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Field investigation of the effect of installation method on the shaft resistance of piles in clay

Kenneth Gavin, David Gallagher, Paul Doherty, and Bryan McCabe

Abstract: This paper presents the results of a series of field experiments performed to study the effect of installation method on the shaft resistance developed by a pile installed in soft clayey silt. Tests were performed on piles that experienced different levels of cyclic loading during installation. The test results indicate that the radial total stress, pore-water pressure, and shear stress on the pile shaft during installation were strongly affected by the installation procedure; all three were found to increase when the jacking stroke length used during installation increased (or the number of cyclic load applications decreased). However, equalized radial effective stresses that control the long-term pile shaft capacity were found to be insensitive to the installation method. A simple expression that requires the results of a cone penetration test, laboratory measurements of the interface friction angle, and the pile geometry is proposed to calculate the shaft resistance.

Key words: piles, clay, shaft resistance, field tests.

Résumé : Cet article présente les résultats d'une série d'essais sur le terrain effectués dans le but d'étudier les effets de la méthode d'installation sur la résistance de l'arbre d'un pieu développée lors de l'installation d'un pieu dans un silt argileux mou. Les essais ont été effectués sur des pieux ayant subi différents niveaux de chargement cyclique durant l'installation. Les résultats des essais indiquent que la contrainte totale radiale, la pression interstitielle, et la contrainte de cisaillement sur l'arbre du pieu lors de l'installation, sont fortement affectés par la procédure d'installation. Ces trois paramètres augmentent lorsque la longueur du levier utilisé pour l'installation augmente (ou lorsque le nombre d'application cyclique de la charge diminue). Cependant, les contraintes effectives radiales égalisées, qui contrôlent la capacité de l'arbre du pieu à long terme, sont demeurées in affectées par la méthode d'installation. Une expression simple, qui nécessite des résultats d'essais de pénétration au cône, de mesures de laboratoire de l'angle de friction à l'interface et de données sur la géométrie du pieu, est proposée afin de calculer la résistance de l'arbre.

Mots-clés : pieux, argile, résistance de l'arbre d'un pieu, essais sur le terrain.

[Traduit par la Rédaction]

Background

The α (total stress) design approach is widely used to calculate the average unit shaft resistance of displacement piles in clay (τ_{av} = shaft load/shaft area),

$$[1] \quad \tau_{av} = \alpha s_u$$

where α is a reduction factor and s_u is the in situ undrained strength of the soil. Although there is myriad guidance on the choice of an appropriate α value, design guides for on-shore piles typically suggest α decreases as the soil strength increases and is affected by the slenderness ratio L/D , where L is length and D is diameter. Randolph and Murphy (1985) developed a design approach that was incorporated into the American Petroleum Institute (API) approach for offshore

pile design (API 1993). They proposed that τ_{av} is linked to the in situ vertical effective stress, σ'_{v0} , and is given as the larger of

$$[2a] \quad \tau_{av} = \sqrt{s_u \sigma'_{v0}}$$

or

$$[2b] \quad \tau_{av} = 0.5 s_u^{0.75} (\sigma'_{v0})^{0.25}$$

Despite the lack of a theoretical framework for the empirical design approaches outlined above, their relatively effective predictive capacity is in keeping with their widespread use in practice. Jardine et al. (2005) applied the API (1993) approach to calculate the shaft resistance of a database of instrumented pile tests and found that the average ratio of predicted to measured resistance was 0.99, with a coefficient of variation (COV) of 0.33. However, Jardine and Chow (2007) note systematic bias in the API approach in soils with high slenderness (L/D) ratios and low residual interface friction angles (δ_r).

