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SYNOPSIS

An instrumented model pile was used to investigate the fundamental behaviour of driven cylindrical steel piles, in clay soils. Data are described from two test-bed sites, one with heavily overconsolidated clay, and one with normally consolidated. These confirm that a residual shear surface is formed adjacent to the pile shaft during installation. Comparisons with other site investigation data and cavity expansion theoretical predictions suggest that stress relief immediately behind the pile tip during installation gives rise to total radial stress and pore pressure measurements on the shaft which are lower than this simple model predicts. However, the data did indicate that the radial effective stress might be successfully predicted.

During reconsolidation, the radial effective stress drops initially, followed by a slow recovery, which was insufficient in the two clays investigated for the final value to reach that during installation. Upon undrained loading, the clay adjacent to the pile did not reach a critical state as many current theories assume, since failure occurred on the residual surface created during installation. The generation of negative excess pore pressures on the shear surface during undrained loading caused an increase in the radial effective stress. In the normally consolidated clay this was solely responsible for the large set-up of pile shaft capacity by comparison with that during installation. The same capacity increase may not be seen in these clays if loading were drained.

INTRODUCTION

The instrumented model pile (IMP) was developed at Oxford University with the intention of investigating the fundamental behaviour of piles in clay. The three phases of the performance of a pile; (1) installation, (2) reconsolidation and (3) loading, were each studied
separately. Both theoretical work and field trials were undertaken at four test bed sites. Data from two of these are described in this paper, one with stiff overconsolidated clay and the other with normally consolidated estuarine clay.

The theoretical work has largely concentrated on the first phase, and for simplicity has considered the behaviour of solid cylindrical driven piles. Carter, Randolph & Wroth (Refs. 1, 2, 3 and 4) idealised the soil movements during the installation of such a pile as purely radial straining. Their series of papers used numerical and analytical methods to investigate predicted installation stresses from undrained cylindrical cavity expansion.

The cavity expansion radial effective stress prediction arises simply from the assumptions that the soil adjacent to the pile shaft is at a critical state under plane strain conditions with a radial major principal stress, and also from the adoption of a Mohr-Coulomb failure criterion:

\[
\sigma'_{r} = [1 + \sqrt{3/M}]s_{uo}
\]  

(1)

where \(\sigma'_{r}\) is the installation radial effective stress  
\(M\) is the critical state line gradient  
\(s_{uo}\) is the initial undrained shear strength of soil

The following modified form of Gibson and Anderson's (Ref. 5) cavity expansion equation was used to give similar pore pressures to those predicted numerically:

\[
u_{s} = u_{o} + (p_{o}' - p_{f}') + s_{uo}\ln(G/s_{uo})
\]  

(2)

where \(u_{s}\) is the pore pressure on shaft during installation  
\(u_{o}\) is the initial insitu pore pressure  
\(p_{o}'\) is the initial mean normal effective stress  
\(p_{f}'\) is the mean normal effective stress at failure  
\(G\) is the shear modulus of soil

The second term represents the change in the mean normal effective stress as the soil is sheared to failure and the third is the increase in
the mean normal total stress due to cavity expansion.

More recent work by Levadoux and Baligh (Ref. 6) used the numerical "strain path" method to investigate the stresses around cone penetrometers. This represents a more realistic modelling of the installation process, but does not allow the simple calculation of the stresses acting on the shaft that is available from cavity expansion theory. They predicted a bulb of higher total and effective radial stresses around the tip. Behind the cone shoulder, along the shaft the radial effective stress was predicted to remain constant whilst the total stresses decreased, as shown in Fig. 1.

Following installation, the outward radial dissipation of installation excess pore pressures during the reconsolidation phase has been modelled both numerically and analytically by Wroth et al. (Refs. 1, 2, 3 and 4). As pore pressures decrease the radial effective stress is predicted to increase monotonically and the radial total stress to decrease.

Most theoretical work on the loading phase has studied undrained conditions since these have been considered to give the lower capacity. Many authors, such as Randolph & Wroth (Ref. 7) have assumed failure of the soil adjacent to the shaft when it is sheared to a critical state. Since in many instances the soil at the end of reconsolidation is predicted to be normally consolidated with the radial effective stress as the major principal stress, loading would result in the generation of positive pore pressures adjacent to the shaft.

In contrast, from the microscopic investigation of clay around model piles (Martins, Ref. 8) and limited field work (Jardine, Ref. 9) researchers at Imperial College have concluded that failure occurs on a residual failure plane created during installation.

