Behaviour of displacement piles in a heavily overconsolidated clay
Le comportement des pieux à déplacement dans une argile fortement surconsolidée

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SYNOPSIS: The paper describes field experiments with heavily instrumented piles jacked into London clay. Measurements of the total and effective stresses developed over the pile shaft are used to analyse the effects of installation, the period of subsequent ground equalisation and the shaft capacities developed during load tests. The pile behaviour is shown to differ in many ways to the predictions made by current theories and, in particular, the rate of jacking during installation is shown to have an unexpectedly strong influence on the shaft friction characteristics.

INTRODUCTION

The development of a satisfactory effective stress theory of displacement pile behaviour has proved to be particularly difficult. The potential advantages of such a design tool have long been recognised (see, for example, Chandler (1968), Burland (1973) or Meyerhof (1976)), but the severe kinematic constraints imposed by displacement piles has made analysis of the stress conditions in the surrounding soil both complex and unreliable. As shown by Kavvadas and Baligh (1982), predictions for the effective stresses are critically dependent on the assumptions made concerning constitutive laws, OCR and external boundary conditions. Experiments, therefore, have a key role in identifying the principal patterns of ground behaviour and in testing theoretical predictions.

CANONS PARK RESEARCH PROJECT

A programme of field research has been undertaken by Imperial College into the behaviour of displacement piles in a mechanically overconsolidated clay. The work has been sponsored jointly by Industry and the UK Government and has involved installing a series of eight 100mm diameter, 5 to 7m long, steel piles into London clay at Canons Park, north London.
This paper presents data from the first three instrumented pile tests in this series and concentrates on describing the processes of installation and subsequent ground equalisation. Reference will also be made to earlier work at Canons Park by the Building Research Establishment (Price and Wardle (1982)), and by Kitching (1983). The results obtained in the later stages of the Canons Park programme, including a detailed description of the pile loading process, will be reported subsequently.

GROUND CONDITIONS

A summary of the ground conditions at Canons Park is given in Figure 1. Beneath a variable layer of gravel exists a typical weathered London clay profile, with Head deposits overlying disturbed London clay and below that intact material. The preconsolidation pressures are thought to equal at least 1MPa, giving minimum OCRs of ≐30 at 2m depth and ≐14 at 6m. The site is gently underdrained with the near surface water table approximately 1m below ground level. Further geotechnical details are given by Jardine (1985) and Bond (1989).

PILE INSTRUMENTATION AND TEST PROCEDURE

Instrumented pile tests in stiff, high OCR soils present some special technical problems. Firstly, as the soil stiffnesses are high, the pore pressure and surface stress transducers must have very low compliances if accurate measurements are to be made. Secondly, the sensors must be exceptionally fast-acting if the installation process is to be followed in detail. Thirdly, the piezometric system may have to cope with negative pore water pressures due to dilatancy during shearing. And, finally, the entire system has to be completely waterproof if reliable long term data are to be obtained. Different trial instrument designs were used in the first two tests (i.e. the 1984 Pilot test and CP1 in 1986) but neither system was satisfactory in the long term. The sensors were completely redesigned for the third test, CP2; the new gauges performed very well and were used successfully in all the following instrumented tests. Figure 2 shows the configuration of transducers used for CP2; more detailed descriptions of the instrumentation are given by Bond (1989) and Bond and Jardine (1989).