Cooke et al. (1979) presented measurements of the shear stress mobilized during the jacked installation of a 168 mm diameter, steel pile in London clay. The distribution of shear stress as the pile was driven to 4.5 m below ground level (bgl) is shown in Fig. 1. It is clear that the shear stress mobilized at any depth reduced as the slenderness ratio increased, a behaviour that is not directly incorporated in

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Fig. 1. Shear stress distribution during pile installation in London clay (Cooke et al. 1979).

or [2]. Heerema (1980) introduced the term *friction fatigue* to describe the reduction in mobilized shear stress developed in a given soil horizon during driving, as the distance to the pile tip (h) increased (i.e., as L/D increased). Randolph (2003) notes that in strain-softening soils, progressive failure at the pile–soil interface could occur, leading to the mobilization of the residual interface friction angle near the top of the pile and peak interface friction angle (δ_p) near the toe. Kolk and van der Velde (1996) suggested an updated form of the API (1993) method to incorporate length effects

$$[3] \quad \tau_{av} = 0.55 s_u \left(\frac{\sigma'_{vo}}{s_u} \right)^{0.3} \left(\frac{L}{40D} \right)^{-0.2}$$

To understand the mechanisms controlling the development of shaft resistance, it is necessary to understand the radial effective stress regime at the pile–soil interface. An important insight into the effective stress response of displacement piles installed in clay has been provided by instrumented test programmes using the Imperial College pile (ICP), which measured the radial stress and pore-water pressure at a number of locations on the pile shaft (Jardine et al. 2005). They confirm that the local peak shaft friction, τ_f , mobilized at the pile–soil interface, obeys the Mohr–Coulomb failure criterion

$$[4] \quad \tau_f = \sigma'_{rf} \tan \delta_f$$

where σ'_{rf} and δ_f are the radial effective stress and interface friction angle, respectively, at ultimate conditions.

Bond and Jardine (1991) reported data from the installation of the ICP into London clay at Canons Park, UK, which are shown in Fig. 2a. They found that the base resistance (q_b) mobilized on the pile during each 200 mm jacking stroke was approximately equal to the q_c value at that depth. The location of the three levels of pressure sensors on the ICP were identified by their distance from the pile toe (h) divided by the diameter (D). The shape of the radial total stress (σ_r) profiles closely matched the base resistance pro-

file, with the values of σ_r nearest the pile toe ($h/D = 4$) being approximately 25% of the cone penetration test (CPT) q_c resistance in that horizon, while friction fatigue was evident as the σ_{ri} value in a given horizon reduced slightly as h/D increased from 4 to 25.

The total radial stresses measured at the end of the installation, normalized by the q_c values at Canons Park, are compared with similar measurements from the installation of the ICP into soft to firm lightly overconsolidated marine clay at Bothkennar in Scotland (Lehane and Jardine 1994a) and also stiff glacial till at Cowden (Lehane and Jardine 1994b), illustrated in Fig. 2b. While the effect of friction fatigue (that is, a reduction of σ_r/q_c as h/D increased) is evident at all sites, it is apparent from Fig. 2b that the ratio σ_r/q_c at a given h/D level was highest for the pile installed in the soft clay and was significantly lower for piles installed in stiff clay. In contrast, a parallel investigation of the radial stress developed during installation of the ICP into sand at two sites in France (a loose to medium dense dune sand at Labenne and dense sand at Dunkirk, reported by Lehane (1992) and Chow (1997)), found that the ratio σ_r/q_c at a given h/D level on the pile was unique and did not depend on the soil state. However, subsequent work by White and Lehane (2004) demonstrated that the ratio σ_r/q_c was also affected by the number of load cycles experienced by the soil at that level.

Whittle (1992) used the strain path method (SPM) to produce an estimate of the normalized installation radial stress profile at Bothkennar, using the MIT-E3 soil model. The radial stress profile predicted using the SPM, shown in Fig. 2b, is clearly significantly higher than the field measurements. Critically, the numerical model suggested that friction fatigue effects were confined to a region within $5D$ from the tip. Randolph (2003) suggested that differences between the theoretical SPM prediction and the field measurements at Bothkennar arose partly because of difficulties in modelling the experimental procedure adopted and partial dissipation of pore-water pressure at points remote from the pile tip.

White and Lehane (2004), Jardine et al. (2005), and Gavin and O'Kelly (2007) have investigated the cause of friction fatigue on piles in sand and have clearly identified the role of (i) the number of installation load cycles and (ii) stress relief in a given horizon as the pile tip passes. These effects combine to reduce the radial stresses in the vicinity and thus reduce the shaft resistance. Since the ICP piles were predominantly installed using the same jacking procedure in all test programmes and in other instrumented pile tests (such as those reported by Coop and Wroth 1989), the sensors were a significant distance from the pile tip, and it is therefore difficult to separate the effects of stress relief and the number of load cycles applied during installation from the available data. Given the uncertainty regarding the mechanisms controlling the friction fatigue process in clay, a series of field tests were performed using instrumented piles in which the sensors were placed relatively close to the pile tip and the mode of installation was varied to examine the importance of these factors in controlling the shaft resistance of piles in clay.

