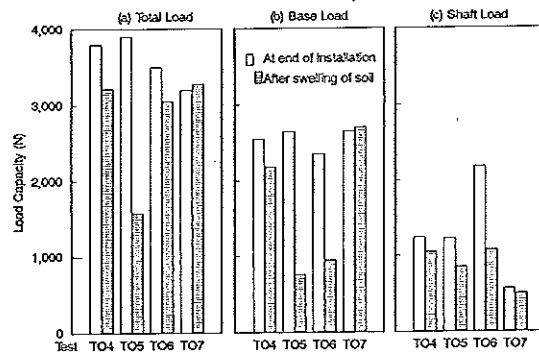
Fig. 6 Summary of Pile Load Capacities.

less than those at the time of installation because of the increase in moisture content of the soil during swelling. The load capacity reduction was more significant for the pile in clay (Tests T04 and T05) than for the piles in the clay/sand profile. In Tests T06, the measurements appeared to be anomalous as they suggested a major reduction in base load capacity but an increase in shaft load capacity during swelling.

Because it was not possible to allow sufficient time for the swelling process to be completed, there was a considerable range of moisture content at various depths within the clay. An example of the moisture content profile with depth is shown in Figure 7, and as would be expected, the moisture content was greater near the drainage boundaries than in the centre.

Pile Head Movements During Soil Swelling

The development of the soil surface movement and pile movement with time after the commencement of swelling is illustrated in Figure 8 for one of the tests (T04). When plotted in terms of log time, the movements develop relatively slowly in the early stages, as in characteristic of consolidation/swelling phenomena, and clearly the swelling process was not complete when the test was terminated. All four tests showed similar characteristics for the development of movements with time.
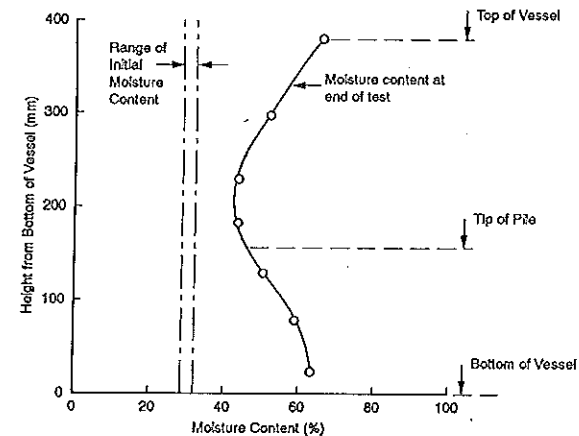
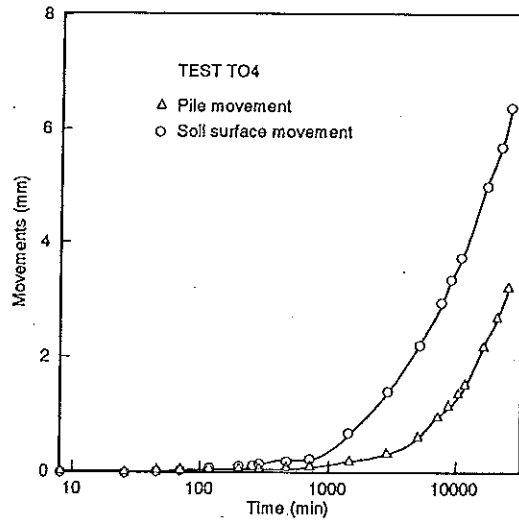**Fig. 7 Example of Variation of Moisture Content with Height (test t04).**

Fig. 8 Surface Soil and Pile Head Movement vs Log-time.

Fig. 9 Pile Head Movement vs Soil Surface Movement.

Figure 9 summarizes the relationship between pile movement and soil movement for all four tests. The relationships for Tests T04 and T05 (where the clay extended to the full depth of the vessel) agree very closely, and each show a trend of continuing pile movement with increasing soil movement. The axial load on the pile in Test T05 appeared to have had a relatively minor effect on the pile movement. In contrast, for the tests in which sand underlaid the clay, (T06 and T07), there was a tendency for the pile movement to stabilize as the soil movement increased, although the pile movement tended to develop more rapidly at early times. Both the pile and soil movements decreased as the depth of clay decreased. Figure 10 summarizes the movements at the conclusion of the tests as a function of the depth of the clay.

