



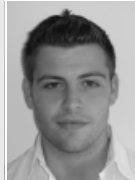
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Field investigation of base resistance of pipe piles in clay

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This paper presents the results of a series of field tests performed to study the effect of soil plugging during installation on the base resistance developed by an open-ended (pipe) pile in clay. Three instrumented pipe piles were installed in soft clay, and the installation resistance and soil plug development were recorded. The tests revealed that the annular base resistance was equal to the cone penetration test q_c resistance and was independent of the soil plug development. In contrast, the soil plug resistance increased from a minimum when the pile was fully coring to a maximum when the pile was fully plugged. A simple expression is proposed that links the plug resistance to the cone penetration test q_c value at the specified depth and the incremental filling ratio value developed during installation. This expression is shown to provide reasonable estimates of the plug resistance developed by a large-diameter pipe pile driven into stiff to hard clay in the North Sea. The proposed relationship will be particularly useful for modelling the installation resistance of monopile foundations.

NOTATION

A_{ann}	annular area
A_b	total base area
C_h	horizontal coefficient of consolidation
C_v	coefficient of consolidation
D	pile diameter
D_{CPT}	cone diameter
D_i	internal pile diameter
h	distance from pile tip
k	permeability constant
k_c	cone reduction factor
L	embedded pile length
L/D	slenderness ratio
L_p	plug length
N_c	bearing capacity factor
N_{kt}	cone factor for undrained shear strength
p_{atm}	atmospheric pressure
Q_b	base load
Q_{plug}	plug load
Q_s	shaft load
Q_T	total pile resistance
q_{ann}	annular end bearing pressure
q_b	pile end bearing pressure
q_{bu}	average ultimate unit end bearing resistance
q_c	cone tip resistance

q_{cnet}	net cone tip resistance
q_{plug}	unit plug resistance; normal plug stress
q_s	closed-ended shaft stress
q_{sav}	average shaft resistance
q_{se}	external open-ended shaft stress
q_{si}	internal open-ended shaft stress
s	displacement
s_u	undrained shear strength averaged over pile shaft
s_{ub}	undrained shear strength acting on pile base
α	empirical constant linking q_s and undrained shear strength
σ'_{v0}	in situ vertical effective stress

1. INTRODUCTION

Open-ended pipe piles are commonly employed in offshore environments where large pile penetrations are necessary to resist uplift. The compressive axial load resistance developed by a pile is derived from a combination of external unit shaft resistance q_{se} and unit base resistance q_b (see Figure 1). For pipe piles the base resistance is composed of two parts: the annular resistance q_{ann} developed beneath the wall of the pile, and the internal shaft resistance q_{si} . The integral of the internal shaft resistance divided by the area beneath the soil plug is termed the unit plug resistance, q_{plug} . The development of the soil plug during the installation of a pipe pile is described by the incremental filling ratio (IFR), given by

$$\text{IFR} = \frac{\Delta L_p}{\Delta L}$$

where ΔL_p is the change in the length of the soil plug over a given increment of pile penetration ΔL . During the initial stages of driving the soil, plug length is coincident with the penetration depth, and the pile is said to be fully coring (IFR = 1). If, at some stage during installation, the internal shear stresses generate sufficient resistance to prevent further soil intrusion, the pile is said to be fully plugged (IFR = 0). Most pipe piles are driven in an intermediate condition: that is, the incremental rate of soil intrusion is somewhat smaller than the pile penetration, and the pile is said to be partially coring ($0 < \text{IFR} < 1$).

Most recent research on open-ended piles in clay (e.g. Jardine *et al.*, 2005; Miller and Lutenecker, 1997; Xu *et al.*, 2006) has concentrated on the development of shaft capacity, with the result that design methods for evaluating the base resistance of open-ended piles in clay have undergone limited development.

