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The Shaft Capacity of Open-Ended Piles in Clay

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ABSTRACT

This paper describes an experimental investigation designed to assess the impact of pile end condition on the capacity of piles installed in soft clay. A series of field tests are described where instrumented open-ended (OE) and closed-ended (CE) model piles were jacked into soft clay. The radial stresses, pore pressures and load distribution were recorded throughout installation, equalization and load testing. While the total stress and pore pressure developed during installation were related to the degree of soil plugging, the radial effective stress which controls the shaft resistance was shown to be independent of the mode of penetration. The long term shaft capacity of the open ended pile was closely comparable to that developed by closed-ended piles, suggesting a limited influence of end condition on the fully equalized shaft resistance. In contrast to the shaft resistance, the base capacity was highly dependent on the degree of plugging.

Notation List

API	American Petroleum Institute
CE	Closed Ended Pile
CEM	Cavity Expansion Method
CPT	Cone Penetration Test
D	Diameter
D_{eq}	Diameter of an Equivalent Closed Ended Pile
D_i	Internal Diameter
G	Shear Modulus
G_{max}	Maximum Shear Modulus
H/H_i	Relaxation Ratio
ICP-05	Imperial College Method
IFR	Incremental Filling Ratio
I_r	Rigidity Index
K_c	Equalised Earth Pressure Coefficient
K_o	Lateral Earth Pressure Coefficient
L	Embedded Pile Length
L_p	Plug Length
MASW	Multi-Channel Analysis of Surface Waves
OCR	Over-Consolidation Ratio
OE	Open Ended Pile
PLR	Plug Length Ratio

R	Radius
R_{eq}	Radius of an Equivalent Closed Ended Pile
R_i	Internal Radius
SPM	Strain Path Method
St	Sensitivity
U_d	Pore Pressure Ratio
UWA-05	University of Western Australia (2005) Method
YSR	Yield Stress Ratio
f_L	Loading Factor for ICP-05
h	Distance from the pile tip
k	Empirical constant for Kulhawy and Mayne (1990) Correlation
q_{ann}	Annular Stress
q_{enet}	Net Cone Tip Resistance
q_{cav}	Average Cone Resistance
q_b	Base Stress
q_{plug}	Plug Stress
q_T	Corrected Cone Tip Resistance
s_u	Undrained Strength
t	Pile Wall Thickness
u	Pore Pressure
u_0	Equilibrium Pore Pressure
u_{max}	Maximum Pore Pressure

α	Empirical Alpha Factor
δ_f	Friction Angle at Failure
δ_r	Residual Interface Friction Angle
ΔL	Change in Pile Length
ΔL_p	Change in Plug Length
Δu	Excess Pore Pressure
τ_{av}	Average External Shear Stress
τ_{ce}	Average External Shear Stress on Closed Ended Pile
τ_{oe}	Average External Shear Stress on Open Ended Pile
τ_f	External Shear Stress at Failure
σ_{v0}	Total Overburden Stress
σ_{ri}	Radial Total Stress at Installation
σ'_{v0}	Effective Overburden Stress
σ'_{vc}	Vertical Effective Yield Stress
σ'_{ri}	Radial Effective Stress at Installation
σ'_r	Radial Effective Stress
σ'_{rc}	Equalised Radial Effective Stress
λ	Empirical Fitting Parameter for CEM
μ	Empirical Fitting Parameter for CEM
ρ	Area Ratio

Introduction

Recent exploitation of offshore resources, such as wind and wave energy, has increased the interest of industry in improving design methods for offshore foundation systems. Large diameter, open ended pipe-piles are particularly useful in the offshore environment, having the combined advantages of high rigidity and the capability to achieve the large design penetrations necessary to resist uplift (or over-turning). A programme of instrumented pile tests undertaken by industry and academic institutions in recent years has led to considerable advances in design techniques for onshore, largely closed-ended piles. Measurements of the effective stress regime at the pile-soil interface, such as those obtained by experiments performed using the closed-ended Imperial College Pile (ICP), by Bond (1989), Lehane (1992) and Chow (1997) have culminated in a new effective stress design method, known as ICP-05 (Jardine et al. 2005). However, there is a dearth of equivalent field test data for open-ended piles installed in clay. As a consequence, many of the current design approaches have varying, and sometimes contradictory recommendations for estimating the shaft resistance developed by open and closed-ended piles. This paper presents field measurements of the shaft stresses developed during installation and load-testing of open and closed-ended model piles installed in soft clayey silt which are used to examine the impact of end condition on the pile resistance.

Background

Paikowsky and Whitman (1990) investigated the effect of soil plugging on the axial resistance developed by open-ended (pipe) piles installed in sand and clay. They described the process of soil plug formation (See Figure 1) wherein; during the initial stages of pile installation, the length of the soil plug (L_p) inside the pipe, equals the pile penetration depth (L) and the pile is said to be coring ($IFR = 100\%$). As the pile penetration depth increases, frictional stresses between the inside wall of the pile and the soil plug may cause partial plugging ($0\% > IFR, 100\%$) and in some cases the pile may become completely plugged ($IFR = 0\%$). They noted that plugging resulted in a large increase in the axial resistance of piles installed in sand and caused a large increase in the zone of excess porewater pressure surrounding piles in clay, causing a delay in set-up effects.

