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# THE DEVELOPMENT OF SHAFT RESISTANCE ON DISPLACEMENT PILES IN CLAY

# LA MOBILISATION DU FROTTEMENT LATERAL SUR LES PIEUX BATTUS DANS DE L'ARGILE

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SYNOPSIS: The Paper summarises the key findings from a recent research programme on pile behaviour in clay soils. An effective stress approach for the design of displacement piles in medium and high plasticity clays is proposed from this which synthesises the measurements of effective stresses acting on pile shafts obtained in the programme with other high quality data reported in the literature. This approach allows the effects on local shaft resistances of clay consistency and sensitivity, interface shearing resistance, progressive failure, and the relative depth of the pile tip to be accounted for within a rational effective stress framework.

## INTRODUCTION

The relatively poor reliability of current design methods for displacement piles (noted by Briaud and Tucker 1988 and others) has prompted organisations such as MIT, Oxford University, NGI and Imperial College to investigate the fundamental processes and soil properties that control pile behaviour. Because of the great shortage of field data and the complexities involved in developing sound theoretical approaches, work has concentrated on measuring the *effective* stresses developed on pile shafts during installation, equalisation and loading to failure. Such measurements have now been made in a wide range of clays, so greatly improving our understanding of shaft friction. Furthermore, data bases have been developed for the key effective stress parameters involved (Karlsrud and Nadim 1990, Azzouz et al 1990, Bond et al 1992, Lehane 1992).

This Paper reviews the principal findings from the field research performed by Imperial College, before presenting an interpretation of the full data base of available high quality measurements. This leads to the formulation of a general approach for evaluating the local shaft friction mobilised by closed-ended displacement piles in medium and high plasticity clays.

## PRINCIPAL FINDINGS OF IMPERIAL COLLEGE FIELD RESEARCH

The Imperial College (IC) field research has, to date, involved jacking closed-ended, 100mm diameter, heavily instrumented steel piles in four soils types: London Clay, medium dense Labenne Sand, stiff glacial Cowden Till and soft marine Bothkennar Clay. These piles measured the radial total stresses, pore pressures and shear stresses acting at various levels along the pile shafts. The instruments performed remarkably well and gave detailed information concerning the effective stress changes during installation, equalisation and load testing. Detailed accounts of the tests are given by Bond and Jardine(1991), Lehane and Jardine (1992a,b) and Lehane et al (1993).

### Installation

The installation process induced radial total stresses ($\sigma_{ri}$) on the pile shaft which depended (primarily) on the initial consistency of the soil and the strain paths imposed by jacking/driving cycles. These strain paths were such that, at each of the sites (including the Labenne Sand site), $\sigma_{ri}$ measured at fixed depths reduced as the pile penetrated to deeper levels.

The pore pressure changes induced during installation also depended on the soil consistency and the relative depth of the pile tip, but showed more complicated trends. This was partly because the shear induced component - associated with distortion - depended on the dilatant or contractant nature of the soil and was, in some cases, rate dependent. The radial effective stresses, calculated from the pore pressure and $\sigma_{ri}$ records were, in general, less than the undisturbed initial horizontal stress ($\sigma'_{h0}$) in the lightly overconsolidated Bothkennar Clay, but exceeded $\sigma'_{h0}$ in the heavily overconsolidated Cowden Till and London Clay.

### Equalisation

Radial total stresses generally reduced during equalisation; the greatest relative reductions took place in the sensitive Bothkennar Clay. In contrast, pore pressures measured at the pile shaft increased immediately after installation and developed their maxima shortly after installation. The subsequent dissipation process to ambient pressures ($u_0$) was controlled by spherical drainage near the pile tip and by radial drainage along most of the shaft length.

The increase in pore pressure shortly after installation gave rise to minima in radial effective stresses, $\sigma'_r$, (and pile capacity). $\sigma'_r$ increased subsequently in Cowden Till and London Clay to reach equalised values ($\sigma'_{rc}$) comparable to those measured during installation. $\sigma'_r$ increased much more rapidly after installation in Bothkennar Clay and attained $\sigma'_{rc}$ values that were over three times the installation $\sigma'_r$ values.

