THE STRESS ACTING ON A CONTINUOUSLY PENETRATING PILE

LES FORCES SUR UN PIEU QUI PENETRE CONTINUELLEMENT

НАПРЯЖЕНИЕ, ДЕЙСТВУЮЩЕЕ НА НЕПРЕРЫВНО ПОГРУЖАЕМУЮ СВАЮ

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SYNOPSIS.. The paper reports the results of field tests on a 100 mm. diameter by 4 m. long steel pile jacked into and out of a London Clay bed at a constant rate. The pile was elaborately instrumented in an attempt to obtain detailed information on the magnitude and distribution of the shear stresses and total radial stresses acting on the moving pile surface. The measurements presented are discussed in relation to the undrained shear strength profile of the clay and previously published results.

INTRODUCTION

The following measurements were obtained as part of a small field-scale research programme concerned with the effect of electro-osmosis on the penetration resistance of steel piles (Johnston 1972). In this connection, the distributions of shear stress and radial stress on the pile surface were of particular interest and the model pile used (100 mm diameter x 4 m long) was elaborately instrumented in an attempt to measure them. Some tests were performed without the application of electro-osmotic potentials, and it is with the results of these that this paper is concerned. They are thought to be of interest since total radial stresses and local shear stresses were measured at eight locations along the pile shaft and axial loads at five locations, both whilst the pile was being jacked into the ground, at a controlled rate of 0.35 mm/sec. and then immediately extracted at a rate of 0.2 mm/sec. Such detailed measurements of local stresses acting on a continuously penetrating pile using purpose made load cells, together with a carefully determined undrained shear strength profile for the subsoil, do not appear to have been reported previously.

SOIL DATA

The site soil profile (Fig. 1a) shows approximately 2 m of compact flint gravel overlying 2.5 m of stiff brown silty clay, followed by at least 25 m. of stiff dark grey silty clay of low sensitivity with small scale fissures at shallow depths, in fact, the London Clay of the Hampshire Basin. The gravel was excavated in a trench some 2m x 2m x 50 m long and all piles were driven from the bottom of the trench which was prevented from drying out by a small inflow of water from the adjacent gravel.

Four shallow shell and auger boreholes sunk along the trench all indicated similar soil profiles and water content distributions (Fig 1c). Some 60 no. 75 mm x 38 mm diameter undisturbed samples of the clay, recovered from depths of between 2 m and 7 m below ground level, were tested in undrained triaxial compression. In all these tests the samples were initially re-consolidated to their mean estimated in-situ effective stress, lubricated enlarged end plattens were used together with secondary plattens supported on ball races to eliminate any side thrust on the triaxial cell piston and the subsequent undrained loading was performed at a relatively low axial strain rate (0.2%/min.) to allow equalisation of pore water pressure changes throughout the specimen. Consequently the undrained shear strength (c_u) profile obtained is thought to be more than usually reliable. However, although all the test results do define a consistent pattern, as Fig. 1b shows, the variation of strength within the 4 m depth of interest was unexpectedly high. Since even the local stresses on the pile shaft will necessarily involve some integrated property of the adjacent soil, the more abrupt changes of strength are unlikely to be reflected in the pile measurements and the mean curve shown in Fig. 1b has been adopted as a reasonable representation of the strength profile. The way in which the variation in c_u is reflected in, for example, the measured local radial stresses (Figs. 5) and the bottom axial load cell readings (Fig. 10) is thought to be quite remarkable and emphasises the importance of reliable strength profile data if pile tests are to be analysed in detail.