There is an abundance of full scale and model pile test data, much of which is of poor quality. Contradictions in existing data are marked and cannot be used to prove conclusively any particular theory of pile behaviour.
THE INSITU MODEL PILE

The Insitu Model Pile (IMP) was originally designed and constructed by Mr. T.J. Freeman at Oxford University. It was subsequently re-instrumented and substantially modified by Coop (Ref. 10). Details of the design may be found in this reference.

The IMP is 80mm in diameter and 1135mm long. It comprises two concentric cylinders attached to a common pile head. The inner is rigidly connected to the tip assembly and so end bearing forces are conducted directly to the IMP head and are not measured. This allows more sensitive measurement of shaft friction in the outer brass cylinder which is made up of interchangeable segments housing the instrumentation. Figure 2 shows a typical IMP arrangement. The actual position of the transducers did not vary, but the specific transducer used at a particular location was changed to verify that the readings obtained were not dependent on the transducer used.

The IMP has two types of pore pressure transducer. Two Druck PDCR-81 transducers were mounted in the shaft, one of which was in the tip assembly. These were supplemented by three strain-gauged diaphragm transducers manufactured at Oxford. Several filter types were used with no apparent influence on readings.

Strain-gauged total radial stress transducers were also manufactured at Oxford, and four were mounted in the IMP shaft. Pairs of either total radial stress or pore pressure transducers were positioned opposite each other to check for variation of stresses around the IMP. No major consistent differences were found, although for the earliest boreholes the data did suggest poor axiality of jacking, which was then rectified.

All of the pressure transducers were of a differential type, and a nitrogen back pressure was applied to the inside of the IMP. This was regulated at the surface and supplied through two gas hoses which also housed the instrumentation cables connecting the IMP with its signal conditioning and data logging systems, also at the surface. The back pressure was used to improve the radial stiffness of the total radial stress transducers. During IMP installation, total radial stress...
transducer compliance is not a problem since the transducer is continually penetrating into fresh soil. Whilst fluctuations in the reading may be subject to a small error, the mean reading will be unaffected. However, during reconsolidation, the IMP is stationary in the ground, and this error may become significant, as will be shown later. To overcome this, the back pressure was made to follow external total radial stress changes, keeping the transducer face effectively stationary.

Three load cells were used to measure axial loads in the outer IMP cylinder, the difference between readings for adjacent cells being used to calculate the unit shaft friction for the two friction sleeves indicated on Fig. 2.

All IMP transducers were calibrated before and after each site visit. Transducer cress-sensitivities were also regularly checked. The pore pressure instrumentation was generally de-aired in the lab before a site visit, and the IMP transported to site in a water filled protective cylinder. It would then be dropped out of this cylinder directly into a water filled augered hole. Transducer zero readings were taken in this starter hole before and after each sounding. The surface of the IMP is principally of smooth machined brass, prepared for each site visit by rubbing down with a fine grade emery cloth.

The IMP monitoring and jacking systems were designed for portability and self-containment, to allow IMP site investigations to be undertaken at any location and by only two people. The system was transported and housed in a Land-Rover and trailer. All of the signal conditioning and data logging equipment was battery powered, and the hydraulic jacking system was run off a petrol motor. Reaction for the jacks was provided by screw pickets. The required loads were minimised by using NVR drill rods to push the IMP into the ground, since these have a smaller diameter than it. A displacement transducer measured the jack stroke to allow calculation of the penetration.

The IMP boreholes were semi-continuous soundings, with breaks between strokes or when an extra drill rod was added. A nominal penetration rate of about 200mm/min was used with a set of transducer readings logged every 0.6 seconds, or at about 2mm intervals.
The IMP is hollow, and can penetrate for a limited distance open-ended. The sample chamber has a plunger which was restrained for closed-ended penetration, and released at a depth where open-ended penetration was required. The ratio of the wall to gross cross-sectional areas is 49%, but an inward tapering cutting shoe and a step to relieve internal friction allow lower apparent area ratios to be achieved. Measurement, of the plunger movement at the surface gave an indication of the quantity of soil entering the sample chamber. However, unless otherwise stated, all of the IMP data presented in this paper are for closed-ended operation.

In several of the boreholes, IMP penetration was halted to allow a reconsolidation phase to be monitored, during which time the drill string would be firmly held at the surface. Following pore pressure dissipation, undrained load tests were performed by means of controlling the hydraulic oil pressure in the jacking system.

FIELD WORK AT MADINGLEY

The four sites used for the IMP field work were carefully chosen. Each had a variety of existing soils investigation information. In several cases the sites had been used as test-beds for experimental work on site investigation instruments or for model pile tests. Two of the sites have normally consolidated estuarine clays and two heavily overconsolidated clays. IMP data from one of the sites with normally consolidated clay and one with overconsolidated clay are described in this paper.