Equalised radial effective stresses ($\sigma'_{rc}$) showed a similar dependence on the soil consistency and on the distance above the pile tip (h) to the installation radial total stresses ($\sigma_{ri}$). This is evident on Figure 1 which shows the mean profiles of $\sigma'_{rc}$ established at Bothkennar and Cowden for two pile slenderness ratios (L/R). The higher stresses developed at any fixed depth on the shorter piles are a direct result of $\sigma'_{rc}$ reducing as 'h' increases.

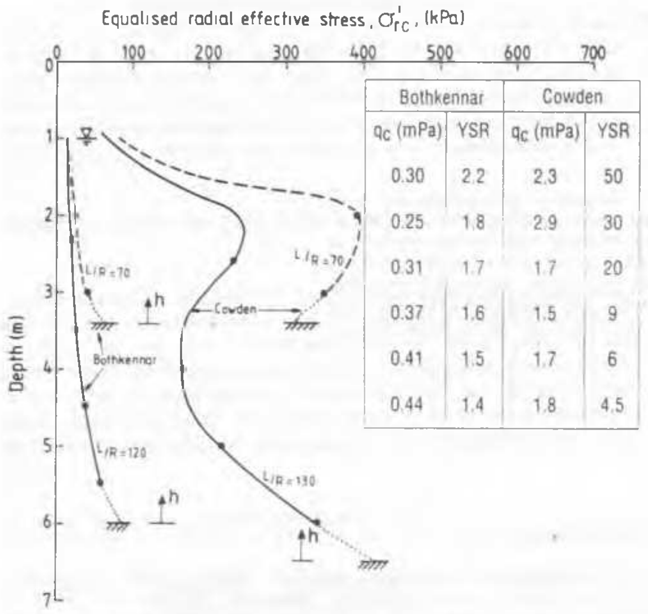


Figure 1 Profile of $\sigma'_{rc}$ in Cowden and Bothkennar clays

**Load testing**

In all cases, the peak local shear stress ($\tau_f$) was related to the radial effective stress acting at failure ($\sigma'_{rf}$) and the angle of friction at the pile-soil interface ($\delta_f$) through the Coulomb failure criterion:

$$\tau_f = \sigma'_{rf} \tan \delta_f \tag{1}$$

This equation may be re-written as:

$$\tau_f = f_L \, K_c \, \sigma'_{v0} \tan \delta_f \tag{2}$$

where $$K_c = \sigma'_{rc}/\sigma'_{v0} \tag{3}$$

$$f_L = \sigma'_{rf}/\sigma'_{rc} \tag{4}$$

$f_L$ (termed the load test coefficient) was typically 0.85 ± 0.15 in first-time load tests performed after full equalisation. The values of $\delta_f$ were far more sensitive to the soil and pile types, varying from ≈13° in London Clay to ≈28° in Bothkennar Clay. $\delta_f$ could, however, be assessed accurately from ring shear interface experiments which modelled (i) the displacement and rate history of soil elements adjacent to a shaft during installation, (ii) the surface properties of the piles, and (iii) the normal effective stress level.

**SYNTHESIS OF IC MEASUREMENTS WITH OTHER FIELD DATA**

**The data base**

Table 1 presents an inventory of soils in which *high quality* measurements of the effective stresses acting on displacement piles have been made. The list includes the average plasticity indices (PI) and 1-D yield stress ratios (YSR) of each material as well as the pile diameters (D) and maximum pile lengths ($L_{max}$) considered. The appropriate references for these case histories are provided by Bond et al (1992) and Lehane (1992).