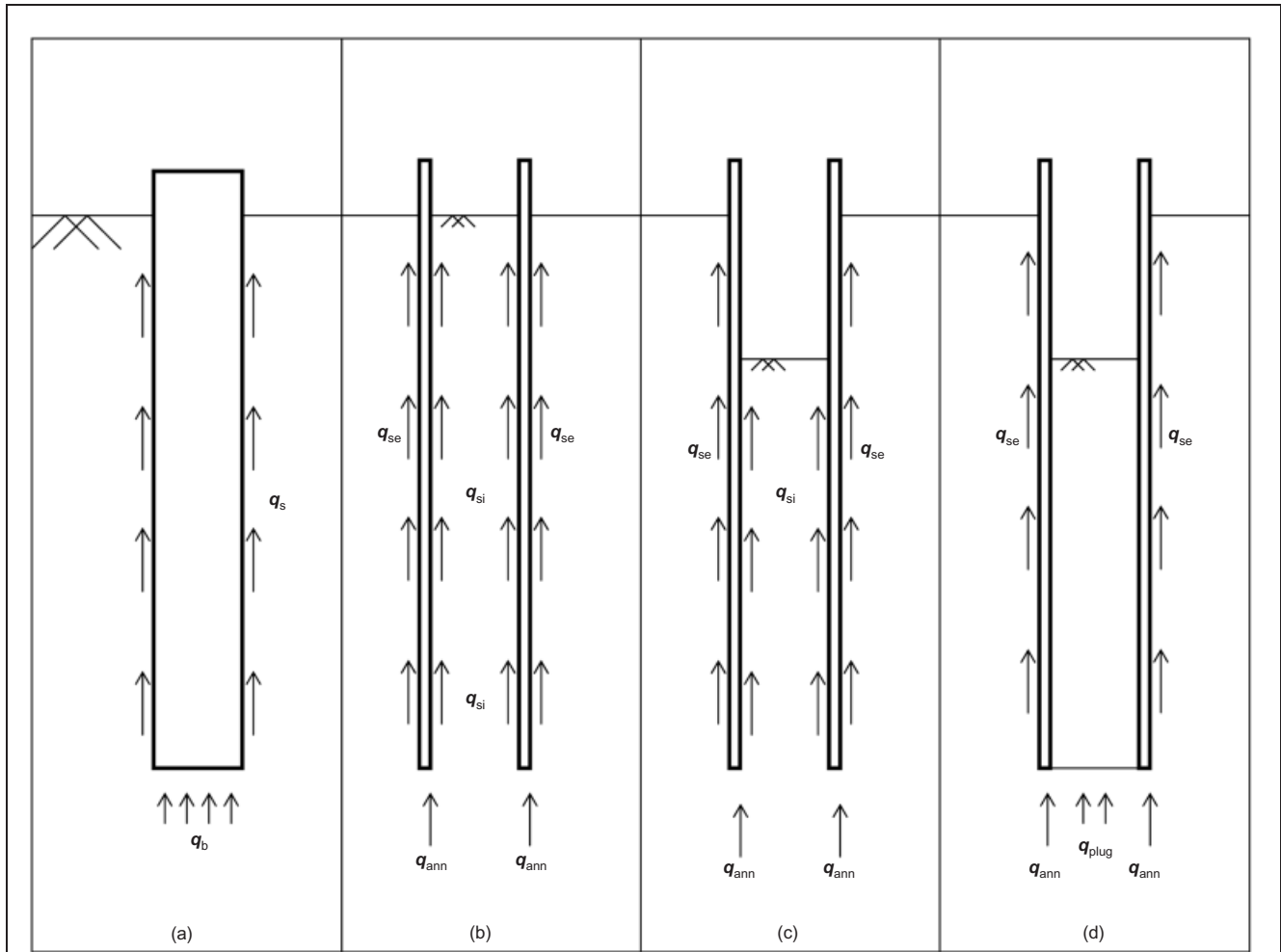


Figure 1. Modes of penetration for open-ended and closed-ended piles: (a) closed-ended; (b) fully coring; (c) partial plugging; (d) fully plugged

This is in stark contrast to open-ended pile design in sand, where concentrated research efforts (Chow, 1997; Lehane and Gavin, 2001; Paik and Salgado, 2003) have identified the critical role of soil plug development during installation on the ultimate base resistance, and have led to improvements in predictive design approaches, such as ICP-05 (Jardine *et al.*, 2005) and UWA-05 (Lehane *et al.*, 2005). The historical lack of research initiatives stems from the general opinion that, for typical pile geometries, the end bearing resistance contributes a relatively minor proportion of the ultimate axial resistance of a pile installed in clay (Jardine *et al.*, 2005). Figure 2 uses traditional total stress design to show the impact of slenderness ratio (length/diameter) on the proportion of total resistance developed by the pile base. It is clear that for a typical closed-ended pile geometry (embedded length $L = 10$ m and diameter $D = 0.3$ m), with a slenderness ratio (L/D) of 30, less than 15% of the total pile resistance Q_T is developed by the base load Q_b . However, for large-diameter monopiles, typical L/D ratios are likely to be significantly less than 30, and up to 45% of the resistance may be developed at the pile base. Furthermore, traditional design may also significantly underestimate the base component, as demonstrated by instrumented field tests in stiff boulder clay (Farrell *et al.*, 1998), where a pile with a slenderness ratio of 24 developed a base load that was 61% of the total undrained resistance.

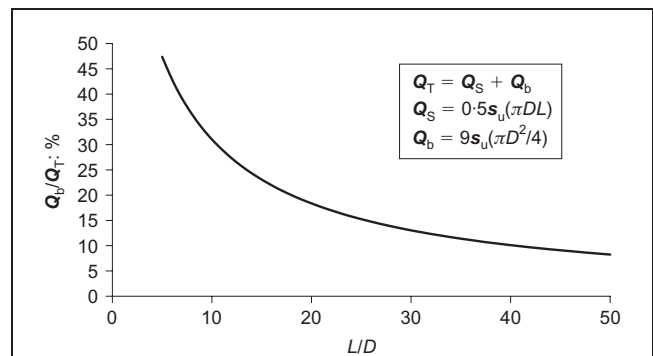


Figure 2. Effect of slenderness ratio on the proportion of base to ultimate pile resistance

Using instrumented field model testing, this paper considers the mechanisms controlling the development of the base resistance of open-ended piles in clay. A simple empirical design approach is proposed for estimating the ultimate base resistance of pipe piles. This method is subsequently tested against a case history involving load tests on full-scale open-ended piles installed in the North Sea.

2. BACKGROUND

The American Petroleum Institute (API RP 2A) method (API, 1993), which is a derivation of the traditional total stress approach, is widely used for the design of offshore piles. The method considers two plug conditions, treating the pile as being either fully plugged (IFR = 0) or unplugged (IFR = 1). The plug condition is determined by assuming that q_{se} and q_{si} are equal. The plug capacity Q_{plug} and normal plug stress q_{plug} are given by

$$2a \quad Q_{plug} = \int_0^{L_p} q_{si} \pi D_i$$

$$2b \quad q_{plug} = \frac{Q_{plug}}{\pi D_i^2}$$

where D_i is the internal diameter of the pile, $q_{si} = q_{se} = \alpha s_u$, α is an empirical reduction factor, and s_u is the average shear strength along the pile shaft. Recommended α values reduce from 1.0 for undrained strength ratios $s_u/\sigma'_{v0} < 0.25$ to a minimum of 0.4 for realistically high strength ratios (Pelletier *et al.*, 1993). The normal plug stress is compared with the average ultimate unit end bearing resistance q_{bu} determined from the bearing capacity equation

$$3 \quad q_{bu} = N_c s_{ub}$$

where N_c , the bearing capacity factor, is 9, and s_{ub} is the undrained strength at the pile base. If the plug stress calculated using Equation 2 exceeds the ultimate bearing capacity of the soil determined using Equation 3 the pile is deemed to be plugged, and the base load is calculated over the entire base area A_b , as

$$4 \quad Q_b = N_c s_{ub} A_b$$

In contrast, if the base resistance exceeds the plug resistance, the pile is assumed to be fully coring and the base resistance is assumed to develop only over the annular area of the pile, A_{ann}

$$5 \quad Q_b = N_c s_{ub} A_{ann}$$

Recent assessment of the predictive capacity of the API approach suggests large scatter in the ratio of predicted to measured base capacity, with N_c factors for closed-ended piles measured as high as 17 for load tests by Farrell *et al.* (1998). Chow (1997) suggests that much of the scatter may result from uncertainty with respect to the design s_u value; however, overall conservatism is apparent throughout the database, with $N_c = 9$ forming an approximate lower bound to the closed ended data. There is also a dearth of information for open-ended piles, which may lead to further uncertainty when applying N_c values to pipe piles with various degrees of plugging. Consideration of the inequality between Equations 2 and 3 suggests that in soil deposits where the undrained strength profile is uniform with depth ($s_{ub} \approx s_u$), and adopting a typical α value of 0.5, results in the plug resistance

exceeding the base resistance when the soil plug length exceeds $6D_i$. On this basis most field-scale piles will be assumed to be fully plugged.

Because of the uncertainties associated with determining a reliable s_u profile in some deposits, direct correlations with in situ tests, such as the cone penetration test (CPT) end resistance q_c , have been suggested by several researchers. Bustamante and Gianselli (1982) proposed the LCPC method: a direct relationship between q_c , averaged $1.5D$ above and below the pile tip, and q_b , given by

$$6 \quad q_b = k_c q_c$$

The cone reduction factor k_c varies, depending on clay strength. No distinction is made between the end bearing resistance of open-ended or closed-ended piles.