Fig. 2. (a) Radial total stress mobilized during pile installation at Canons Park (Bond and Jardine 1991). (b) Normalized radial stresses from ICP pile tests in clay (Lehane and Jardine 1994a).

Site conditions

The pile tests considered in this paper were conducted at the Kinnegar geotechnical research site on the shores of Belfast Lough, Northern Ireland. The site is located near Belfast harbour, and the water table is tidal varying between 0.8 and 1.3 m bgl. The stratification consists of four distinct soil layers: a layer of fill typically 1 m thick, mainly consisting of building rubble and gravel over a natural sandy silt deposit of variable thickness (approximately 0.7 to 2 m thick), which in turn overlies 6 m of soft clayey silt. The silt deposit is known locally as *sleeach* and will be considered in this paper as a two-layer deposit with the upper higher permeability sandy *sleeach* and the lower clayey *sleeach*. The *sleeach* is underlain by a uniform fine to medium dense sand.

The upper sandy *sleeach*, located between 1 and 3 m bgl, has proportions of sand and clay of 20% and 10%, respectively, while below this level the proportions reverse with a maximum clay fraction of 38%. The natural moisture content of the lower *sleeach* is $60\% \pm 10\%$, with a liquid limit and plasticity index, I_p , of $65\% \pm 10\%$ and $35\% \pm 5\%$, respectively. Despite the majority silt fraction, the material plots above the A-line in the Casagrande chart, thus identifying it as an intermediate to high plasticity clay. Oedometer tests indicated a range in permeability from 1.5×10^{-10} to 5×10^{-10} m/s and vertical coefficients of consolidation (c_v) reducing with stress level from 3 m²/year to about 0.5 m²/year at an effective stress of 100 kPa. These parameters are more typical of clay than silt and indicate the dominant nature of the clay size fraction in influencing the engineering behaviour. The *sleeach* can be termed lightly overconsolidated, with an overconsolidation ratio (OCR) decreasing with depth from ~ 1.6 at 3 m to 1 at 8 m.

A number of CPTs were performed at the site. Profiles of the net CPT end resistance ($q_{net} = q_t - \sigma_{vo}$) are shown in Fig. 3a. The q_{net} profiles were noticeably variable near ground level and became much more consistent when the cone entered the lower *sleeach*, where the q_{net} 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 with values backfigured from the q_{net} profile (assuming $N_{kt} = 12$) and those measured in triaxial compression tests in Fig. 3b. Values of the small strain shear modulus (G_{max}) were measured using two techniques: the seismic cone and the multichannel analysis of surface waves (MASW) method (Donohue 2005), shown in Fig. 3c, while the stiffness-strain response observed in triaxial compression tests with Hall effect gauges are shown in Fig. 4, where, for example, G_{25} corresponds to the shear modulus mobilized when the stress level was 25% of the maximum stress.

A number of ring shear tests were performed on samples of *sleeach* taken from the Kinnegar test site. Strick van Linschoten (2004) performed tests where *sleeach* was sheared against a rough concrete interface and reported peak interface friction angles (δ_p) of $16^\circ \pm 0.2^\circ$. Post peak softening generally resulted in a reduction of 1° to residual interface friction angles (δ_r). Doherty and Gavin (2009) performed tests using a smooth stainless steel interface and reported δ_p and δ_r values of 11° and 10° , respectively. Further details of the geotechnical properties of the *sleeach* can be found in McCabe (2002), Lehane (2003), and Gallagher (2006).