The development of the soil core during installation is quantified by the Plug Length Ratio (PLR) or the Incremental Filling Ratio (IFR):

$$[1a] \quad PLR = L_p/L$$

$$[1b] \quad IFR = \Delta L_p/\Delta L$$

Lehane and Gavin (2001), Gavin and Lehane (2003) and Foye et al. (2009) demonstrated experimentally that plugging increased both the shaft and base resistance of piles installed in sand and proposed correlations between pile resistance and IFR. Both Jardine

et al (2005) and Lehane et al (2005) incorporated plugging into design practice in the ICP-05 and UWA-05 design approaches, for piles in sand, which are included in the commentary of the latest American Petroleum Institute (API) design code. The most significant effect of plugging for piles in sand is the increase in base resistance, with a five to seven-fold increase in the ultimate base resistance mobilised as a pile moved from the coring to fully-plugged condition in sandy soil. In general, the base resistance amounts to a much smaller proportion of the total capacity for closed ended piles in clay. This may explain the historical lack of research examining the effects of plugging on the resistance of piles in clay.

Karlsrud and Haugen (1981) performed field tests in which they compared the resistance developed during installation of open-ended and closed-ended piles. The piles which were 153 mm diameter (the open-ended pile had a wall thickness of 4.5 mm) were jacked into over-consolidated clay. The ratio of the total resistance developed by the open and closed-ended piles is shown in Figure 2 alongside the PLR and IFR profiles measured during pile installation. During the early stages of installation when the ratio of pile penetration length to the pile diameter (L/D) was less than five, the resistance was much lower on the open-ended pile ($< 60\%$). As the pile penetration increased, the ratio of mobilised capacity increased significantly with both piles mobilising equal resistance above L/D values of 15. The open-ended pile remained fully coring ($IFR = PLR = 100\%$) until $L/D = 12$ and became fully plugged ($IFR = 0\%$) at $L/D = 21$. The data suggests that changes in the installation resistance developed by the open-ended pile were not well correlated to either PLR or IFR.

Miller and Lutenegger (1997) investigated the effect of pile plugging on the equalised τ_{av} values developed during field load tests performed on open and closed-ended model piles driven or jacked into over-consolidated clay. Consideration of their data in Figure 3 shows that α ($= \tau_{av}$ normalised by the average undrained shear strength s_u , along the pile shaft) depended on the mode of installation, with jacked piles developing much higher τ_{av} values. In contrast to the findings of Karlsrud and Haugen (1981), the degree of plugging experienced (quantified through PLR) strongly influenced the mobilised shaft resistance, with α increasing linearly as PLR reduced. While there was considerable scatter in the data, it is worth noting that this effect was much more pronounced for jacked in place piles.

The API 2007 design method uses a total stress approach where τ_{av} is linked to the in-situ vertical effective stress, σ'_{vo} and the undrained strength, s_u and is given as the larger of:

$$[2a] \quad \tau_{av} = \sqrt{s_u \sigma'_{vo}} \quad \text{or}$$

$$[2b] \quad \tau_{av} = 0.5 s_u^{0.75} \sigma'_{vo}{}^{0.25}$$

Where no distinction is made between the shaft resistance developed by open and closed-ended piles.

Karlsrud et al. (2005) proposed an alternative total stress approach known as the NGI-99 method, in which α varies with the undrained strength ratio (s_u/σ'_{vo}) and the plasticity

index (PI) (See Figure 4). The shaft capacity developed on open and closed-ended piles is identical for $s_u/\sigma'_{vo} < 1$, whilst for $s_u/\sigma'_{vo} > 1$, closed-ended piles are assumed to develop higher shaft resistance (with the maximum difference being 20% for heavily over-consolidated clay).

Jardine et al. (2005) proposed an effective stress design approach known as ICP-05, where the local peak shaft resistance τ_f is given as:

$$[3] \quad \tau_f = f_L K_c \sigma'_{vo} \tan \delta_f$$

where f_L is a loading factor (of 0.8), K_c is an earth pressure coefficient, and δ_f is the interface friction angle at failure. Lehane (1992) compiled a database of K_c measurements from instrumented pile load tests performed on closed-ended piles, which was later updated by Chow (1997). Using this database, Jardine et al (2005) proposed an empirical correlation to estimate K_c , which they found to be dependent on the Yield Stress Ratio (YSR), the sensitivity (S_t) and a geometric term (h , the distance from the pile tip normalized by the pile radius, R) which described the effect of friction fatigue. Hereema (1980) introduced this term to describe the reduction in shear stress which occurs in a given soil horizon, as the distance h increased, a feature of behavior noted in many field tests (See Cooke et al 1979, Bond and Jardine 1991 and others).