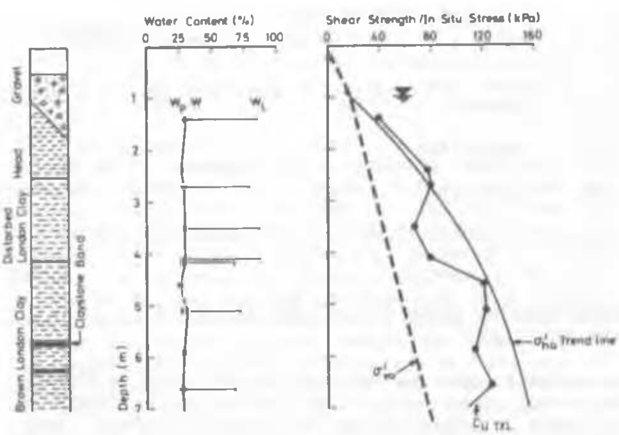

Figure 1 - Ground conditions at Canons Park

A summary of the three tests reported in this paper is given in Table 1. In each case, the instrumented piles were jacked hydraulically through cased boreholes to their final penetrations, using a large loading frame and kentledge. Installation proceeded with a series of 225mm pushes separated by pause periods (≙ 2 mins) during which the jack and its cross-head beam were reset. This arrangement allowed the rate of penetration to be controlled, up to a maximum velocity of 600mm/minute. (In each installation the initial texture of the pile's surface was also controlled by grit blasting to give a CLA roughness of approximately 10μm.)

The pile sensors were logged rapidly during installation with data collection continuing at a decreasing rate until first loading. The tests involved applying load increments at a fixed rate until a creep-yield point was reached, after which the experiments continued using an approximately constant rate of displacement until the required total moment had been achieved.

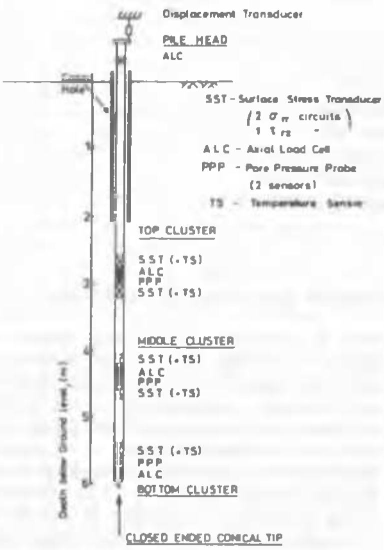

Figure 2 - Configuration of Instrumented Pile

Table 1 : Summary of Tests

Test	Loading Type	Jacking Rate on Installation (mm/minute)	Final depth to toe (metres)	Pause before 1st load (days)
Pilot	Tension	500	5.20	79
CP1	Compression	95	5.28	79
CP2	Compression	425	5.95	63

OBSERVATIONS MADE DURING INSTALLATION FOR TEST CP2

As mentioned earlier, the data from Test CP2 should be regarded as the most reliable and it is this Test that will be reported in most detail. Dealing first with the piezometric response, Figure 3 shows the traces produced by the bottom cluster of instruments during penetration. These two independent sensors show close agreement and confirm the trends indicated by the earlier tests. Although positive pressures (up to 350 kPa) were generated near the tip in the Head layers, considerable suctions (up to 0.5 atm) were noted at most levels as the pile penetrated through the London clay. The traces indicated a marked change in the soil's response below 5.3 metres, where the pore water pressures