The Madingley site, on the outskirts of Cambridge has been frequently used as a test-bed, and extensive site-investigation data are available (Ref. 11). Six successful IMP soundings were made at this site.

Figure 3 shows an undrained shear strength profile for the stiff, heavily overconsolidated Gault clay which outcrops at Madingley. The values of $s_{uo}$ chosen for analysis are based on the triaxial and pressuremeter data (Refs. 11 and 12) which are believed to be the most
reliable. Insitu stresses are indicated on Fig. 4. The vertical effective stresses have been estimated from unit weights quoted in Reference 11. The horizontal effective stresses have been calculated using the measured total horizontal stresses from self-boring pressuremeter tests conducted by PM Insitu (Ref. 12), deducting ambient pore pressure readings taken from piezometers by Clegg (Ref. 15) at the time of the pressuremeter investigation. Pore pressures were taken from these standpipes on each IMP site visit, and a typical ambient pore pressure distribution is shown on Fig. 4.

Phase 1: Installation

Figure 5 shows summary plots of all the pore pressure and total radial stress measurements made by each of the IMP transducers during continuous penetration in five of the soundings. The boreholes start at different penetrations, and data from about the top 0.2m of each are omitted since readings were locally affected by stress relief around the base of the pre-augered hole. Data are also omitted from sections of the soundings where penetration has been either non-continuous or open-ended.

Total radial stresses measured by the lower transducers were similar for all of the boreholes, and always exceeded those measured by the upper. For the first borehole, the difference was about 200kPa, reducing to under 50 kPa for the later ones. Pore pressures generally showed similar trends and it is believed that this feature may have arisen from poor assembly of the IMP for the early boreholes, which gave rise to protuberances on the shaft so causing the relief of the radial stresses by the time the upper transducer set reached a given element of soil. However, the radial effective stress was not affected by this, and for the data shown in Fig. 5 there were no consistent differences between the radial effective stresses for the two transducer sets.

At any particular depth, the radial effective stresses shown have been calculated for each of the two principal transducer sets by taking a running average integrated over a length of the borehole equal to the length of the corresponding friction sleeve, and centered at the depth at which the data point is plotted. By this means, the data may be correctly compared with the values of unit shaft friction calculated for
the two friction sleeves, for which each data point is also plotted at the depth of the centre of the sleeve. Figure 6 shows data from both friction sleeves, but again there was no consistent difference between the two.

Values of the ratio of unit shaft friction to radial effective stress, \( \tau / \sigma_r' \) for the installation phase in most of the boreholes are presented in Fig. 6. Again, data are plotted for both friction sleeves. Since the upper friction sleeve lies between the two sets of pressure transducers, the \( \sigma_r' \) value used to calculate the ratio for this sleeve was interpolated between that calculated for each of the transducer sets. The cavity expansion predictions shown on Fig. 5 have been derived from Equations 1 and 2 using the soil strengths shown on Fig. 3. The large strain shear moduli from the pressuremeter investigation of this site were believed to be appropriate for these calculations and a value of 10MPa at 2m was chosen, increasing linearly to a constant value of 40MPa below 7m. The mean normal effective stress at failure was calculated as:

\[
Pf' = (\sqrt{3}/M_{ps}) s_{uops}
\]  

where \( s_{uops} \) is the undrained shear strength measured in plane strain.

The plane strain value of the critical state line gradient, \( M \) was calculated as follows:

\[
M_{ps} = \sqrt{3} \sin \phi'_{ps}
\]

where \( \phi'_{ps} \) is the friction angle measured in plane strain.

As Wroth (Ref. 17) proposed, \( \phi'_{ps} \) has been estimated as 9/8\( \phi'_{tc} \) (triaxial compression). A value of \( \phi'_{ps} \) of 24° has been selected from data presented by Clegg (Ref. 15).

The initial pore pressure \( u_0 \), as measured by the standpipe piezometers at the time of the IMP boreholes, was added to the increase predicted by cavity expansion to obtain the total pore pressure which is compared on Fig. 5 to the IMP measurements. The radial effective stress from Equation 1 was then added to this to obtain the total radial
stresses also shown on Fig. 5.

Since there is friction on the pile/soil interface during installation, the major principal stress cannot be radial as assumed by Equation 1. However, using a solution by Sagaseta (Ref. 18), Coop (Ref. 10) has shown that neither this nor the initial insitu stress anisotropy has a major influence on the predicted radial stresses.