$$[4] \quad K_c = [2.2 + 0.016 YSR - 0.87 \log_{10} S_t] YSR^{0.42} \left(\frac{h}{R} \right)^{-0.2}$$

However, the database of K_c measurements for open-ended piles was relatively small, precluding the development of a direct correlation (such as Eqn 4). Chin (1986) used the Strain Path Method (SPM) to investigate differences between the strain fields induced by installation of closed and open-ended piles. The open ended pile geometry modelled was typical of an offshore pile with a pile diameter, D to wall thickness t , $D/t = 40$. Comparisons of the octahedral shear strains developed by the piles revealed that the strains were comparable when the equivalent radius R_{eq} factor was used to normalise the open-ended pile measurements, whilst the radius of the plastic zone was found to be proportional to the area displacement ratio, ρ :

$$[5a] \quad R_{eq} = \sqrt{R^2 - R_i^2}$$

$$[5b] \quad \rho = 1 - IFR \left(\frac{R_i}{R} \right)^2$$

Where R_i is the internal radius of the open-ended pile.

On this basis, Chow (1997) suggested that the Imperial College design approach for closed-ended piles in clay (Eqn 4) could be extended to open-ended piles by substituting R_{eq} for R . The method inherently assumes that pile installation takes place in a fully coring manner ($IFR = 100\%$) and that friction fatigue is more extreme on open ended piles.

While the ICP-05 approach for open ended piles has yet to be verified by rigorous experimental evidence, the use of an R_{eq} reduction factor which links the behaviour of

closed and open-ended piles is supported by the experimental work of Xu et al. (2006). They reported measurements of the radial total stress and porewater pressure set-up in the soil mass remote from the pile-soil interface, during installation of two 1.02 m diameter steel pipe piles. The piles which had a D/t ratio of 7.4 remained fully coring (with PLR = IFR = 100%) throughout installation. They demonstrated that reasonable estimates of both the radial total stress and pore pressure were achieved when the reduced radius R_{eq} term was used in the cavity expansion method.

Randolph (2003) proposed a simple generalized form of the cavity expansion method (CEM) for determining the radial effective stress acting on the pile shaft both immediately after installation and following complete consolidation. The technique explicitly considers the effects of a transition from fully coring to fully plugged installation for open-ended piles. The concept, originally proposed by Randolph (2003) was formalized by Chen and Randolph (2007) allowing equation 6 to be developed.

$$[6] \quad \sigma_{ri} = u_0 + \frac{1}{S_t} \frac{s_u}{\tan \delta_r} + \left(1 - \frac{1}{S_t}\right) \frac{1+2K_0}{3} \sigma'_{v0} + s_u \ln(\rho I_r)$$

where the first component on the right hand side represents the initial pore pressure, u_0 , the second reflects the external radial effective stress, σ'_{ri} , the third accounts for shear induced excess pore pressure and the fourth is the expansion induced excess pore pressure. The rigidity index, I_r , is the ratio of the shear modulus, G , to the shear strength, s_u .

Lehane (1992) reported measurements of radial total stress measured following installation of the Imperial College Pile (ICP) at three clay sites. He found in all cases that the radial total reduced during consolidation (a process termed relaxation). The observed relaxation behavior is in keeping with radial consolidation models (Fahey and Lee Goh, 1995), that predict zones of different stiffness adjacent to the pile (where consolidation occurs), and remote from the pile,(where swelling occurs to accommodate the radial pore water flow). Randolph (2003) proposed equation 7 to capture this relaxation gradient, which is dependent on the pre-consolidation stress, σ'_{vc}

$$[7] \quad -\frac{d\sigma'_r}{du} = \lambda e^{-\mu(\sigma'_r - \sigma'_{ri})/\sigma'_{vc}}$$

By integrating the relaxation gradient over the change in pore pressure from the maximum to the equilibrium values allows the total magnitude of stress reduction to be determined. Therefore, after consolidation, the equalized radial effective stress can be estimated from the earth pressure coefficient K_c :

$$[8] \quad K_c = \frac{\sigma'_{rc}}{\sigma'_{v0}} = \frac{\sigma'_{ri}}{\sigma'_{v0}} + \frac{YSR}{\mu} \ln \left(1 + \frac{\lambda \mu}{YSR} \frac{\Delta u_i}{\sigma'_{v0}} \right)$$

Randolph (2003) suggests values of 1 and 5 respectively for the empirical parameters λ and μ in order to match radial effective stresses profiles on the ICP reported by Lehane (1992). Chen and Randolph (2007) show this approach provided a slight over-prediction of the external radial stress changes during suction caisson installation in centrifuge

model tests. However, it gave reasonable estimates of the external shaft friction following complete equalization, although under-predicting the result for the sensitive clay (Chen and Randolph, 2007). The CEM method was applied by Doherty and Gavin (2010) to predict the radial total stress, porewater pressure and radial effective stress mobilized during installation of the NGI closed-ended test pile in over-consolidated clay at Haga, reported by Karlsrud and Haugen (1981). They noted that the method predicted reasonable estimates of all values measured near the pile tip. However, at points remote from the pile, the stresses were overestimated.