**Table 1** Typical average parameters at test sites

| Clay | PI (%) | YSR | $L_{max}$ | D (mm) | Symbol used in figures |
|---|---|---|---|---|---|
| Cowden | 20 | 10 | 6.4 | 102 | ■ |
| Bothkennar | 40 | 1.7 | 6.0 | 102 | ● |
| Labenne | 0 | ≈1.5 | 6.0 | 102 | (sand) |
| London clay | 45 | 30 | 6.0 | 102 | o |
| Boston Blue | 22 | 1.5 | 40 | 38 | + |
| Empire | 52 | 1.6 | 75 | 38 | ▵ |
| Gault | 48 | 20 | 9 | 80 | ▿ |
| Huntspill | 35 | 1.6 | 9 | 80 | □ |
| Tilbrook | 25 | 10 | 25 | 219 | ▾ |
| Haga | 15 | 5 | 5.2 | 153 | ◊ |
| Onsφy | 40 | 1.3 | 35 | 219 | x |
| Pentre | 15 | 1.6 | 30 | 219 | (permeable silt) |
| Lierstranda | 15 | 1.1 | 35 | 219 | (permeable silt) |
| Tokyo | 55 | 6 | 5.6 | 300 | ♦ |
| Rio de Jan. | 60 | 1.8 | 6.7 | 220 | ◐ |

For the reasons outlined below, the tests performed in the very low plasticity, permeable silts at Lierstranda and Pentre have been omitted from the correlations developed.

**Installation**

The most reliable and abundant sets of records in the data base are those of the radial total stresses measured during jacked pile installation ($\sigma_{ri}$). A detailed examination of these data revealed that, as in the IC tests, $\sigma_{ri}$ - at any fixed depth - depended on the soil consistency (as expressed, for example, by the undrained strength, the CPT $q_c$ value or the yield stress ratio) and reduced with the distance above the pile tip (h).

Statistical analyses indicated that the most consistent trends emerged when $\sigma_{ri}$ was related to the 1-D yield stress ratio (YSR) and the distance from the tip of each pile (h) normalised by its pile radius (R) i.e. h/R (this normalisation is in keeping with the trends predicted by the Strain Path Method, Baligh 1985). The best fit relationship obtained was:

$$H_i = (\sigma_{ri} - u_0)/\sigma'_{v0} = 3.7 \; YSR^{0.42} \; (h/R)^{-0.2} \tag{5}$$

The $H_i$ values predicted by this expression (at h/R= 30) are compared with the data base of field $H_i$ measurements on Figure 2. Only $H_i$ ratios recorded at h/R values greater than 20 (where h/R effects become less important) are shown. It is apparent that most of these fall within 20% of the predicted trend. All are lower that the Cavity Expansion Method prediction (Wroth et al 1979), but higher than the undisturbed in situ lateral stress ratio, $K_0$.