Jardine *et al.* (2005) proposed an alternative CPT design method known as IC-05. Plugging during static loading is assumed to occur if

$$7 \quad \frac{D_i}{D_{CPT}} + 0.45 \frac{q_c}{p_{atm}} < 36$$

where D_{CPT} is the diameter of the CPT cone (0.036 m) and p_{atm} is 100 kPa.

The base load is given by Equation 8a for plugged piles and Equation 8b for unplugged piles, where $k_c = 0.45$ or 0.65 for undrained or drained loading of plugged piles, and 1.0 or 1.6 for undrained or drained loading of unplugged piles respectively.

$$8a \quad Q_b = k_c q_c A_b$$

$$8b \quad Q_b = k_c q_c A_{ann}$$

It is clear from the current literature that although the effects of plug development on the base resistance developed by pipe piles in clay are incorporated into some design methods, these tend to consider only the extreme case of fully plugged or unplugged piles. The techniques used to assess the likelihood of piles becoming fully plugged are highly empirical, and there is a dearth of measurements of plug response with which to calibrate these methods. With this in mind, a series of instrumented open-ended pile tests were performed to investigate the mechanisms of plug development and quantify the base stress changes that occurred during soil plugging.

3. SOIL CONDITIONS

The pile tests considered in this paper were conducted at the Kinnegar geotechnical research site on the shores of Belfast Lough. Soil deposits at the site comprise a layer of fill typically 1 m thick, consisting mainly of building rubble, over a natural soft sandy silt deposit of variable thickness (approximately 0.7–2 m thick), which in turn overlies 6 m of soft clayey silt. The soft silt deposits are locally termed sleet, and are the

strata of primary concern for the pile tests considered. The soft deposits contain sandy silt overlying clayey silt, and are referred to as the upper and lower sleetch respectively. The sleetch is underlain by a uniform medium dense sand. The site is tidal, with a water table fluctuating between 0.8 and 1.3 m below ground level.

The geotechnical properties of the sleetch have been the subject of extensive laboratory and field investigations described by McCabe (2002), Lehane *et al.* (2003) and Gallagher (2006). The upper sandy sleetch, located between 1 m and 3 m below ground level (bgl), has proportions of sand to clay of 20% and 10% respectively; below this level the proportions reverse, with a maximum clay fraction of 38%. The natural moisture content of the lower sleetch is $60 \pm 10\%$, with a liquid limit and plasticity index of $65 \pm 10\%$ and $35 \pm 5\%$ respectively. The material plots above the A-line in the Casagrande chart, identifying it as an intermediate- to high-plasticity clay. The low permeability ($k = 1.5 \times 10^{-10}$ to 5×10^{-10} m/s), vertical coefficients of consolidation C_v in the range $0.5\text{--}3$ m²/year (at an effective stress of 100 kPa) and coefficient of consolidation in the horizontal direction, C_h , that ranged from 7 m²/year to 12 m²/year, are more typical of clay than of silt, and indicate the dominant nature of the clay size fraction in influencing the engineering behaviour. The sleetch is lightly overconsolidated, with an overconsolidation ratio (OCR) decreasing with depth from 1.6 at 3 m to 1 at 8 m.

Cone penetration tests (CPT) and piezocone tests (CPTu) were performed at the site, with a typical trace shown in Figure 3(a). The CPT profiles were noticeably variable near ground level, but became much more uniform when the cone entered the lower sleetch, where the q_{cnet} value is seen to increase gradually from 175 kPa at ~ 2.3 m bgl to 240 kPa at 6 m bgl. The undrained strength (s_u) profile measured in vane tests is compared in Figure 3(b) with values back-figured from the q_{cnet} profile (assuming $N_{kt} = 12.5$), and those measured in triaxial compression tests.

4. INSTRUMENTATION AND EXPERIMENTAL PROCEDURE

Several instrumented pile tests have been performed by University College Dublin (UCD) researchers at the Kinnegar test site between 2003 and 2008. The piles were developed specifically to investigate the influence of penetration mode on pile capacity in clay, and therefore adopted two different end conditions: a closed-ended (CE) and an open-ended (OE) geometry. In total, seven pile installations were conducted, the locations of which are illustrated in Figure 4. The primary focus of this paper is on the base resistance developed by open-ended piles, but the results of some closed-ended pile tests are used for comparative reference.

The UCD open-ended model pile (UCD-OE) comprises a heavily instrumented section 2 m long, capable of penetrations up to a maximum depth of 6 m through the addition of extension pieces of equal external diameter. A twin-wall construction was adopted for the heavily instrumented lower section to allow the resistance developed by the pile annulus, shaft and plug to be determined independently of each other. This involved joining two steel tubes: a larger outer tube with an external diameter of 168 mm and wall thickness of 3.34 mm,

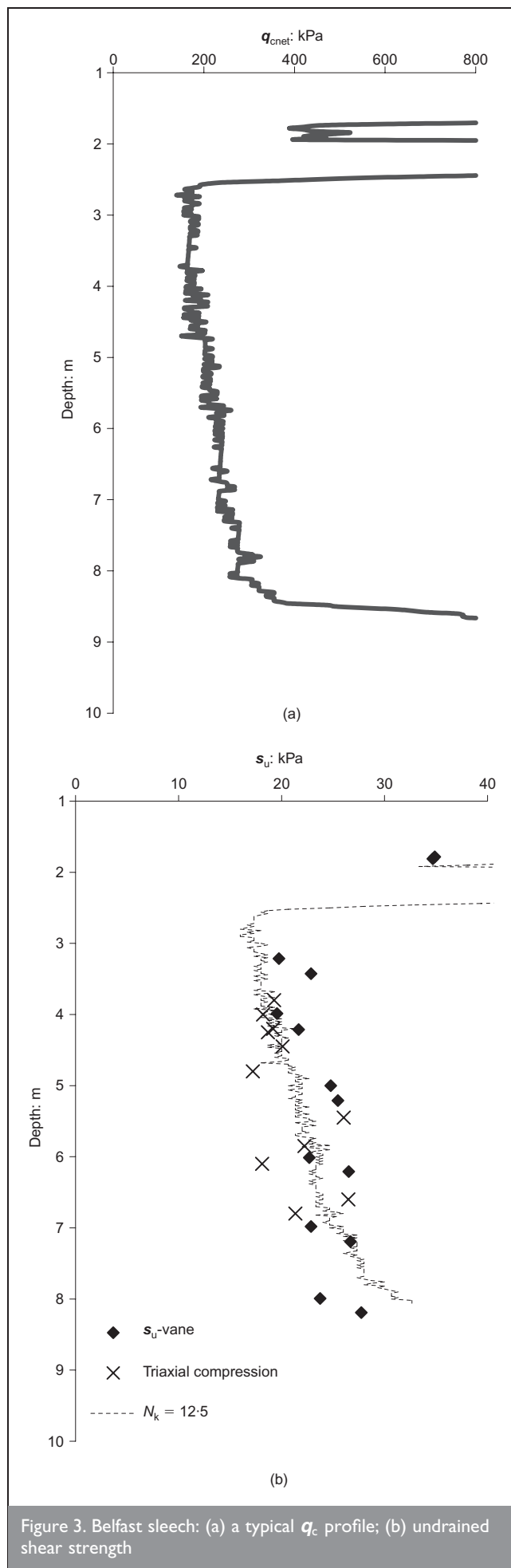


Figure 3. Belfast sleetch: (a) a typical q_c profile; (b) undrained shear strength