Significant uncertainties exist in our understanding of the mechanisms controlling the shaft resistance developed by open-ended piles in clay. These uncertainties are reflected in the wide variation in design recommendations for the treatment of open-ended piles. This paper describes a programme of pile testing performed at a soft clay test site located in Kinnegar, Belfast. The test programme involved the installation and load testing of seven piles, four closed-ended and three open-ended, and thus allows direct comparison of the resistance developed by both pile types.

Experimental Procedure

The experimental program spanned between 2003 and 2008, and included tests performed using two model piles; a closed-ended pile (described in Gavin and O’Kelly, 2007) and an open-ended pile (described by Igoe et al. 2010).

The UCD Closed-Ended (CE) pile was constructed using a 73 mm external diameter stainless steel tube and consisted of a 1.7 m long instrumented section which housed total stress and pore pressure sensors. These sensors were located diametrically opposite each other at normalized distances of $h/D = 1.5, 5.5$ and 10 from the pile tip. A total stress sensor was also incorporated into the base plate ($h/D = 0$) which provided a continuous measurement of the pile tip resistance throughout installation. One metre long extension pieces (made from the same stainless steel tube) were added to the instrumented unit to achieve penetration depths of up to 7 m below ground level (bgl).

The 168 mm diameter UCD Open Ended (OE) pile comprised of a 2 m long, instrumented section capable of penetrations up to a maximum depth of 7 m through the addition of 1m long extension pieces. The twin wall form of construction, previously used by Paik and Lee (1993), was adopted in order to provide direct measurement of the plug load (internal shear stress), annular base resistance and external shear stress. The twin-walled form of construction involved placing a smaller diameter tube, in this case a 154 mm external diameter tube with a wall thickness, $t = 2\text{mm}$, within a larger tube, with an external diameter of 168 mm and a wall thickness of 3.34 mm (See Figure 5). The piles were joined at the top and the gap at the base was filled with a flexible sealant, which prevented load transfer. All instrumentation was housed within the 3.66 mm wide void space between the two tubes which allowed the stress distribution on the annulus, shaft and plug to be independently determined. The pile had a total wall thickness of 9 mm and a D/t ratio of 18.7. The instrumentation layout mirrored the CE pile. Kyowa PS-5KA sub-miniature pressure transducers were used to measure the radial stress and pore

pressure, which were located at h/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.

A total of seven pile installations were performed, four using the closed-ended pile (identified as CE1 to CE4), and three using the open-ended piles (OE1 to OE3). Installation depths are illustrated in Figure 6, with further details provided in Table 1. The pile locations are shown on Figure 7. A hydraulic cylinder was used to jack the piles from the base of preformed starter holes to their final depths in strokes ranging from 100mm to 250mm length. Reaction was provided by a specially fabricated steel frame anchored to a concrete kentledge founded on 10 m deep piles bearing in the underlying dense sand. Additional details on the reaction system used at the Belfast test site are described by Doherty et al (2010). The pile head accommodated a hollow load cell between the hydraulic jack and the driving cap, providing a continuous load profile during installation. A system 5000 datalogger was used to record the load cell, strain gauges and pressure sensor data at 0.1 second intervals throughout installation and load testing. In addition, two displacement transducers fixed to an independent reference beam were used to measure the vertical movements during load testing. The piles were subjected to various load tests including static, cyclic and rapid testing at a constant rate of penetration. The load test chronology is included in Table 2.

Soil Conditions

The experimental programme described in this paper was undertaken at the Kinnegar geotechnical research site, located 10 km north-east of Belfast city centre in Northern Ireland. This geotechnical research site has been the subject of extensive investigations over the past fifteen years, as shown by the various in-situ tests depicted in Figure 7. The soil stratigraphy has been described in detail by McCabe (2002), Gallagher (2006) and Doherty et al. (2010) and therefore only a brief summary of the relevant geotechnical properties is provided here. The stratigraphy of primary concern is an approximately 7 m thick layer of estuarine deposit, known locally as *Belfast sleetch*. The sleetch can be subdivided into an upper and lower layer, with the upper sleetch comprising of a sandy silt deposit of variable thickness between 0.7 and 1.5 m depth and the lower sleetch consisting of clayey silt. At the Kinnegar site the sleetch is overlain by a layer of general fill extending to between 1 m and 1.5 m depth, and is underlain by a deep deposit of uniform medium dense sand. The water table varies both seasonally and tidally between 1.0 m and 1.5 m below ground level.