PILE AND TEST DETAILS

Fig. 2 shows the location, along the 4 m pile length of the 5 axial load cells and the 8 local cells. The latter measured independently both the local total radial stress and the local shear stress at

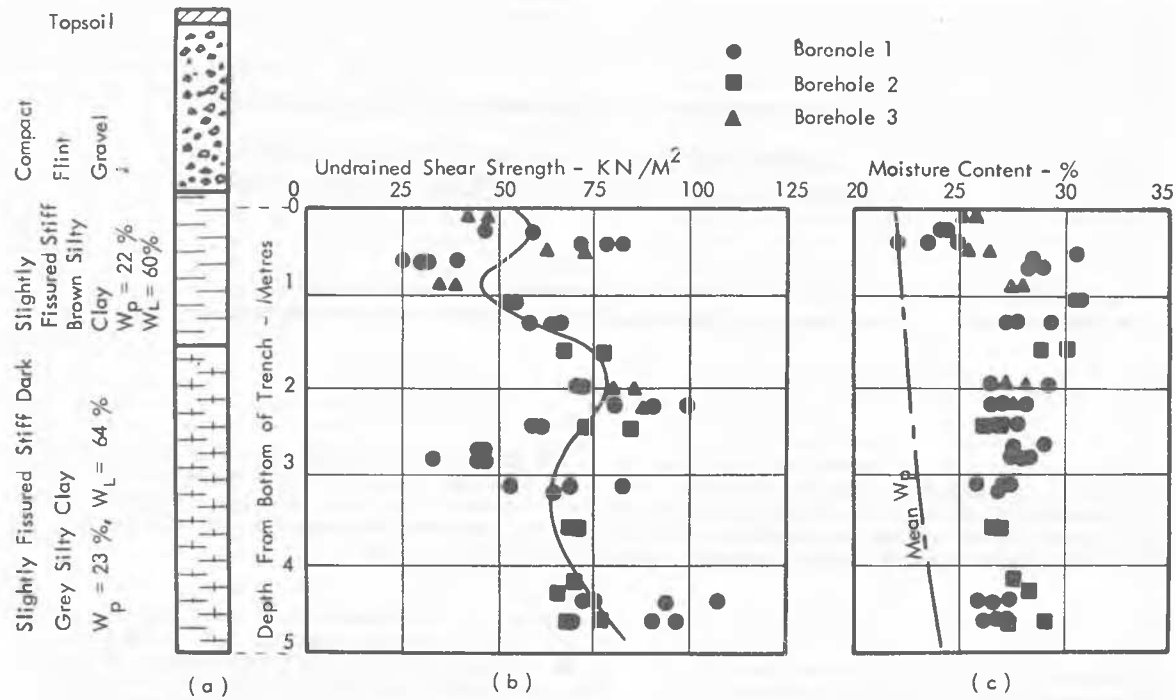

Fig 1 SOIL PROFILE DETAILS

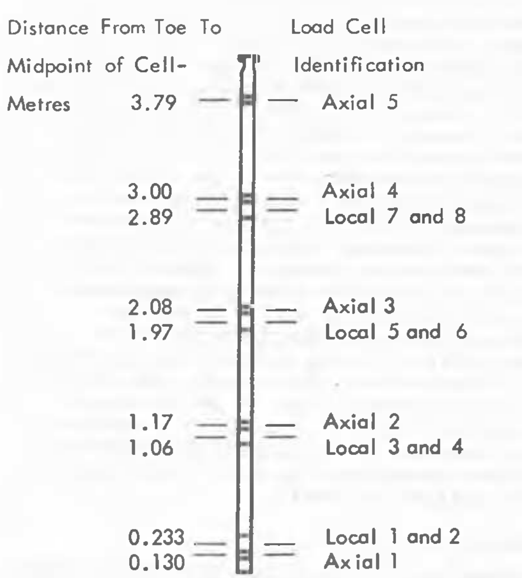

Fig 2. IDENTIFICATION AND LOCATION OF LOAD CELLS

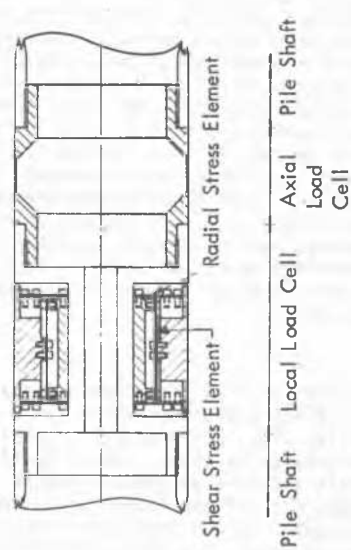

Fig 3 LOAD CELL DETAILS

opposite sides of the 100 mm diameter pile. The purpose made axial cells were of a simple design, Fig. 3 upper part; each incorporated 8 foil resistance strain gauges and were independently calibrated to allow their cross-sensitivity to radial stress to be corrected from the adjacent local radial stress measurements. Axial load changes of the order of ± 200 N could be detected. The local cells were much more complicated, Fig. 3 lower part; each incorporated 20 similar strain gauges and were an adaptation of the Cambridge earth pressure cell design (Arthur and Roscoe, 1961). In order to function satisfactorily as boundary cells they have not only to be fitted accurately into the pile face but also need to have the maximum tolerable stiffness. The cells used had stiffnesses of about 580 kN/mm. and 30 kN/mm. and stress sensitivities of ± 15 kN/m^2 and ± 3 kN/m^2, in the radial and axial shear directions respectively. All cells were thoroughly sealed and waterproofed and, apart from one axial cell damaged mechanically during driving and one pair of local cells in which the waterproofing failed, they all functioned satisfactorily throughout 13 cycles of driving and extraction (including for one test of 5 days burial) and subsequently gave calibration curves very close to their original values.

Each 1 m. pile section was jacked at a constant rate by a double acting 15 ton hydraulic ram supported in gimbals on a kentledge beam system which spanned the trench between two rail mounted bogies.

TEST RESULTS

The test results presented refer particularly to two penetration and extraction cycles of the pile in positions spaced about 3 m apart along the trench, close to boreholes 1 and 3. The total recorded axial loads for the two complete cycles are shown in Fig. 4, excluding the unloading sequences which occurred during the addition of each 1 m. pile section.