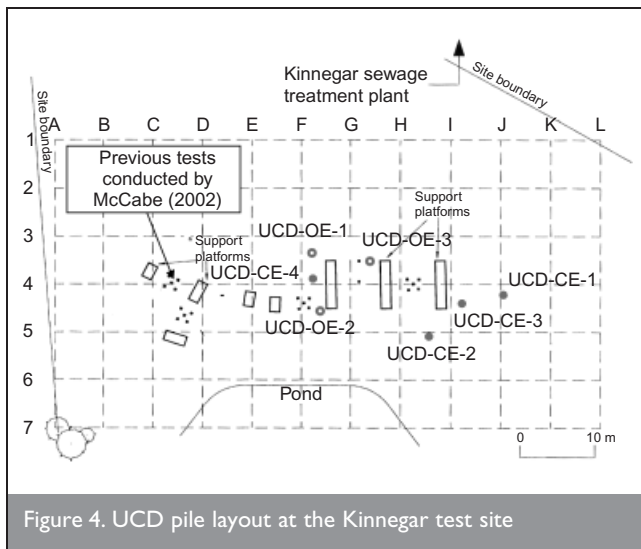


Figure 4. UCD pile layout at the Kinnegar test site

and an inner tube with an external diameter of 154 mm and a wall thickness of 2 mm. The void space between the tubes that accommodated the instrumentation was 3.66 mm: thus the pile has a total wall thickness of 9 mm and a D/t ratio of 18.7. The instrumentation layout is shown schematically in Figure 5. Kyowa PS-5KA sub-miniature pressure transducers were used to measure the radial stress and pore pressure at multiple locations. They were positioned diametrically opposite each other at three levels along the pile shaft, at a distance h from the pile tip normalised by the pile diameter D of 1.5, 5.5 and 10.5. A pair of Kyowa PS-5KB total stress and pore pressure sensors were used in the annulus. TML brand PFL-10-11, 120 Ω uniaxial foil strain gauges were used to measure the axial load distribution in each tube. Additional information on

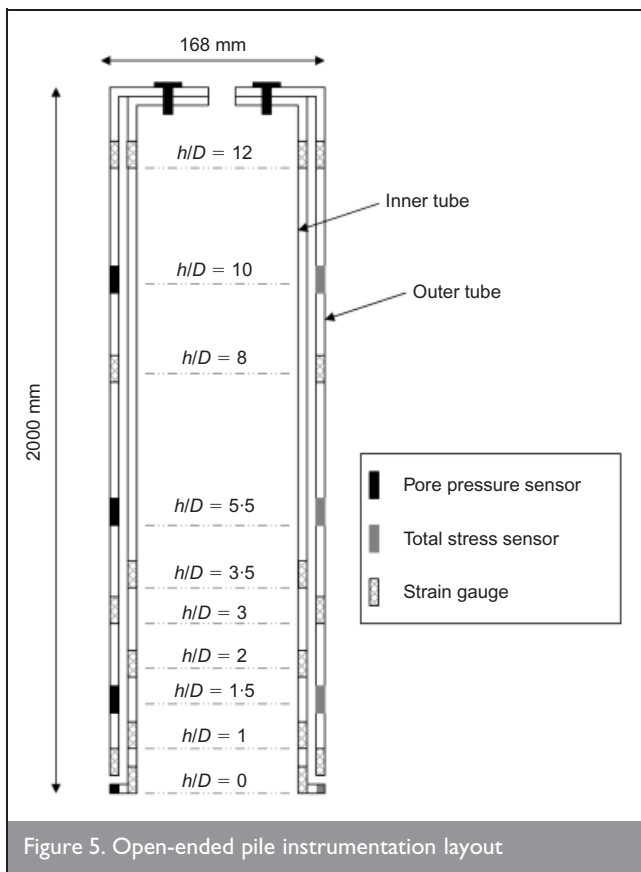


Figure 5. Open-ended pile instrumentation layout

the instrumentation layout, construction details and design requirements are provided by Gallagher (2006).

The UCD instrumented closed-ended stainless steel tubular model pile consists of an instrumented stainless steel tube of 73 mm external diameter and 1.7 m long, capable of penetrations up to a maximum depth of 7 m through the addition of extension pieces of equal external diameter. The 1.7 m lower section adopted an instrumentation layout that included total stress and pore pressure sensors at three separate locations along the pile shaft, at h/D of 1.5, 5.5 and 10, mimicking the OE pile. Base stress measurements were performed using a pressure sensor mounted in the pile tip plate.

Three separate installations of the UCD OE pile (termed UCD-OE-1 to UCD-OE-3) were performed, and four installations of the UCD CE instrumented model pile were conducted (termed UCD-CE-1 to UCD-CE-4). A schematic of all the piles, indicating their final penetration depth and instrument locations, is shown in Figure 6.

The piles were installed using a hydraulic jack, from the base of pre-formed starter holes to their final depths in strokes of 100 mm length for UCD-OE-1 and UCD-OE-2; the jacking stroke length was increased to 250 mm for UCD-OE-3. The vertical reaction was provided by a specially fabricated steel frame anchored to a concrete kentledge founded on deep piles bearing in the underlying dense sand. The pile head accommodated a hollow load cell between the hydraulic jack and the driving cap, providing a continuous load profile during installation. A schematic illustration of the test set-up and load cell detail is presented in Figure 7. The load cell, strain gauges and pressure sensor data were logged at 0.1 s intervals throughout installation, using a System 5000 datalogger.

5. EXPERIMENTAL RESULTS

5.1. Plug development

The soil plug length (as reflected by the IFR value) measured during installation of the three pipe piles is shown in Figure 8. The plug development of all three piles was similar, reflecting the consistent installation procedure used. Paikowsky and Whitman (1990) conducted a comprehensive study of plug development, and noted that the plugging behaviour of driven piles was difficult to predict. However, it was observed that during quasi-static penetration, which occurs during free run (at the start of installation, when the pile penetrates under self-weight), piles became fully plugged within a penetration of 10–20 pile diameters in a range of soil types from soft to stiff clay. The UCD piles that were jacked into place remained fully coring (IFR = 1.0) until L/D_i ratios in the range 7–10. Thereafter, the piles penetrated in a partially plugged mode until the normalised penetration exceeded 16–18, and they became fully plugged (IFR = 0). Pile UCD-OE-1, which had a final penetration depth z of 4.04 m, remained only partially plugged throughout installation, with a final IFR value of 0.44.