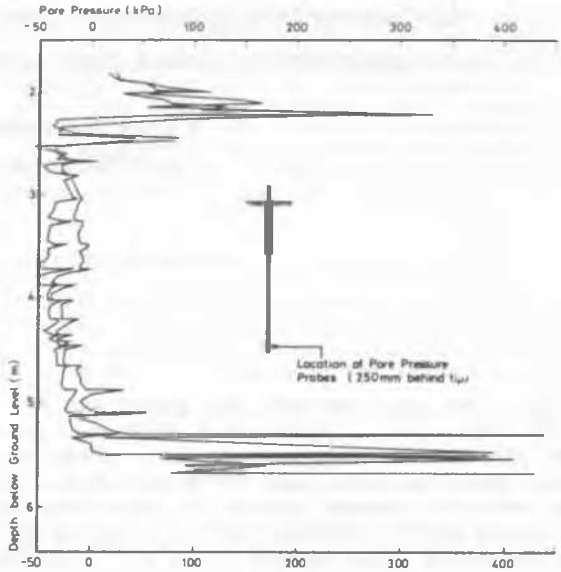

Figure 3 - Pore pressure traces for CP2 Installation

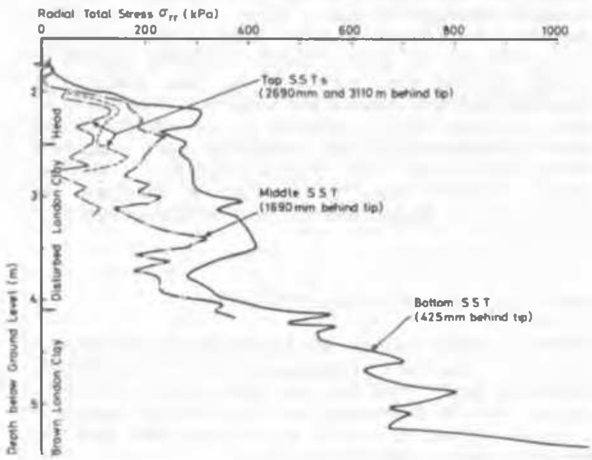

Figure 4 - σ_{rr} measurements; CP2 Installation

increased again to give maxima of up to 450 kPa. It was found that the pressures also varied with time: the lowest values were developed during steady penetration and the highest were recorded at the end of the 2 minute pause periods. (These time dependent changes appear as sharp spikes on Figure 3).

The pore-pressure profile recorded immediately after installation is indicated in Figure 5. Although the pore pressures were changing rapidly near the pile tip, small or negative pore water pressures were acting over most of the pile shaft at this stage - in contrast to the large positive values predicted by most theories. For comparison, the cavity expansion analysis described by Wroth et al. (1979) predicted a pore pressure increase of ≙3.5 Cu (or 250-450 kPa) for piles installed in London clay.

The radial total stresses measured during jacking are shown on Figure 4. The values of σ_{rr} recorded at fixed soil horizons fell sharply as the pile advanced and σ_{rr} continued to fall when the tip had reached a level 20 or more diameters below the given horizon.