Figure 11 plots the ratio of pile movement to soil surface movement against time. For tests T04 and T05 with a full depth of clay, this ratio increased continuously with time. For tests T06 and T07, the ratio reached a peak value and then decreased with time, reflecting the tendency for the pile movement to reach a constant value.
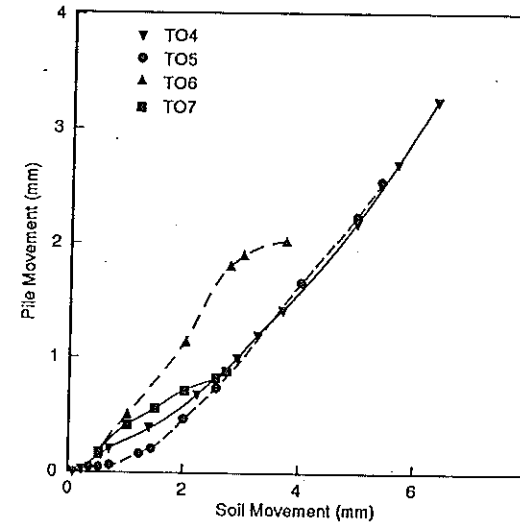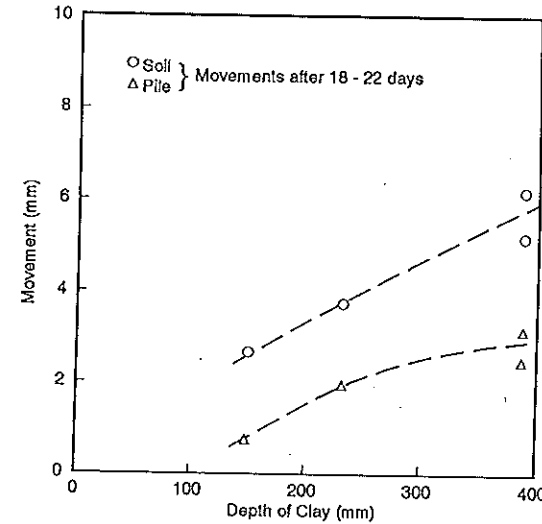

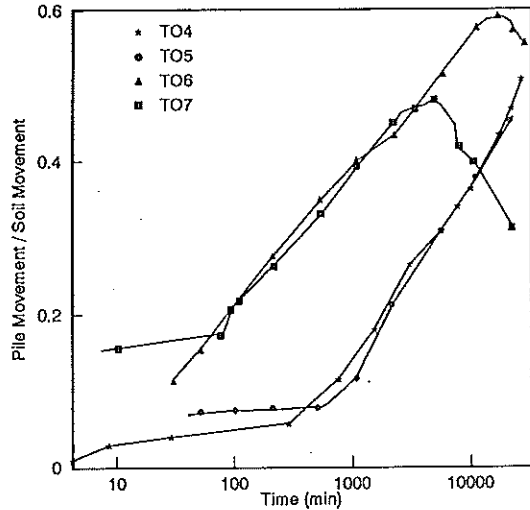Fig. 10 Soil and Pile Movement as a Function of Clay Depth.

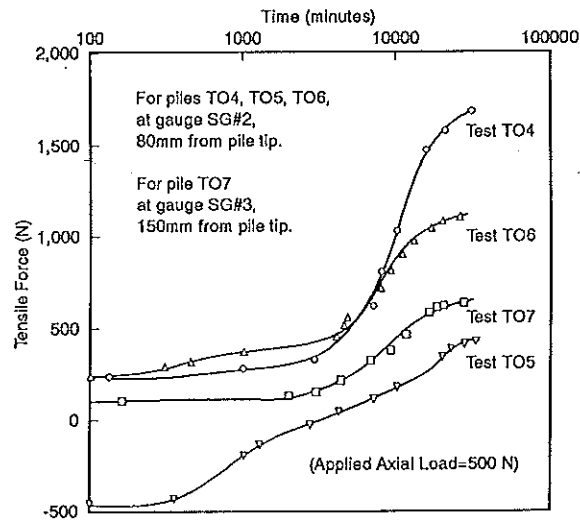Fig. 11 Ratio of Pile Head Movement to Soil Surface Movement vs Log-time.