The sleetch is primarily silt sized, the upper layer has a 20% sand content and a clay fraction of 10%, whilst the lower sleetch has a reduced sand content and higher clay fraction (up to a maximum of 38%) . The average bulk unit weight of the sleetch is 16.2 kN/m³. The index properties are illustrated in Figure 8, with Atterberg tests yielding a liquid limit of $65 \pm 10\%$ and plasticity index of $35 \pm 5\%$, for the lower sleetch, with a natural moisture content of $60 \pm 10\%$. Despite the large silt fraction, the material falls

above the A-line in the Casagrande chart identifying it as an intermediate to high plasticity clay. Oedometer tests indicated a range in permeability from 1.5×10^{-10} m/sec to 5×10^{-10} m/sec and vertical coefficients of consolidation (c_v) reducing with stress level from $3 \text{ m}^2/\text{year}$ to about $0.5 \text{ m}^2/\text{year}$ at an effective stress of 100kPa. Multiple piezocone dissipation tests were performed with the horizontal coefficient of consolidation ranging between 7 and $12 \text{ m}^2/\text{year}$, indicating a slightly higher degree of dissipation in the radial direction in comparison to the vertical.

Cone Penetration Tests (CPT) and piezocone tests (CPTu) were conducted at the site, yielding q_{cnet} profiles ($q_{\text{cnet}} = q_T - \sigma_{v0}$) which, despite initial variability in the fill and upper sleet, reveal the lower sleet to be a uniform deposit in which the q_{cnet} value rises from 175 kPa at $\approx 2.3 \text{ m}$ bgl to 240 kPa at 6m below ground level (bgl). The undrained strength (s_u) profile measured from in-situ vane and triaxial compression tests is shown alongside the net cone resistance in Figure 8, where s_u values are seen to increase with depth from approximately 18kPa at 3m to 26kPa at 8m depth. Additional in-situ tests performed at Kinnegar included the seismic cone and the Multiple Analysis of Surface Waves (MASW) which measured small strain stiffness, G_{max} of approximately 11MPa.

The stress history of the sleet was determined by one dimensional compression tests on intact samples of the Belfast sleet. The vertical preconsolidation stress and in-situ stress profiles are shown in Figure 9, alongside the undrained strength ratio (s_u/σ'_{v0}) and the Over-Consolidation Ratio (OCR). The sleet is lightly over-consolidated exhibiting an OCR which decreases with depth from 1.6 at 3m to 1 at 8m depth. The OCR profile

determined from the oedometer tests provides a reasonable comparison with the correlation suggestion by Kulhawy and Mayne (1990): $OCR = k((q_T - \sigma_{v0}) / \sigma'_{v0})$, where k has been shown to vary from 0.2 to 0.5. An empirical k value of 0.25 provides the trend shown in Figure 9b. The soil sensitivity was determined using the oedometer tests and the procedures recommended by Chow (1997). The relative change in void ratio (ΔI_{vy}) for the Belfast sleet was determined to be 0.3 from comparison of the void ratio at yield on samples of natural sleet (e_y) and remoulded samples e_y^* . The relative change in void ratio is defined as: $\Delta I_{vy} = (e_y - e_y^*) / C_c$, where C_c is the coefficient of consolidation on the intrinsic compression line. The sensitivity is estimated from ΔI_{vy} to have a value of 2. Multiple oedometer tests at depths ranging from 1.8 m to 6.1 m indicate the sensitivity is relatively constant throughout the sleet and is independent of depth.

Experimental Results

Soil plug development during installation

The soil plug formation during installation of the open-ended piles, illustrated in Figure 10, shows that the degree of plugging experienced was independent of the jacking stroke length as all piles exhibited similar plugging response. The piles remained fully coring (IFR = 100%) over the initial 0.7 – 1.4 m penetration, thereafter partial plugging commenced. At the final penetration depth of pile OE1 (4.04 m bgl), the IFR value was 44%. Pile OE2 and OE3 became fully plugged (IFR = 0%) at penetration depths of 4.8 m and 5.3 m bgl respectively.

Average shear stress during installation

Gavin et al. (2010) examined the effect of installation method on the shaft resistance developed by closed-ended piles installed at Kinnegar. They varied the jacking stroke length (number of installation load cycles) and installation rate and found that radial total stress, porewater pressure and shaft resistance mobilized during installation (see Figure 9a) increased as the jacking stroke length and installation rate increased. Shear induced dilation was measured at the pile-soil interface, where pore pressures decreased in proportion to the shearing rate. This dilatatory response led to a viscous increase in effective stress during shearing. The authors found that long-term radial effective stresses measured on piles installed in the sileech were insensitive to the number of jacking strokes applied during installation. They concluded that long term friction fatigue effects were minimal because the largely undrained nature of pile installation in this deposit prevented significant volume change occurring.