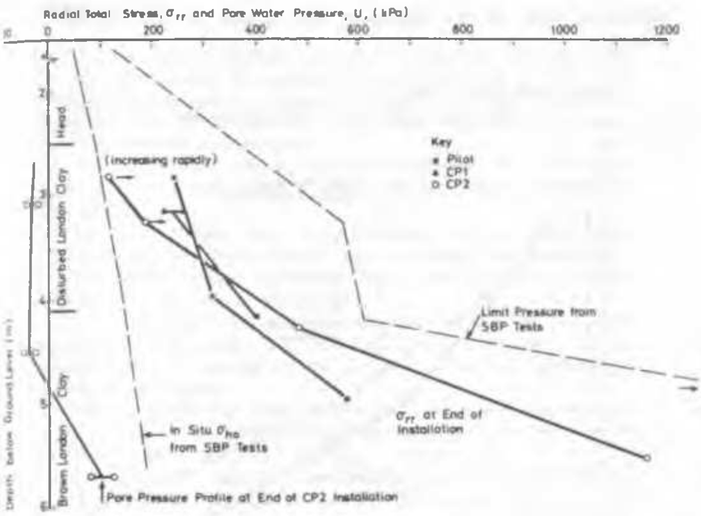

Figure 5 – Profiles of σ_{rr} and pwp at end of Installation

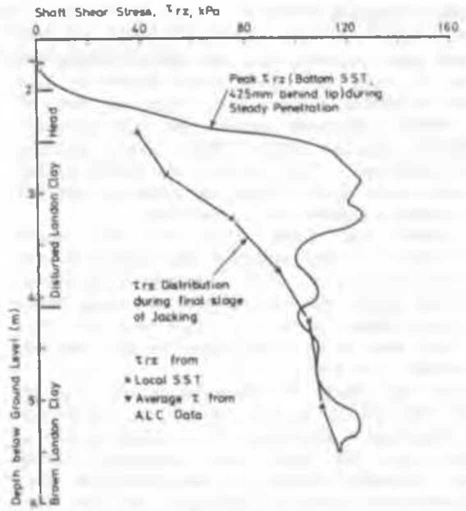

Figure 6 – Shear stress measurements; CP2 Installation

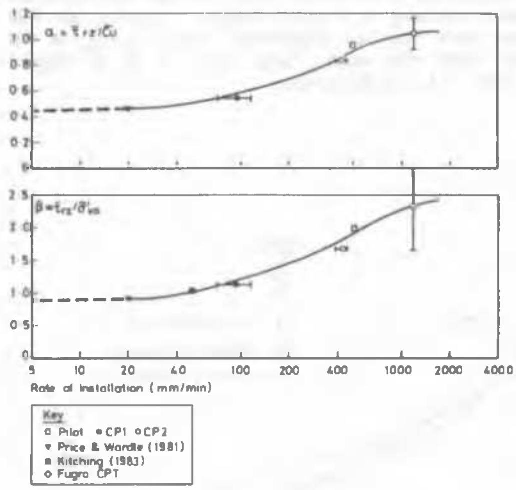

Figure 7 – Rate dependency of Installation shaft resistance

A summary profile for σ_{rr} at the end of installation is given on Figure 5, which also shows the data from the two earlier experiments. Despite the differences in instrumentation and jacking rates, the agreement between the three tests is good and in each case σ_{rr} was found to vary rapidly with depth. Also shown are the limit pressures interpreted from Self Boring Pressuremeter (SBP) tests performed a few metres from the pile; the pile installation process evidently develops smaller radial total stresses than the monotonic expansion of a cylindrical cavity.
Figure 6 illustrates the shear stresses developed during installation using two different plots. The 'peak curve' is the envelope of the τ_{rz} values recorded during penetration at the lowest instrument cluster. As with the σ_{rr} measurements, the shear stresses fell with increasing pile penetration and this may be seen from the 'final' distribution of the maximum τ_{rz} values recorded during the last stage of jacking. This 'final' plot combines the local shear stress data, recorded at four shaft levels, with the mid-section average shear stresses calculated from the gradients in total axial load; the plot also shows the consistency obtained from the two independent methods of shear stress measurement.

EFFECT OF RATE ON FRICTIONAL RESISTANCE

Pile CP1 was installed with a far slower rate of jacking than the other two experiments (see Table 1). This change in rate appears to have had no strong influence on the observed pore water pressures or radial total stresses. However, the effect on the frictional resistances was striking: the average value of τ_{rz}, towards the end of jacking, was approximately 40% less than that seen in the faster tests. Further information has been interpreted from the more slowly jacked pile tests reported by Price and Wardle (1982) and Kitching (1983) and the combined data is shown on Figure 7 as plots of α and β (the skin friction coefficients) against the rate of jacking. Also shown is the range of sleeve friction results noted in Fugro cone soundings performed at Canons Park. The test data show a slight rate dependency ($\triangleq$5% / log cycle) for velocities below 40mm/minute but at faster rates the interpreted curve climbs steeply, giving a maximum gradient of $\triangleq$100% per log cycle.
This remarkable rate effect can be more easily understood by comparing the interface friction angles, $\delta' = \tan^{-1}(\tau_{rz}/\sigma'_{rr})$ with those seen in comparable laboratory shear tests. In the pile tests, the values of δ' varied with depth and stress level, but the CP1 measurements from the middle cluster indicate the range $14^\circ > \delta' > 9^\circ$ (for slow jacking), which conforms with the results obtained from slow interface ring shear tests on London clay by Lemos (1986). Similarly, the corresponding range from CP2 during fast jacking ($17^\circ > \delta' > 14^\circ$) compares well with the trends of Lemos' more rapid tests. The ring shear test characteristics were interpreted by Lemos as showing evidence of a change, with rate, in the fundamental mechanism of residual friction. It was thought that sliding predominates at slow rates but that at very fast rates the clay experiences more turbulent shear; between those extremes lies a region of transitional behaviour, as described by Lupini, Skinner and Vaughan (1981). It is suggested that the same conclusions may be drawn from the Canons Park tests, i.e. that a transition band of penetration rates exists (40 – 1000 mm/minute) below which δ' approaches a low residual value and above which δ' tends to a maximum that may approach φ' critical state (23°). The rate of penetration is therefore seen as controlling the soil fabric close to the shaft and, if this is correct, the effects of jacking rate are likely to persist and to have a strong influence on the peak skin frictions developed in subsequent monotonic load tests. However, because load tests are carried out at relatively slow rates of penetration (typically <0.1mm/minute at peak) viscous effects should lead to lower δ' values than those noted during installation.