The radial effective stress predictions given on Fig. 5 fall within the scatter of IMP data, and have a similarly shaped profile, confirming that the installation radial effective stress is a function of $s_{uo}$. However, the predictions are around 20% higher than the mean of the IMP data.

The pressuremeter and triaxial test undrained shear strength data for this site are inseparable. To cover any uncertainty about the chosen $s_{uo}$ values, the extreme assumptions have been made, firstly that the profile is representative of a triaxial compression shear strength, and then that it was plane strain. The following equation arising from the extended Von Mises failure criterion was used to relate the two:

$$s_{uops} = (2/3)s_{uotc}$$

where $s_{uotc}$ is the triaxial compression undrained shear strength

$s_{uops}$ is the plane strain undrained shear strength

The shaded bands for the predictions span the two assumptions. For the total radial stresses and pore pressures the uncertainty is small relative to the large differences between the predictions and IMP measurements. However, measurements again show similar profile shapes to the predictions. Both are also similar to the $s_{uo}$ profile shape indicating that in this type of soil the third term of Equation 2 dominates.

Self boring pressuremeter data for this site (Ref. 12) showed terminal pressures at the maximum cavity strain reached which are around $100kPa$ higher than the IMP total radial stresses, but below the predicted cavity expansion limit pressures. The inflation curves indicated that limit pressures had not been reached and this is confirmed by data from
the prototype cone pressuremeter (Ref. 16). This instrument is installed as a normal cone penetrometer and then a pressuremeter located on the shaft is inflated. Limit pressures are quickly reached and these data agree well with cavity expansion predictions. If cavity expansion correctly modelled the installation of such an instrument, the limit pressure would be reached immediately, and it is interesting that the lift off pressures agree well with the IMP total radial stress measurements during installation.

The values of $\tau/\sigma'_r$ appear to be residual values. Lemos (Ref. 19) suggested that the measured friction angle in an interface ring shear test might be influenced by, among other things, the interface roughness and the speed of shearing. A series of Bromhead ring shear tests was therefore conducted; two tests with the conventional platens, and two using machined brass top platens prepared in a similar way to the IMP shaft. The tests were conducted on a soil sample retrieved by the IMP from 3.5m depth. Two separate machines were used, and for each test shearing speeds of 0.59mm/min and 0.02mm/min were investigated for normal stresses ($\sigma_n'$) of both 223kPa and 591kPa. These are similar to the effective radial stresses measured by the IMP.

The normal stress and shearing speed were not found to influence consistently the ratios of $\tau/\sigma'_n$ for these simple tests. The smoothness of the interface had only a minor effect, reducing the ratio from 0.17 for the conventional apparatus to 0.15 for the smooth brass interfaces. The latter value is plotted on Fig. 6 at the soil sample depth. It is clear that the values of $\tau/\sigma'_r$ determined from effective stresses during the relatively rapid shearing of 200mm/min agree well for this soil with the slow, drained value obtained from laboratory interface ring shear tests.

Extraction of the IMP from its borehole was conducted at around 1300mm/min, but still no influence was found on the $\tau/\sigma'_r$ ratio when measured in terms of effective stresses. Upon retrieval, the IMP was always found to be covered only in loose, wet mud, underneath which the shaft was highly polished. However, a section of the shaft was knurled for the final borehole, the data from which have not been presented in this paper. Upon extraction the roughened section was covered in 4.5mm of firm clay. Total radial stresses from transducers above the roughened
section during penetration indicated lower readings than for other boreholes due to the relief of the radial stress behind the coating. Clearly the residual plane was at some distance from the IMP. Since the shear stress in the soil is inversely proportional to the radius from the IMP centreline, there may have been a minimum radial effective stress at some distance from the shaft. This would not be expected from cavity expansion, since the effective stresses are predicted to be constant in a zone of soil up to several pile radii from the shaft where the soil has reached a critical state.

Open-ended penetration was conducted in a number of boreholes. As previously described, the IMP cutting shoe encourages soil into the instrument, and it is thereby intended to simulate the installation of open-ended cylindrical piles which have lower ratios of wall area to gross cross-sectional area. The "apparent area ratio", that the IMP achieves is calculated as the proportion of the soil immediately below the shoe which is pushed outwards rather than entering the instrument’s sample chamber, since it is assumed that the soil displaced by the walls of an open-ended tubular pile would only move outwards. Unfortunately, the lowest area ratios achieved at this site were only about 40%. Measurements also showed great variability, with such a minimum only being maintained for a few centimetres before internal friction caused the ratio to rise steeply with further penetration. Numerical cavity expansion predictions by Carter, Randolph & Wroth (Ref. 20) predict that the effects on the radial stresses would be small for the IMP’s high area ratios. This was confirmed by the fact that within the data scatter it is not possible to discern any clear effects on stress measurements due to open-ended penetration, and these data are omitted from the data presented in this paper.