The average shaft resistance (τ_{av}) mobilized during installation of the open-ended piles is shown in Figure 11b. τ_{av} was seen to be relatively constant at 4 to 6 kPa throughout installation, suggesting that α values in the range 0.2 – 0.3, which were independent of the degree of plugging or jacking stroke length. The τ_{av} values are similar to those measured on the closed-ended piles installed using 100 mm jacking strokes at the lowest installation rate (See Figure 11a). Note that some discrepancies in average shaft resistance were observed between CE1 and CE2, which are attributed to local variations in the thickness of the fill and upper sandy sileech.

Radial stresses and pore pressures during installation

Typical profiles of radial total stress and porewater pressure developed during installation of an open-ended (OE3) and closed-ended pile (CE4) are compared in Figure 12. The longer jacking strokes adopted for installation of the open-ended piles were chosen such that the number of installation load cycles (jacking strokes length/pile diameter) were similar for the open and closed-ended piles considered. In addition, CE4 is the closest closed ended pile to the open ended test, minimizing any inherent variability in soil properties. As a result, these piles offer the best comparison between closed and open ended penetration and this is confirmed by the similar average shaft shear stress profiles. The following are noteworthy:

- (i) The stresses on both piles increased rapidly until the pile tip reached the lower sloop (at approximately 2 m bgl). Thereafter, the rate of increase with depth slowed, reflecting the gradual increase in soil strength evident in the in-situ strength profiles.
- (ii) Friction fatigue is evident on both the open and closed-ended piles, with the highest radial stress and pore pressures measured by the sensors close to the pile tip (at $h/D = 1.5$), and both values steadily reducing as h/D increased.
- (iii) At a given depth, both the radial stress and pore pressure were lowest on the open-ended pile until the pile became fully plugged. Radial effective stresses (σ'_{ri}) mobilized during the installation of a typical open (OE3) and closed-ended (CE4) pile are compared in Figure 13a. Although there was significant scatter in the data with σ'_{ri} values varying from 10 to 40 kPa, there was no apparent effect of soil plugging on the σ'_{ri} values developed on the open-

ended piles. The strain gauges allowed the distribution of local shear stress to be determined. The mobilized friction angle can then be determined from the peak radial effective stresses and the local shear stresses at the same location on the pile shaft. Typical values measured during the installation of pile UCDCE4 are shown in Figure 13b. These show that the average friction angle of 14 degrees, agreed reasonably well with the residual friction angles determined from interface shear tests, which decreased slightly with depth to consistent values of between 10 and 11 degrees below 3.5 m.

The effect of soil core development on the pile-soil stresses mobilized during installation is considered in Figure 14 which shows the radial total stress, pore pressure and radial effective stress, normalized by q_{net} and plotted as a function of IFR. A clear trend for both the radial total stress and pore pressure to increase as IFR reduced, reaching maxima when the piles became fully plugged (IFR = 0) is noted from the data. By contrast, the radial effective stresses were seen to be independent of IFR. This latter trend explains the similarity between the average shaft resistance mobilized by the open and closed-ended piles installed at comparable rates using incremental jacking.

Base resistance during installation

The unit base resistance ($q_b = \text{base load/base area}$) developed by an open-ended pile is generated by two components; namely the annular stress (q_{ann}) mobilized beneath the pile wall and the plug stress (q_{plug}) which is controlled by the internal shear stresses which develop between the soil plug and pile wall (Paikowsky and Whitman 1990). The

variation of q_{plug} , q_{ann} and q_b during the installation of typical open-ended and closed-ended piles (OE2 and CE1 respectively) are compared in Figure 15. It is apparent that both the base resistance developed by the closed-ended pile and the annular stress on the open-ended piles were similar to q_T . By contrast, the plug stress was low during the initial stages of installation and increased significantly as the pile penetration increased (or as IFR decreased). Doherty et al. (2010) noted that whilst the plug stress developed during jacked installation of the pile in Belfast sleech varied linearly with IFR, the annular stress was independent of IFR (See Figure 16). The following expressions were proposed to calculate the separate components of the ultimate base resistance mobilized during installation of open-ended piles:

$$[9a] \quad q_{\text{plug}} = q_T (0.8 - 0.6 \text{ IFR}), > q_{\text{plugmin}}$$

$$[9b] \quad q_{\text{plugmin}} = 0.2 q_T$$

$$[9c] \quad q_{\text{ann}} = q_T$$

Pore pressure and radial stress during equalization

During equalization, the excess pore pressure generated during installation decreased to hydrostatic values. This process took 7 days for the closed-ended pile and 16 days for the open-ended piles. When the data are plotted against normalized time ($T_{\text{eq}} = t \cdot c_h / D_{\text{eq}}^2$) the response of both piles is quite comparable (See Figure 17a), where the pore pressure ratio, U_d is given as:

$$[10] U_d = \frac{u - u_0}{u_{\max} - u_0}$$

Where: t is the time since pile installation, c_h is the horizontal coefficient of consolidation, D_{eq} is the equivalent diameter of a closed ended pile displacing the same volume of soil as the plugged or partially plugged open ended pile (taking into account the incremental filling ratio at the end of installation: $D_{eq} = D_x \rho^{0.5}$), u_{\max} is the maximum recorded pore pressure and u is the pore pressure at time t . For normalization in Figure 17, the horizontal coefficient of consolidation was taken as the average value determined from piezocone dissipation tests, of $9.5 \text{ m}^2/\text{year}$.