5.2. Shaft resistance

Although this paper primarily addresses the effects of plugging on the base resistance developed by pipe piles in clay, it is instructive to compare the external and internal shaft

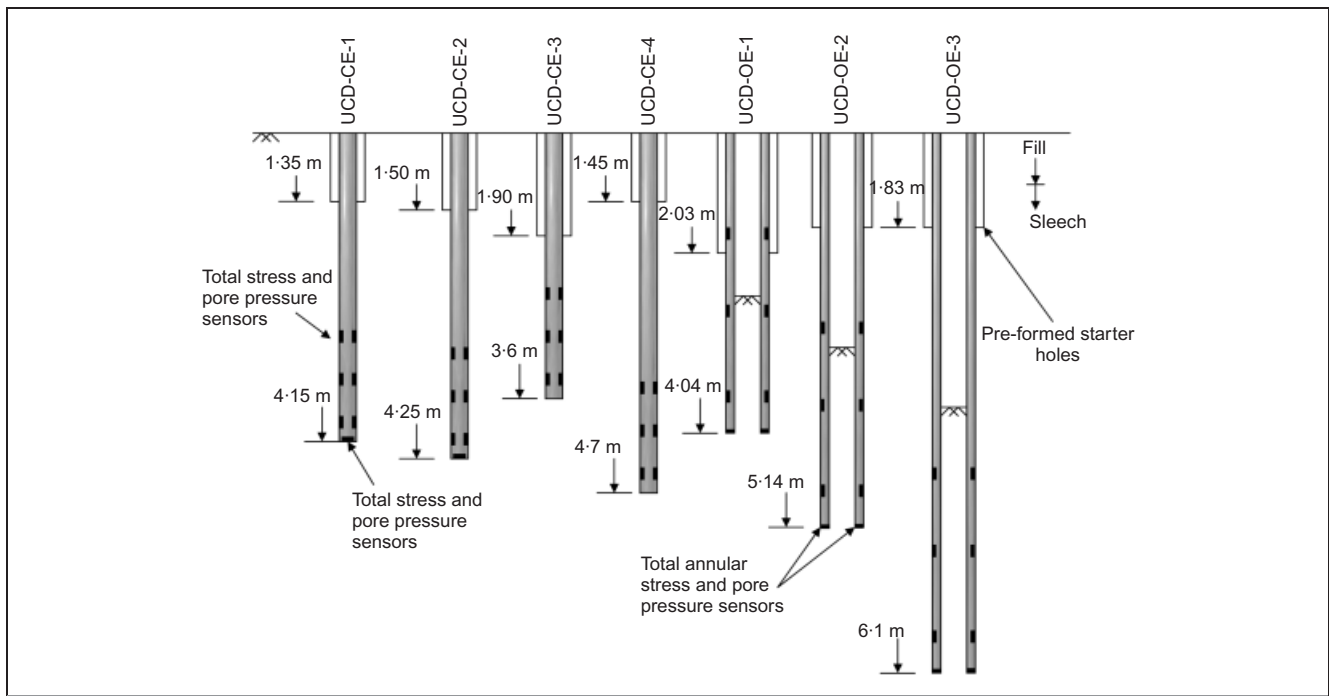


Figure 6. Relative pile lengths for UCD tests

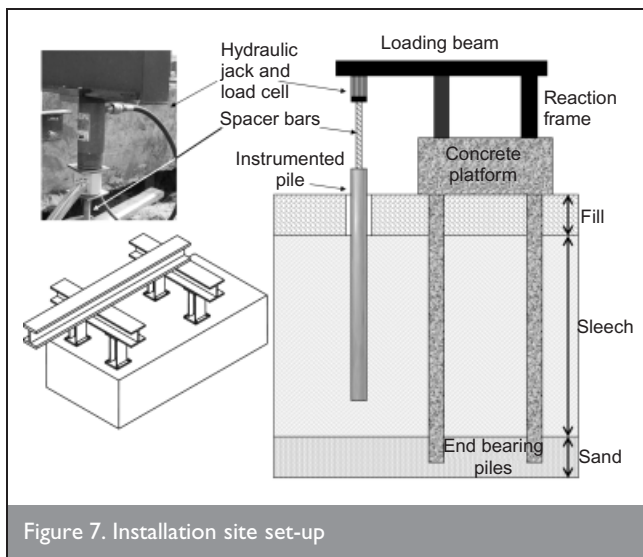


Figure 7. Installation site set-up

resistance developed by pipe piles with that mobilised by closed-ended piles installed in the same deposit. The average shaft resistance q_{sav} (= total shaft load/area of shaft) developed on the external shaft area of the open (q_{se}) and closed-ended piles (q_s), and the internal shaft (q_{si}) area of the open-ended pile, are compared in Figure 9. The initial variability over the first metre of penetration may in part be related to the site variability and the transition from the upper to the lower sleech. Despite the scatter, the data suggest that the closed-ended piles form an approximate upper bound to the shaft stresses, and q_s reduces slightly with depth or with increasing pile length. The external shear stresses on the open-ended pile are slightly lower than those on the closed-ended piles for depths to 4 m bgl, although other factors, such as rate of installation and the occurrence of pause periods during installation, may also affect these results, as discussed by Gavin *et al.* (2010). The average internal shear stresses provide a lower bound to the data over the initial 2 m of penetration