Phase 2: Reconsolidation

Figure 7 shows typical reconsolidation data for Madingley from Borehole G with the IMP tip at 5.79m. Soon after halting penetration, large positive pore pressures build up along the length of the IMP, and there then follows a period of stabilisation, culminating after 5-6 hours with a uniform pore pressure distribution along and around the IMP. The maximum pore pressures registered are still well below cavity expansion
predictions.

The source of these pore pressures is uncertain, but since the tip transducer registers the increase last, they cannot result from high pore pressures near the tip spreading back along the shaft. It is more likely that there is a pore pressure maximum during installation which is radially remote from the shaft. For this borehole the conventional, smooth walled IMP was used but tests with the rough walled modified IMP previously described provided additional evidence for this conclusion.

Total radial stresses show a slight decline during the initial stage of reconsolidation, leading to a large reduction in the effective radial stress, and a minimum was reached after about one hour. The behaviour is quite different from the expected monotonic rise in radial effective stress.

In one borehole a load test was conducted at the point of minimum radial effective stress during reconsolidation. A decrease of unit shaft friction of 34% was observed compared to the value immediately before halting installation. This agreed well with a radial effective stress which at failure was 30% lower than during penetration.

When pore pressures start to decay the rate of decrease of total radial stress increases and the recovery of the radial effective stress is insufficient for it to return even to installation values. Final pore pressures agree well with standpipe measurements, but it is interesting that once primary consolidation is complete the radial total stresses continue to creep downwards.

During installation, the two transducer sets had given similar radial effective stresses despite the fact that the total stresses measured at a given depth decreased as the IMP penetrated past that point. After complete reconsolidation both the effective and total stresses are similar for the two transducer sets showing the variation of total stress along the IMP shaft was a feature only of installation.

The theoretical pore pressure dissipation curve shown on Fig. 7 is based on predictions by Wroth et al (Ref. 4) and represents the behaviour that might have been expected if the initial pore pressure distribution
had conformed to cavity expansion predictions. The shaded band covers the two $s_{uo}$ assumptions and the various transducer depths. A Poisson’s ratio of 0.2 was assumed with a permeability of $2.7 \times 10^{-10} \text{m/s}$ based on consolidation coefficients from Kay and Parry (Ref. 13) insitu screw plate data.

Although pore pressures adjacent to the shaft during installation are lower than expected, there are extensive positive pore pressures in the soil around the instrument which take an order of magnitude longer to dissipate than might have been predicted. Quicker dissipation would have been expected since the prediction assumes an infinitely long solid pile. The soil adjacent to the IMP during reconsolidation had also been penetrated open-ended although with rather high area ratios as previously discussed.

A failure in the back pressure system at $\log_{10} t = 5.3$ resulted in a decrease of 130kPa in the back pressure until the fault could be repaired. The resultant dip of around 20kPa in the total radial stress readings illustrates the influence of transducer compliance.

**Phase 3: Load Tests**

Following the complete reconsolidation allowed in Borehole G, load tests were performed. These were nominally undrained, failure being achieved in around 90 seconds. Figure 8 shows the unit shaft friction, pore pressures and total radial stresses measured by the IMP together with the calculated $\tau/\sigma'_r$ ratio mobilised on the shaft. The most interesting feature of this test is the sudden drop in pore pressure at failure. Total stress transducers show little response with the exception of those mounted in the upper transducer set. The steep decreases these transducers measure during the load test result solely from the transducers moving into an horizon of softened soil created by the IMP during the penetration phase. Immediately prior to penetrating open-ended, the IMP had been extracted by about 30 mm to allow the release of the mechanism which seals the sample chamber. This process causes local softening below the tip of the instrument which appears as a dip in the profile of radial stress measurements.
The mobilised $\tau/\sigma'_{x}$ ratios are the same as those calculated for the installation phase and also for the slow drained interface ring shear test.

The unit shaft friction at failure was 66kPa for both friction sleeves, which is, perhaps coincidentally, similar to that when penetration was halted, for which the lower and upper sleeves had given 69 and 64kPa respectively. It is clear that for this clay there is no increase in shaft frictional capacity, or set-up associated with reconsolidation. This arises from the fact that the expected monotonic increase in the radial effective stress was not observed.