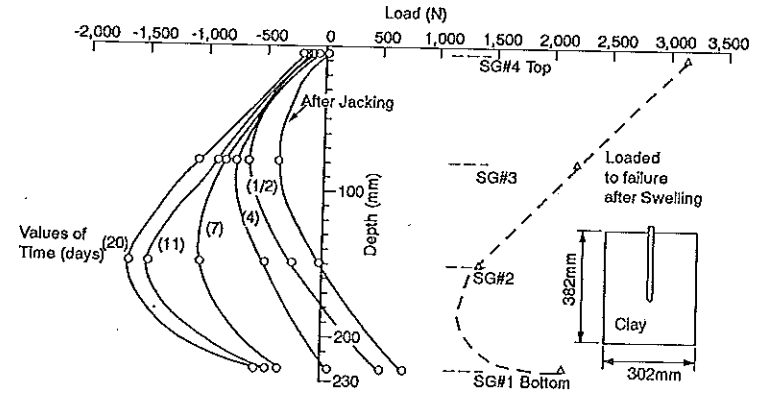Fig. 12 Development of Tensile Force in Pile During Swelling of Soil.

Fig. 13 Load Distributions During and after Swelling Stages (test t04).
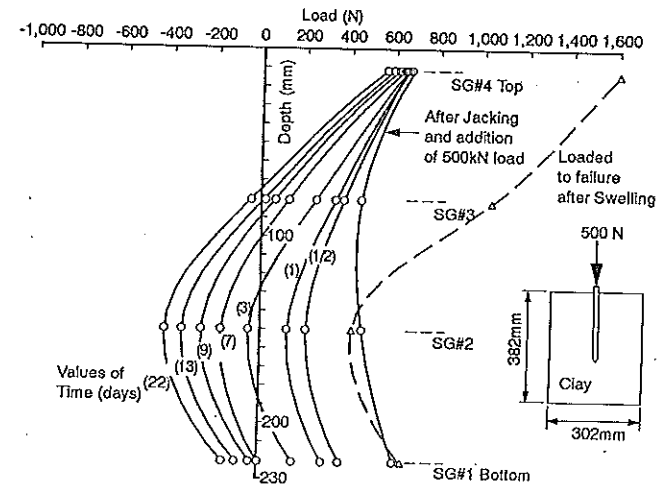

Fig. 14 Load Distributions During and After Swelling Stages (test t05).

Pile Forces Induced During Soil Swelling

The development of tensile force in the pile with time after commencement of swelling is shown in Figure 12 for the four tests. In each case, the force plotted is for the strain gauge at which the maximum final value was
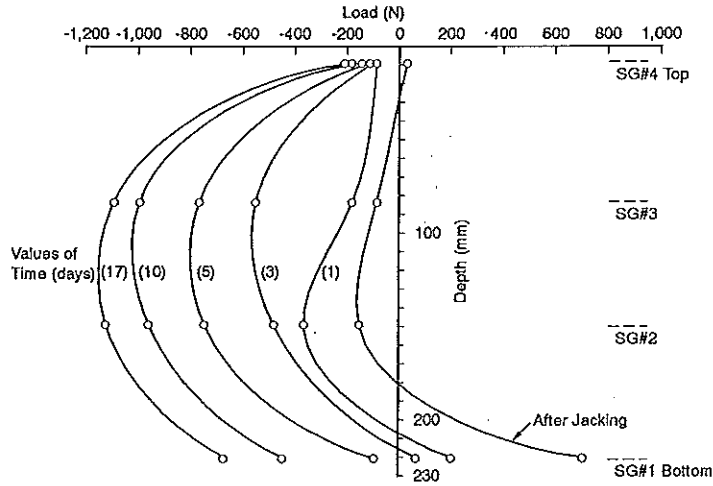
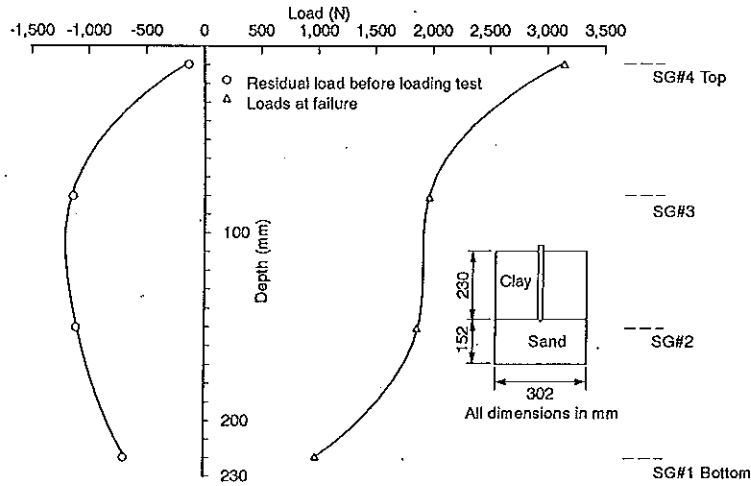**Fig. 15a Load Distributions During Swelling (test t06).**