THE EQUALISATION PERIOD

The changes in pore water pressures after the end of jacking are illustrated in Figure 8, using the typical traces shown by Test CP2. The most important changes occur over the first 24 hours. At the bottom instrument position, the pore pressures climb rapidly, reaching maxima within ≈300s, before decaying towards hydrostatic conditions. The traces for the middle probes climbed to their peaks more slowly, whilst the pressures recorded at the top probes remained negative for at least 24h.
The change in radial total stress with time also varied systematically with depth; a rapid reduction was noted over the first 24 hours at the deepest level, whilst a marked increase was found at the shallowest depth, with smaller gains and losses being recorded at the intermediate levels. Slight (i.e. < 5%) reductions in σ_{rr} were seen at all levels when the pile was left undisturbed for a further 2 months.
Figure 9 summarizes the effects of these changes on σ'_{rr} by plotting profiles for the end of jacking and for the equilibrium condition that had developed 3 days later; it is evident that σ'_{rr} reduced with time from the high values developed during installation. The equalisation process, in this high OCR clay, therefore involves important features of swelling at the pile face, and differs considerably from the simpler process of monotonic consolidation predicted by most theories. Figure 9 also shows the final valves of σ'_{rr} expected from the cavity expansion analysis of Wroth et al. (1979); the field measurements tend towards the prediction at the lowest levels, but fall well below the expected values for most of the pile's length. Equally, the field measurements show that the equilibrium ratio $\sigma'_{rr}/\sigma'_{ho}$ is not constant but varies with depth, rising from 1.5 at the highest transducer level to 5.2 at the lowest.

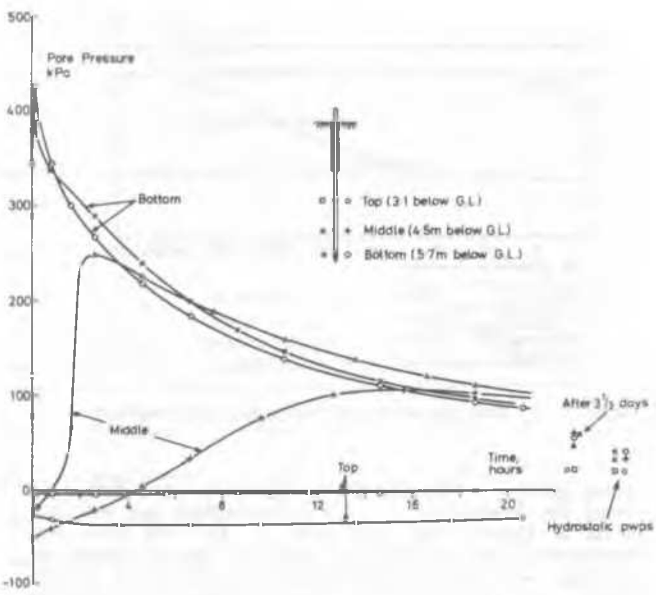

Figure 8 – Equalisation of pwp's after CP2 Installation

PILE LOADING

Although the detailed measurements made during pile loading will be reported in a later paper, it is appropriate to consider here some general aspects of the measured shaft capacities. The three instrumented tests, and the earlier experiments by Price and Wardle (1982) and Kitching (1983), covered both tension and compression loadings for similar piles, jacked in at a variety of rates and tested at various dates after installation. The effect of these variables on shaft resistance is most easily discussed by comparing the overall values of α and β for each test, as differences exist in the diameters and lengths of the various piles.