Towards the end of the load test, and then after unloading, positive changes in pore pressure were observed by some transducers. Clearly the pore pressure distribution around the IMP during loading is complex. During the period of stabilisation allowed before the next load test, positive or negative pore pressures generated by loading dissipated within a few hours emphasising their more localised nature by comparison with those generated during installation.

Several monotonic load tests were conducted, both tensile and compressive, and similar unit shaft frictions were measured for both.

FIELD WORK AT HUNTSPILL

One of the two sites chosen with normally consolidated clay was at Huntspill. This site is located on the Somerset Levels, about two miles South of Burnham-On-Sea, and is adjacent to the River Parrett. The land is owned by Somerset County Council and was extensively investigated for use as a waste disposal site, under the supervision of Aspinwall and Co. (Refs. 21, 22 and 23). The soils are soft silty clays which are geologically recent, but agricultural drainage has given rise to a pronounced overconsolidated crust which can be clearly seen in the $s_{uo}$ profile shown in Fig. 9. The chosen profile has been derived from end resistance data from a cone penetrometer sounding adjacent to the IMP boreholes. A cone factor, $N_{k}$, of 15 was derived from a correlation of end resistance with field vane and laboratory triaxial undrained shear strengths over the whole site. In the calculation of cavity expansion
stresses, this profile has been assumed to represent \( \sigma_{ouc} \).

The insitu pore pressures shown on Fig. 10 are based on standing water levels in the boreholes, but take into account some slight downward drainage which Aspinwall (Ref. 23) showed to exist below 4m. Vertical effective stresses are based on unit weights presented by Aspinwall (Ref. 22). Estimates of overconsolidation ratio made from the oedometer data in this reference agree well with the trend reported by Cook & Roy (Ref. 24) for the Somerset alluvium and also shown on Fig. 10. The latter has been used to derive estimates of insitu horizontal stresses by means of Mayne & Kulhawy's equation:

\[
K_o = (1 - \sin \phi')OCR \sin \phi'
\]

(6)

where \( K_o \) is the insitu earth pressure coefficient.

A value of \( \phi_{tc} \) of 27.5\(^\circ\) was estimated from plasticity index data from the Aspinwall investigation using the relationship given by Kenney (Ref. 25).

The stresses measured by the IMP during installation in one of the boreholes at this site are shown on Fig. 11 along with cavity expansion predictions. As before, only closed-ended continuous penetration data are presented. Radial effective stresses and unit shaft frictions have been calculated in the same way as for Madingley, and are illustrated in Figs. 11 and 12. Lower friction sleeve data are omitted below 5.5m tip penetration, since during the reconsolidation allowed at this depth, the lowermost load cell drifted badly.

In calculating the cavity expansion stresses using Equations 1 and 2, a ratio of shear modulus to undrained shear strength of 93 has been estimated based on pressuremeter data at a similar site. A factor of two on this estimate only alters the predicted total radial stress by 8%.

The data are presented at an enlarged scale relative to that for Madingley. In this soil which is essentially normally-consolidated, the insitu stresses are a much more important component of the cavity expansion stresses, and the linear increase of both predicted and measured stresses with depth results from both the increase of insitu
stresses and the corresponding increase of undrained shear strength.

The IMP sounding starts towards the end cf the overconsolidated crust. The shear strength and hence the predicted stresses decrease rapidly with depth until about 3m depth below which a linear, normally consolidated increase with depth is predicted. IMP measurements show a similar pattern, although as for the Gault clay, they are less than predicted. The exception to this is the tip pore pressure transducer which, below 4m gave readings about 60kPa greater than those made further back along the IMP shaft. Otherwise, the pore pressure predictions form an approximate upper bound to the measurements.

The pair of total radial stress transducers which give the higher readings on Fig. 11 are from the lower transducer set. Those measured at a given depth by the upper set are about 40-50kPa less. However, the opposite change in pore pressures is seen, those measured by the upper set of transducers being slightly greater than those by the lower. Figure 11 therefore shows that whilst the lower transducer set gives a very good correlation with the radial effective stress predictions, the upper set does not. It is likely that the reason for this is that significant reconsolidation of the soil takes place during the time between the two transducer sets reaching a given depth. Although the nominal penetration rate is 200mm/min, allowing for drill rod changes and jack upstrokes, a mean rate of only 70mm/min was achieved, giving about 10 mins between the two pressure transducer sets reaching a given depth. This did not appear to be a problem at Madingly where the radial effective stresses measured by the two transducer sets were indistinguishable. However, the reconsolidation data for Huntspill, shown on Fig. 13 again indicates an initial phase during which the pore pressures rise whilst the total radial stresses drop, leading to a substantial reduction in the effective radial stress. In this more permeable clay, a delay of ten minutes is of the same order of magnitude as the time taken to develop the minimum radial effective stress. It is therefore not possible to separate any genuine radial stress changes along the shaft of the instrument from the effects of reconsolidation due to slow installation.