The total stress ratio H/H_i (defined in equation 10) also decreased during equalization as shown in Figure 17b. This stress reduction, known as relaxation, is a common feature of displacement pile behavior in clay (see Bond 1991, Lehane and Jardine 1994 and others). The degree of relaxation was highest for the closed ended pile, with fully equalized values being approximately 45% of those measured at the end of installation.

$$[11] \frac{H}{H_i} = \frac{\sigma_r - u_0}{\sigma_{ri} - u_0}$$

In general σ'_r values tended to increase during equalization, resulting in set-up as shown by Figure 17c. At some sensor levels, relaxation occurred for a short period immediately following installation. This occurred for piles which were installed at the highest rates, where fast loading during installation caused large shear induced reductions in pore water pressure and therefore high radial effective stress and shear stress (See Gavin et al. 2010).

The shear induced pore pressures dissipated rapidly, after which set-up occurred. Notwithstanding variations in relaxation effects observed for total and effective radial stresses, the radial effective stress at the end of equalization (σ'_{rc}), shown in Figure 18 were remarkably similar for both open and closed-ended piles. However, it is worth noting that the tendency for plugging to occur towards the end of pile installation for OE2 and OE3, resulted in limited σ'_{rc} data being available at high IFR values.

The equalized radial effective stresses in Figure 18 also show good agreement with those made on a 6 m long 250 mm square driven precast piles installed at the same site and described by McCabe and Lehane (2006). Comparison is also made in Figure 15 to the σ'_{rc} values predicted using the cavity expansion approach method (CEM) proposed by Chen and Randolph (2007). Equations 6 and 8 were used with the relevant soil parameters for Belfast sileach to predict the equalized radial effective stresses. The σ'_{rc} values calculated using two possible values for the rigidity indices, are shown to slightly over predict the measured data for both open and closed ended piles. Friction fatigue, which is not considered in CEM, may be partly responsible for the difference in values. The magnitude of friction fatigue is however expected to be limited by the relatively undrained nature of the installation process, which limits volume changes and prevents contractions of the shear zone. The CEM method incorporates two fitting parameters, λ and μ , which were selected by Randolph (2003) to provide good agreement with results of pile tests performed with the Imperial College pile by Lehane (1992). Radial effective stress measurements at the three sites (Cowden, Bothkennar and Canons Park) at which the Imperial College pile was installed are included in Figure 18. Despite the fact that

they show reasonable agreement with the measurements from Belfast, there was a noticeable difference in friction fatigue effects at the sites considered. As such it is unreasonable to expect that unique values of the parameters λ and μ should apply. For the Belfast data it is interesting to note that the CEM method only predicts a 5% decrease in radial effective stress from closed ended to open ended penetration, which is comparable to the measured differences.

The model piles described in this paper were subjected to a range of load tests including static, rapid constant rate of penetration (CRP) and cyclic loading which are discussed in Doherty and Gavin (2010). Of the three open-ended pile tests described in this paper, only one (OE1 – which experienced the least plugging during installation) had a static load test performed before cyclic load testing. The τ_{av} profile measured during this test is shown in Figure 19, where the initial stiffness and ultimate shaft resistance are seen to be comparable to values mobilized by the comparative closed-closed-ended piles at the same site.

Discussion

The test results described in this paper demonstrated clearly that both the radial total stress and excess pore water pressure were affected by the degree of plugging experienced during installation. Somewhat surprisingly however, the shaft resistance (which is controlled by both the radial total stress and pore water pressure) developed in both the short-term (undrained installation) and long-term (static load tests) tests was independent of IFR. Whilst the annular resistance mobilized during pile installation was

independent of the degree of plugging experienced and could be predicted using the CPT q_c profile, the plug stress was directly correlated to the CPT q_c value and IFR. It is worth noting that whilst these findings may be applicable to soils with low over-consolidation ratios, some researchers suggest that differences might be more significant for high OCR soils. It is therefore instructive to compare the findings to measurements made from large-scale pile tests.