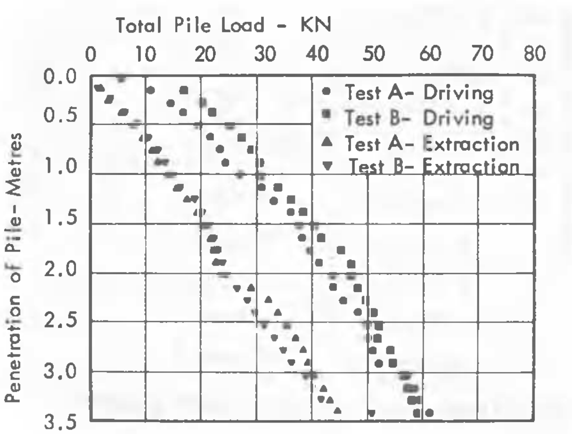

Fig 4 TOTAL PILE LOAD- DRIVING AND EXTRACTION

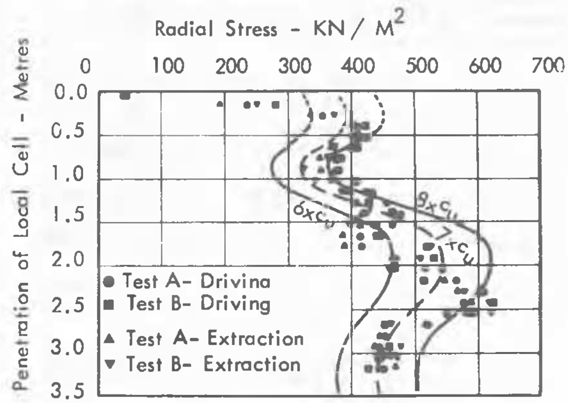

Fig 5a RADIAL STRESS ON LOCAL CELLS 1 and 2

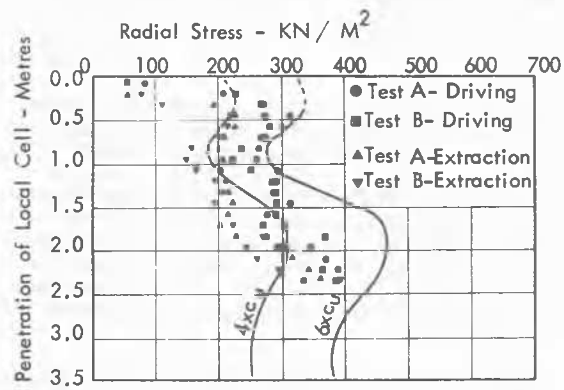

Fig 5b RADIAL STRESS ON LOCAL CELLS 3 and 4

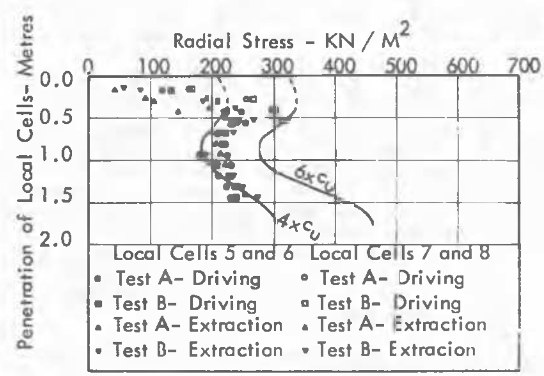

Fig 5c RADIAL STESS ON LOCAL CELLS 5 and 6,7 and 8

The subsequent figures all refer to more detailed measurements of the loads and stresses acting on the pile during these tests.

Figs. 5a, b, c summarise the complete set of total radial stress readings obtained from all the local cells 1 to 8 during essentially continuous driving and extraction. Superimposed on the experimental points are curves showing multiples of the undrained shear strength based on Fig. 1b. From Figs. 5 it is evident that as any particular load cell penetrates the ground, the total radial stress acting on it is strongly influenced by the c_u value of the adjacent soil. Also the radial stress is consistently around 30% to 35% higher on local cells 1 and 2, near the pile toe, than on the others which show a slight progressive decrease of recorded stress with increase in distance from the toe. As the pile is withdrawn, the radial stress acting on each cell generally reduces by a very small amount. (Here Fig. 5b, cells 3 and 4, shows anomolously low values which are similarly at variance with the general trend in Fig. 9b). It is also interesting to note that within the experimental scatter, it is impossible to distinguish between the readings of diametrically opposed cells (e.g. 1 and 2) which suggests that the stress distributions do not depart significantly from radial symmetry.

More specifically at penetration depths below about 4 pile diameters the numerical value of the total radial stress is seen to range between 4 and 8 times the value of c_u. These values correspond quite reasonably with theoretical predictions of the increase in radial stress due to the pile (5 to 7 x c_u) obtained from an elasto-plastic analysis of the expansion of a deep cylindrical cavity from zero radius to the pile radius (Butterfield and Banerjee, 1970) and with the field measurements of Koisumi and Ito (1967).

The actual distribution of total radial stress along the pile at two typical penetration stages (3.3 m. and 2.3 m), driving and extraction, is shown in Figs. 6a and b. To the right of each figure the measurements are plotted in dimensionless form by dividing the cell readings by the local value of c_u. In view of Fig. 5, these are obviously the more significant curves and suggest the typical total radial stress distribution to be expected in a homogeneous clay bed although the rapid variation of c_u along the pile length unfortunately reduces the reliability of the results.