Fig. 15b Load Distribution During Loading to Failure (test t06).

measured. As would be expected, the tensile force increased with time, and the force-time relationship had the characteristic shape of a consolidation/swelling phenomenon. In all tests, the tensile force appears to have almost reached a maximum at the time the test was terminated. The maximum tensile force developed appeared to decrease as the depth of swelling clay decreased. In Test T05, the initial compressive forces in the pile became tensile as swelling of the clay proceeded. The incremental tensile force was less than in Tests T04, which had no initial axial load, but this may have been due to the final moisture content of the soil in Test T05 being greater than in Test T04, rather than to the presence of axial load on the pile in Test T05.

Figures 13 to 16 show measured distributions of load in the pile during the period of swelling of the clay and then at the end of the subsequent loading test to failure. The following observations may be made from these Figures.

i) there are significant residual forces left in the pile after installation by jacking,

ii) the tensile forces induced in the pile by swelling of the clay increase markedly with time, especially in the lower half of the pile,
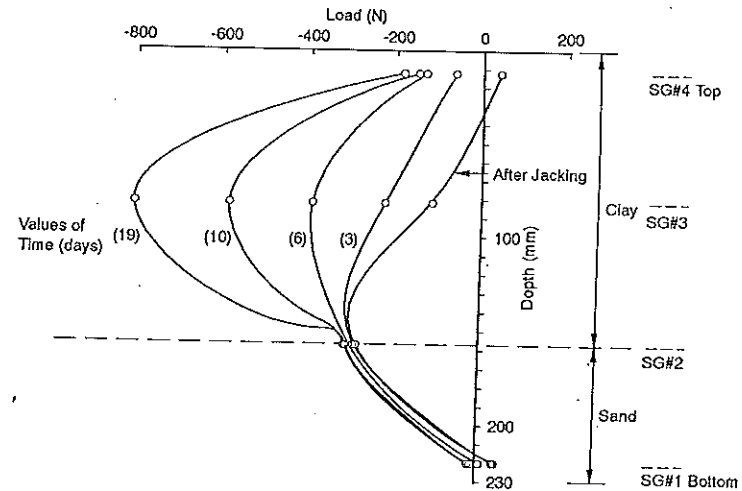

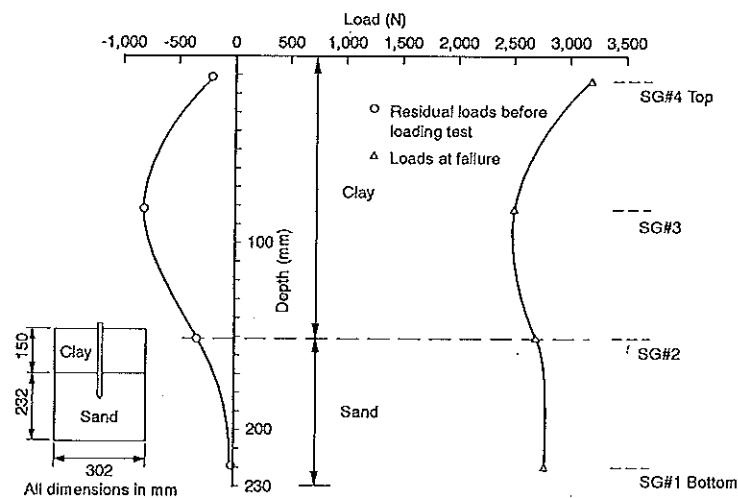**Fig. 16a Load Distributions During Swelling (test t07).**