Short-term resistance

The installation and load testing of two 18.3 m long, 762 mm diameter open-ended piles into North Sea clay is described by Clarke et al (1985). Ground conditions at the site comprised 13 m of very stiff to hard glacial till (with s_u in the range 200 to 700 kPa), over Lias clay (with $s_u > 400$ kPa). One pile (Pile A) had a 457 mm long, 19 mm wide, driving shoe at the pile toe, which increased the pile wall thickness to 51 mm. The presence of the shoe resulted in significant reduction in the amount of plugging experienced during pile installation (See Figure 20a); with pile A remaining near fully coring (with IFR in the range 80 – 100%) throughout installation. In contrast, pile B experienced significant plugging. Static compression and tension load tests were performed on the piles at 3 m intervals during installation (i.e. at pile penetrations of 3 m, 6 m, 9 m etc). Compression load tests were performed within 3 to 12 hours after the end of driving, whilst tension tests were performed within 3 hours of the compression tests, resulting in all load tests being essentially undrained. Pile tests performed in the nearby Cowden Till (Lehane et al. 1994) which has a similar geological history suggest that less than 5% of the excess pore pressure generated during installation would have dissipated prior to the load tests. A

delay before testing the pile at a penetration depth of 6 m caused the results of this test to be omitted from consideration herein.

The average shaft resistances developed during the tension load tests, normalized by the average cone resistances, are shown in Figure 20b. It is clear that τ_{av}/q_{cav} reduced as the pile slenderness increased, suggesting that friction fatigue effects were significant. Despite the large differences in IFR values, the τ_{av}/q_{cav} ratios at all levels (except at 9 m bgl) were remarkably similar. At 9 m bgl, the pile with the highest IFR value mobilized the largest shaft resistance. The τ_{av}/q_{cav} values are seen to be compatible with those measured at Belfast, although this is likely to be somewhat fortuitous given the vastly different soil state, interface friction angles and installation methods.

Long-Term resistance

Chow compiled a database of case histories where load tests were performed on open and closed-ended piles at the same site, after full equalization of excess pore pressures. The τ_{av} values are compared to the Kinnegar data in Figure 21. The ratio of τ_{av} mobilized by open and closed-ended piles (τ_{avo}/τ_{avc}), which was $\approx 95\%$ at Kinnegar, is consistent with data from other low OCR soils such as Onsoy and Lierstranda. The data scatter increased with OCR, and there appears to be a trend for τ_{avo}/τ_{avc} to reduce as OCR increased. A possible reason for this trend was suggested by the radial total stress and pore pressure measurements made during pile installation at Kinnegar, with both values increasing linearly as IFR reduced. Piles installed in soft clay typically develop relatively high porewater pressures which approach the total radial stress and therefore radial effective

stress values are low (See Lehane and Jardine 1994 and Gavin et al. 2010). On the other hand, piles installed in heavily over-consolidated soils develop low (sometimes negative) porewater pressure in the interface shear zone adjacent to the pile shaft, resulting in high radial effective stresses. If the radial total stresses developed by piles installed in over-consolidated deposits increased with IFR in a manner similar to that seen in Kinnegar, it follows that the radial effective stresses would therefore be much higher on fully plugged (or closed-ended) piles, than on piles with high IFR values.

The NGI-99 design approach is seen to produce a reasonably conservative lower-bound to the overconsolidated data. However, it is worth noting that the lowest ratio of $\tau_{avo}/\tau_{avc} = 0.74$ was recorded for piles installed in Tilbrook Grange, where errors may have been caused by the test procedure adopted. The open-ended pile tested at this site was previously used as a borehole casing for the installation of a closed-ended pile. The installation of this secondary pile may have caused disturbance to the in-situ radial stress, reducing the measured shaft friction on the open-ended pile and possibly leading to a reduced ratio of τ_{avo}/τ_{avc} .

Conclusions

A series of tests involving the installation and load testing of piles at a soft clay test bed site located near Belfast in Northern Ireland were described. The main conclusions from the test programme can be summarized below:

1. The installation radial total stress and excess pore pressure were directly related to the degree of soil plugging and increased linearly as IFR reduced.
2. The radial effective stresses mobilized during installation and at the end of equalization were independent of IFR. As a result both the short-term and long-term shaft resistance mobilized by open and closed-ended piles were closely comparable, with the shaft resistance mobilized by an open-ended pile being approximately 5% lower than a closed-ended pile.
3. Whilst the annular resistance developed at the pile base was independent of IFR, the plug stress varied linearly with IFR.
4. The impact of pile end condition on the capacity of piles in highly overconsolidated soils should be examined in future experimental programmes.

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Table 1: Model Pile Details

Pile ID	Diameter (mm)	Starter Hole Depth (m)	Tip Depth (m)	PLR (%)	JS Length (mm)	Installation Rate* Mm/s	No. of strokes	Total time (mins)
CE1	73	1.35	4.15	-	100	20	28	155
CE2	73	1.5	4.25	-	100-400	20	18	292
CE3	73	1.9	3.6	-	1700	34	1	1
CE4	73	1.45	4.7	-	100	1.1	33	290

OE1	168	2.03	4.04	84	100	1	21	240
OE2	168	1.83	5.14	48	100	0.7	34	381
OE3	168	1.83	6.1	41	250	10	17	167

*Refers to the average installation rate during active jacking, does not consider the pause periods

Table 2: Pile Load Test Chronology

PILE ID	Installation Date	Virgin Load Test Date	Loading History
CE4	Installation Date		
OE1			
OE2			
OE3			

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