Figure 12 shows that the two friction sleeves gave very similar unit shaft frictions and therefore quite different ratios of unit shaft
friction to radial effective stress. However, all IMP values of $\tau/\sigma'_{n}$ are very much lower than measured in the Bromhead ring shear apparatus. The same programme of ring shear tests as for Madingley was performed, but under normal stresses of 28 and 126 kPa. Again the speed of shearing had no conclusive effect, but $\tau/\sigma'_{n}$ ratios of about 0.36 were obtained for the lower normal stress test with conventional platens, compared to 0.28 for the higher. At these low stresses this may result from friction in the machine. If this is assumed to be the case, and the intercept on the $\tau:\sigma'_{n}$ graph is neglected, a $\tau/\sigma'_{n}$ ratio of 0.26 was obtained for the standard rough platen. As for the Gault clay, there was a small reduction when a smooth brass top platen was used, for which a value of 0.22 was obtained. However this is still much higher than any values calculated from IMP measurements.

As already discussed, the data for the reconsolidation allowed at 5.51 m penetration are similar to those for the Gault clay. Final radial effective stresses are below those during installation, and for the upper transducer set remain below insitu horizontal effective stresses. Final pore pressures again agree well with estimates of ambient values.

Dissipation curves have again been predicted using the elastic solution of Wroth et al. (Ref. 4) for the dissipation of cavity expansion pore pressures. The soil permeability used was $2 \times 10^{-9}$ m/s based on an adjacent insitu test by Somerset County Council (Ref. 21). The shaded band covers the various transducer depths. This prediction is based on an initial pore pressure distribution which is clearly incorrect, but it serves to show that since measured dissipation times are similar to those predicted, there are extensive positive pore pressures generated around the instrument during installation. As at Madingley, shorter dissipation times would have been expected due to the existence of non-radial drainage paths and the fact that the IMP again approached the reconsolidation depth open-ended. This time a minimum area ratio of 30% was achieved over a 200 mm section of borehole which was adjacent to the centre of the IMP, between the two transducer sets, during reconsolidation.

Figure 14 summarises data from the first undrained load test after complete reconsolidation. A unit shaft friction of about 11 kPa was developed, representing a 160% set-up when compared to the unit shaft
friction immediately before halting installation. It is clear that this cannot have resulted from a radial effective stress increase during reconsolidation, and Fig. 14 indicates that such an increase actually occurs during undrained loading. As in the Gault clay, pore pressures drop dramatically as failure is approached. The resulting radial effective stress increase is very much more important in the normally consolidated clay. Total radial stresses are fairly constant until the IMP starts to move, then showing a slight increase, perhaps as the transducers move into slightly stiffer soil. A subsequent tensile load test showed the reverse total radial stress change but similar unit shaft frictions. Another feature in common with the Gault clay tests is the increase in pore pressure shown by some transducers towards the end and immediately after the load test.

Mobilised $r/\sigma'_r$ ratios are also shown on Fig. 14. At failure the ratio is about 0.16, quite similar to values calculated from installation phase data at this site, and very much lower than the laboratory ring shear data previously described. However, immediately prior to failure, a ratio of about 0.25 was developed, which is similar to the ring shear data. This may indicate that stresses measured at the IMP shaft are not representative of those on the failure plane which may therefore be at some small distance from the shaft. In preparing the IMP for this borehole, four black marker pen lines had been drawn along the length of the instrument to provide some qualitative assessment of the abrasion to the IMP shaft. Similar lines were completely erased for the smooth walled IMP tests at Madingley, and when retrieved, the IMP shaft was highly polished. However, at Huntspill, the clay adhering to the shaft was noticeably firmer than at other sites, and although abrasion had polished the shaft, clear remnants of the marker pen lines remained. This perhaps indicates that principal displacements had not been along the shaft surface.

CONCLUSIONS

The IMP work has confirmed that in clays a residual friction surface is formed along or near the shaft of a pile during installation. Residual friction angles developed on the failure surface are the same for installation and load tests. For Gault clay, these angles agreed
well with the slow drained residual friction angle measured in the laboratory. For the alluvial clay they did not, and mobilised friction angles on the IMP shaft were consistently low, only reaching similar values to those of the ring shear immediately before failure in the load test. A possible explanation for this is that the failure surface is remote from the shaft, and this certainly did appear to be the case in the Gault clay when a knurled shaft was used.