Load (N)

Fig. 16b Load Distribution Profile after Loading to Failure (test t07).

iii) the maximum tensile force occurred at a depth of about 0.5 to 0.75 times the depth of the swelling clay layer,

iv) the load distribution at the end of the loading test suggests that the skin friction was not uniform along the pile, and that along the lower-most portion of the pile it was tensile, i.e. the load increased with depth. This does not appear to be logical and may indicate mal-functioning of strain gauge SG1.

v) in Test T07, there was almost no time-dependency of the force for that portion of the pile within the sand layer.

Comparisons Between Model Pile Test Behaviour and Theory

One of the objectives of the experimental programme was to obtain data on pile behaviour which could be compared with theoretical predictions of pile response in swelling soils. The theoretical analysis used in this study has been described by Poulos and Davis (1980), and employs a simplified boundary element approach. The analysis is implemented via a computer program PIES (Poulos, 1989) and has the ability to consider the following factors:

i) the behaviour of a single pile or a pile within a group subjected to axial

load and/or externally imposed vertical soil movements (either heave or consolidation),

ii) nonlinear response of the pile-soil interface via an elastic-plastic or hyperbolic relationship between soil stiffness and interface stress level, including the effects of pile-soil slip and end-bearing failure,

iii) variable soil stiffness and pile-soil skin friction with depth,

iv) piles of non-uniform cross-section or stiffness,

v) sequential applications of loads and/or external soil movements, with the soil stiffness and pile resistance varying at the end of each sequence.

In using PIES to analyse the model tests, the following sequence of loading was applied:

i) an initial compressive load almost equal to the ultimate capacity, to represent the final stage of penentration during installation,

ii) unloading to zero load, to represent the removal of the jacking force and to determine the residual forces in the pile. (Poulos, 1987),

iii) application of the static load (for test T05 only, where 500 kN was applied),

iv) application of the soil movements along the pile, with the soil surface being increased by 0.5 mm during each increment, up to a total soil surface movement of 8 mm. The distribution of soil movement was assumed to decrease linearly with depth from a maximum at the clay surface, to zero at the base of the clay layer; the measurements of soil movement versus depth indicated that this was an appropriate assumption.

Apart from the dimensions of the pile and the soil layer, the analysis required the following key parameters:

i) Young's modulus of the soil, E_s,

ii) the limiting skin friction f_s in tension and compression,

iii) the limiting end-bearing resistance f_b of the pile in tension and compression,

iv) the magnitude and distribution with depth of the soil movement due to swelling or shrinking,

v) Young's modulus of the pile material, E_p.

Table 3 shows the values adopted for these parameters and the method by which they were assessed. The values of E_s, used was the average of values

Table 3 Parameters Adopted for Theoretical Analyses.

Parameter	Value adopted	Method of Estimation	Remarks
Soil Young's Modulus, E_s	24 MPa	Backfigured from load-settlement curves before and after swelling for Test T04	Values before and after swelling were 30 MPa and 18 MPa respectively
Limiting skin friction, f_s	For clay, 130 kPa. For sand, 55 kPa.	Interpreted from measured residual stresses and stresses at failure. For clay, average value for the four tests adopted.	Assumed the same for tension and compression.
Limiting end bearing resistance, f_b	For clay, 900 kPa. For sand, 2.0 MPa (T06) 3.0 MPa (T07)	Interpreted from measured pile load distributions after installation, and after loading to failure.	Value in tension assumed to be negligible.
Soil Movement Profile with Depth	—	Increasing values of soil surface movement adopted, assuming a linear distribution with depth.	
Young's Modulus of Pile, E_p	70 000 MPa	Typical value for aluminium	