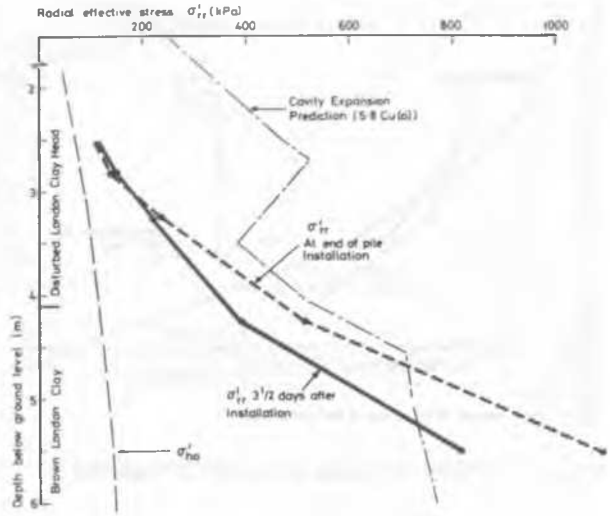

Figure 9 – Radial effective stress profiles; CP2

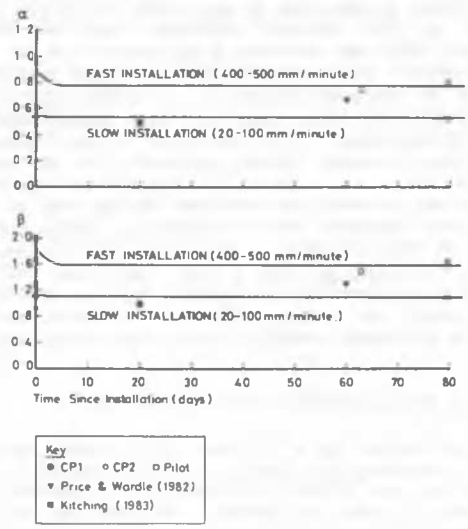

Figure 10 – Variations of shaft capacity with time.

It is interesting that in most cases the long term shaft capacity fell below the maximum developed by the same pile during installation; this tendency for negative set-up is demonstrated by the scatter diagrams and interpreted trend lines given in Figure 10. The plot also shows that jacking rate is the most important single parameter in controlling long term capacity. A pile installed at 500mm/minute appears to have 60% more shaft capacity than one jacked in at 20mm/minute; this lends further support to the earlier conclusion that the interface friction angle is critically dependent on the pile velocity during jacking.

CONCLUSIONS

The Canons Park programme has shown that the effective stress processes that govern displacement pile behaviour (in a high OCR medium plasticity clay) are quite different to those anticipated theoretically or those expected from tests on other soil types. The most notable findings are:

1. The soil dilates strongly during installation, with substantial increases in σ'_{rr} and negative pore pressures being recorded at most depths;
2. The total stresses (σ_{rr}, τ_{rz}) developed at any given depth below ground level reduce sharply with increasing pile penetration;
3. The shaft friction coefficients (α, β and δ') are critically dependent on the rate of penetration;
4. A clear range of jacking velocities exists (40 to 1000 mm/minute) over which the rate effects are steepest; it is probable that this corresponds to a transition in the fundamental mechanism of friction;
5. Pore pressures may show temporary maxima after installation;
6. σ'_{rr} falls as the pore pressures equalise, but the final stresses remain far higher than the initial σ'_{h0} values;
7. The long term capacities are slightly lower than those at the end of jacking (i.e. a negative set-up effect is observed), and
8. The most important factor in determining ultimate pile capacity is the rate of penetration during installation.

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