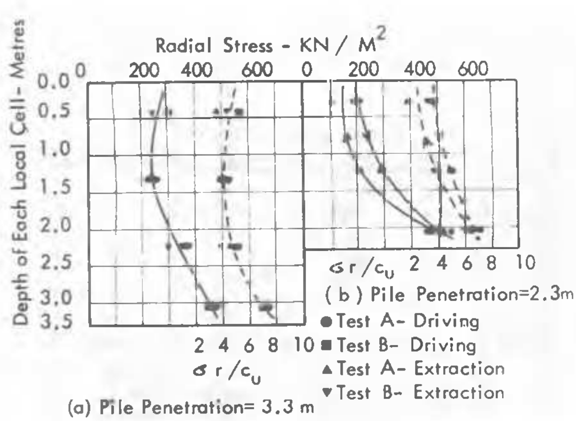

Fig 6 TYPICAL DISTRIBUTION OF RADIAL STRESS ON THE PILE SHAFT

The load carried between the pile head and each of

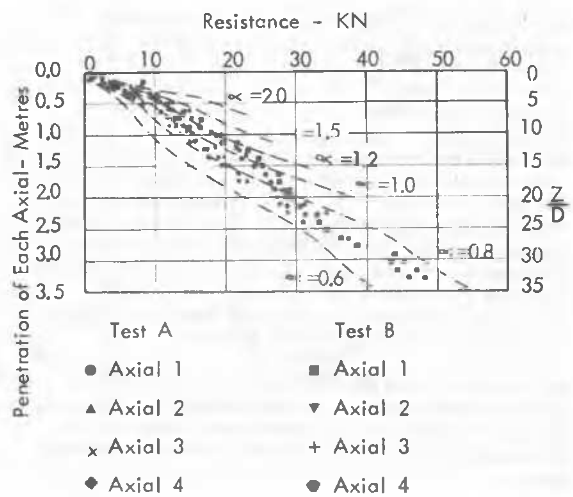

Fig 7a LOAD CARRIED BY PILE SHAFT BETWEEN SURFACE AND INDIVIDUAL AXIAL CELLS -DRIVING

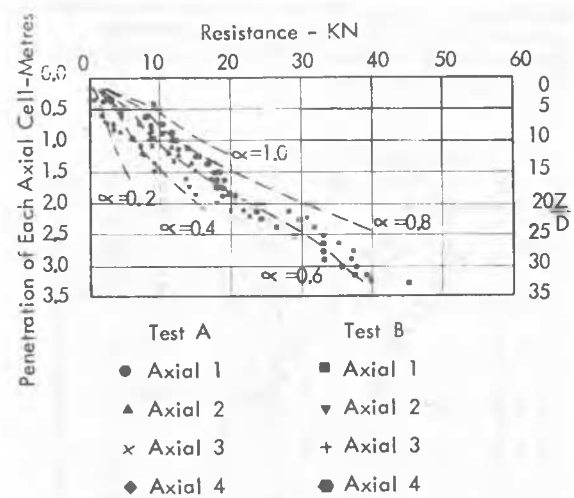

Fig 7b LOAD CARRIED BY PILE SHAFT BETWEEN SURFACE AND INDIVDUAL AXIAL CELL -EXTRACTION

the axial cells, nos. 1 to 4, is given by the points marked in Fig. 7a for driving, and Fig. 7b for extraction. The dashed curves represent the load which would be carried by the pile if the α* value, marked on the curves in conjunction with the c_u values of Fig. 1b, applied uniformly along the embedded length. It is therefore evident that the mean α value applicable at any penetration is rather lower for extraction than for driving and that α is higher for shorter piles than for long ones.

Furthermore, α not only varies along the shaft but in general appears to reduce very slightly with increasing distance from the pile toe, although remaining consistently higher in the upper 15 diameters of pile length at all stages. It is interesting to note how even rapidly varying values of c_u, when integrated along the shaft length, produce relatively smooth shaft resistance curves (dashed lines, Fig. 7) and that at full penetration the α values approach about 0.7 and 0.6 for the shaft above axial cell 1 during driving and extraction respectively.

At any penetration stage the difference between the readings of neighbouring axial cells enables the mean pile interface shear stresses to be calculated over the intervening length of pile. The results of such calculations during driving are plotted as the stepped dashed lines in Fig. 8 for penetrations of 3.3 m. and 2.3 m. Simultaneously recorded local shear stresses are also shown and it is encouraging to note the close similarity between the two sets of values since the measurement of truely local resultant stresses on pile surfaces has not been accomplished previously.