backfigured from pile load tests before and after the swelling of the clay. The values of f_s and f_b were assessed from the local gradients of the measured residual force distribution after installation and the load distribution at failure after swelling. For the clay, the values of f_s so obtained varied between 104 and 160 kPa, with an average value of 130 kPa being adopted. The use of constant values of E_s, f_s and f_b was a simplifying approximation as all three parameters tended to decrease as swelling proceeded and the moisture content of the clay increased; however, there was insufficient data to define adequately the variation of these parameters with moisture content.
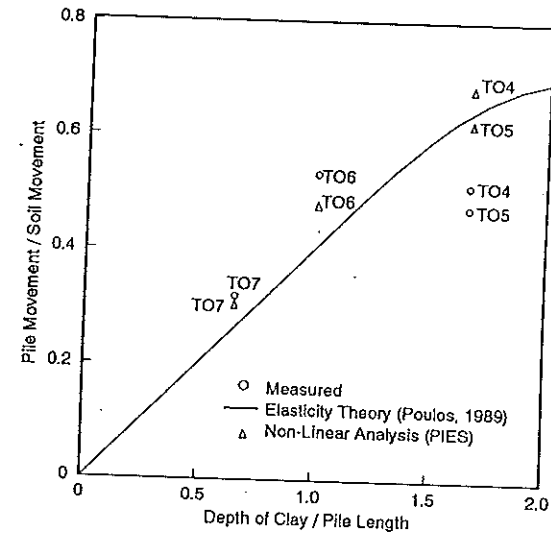

Fig. 17 Comparison Between Theory and Measurement—ratio of Pile Movement to Soil Surface Movement at the End of Each Test.

Figure 17 summarizes comparisons between the measured and theoretical ratios of pile to soil surface movement, S_p/S_o, at the end of each test. Theoretical solutions have been obtained both from a non-linear analysis using program PIES, and from parametric solutions from elasticity theory presented by Poulos (1989). Both theoretical solutions agree reasonably closely and are also in fair agreement with the measured values. The theory predicts quite well the decrease in relative pile movement with decreasing thickness of soil layer relative to pile length.
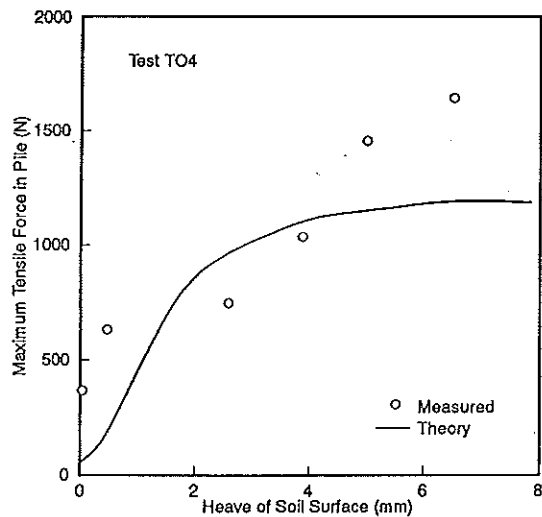
Fig. 18 Development of Maximum Pile Tension with Soil Surface Heave (test T04).
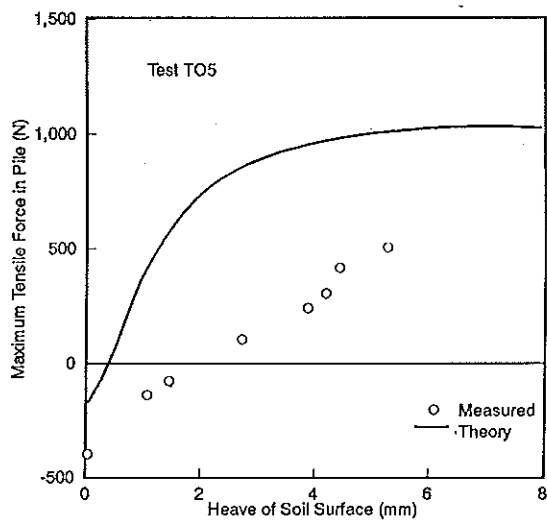