It is known that cavity expansion does not accurately model the installation process, and could not successfully predict the installation stresses. It appears that there is stress relief immediately behind the pile tip as shown by the predictions of LeVadoux and Baligh using their strain path method for cone penetrometers, data from which are shown in Fig. 1. This results in the radial total stress and pore pressures on the shaft being less than predicted. The severity of this stress relief increases with soil stiffness and the agreement between measurements and predictions of total radial stresses and pore pressures is much better in the alluvial clay investigated than in the Gault clay. Total radial stresses and pore pressures appear to be similarly affected by this stress relief, and cavity expansion provides a reasonable prediction of the radial effective stress providing installation is sufficiently rapid to avoid consolidation effects. The method could therefore be simply used to predict the soil resistance at time of driving for full size piles, providing the residual friction angle of the clay was known. However, such factors as pile driving whip and the poorer fabrication tolerances of offshore piles by comparison with the IMP may well influence the radial stress developed. Partial displacement driven piles are also not likely to be driven quickly enough to avoid consolidation effects.

The IMP data confirm that the installation radial effective stress is a function of the initial undrained shear strength, but that the total radial stress and pore pressure depend additionally on the initial insitu stresses. In heavily overconsolidated soils, the latter component is small and it is clear that the normalisation with respect to the initial vertical effective stress as is frequently used would be misleading.

During reconsolidation, there was no monotonic rise in the radial effective stress, but instead, a steep initial decline was seen followed
by a slow recovery, which in the two clays described resulted in final values which were similar to, or even less than installation values. Full reconsolidation was found to take longer than could have been anticipated and it is likely that many test piles are therefore loaded prematurely.

Dilation on the residual surface during undrained loading results in a pore pressure decrease, whilst total radial stresses remain unaffected. During installation, similar localised decreases in pore pressure adjacent to the shaft may be superimposed on the more extensive positive pore pressures resulting from the radial straining. This could explain the initial rise in pore pressure measured at the shaft when driving is halted. Since the dilative pore pressures must be generated just once at the start of loading, towards the end of the load tests the pore pressure transducers often show increases in pore pressure presumably resulting from the shear loading of the soil beyond the residual failure surface. The resultant radial effective stress change should affect the mobilised shaft friction, and it is likely that for nominally undrained tests, the pore pressure at the failure surface, and hence the unit shaft friction will be highly sensitive to loading rate. More consistent data would be obtained from fully drained tests.

The magnitude of the increase in radial effective stress during loading is broadly similar in the normally consolidated and the heavily overconsolidated clays. It is therefore relatively much more important in the former and entirely accounts for the set-up of pile capacity. Since the same radial effective stress increase would not be expected for fully drained loading, the pile capacity under these conditions may be very much lower and requires investigation. If the magnitude of the radial effective stress increase is insensitive to stress level, then factors used in pile axial capacity determination, such as $a$, $b$ and $\lambda$ would be expected to decrease with depth, which has often been observed.

Many pile tests, including some of the IMP work, have been carried in older sedimentary stiff clays, which have very low residual friction angles. However, North Sea boulder clays have residual friction angles much closer to the peak friction angle, and extrapolation of data from one type of clay to another, without accounting for the difference in residual friction angle is unsound. Further research is required to
check if the mechanisms of pile behaviour are the same in the latter type of clay.

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Fig. 1. Strain path predictions for stresses around a penetrating cone (After Levaudoux & Haligh 1980)
Fig. 2. The In situ Model Pile
Fig. 3. Madingley soil strength data

- Self-boring pressuremeter (PM in-situ (Ref. 12))
- Kay & Parry screw plate (Ref. 13)
- Trend of Clegg's undisturbed triax. comp. data (Ref. 15)
- BRE dilatometer (Ref. 14)
- Fugro undisturbed triaxial compression (Ref. 11)
- Selected profile
Fig. 4. In situ stresses at Madingley
Fig. 5. Comparison of installation stresses and cavity expansion predictions for Maidingley.
Fig. 6. Madingley shaft friction data
Fig. 7. Madingley reconsolidation data
Fig. 8. Madingley load test data
$s_u_0$ (kPa)

Depth (m)

$S_{uo}$ derived from Fugro CPT using $N_k=15$

+ Somerset CC unconsolidated undrained triaxial test

Fig. 9. Undrained shear strength data, Huntspill
Fig. 10. In situ stresses at Huntspill
Fig. 11. Comparison of installation stresses and cavity expansion predictions for Huntspill
Fig. 12. Huntspill shaft friction data
Fig. 13. Huntspill reconsolidation data
Fig. 14. Huntspill load test data