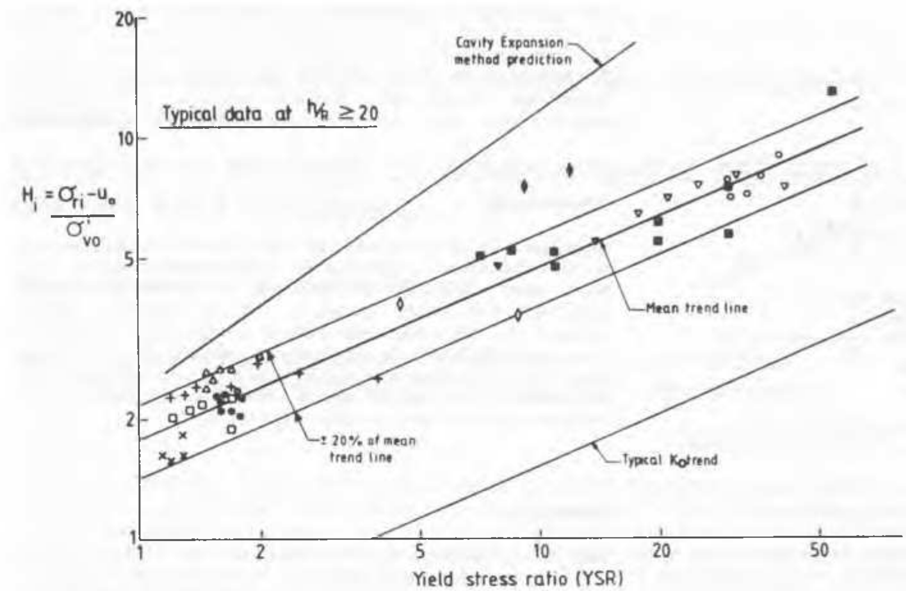


**Figure 2** Data base of $H_i$ measurements at h/R >20

Part of the scatter on Figure 2 occurs because the reduction of $H_i$ with h/R is not unique and depends both on the soil type and installation method (Lehane 1992). Partial drainage during installation also contributes to the spread of data. This is demonstrated on Figure 3 which compares the variations of $H_i$ with h/R for the lightly overconsolidated clays included in the data base with those measured in the medium dense Labenne sand and low plasticity Pentre silt: $H_i$ is evidently far smaller in the loose sands and silts.

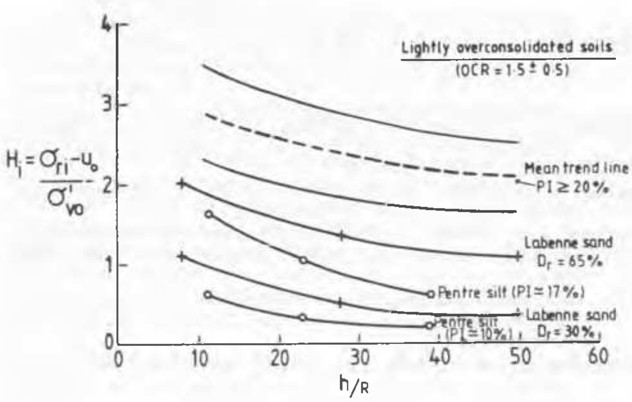


**Figure 3** Variation of $H_i$ with h/R in lightly overconsolidated soils

**Equalisation**

The stresses in the ground surrounding a pile undergo complex changes after installation, with the radial total stresses declining as pore pressures equalise. Field measurements indicate that the relative reduction in radial total stress, as defined below, increases with clay sensitivity ($S_t$) and reduces with the clay's YSR:

$$(\sigma_{rc} - u_0)/\ (\sigma_{ri} - u_0) = K_c/H_i \ = f\ (\ S_t\ ,\ YSR) \qquad (6)$$

For sensitive soils, $S_t$ and YSR can be expressed approximately in terms of the relative void index ($I_{vr}$) defined as:

$$I_{vr} = (e - e_{ICL})/C_c^*, \qquad (7)$$

where e is the in-situ void ratio, $e_{ICL}$ is the void ratio at the same vertical effective stress on the 1-D virgin consolidation line of reconstituted material (or ICL) and $C_c^*$ is the slope of the ICL. In the absence of measured data, $e_{ICL}$ and $C_c^*$ may be estimated using the trends proposed by Burland (1990).

Following from (6) and (7), it was considered that $K_c/H_i$ may be related empirically to $I_{vr}$. This relationship is explored in Figure 4 which compares the average $K_c/H_i$ ratios taken from the data base with the appropriate $I_{vr}$ values. A clear trend for $K_c/H_i$ to reduce with $I_{vr}$ is seen (which is independent of h/R) and the following best fit line predicts the measured $K_c/H_i$ values to within 20%.

$$K_c/H_i = 0.53 - 0.17\ I_{vr} \qquad (8)$$

Other factors such as stiffness and effective stress strength parameters may affect the magnitude of $K_c/H_i$, but the current sparsity of field measurements precludes refinement of this expression.

**Radial effective stresses after equalisation**

Combining (5) and (8), the radial effective stresses acting on a displacement pile after full equalisation may be expressed as:

$$K_c = \sigma'_{rc}/\sigma'_{v0} = f\ (YSR,\ h/R,\ S_t)$$
$$= YSR^{0.42}\ [2 - 0.63\ I_{vr}]\ (h/R)^{-0.2} \qquad (9)$$

This relationship predicts over 80% of the $K_c$ values in the data base to within 20%. As for the $H_i$ data, one of the primary factors causing the deviation between predictions and measurements is the variable dependence of $K_c$ on h/R. For example, the best fit exponents of h/R for the data measured in the IC tests in Bothkennar Clay and Cowden Till were -0.25 and -0.35 respectively.