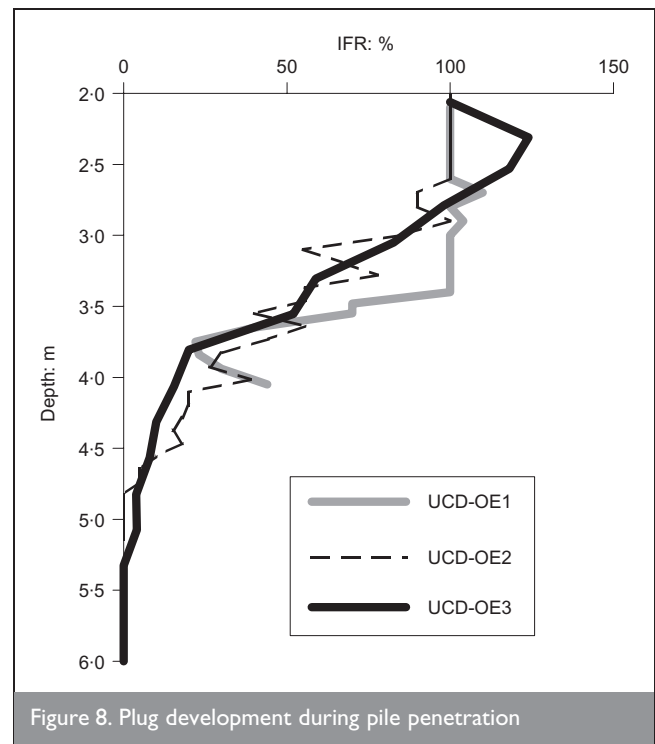
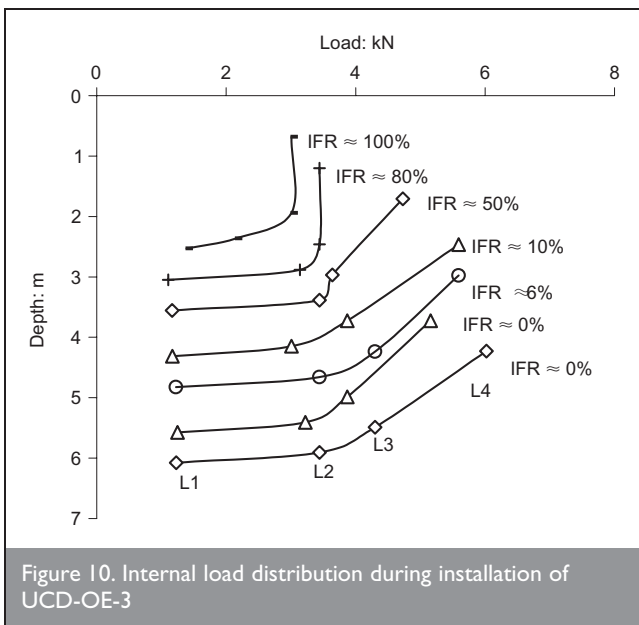
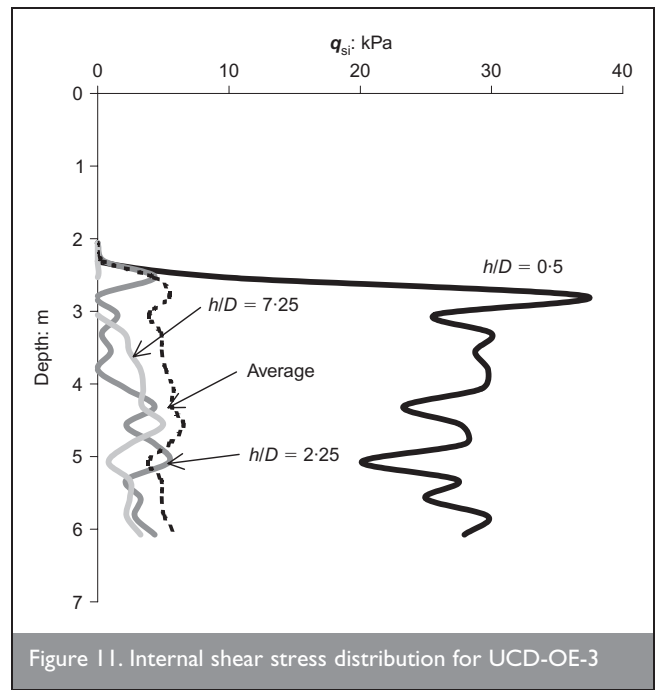
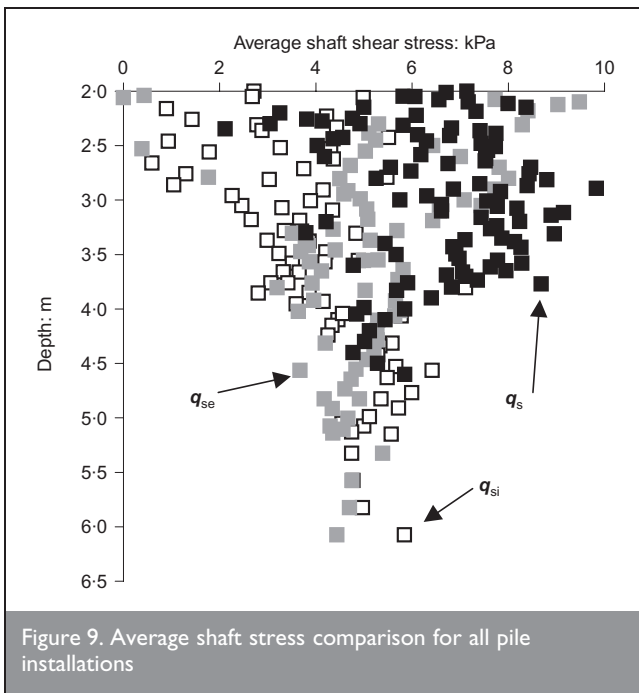


Figure 8. Plug development during pile penetration

(when the pile is unplugged). These observations ($q_{si} < q_{se} < q_s$) contrast with the general assumption of the API method (Equations 2 and 3), which suggests that the internal and external shear stresses mobilised by pipe piles are equal. For deeper pile penetrations (> 4 m bgl), when the pipe piles are fully plugged, the shear stresses developed on all piles are approximately equal ($q_{si} \approx q_{se} \approx q_s$).

5.3. Base resistance

The twin-walled construction of the pile meant that the total base load was easily discernible, as it was exclusively transferred to the inner pile wall. The distribution of load on



the inner wall of the pile UCD-OE-3 during installation is shown in Figure 10. It is clear that the annular load developed at the pile toe (level one, L1) remained practically constant throughout installation. In contrast, the load developed by the pile plug (difference between L4 and L1) increased significantly as the soil core length increased and the IFR value reduced.

The distribution of internal shear stress during installation of UCD-OE-3 is shown in Figure 11. The tendency for high plug resistance to be concentrated near the toe of the pile is illustrated by the shear stress value at $h/D_i = 0.5$, which developed to a maximum of ~ 35 kPa soon after the start of installation, and thereafter remained relatively constant at 20–30 kPa. The increase in internal plug resistance occurs as load was transferred to the upper plug (at $h/D_i > 2$), with the q_{si} value at $h/D = 0.5$ remaining relatively independent of IFR. Lehane and Gavin (2001) compiled a database of similar measurements for pipe piles in sand, and noted that the

internal shear stresses near the pile tip were directly related to the degree of plugging, and the majority of load was shed over the bottom $3D$. This suggests that the mechanism of plug development during undrained installation of pipe piles in clay is slightly different from that for the drained installation of piles in sand, allowing more load transfer in the upper regions of the plug. This is perhaps unsurprising, as the base resistance developed by pipe piles in sand is controlled by the very large effective stresses developed near the pile toe, and by arching of radial effective stresses.