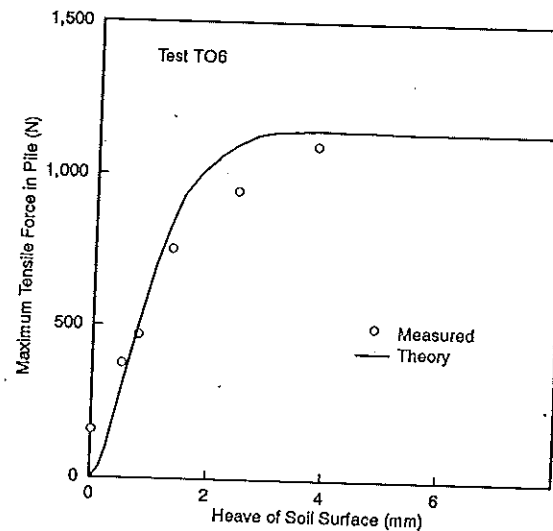Fig. 19 Development of Maximum Pile Tension with Soil Surface Heave (test T05).

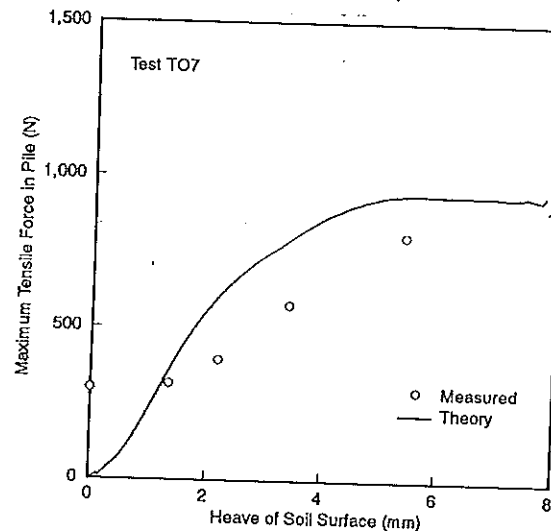Fig. 20 Development of Maximum Pile Tension with Soil Surface Heave (test T06).

Fig. 21 Development of Maximum Pile Tension with Soil Surface Heave (test T07).
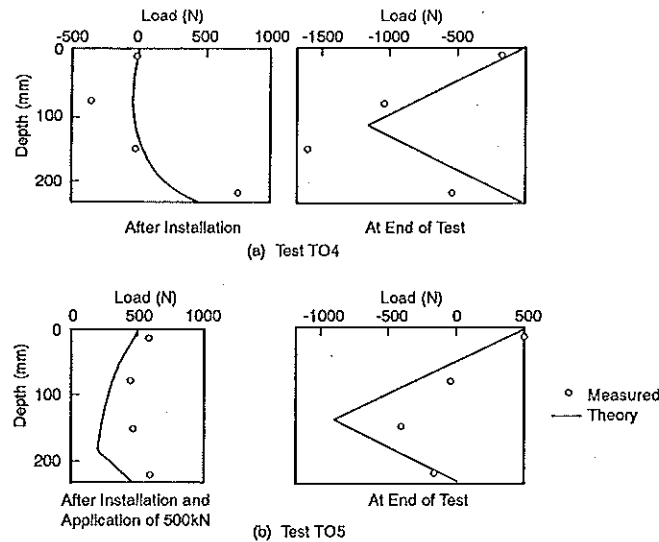
Fig. 22 Measured and Theoretical Load Distributions. Tests T04 and T05.

Figures 18 to 21 compare predicted and measured maximum tensile forces in the pile as a function of soil surface heave. The theory predicts a generally similar magnitude and rate of increase of tensile force with soil surface heave, although the agreement is not close in Tests T04 and T05. Using the parameters in Table 3, the theory overpredicts the maximum tensile force for Test T05, and underpredicts it for Test T04, and clearly, better agreement could have been obtained by choosing different values of f_s and f_b for each of these tests. Nevertheless, the agreement overall is fair, especially for Tests T06 and T07, despite the simplying assumptions made in the theoretical analysis.