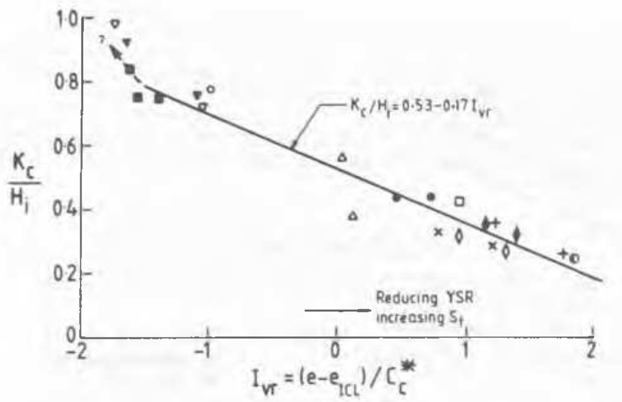


Figure 4 Dependence of $K_c/H_i$ on $I_{vr}$ (for all h/R ratios)

Important implications of expression (9) include:

- $K_c$ increases significantly with the yield stress ratio; $K_c$ in lightly overconsolidated clays (YSR ≤2) is likely to be about three times less than in heavily overconsolidated clays (YSR ≥20).
- The most pronounced reductions of $K_c$ with h/R occur close to the pile tip (h/R < 20).
- The $K_c$ values for a very sensitive, low YSR clay (with $I_{vr} \approx +1.5$) are expected to be half those developed in a comparable, but insensitive clay (with $I_{vr} = 0$).
- At locations away from the pile tip (h/R <5), the Cavity Expansion Method is likely to over-predict $K_c$ values greatly.

Lehane (1992) showed that expression (9) is consistent with trends predicted by Strain Path Method calculations performed with the MIT-E3 soil model (Baligh 1985, Whittle et al 1988). This analytical approach is the most promising proposed to date, but (in its current formulation) tends to under-estimate the reduction of $K_c$ with h/R and, in certain instances, the average $K_c$ value.

### Local shaft friction

The load test coefficient ($f_L$) recorded in IC tests and in the few other case histories which obtained reliable $f_L$ measurements fell in the range 0.75 to 1.0. Combining (2) and (9) and assigning $f_L$ = 0.8, the following expression for the local shaft friction ($\tau_f$) that can be mobilised after full equalisation is obtained:

$$\tau_f = \sigma'_{v0}\ YSR^{0.42}\ (h/R)^{-0.2}\ [1.6 - 0.5\ I_{vr}]\ \tan\delta_f \qquad (10)$$

Field and laboratory studies have shown that the angle $\delta_f$ is usually lower than the constant volume soil-soil friction angle ($\phi'_{cv}$), often as a result of soil fabric re-orientation close to the pile shaft. The re-orientation process can be strongly rate dependent and lead to brittle $\delta$ characteristics in (slow) tests performed on fast-jacked piles; see Bond and Jardine (1991). Consequently, the values of $\delta_f$ selected for various levels/soil types along the shaft must allow for possible post-peak degradation of $\delta$. The degree of degradation may be assessed from appropriate ring shear interface tests.

Lehane (1992) has shown that equation (10) is compatible with the general trends of established data bases for large un-instrumented driven piles; these trends have been used to establish correlations for the $\alpha$ and $\beta$ design methods. However (10) represents a significant improvement on these methods as it provides a design approach which is based on the local variations of effective stresses along the pile shaft. In addition to soil consistency (expressed as YSR and $\sigma'_{v0}$), the approach incorporates three other factors which the instrumented pile test data have shown to have a critical affect on shaft capacity: (i) the interface frictional characteristics, (ii) the clay's sensitivity, and (iii) the relative depth of the pile tip.

### CONCLUSIONS

High quality measurements of the effective stresses acting on the shafts of closed-ended displacement piles in clays have been used to develop an effective stress design approach for shaft capacity. This approach takes due account of the changing stress conditions in the pile's life and allows the principal parameters that control capacity to be dealt with in a simple and rational manner. Refinements and improvements should be possible as further reliable field data become available.

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