The dimensionless curves to the right of Figs. 8 have been produced by the same means as those in

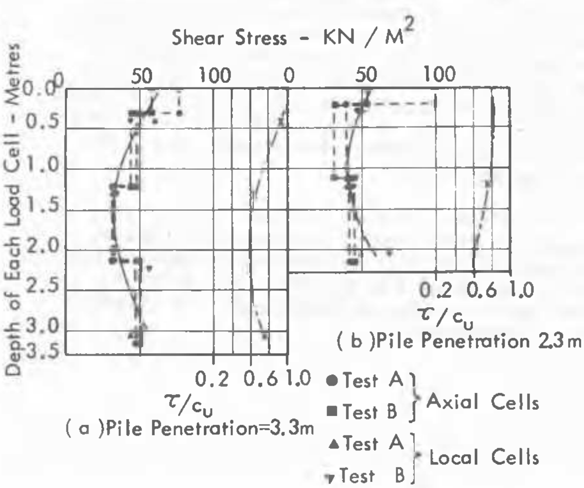

Fig 8 TYPICAL DISTRIBUTION OF SHEAR STRESS ON THE PILE SHAFT - DRIVING

* α = Adhesion Factor, τ/c_u.

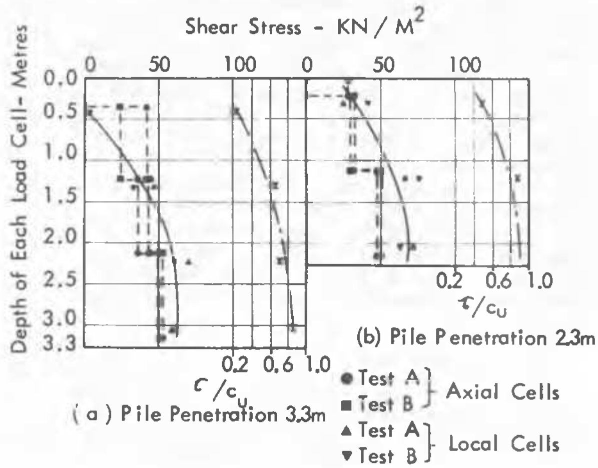

Fig 9 TYPICAL DISTIBUTION OF SHEAR STRESS ON THE PILE SHAFT - EXTRACTION

Figs. 6 and similar comments are applicable to them. Figs. 9 are analogous results for the pile during extraction from the same depths. The agreement between the two sets of values is less good in Fig. 9b (see also note referring to Fig. 5b) but all the results obtained generally support a reduction of interface shear stress at the pile head between driving and extraction.

The simultaneous local measurement of shear and radial stress allows an effective interface friction angle (δ) to be assessed and from some 200 values obtained 46% of them fall within the range $\delta = 10^{o} \pm 2^{o}$ (Fig. 11). If the results obtained from the extremities of the pile, local load cells 1 and 2 and any local cell within 4D of the surface, are discounted, 53% of the values obtained fall within the range $\delta = 10^{o} \pm 2^{o}$ and 72% within the range $\delta = 10^{o} \pm 3^{o}$. This result establishes that, for the conditions of these tests, an appreciable and essentially constant proportion of the total radial stress was in fact effective across the interface and offers an alternative interpretation of the "skin friction" acting on the pile as indeed the frictional shear component of a radial stress of the order of $5 \times c_u$ associated with an effective friction angle (δ) of about 10^{o}.

Although the measurement of the toe load on the pile was of secondary interest in the tests, since its magnitude is known not to be affected by electro-osmotic potentials (Johnston, 1972), it should be possible to estimate its value from the readings of axial cell 1 and local cells 1 and 2. The expected result was that if at any penetration (Z), the local shear force developed over the bottom 0.13m. of pile shaft was calculated from the local cell (1 and 2) shear stress readings and added to the conventional end load estimate $(9.c_u.A + \gamma.Z.A)$, where A is the area of the toe, then the result should give the reading of axial cell 1 corrected for the local radial stress cross-sensitivity via load cells 1 and 2. It should be added that a precisely similar procedure consistently and correctly produced zero end load for all extraction test measurements. However, the results for driving are shown in Fig. 10 where the difference between the curve (c) constructed as above and the corrected readings of axial cell 1 is both large and fairly constant with depth. If the well established values of $N_c = 9$, $N_q = 1$ are accepted as being applicable to a continuously penetrating pile in undrained conditions with $\phi = 0^o$, then there appear to be two possible explanations of the discrepancy. Either sufficient consolidation of the soil near the toe is occurring during driving to produce a mobilised