Although quantification of the exact value and distribution of internal shear stress is an extremely complex problem, the normal plug stress q_{plug} is given by the integral of the internal shear stress, and simple empirical correlations have been developed between q_c , IFR and q_{plug} for piles in sand. Similar relationships are now considered for pipe piles in clay. The variation of q_{plug} and q_{ann} during the installation of a typical pile (UCD-OE-2) is compared with the q_b value for a closed-ended pile (UCD-CE-1) and CPT q_c values in Figure 12. The plug stress is low during the initial stages of installation, and increases significantly as IFR reduces and the pile becomes fully plugged. In contrast, the q_{ann} profile is similar to those of q_c and q_b throughout installation, suggesting that it is independent of the degree of plugging experienced by the pile.

The data from all open-ended pile installations are considered in Figure 13, where the variation of the normalised maximum plug resistance (q_{plug}/q_c) and annular resistance (q_{ann}/q_c) mobilised during each jacking stroke are plotted against the corresponding IFR value. As suggested previously by Figure 12, the annular stress is independent of the IFR, and can be approximated by the CPT q_c resistance. In contrast, the data suggest a direct relationship between q_{plug} , q_c and the IFR of the form

$$9a \quad q_{plug} = q_c(0.8 - 0.6IFR) > q_{plugmin}$$

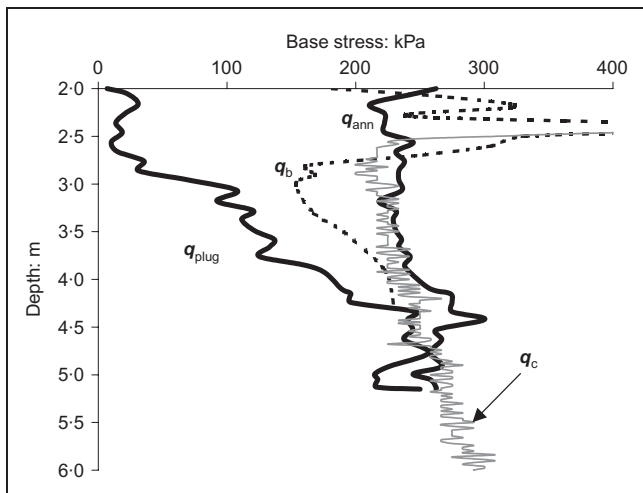


Figure 12. A comparison of the stresses developed under the pile tip

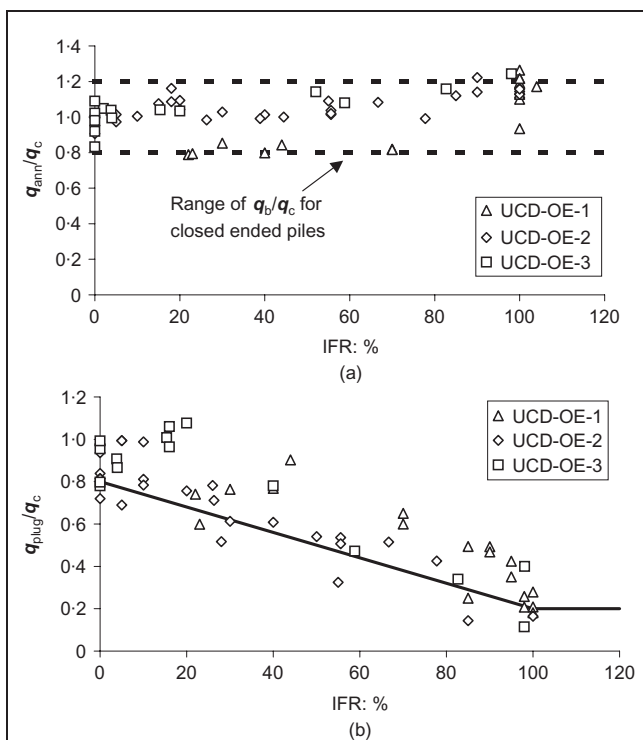


Figure 13. (a) Annular stress and (b) plug stress dependence on IFR

9b $q_{\text{plugmin}} = 0.2 q_c$

9c $q_{\text{ann}} = q_b = q_c$

Equation 9 implies that the plug resistance of a pipe pile increases by 400% as the pile moves from the fully coring to the fully plugged condition. The base resistance measurements in Figure 13 correspond to maximum values mobilised at relatively large pile head displacement (typically at $s/D \approx 67\%$), but the base stress mobilisation curves during individual jacking strokes indicated full base mobilisation before $s/D = 10\%$ movement.

6. CASE HISTORY

Clarke *et al.* (1985) describe the installation and static load testing of two pipe piles 762 mm in diameter and 18.3 m long, installed in the North Sea at the British Petroleum 'WC' block platform. One pile (pile A) had a driving shoe 457 mm long and 19 mm thick, which increased the annular thickness of the pile to 51 mm; a second pile (pile B) was driven without a shoe. The ground conditions at the site consisted of approximately 13 m of very stiff to hard glacial till over a layer of hard Lias clay, which extended to the base of the borehole (24.4 m bgl). Both the till and the Lias clay had occasional partings of fine sand and silt. The glacial till had a plastic limit of 15 and a plasticity index in the range 20–25. The natural moisture content reduced from 16% near the top of the deposit to 12% at depth. The s_u and q_c profiles are shown in Figure 14. The s_u and q_c values were quite variable in the till, where s_u increased with depth from 200 kPa to 700 kPa and q_c ranged from 3 MPa to 12 MPa, with inferred N_{kt} ($= q_c/s_u$) factors in the range 15–25.

The Lias clay had a plastic limit of 27–28 and a plasticity index that increased from a value of 20 in the depth range