Figures 22 and 23 compare measured and theoretical distributions of load with depth for two stages of each test:

a) at the end of installation (prior to swelling of the clay), and
b) at the end of the test.

The predictions of residual forces in the pile after installation are generally not satisfactory, especially in the case of Test T07, where the shaps of the
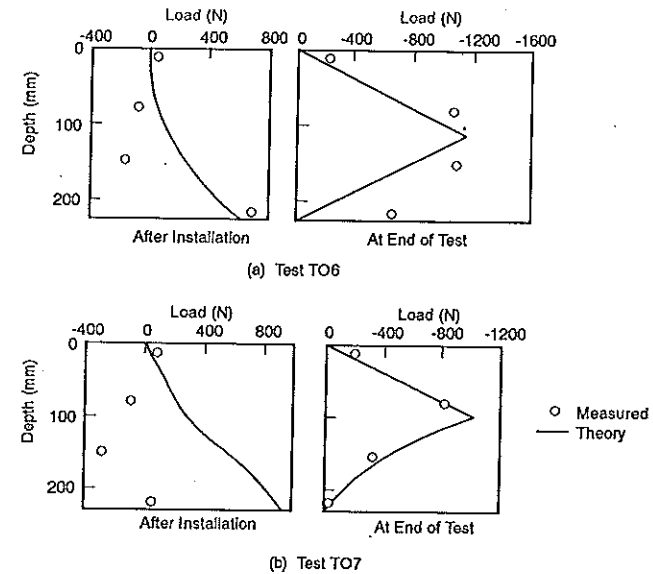
Fig. 23 Measured and Theoretical Load Distributions. Tests t06 and t07.

theoretical and measured distributions differ considerably. The assumption of the same value of Young's modulus for both the clay and sand may contribute to this difference, and the residual load distribution depends critically on the appropriate choice of all three parameters, f_s f_b and E_s. It is also influenced significantly by the distribution of E_s along the pile. At the end of the test, the load distribution is depended primarily on the values of f_s and f_b, and is much less influenced by the choice of Young's Modulus. Here, the theory shows better agreement with the measurements, and in all cases, the general characteristics of behaviour are well-reproduced by the theory.

The comparisons shown in Figures 17 to 23 suggest that the theory can give realistic predictions of relative pile movement and also of the development of maximum pile tension during swelling of the soil, provided that appropriate values of limiting shaft friction f_s and end bearing resistance f_b are chosen. However, detailed distributions of load along the pile at early and intermediate stages of swelling may be inaccurate unless reasonable predictions of the residual loads after installation can be made.

CONCLUSIONS

Test have been carried out on model piles in an expansive clay to measure the movement and tensile force developed in the pile by the swelling of the clay. Measurements have also been made of the distribution with depth of swelling movement in the clay at various times after commencement of swelling.

The main conclusions from the model tests may be summarized as follows:

1) from relatively early times, the soil swelling profile appears to vary almost linearly with depth from a maximum at the surface to zero at the base of the clay.

2) the pile head movement increases as the soil heave increases, but tends to approach a constant value if the pile is founded in a stable sand layer below the swelling clay.

3) swelling of the soil may induce substantial tensile forces in the pile. The maximum tensile force tends to increase with increasing depth of the swelling clay relative to the pile length, with increasing amount of soil movement, and with increasing limiting shaft friction and end bearing capacities of the pile.

Comparisons have been made between the measured behaviour and that predicted from a theoretical approach based on a boundary element analysis implemented via the program PIES. It has been found that the theory is capable of giving realistic estimates of relative pile movement and the distribution and magnitude of tension in the pile during the later stages of soil swelling, provided that appropriate values of limiting shaft friction and end bearing resistance are chosen. However, it has been found to be difficult to accurately predict the distribution of tensile force within the pile during the early stages of soil swelling because of the difficulty of predicting the residual forces remaining in the pile after installation.

Considerable scope exists for modifications and improvements to the theoretical analysis to model better the residual forces in the pile after installation and to incorporate such aspects as the change in soil properties within the clay during the swelling process.

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KEY WORDS

expansive soils; foundations; model testing; piers; piles; soil mechanics.