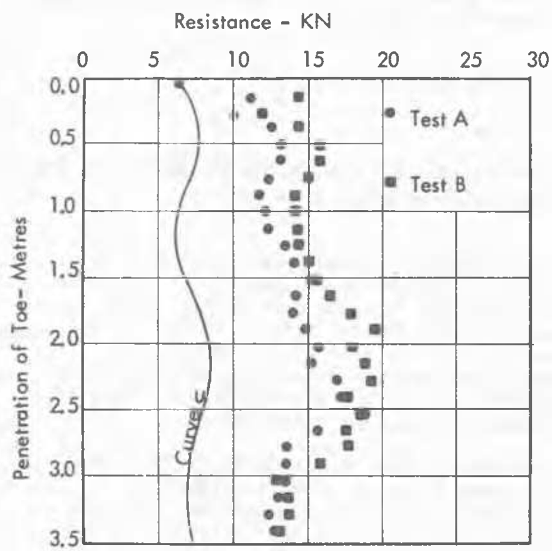

Fig 10 LOAD CARRIED BY PILE TOE - DRIVING

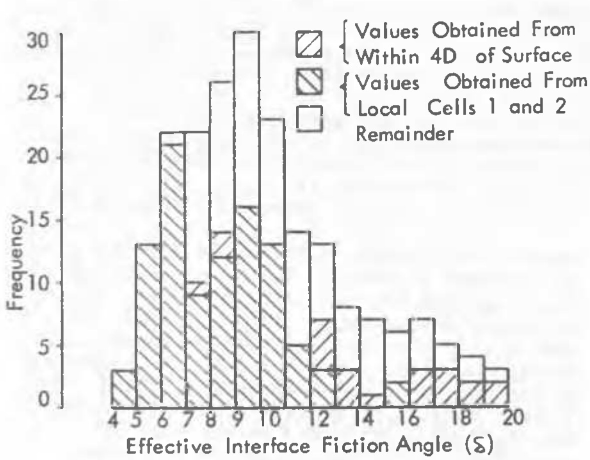

Fig 11 FREQUENCY DISTRIBUTION OF EFFECTIVE INTERFACE FRICTION ANGLE -DRIVING AND EXTRACTION

ϕ value of about 8^o or there is a very rapid increase in the effective α value on the pile shaft below axial cell 1. The former is unlikely since the penetration rate used of 0.35 mm/sec. on the 100 mm. diameter pile was comparatively quicker than that used by Whitaker and Cooke (1966) of 0.015 mm/sec. on 800 mm. diameter piles in London clay on which N_c values of around 9 were measured. Unless the total radial stress acting below local cells 1 and 2 reduces rapidly to virtually zero at the level of axial cell 1, then the second explanation requires a mean α value of about 3.5 over the bottom 0.13 m. of shaft. The evidence for this is unfortunately not conclusive although in principle high stresses could develop near the stress field singularity at the edge of the pile toe and measurements on the same pile system at very small penetrations (up to 0.5 m) show that high shear stresses do indeed develop there (Johnston, 1972).

CONCLUSIONS

Results of detailed local stress measurements made during continuous driving and extraction tests on a 100 mm. diameter x 4 m. long steel pile in London clay show that:

1. Radial total stress develops symmetrically on the pile-soil interface varying in magnitude from $4 \times c_u$ to $8 \times c_u$, the upper limit occurring near the pile toe, during driving and the lower near the pile head during extraction.

2. The successful measurement of local shear and radial stresses on the pile surface enables an effective interface friction angle to be measured. This varied between the extremes of 4^o and 20^o but 72% of the 200 values fell within the range of $10^o \pm 3^o$.

3. The α value applicable varied along the pile shaft, being generally higher on the upper 2 m. of the pile ranging from 0.7 to 1.2 during driving and from 0.6 to 0.8 during extraction. Some evidence was obtained supporting the existence of a zone, around one pile diameter in length from the pile toe, on which, during driving, the interface shear stresses reach about four times their average value.

4. There is remarkably good correlation between the local radial stresses developed at any point on the pile surface during driving and the c_u value of the adjacent soil, a result which may find some application in site investigation procedures.

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