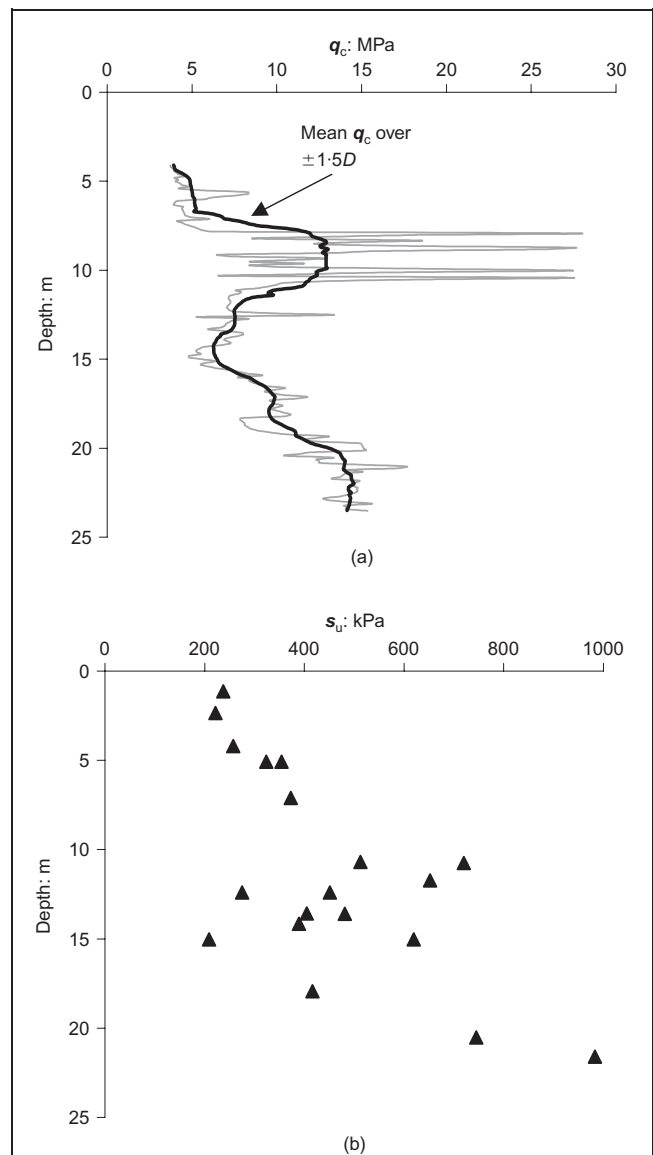


Figure 14. West Sole load tests: (a) cone resistance; (b) undrained shear strength (adapted from Clarke *et al.*, 1985)

13–18 m bgl to 25 below this depth. The natural moisture content was consistent with depth, at 17–18%. Although large scatter was evident in the measurements of s_{u1} , values were typically > 400 kPa, with a maximum recorded value of 983 kPa. CPT q_c values varied with depth from ~ 8 MPa at the top of the deposit to ~ 14 MPa at 24 m bgl.

Although the piles were uninstrumented, both static compression and tension load tests were performed on each pile at intervals of 3 m of pile penetration during installation (i.e. at pile penetration depths of 3 m, 6 m, 9 m, 12 m, 15 m and 18 m). The compression load tests were typically performed within 3–12 h of the end of driving, and tension load tests typically followed within 3 h, resulting in the tests being essentially undrained. Delays in testing were noted for two tests: the first compression load test at 6 m bgl on pile A, which was performed 121 h after driving, and the first compression load test on pile B at 3 m bgl, which was performed 64 h after driving.

The IFR profiles, which were obtained by measuring the plug height prior to each static load test, are shown in Figure 15. These show that the presence of a driving shoe was effective in reducing plugging, with pile A exhibiting an IFR value in the range 0.8–1.0 throughout installation. In contrast, significant plugging occurred throughout the installation of pile B, with IFR values in the range 0.19–0.82. By measuring both the compression resistance Q_c and tension resistance Q_T of the piles, Clarke *et al.* (1985) interpreted the unit base resistance q_b as the difference between these measurements divided by the overall pile base area. By adopting the additional assumption that the annular resistance developed during the load tests was equal to the q_c value averaged over the zone $1.5D$ above and below the pile base, the q_{plug} values are inferred at each loading level. Although the accuracy of these assumptions is somewhat debatable, and specifically the inherent assumption that the shaft resistances in tension and compression are equal, and are unchanged by a previous load testing, the resulting q_{plug}/q_c values demonstrate a definite trend to increase in line with the IFR values, as illustrated in Figure 16. The West Sole data are seen to be a reasonable fit to Equation 9 developed from the UCD open-ended piles tests, despite the obvious difference of soil state at the two sites. The tendency for the

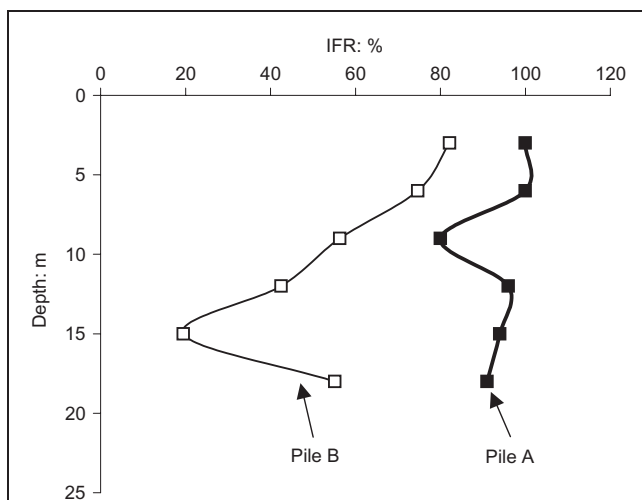


Figure 15. Incremental filling ratio back-figured for West Sole piles

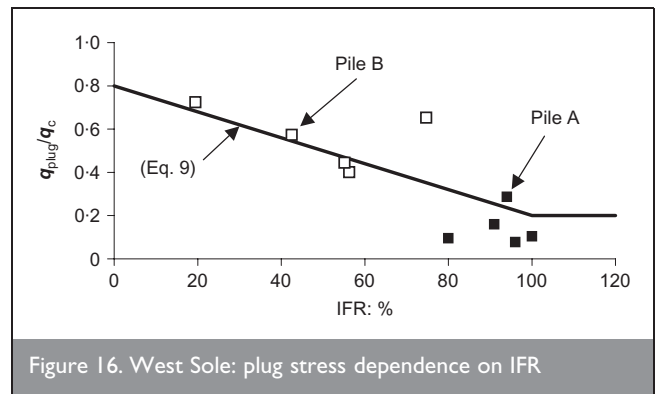


Figure 16. West Sole: plug stress dependence on IFR

normalised plug resistance measured on the full-scale field piles to increase with IFR gives confidence in the wider application of Equation 9.

7. CONCLUSIONS

Field tests that examined the effect of soil plugging on the base resistance developed during the installation of pipe piles in clay were presented. The following observations were made.

- The annular resistance of a pipe pile in clay was equal to both the closed-ended base stress and the CPT q_c value, and was independent of the degree of plugging throughout installation.
- The plug capacity of a pipe pile in clay was found to be dependent on the plug formation, and when normalised by the CPT q_c value acting at the pile tip could be related directly to the IFR value.
- The average internal shear stresses developed during installation formed a lower bound to the corresponding external shaft measurements at shallow penetrations, with both values tending toward a constant 5 kPa at depth. The distribution of shear stress within the pile plug suggests a mechanism for plug formation during undrained plugging in clay different from that for drained plugging in sand, as the internal shaft stress at the pile tip was found to be relatively independent of IFR.
- A simple predictive equation based on the observations for Belfast soft clay was applied to a case history of offshore load tests with reasonable accuracy, implying a more widespread dependence of base stress on plug formation.

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