Test piles

The instrumented closed-ended stainless steel tubular model pile of 73.0 mm external diameter was developed at University College Dublin (UCD). The 1.7 m long lower section contained four instrumented units, which included total earth pressure sensors and pore pressure transducers manufactured by Entran Ltd., UK. A schematic of the piles is shown in Fig. 5. The pore pressure transducers were mounted in predrilled holes in the wall thickness of the instrumented units. A porous ceramic disc of 8.5 mm in diameter and 2.5 mm in thickness was mounted flush with the outer wall surface in front of each of the pore pressure transducers. The earth pressure sensors and pore pressure transducers were located diametrically opposite in the

Fig. 3. In-situ test profiles at Kinnegar: (a) cone penetration test end resistance; (b) undrained strength profile; (c) small-strain shear stiffness.

Fig. 4. Stiffness of sleech.

instrumented units at $h/D = 1.5, 5.5$, and 10.0 . In addition, total stress and pore-water pressure sensors were placed in the base plate to measure the end resistance. Spacer sections, 1.0 m in length and machined from the same steel pipe were attached to the lower instrumented section, allowing installation of the pile to depths of up to 4.7 m. Electrical resistance strain gauges were bonded to the inner wall of the pile at the top and bottom of the instrumented section and at the top of all spacer sections to allow the load distribution in the pile to be determined.

Four separate installations of the UCD instrumented model pile were carried out. These are referred to as UCD1, UCD2, UCD3, and UCD4, corresponding to pile embedments of $4.15, 4.25, 3.6$, and 4.7 m bgl., respectively. Piles UCD1 and UCD4 were installed in 100 mm jacking increments, while the jacking increments used to install pile UCD2 varied from 100 to 400 mm. UCD3 was installed in one continuous stroke. Pause periods between jacking strokes were typically less than five minutes, except on the occasions when extra spacer sections were added to the pile shaft. In this instance, pause periods increased to approximately 15 – 20 min. Using the coefficient of horizontal drain-

age (c_h) measured in piezocone dissipation tests and assuming a rigidity index (G/s_u) of 100 , the degree of dissipation of excess pore-water pressure was estimated to be 5% for a typical pause period and 20% for the extended pause periods, as per the method proposed by Randolph (2003). All instruments were logged at 0.1 s intervals throughout installation and load testing, using a System 5000 data logger. Two displacement transducers, supported by an independent reference beam, were attached to the pile head during load tests. Full details of the installation procedure and the static load tests performed on the piles are given in Table 1. Generally the response of the pressure sensors during testing was excellent. However, a leak that developed during the first installation of the pile (UCD1) resulted in the loss of data from most of the sensors, with the exception of the total stress sensors at $h/D = 5.5$ and 10 .

In this paper, measurements made with the UCD instrumented pile are compared with installation data from a full-scale instrumented pile installed at the same Kinnegar test site reported by McCabe and Lehane (2006). This 250 mm square precast concrete pile was installed to a 6 m embedment. Flatjack sensors were cast flush with the pile shaft at 3.25 and 5.25 m bgl., which measured total stresses during installation and equalization. The full-scale concrete pile (referred to as TCD1 in Table 1) was driven through the fill layer and penetrated under self-weight through the sleech. The load-displacement response of a second (identical) uninstrumented pile, referred to as TCD2, is examined in the section on load testing.

Average shear stress

The average shear stress (τ_{av} = shaft load/shaft area) mobilized during pile installation is shown in Fig. 6. While τ_{av} was quite variable in the upper sandy sleech, it reduced and became more uniform (for a given pile) when the piles entered the lower sleech. Values of τ_{av} generally reduced with increasing pile penetration, with the result that α ($= \tau_{av}/s_u$) reduced from maximum values of unity when the pile first

Fig. 5. Schematic of test piles.

Table 1. Details of pile tests at Kinnegar.

Pile No.	Type	B (mm)	Final depth bgl (m)	Notes
UCD1	Steel-circular	73	4.15	Installed in 100 mm jacking strokes. Static maintained load test performed 0.81 days after installation.
UCD2	Steel-circular	73	4.25	Installed in 100 mm jacking strokes to 2.9 m bgl, 200 mm strokes to 3.45 m bgl, and 400 mm strokes to 4.25 m bgl. Rate of installation was 20 mm/s. Two static load tests performed 9.5 days after installation. First test was a maintained load test, while a second was constant rate of penetration test performed at 0.44 mm/s
UCD3	Steel-circular	73	3.6	Installed in a single jacking strokes at a rate of 34 mm/s. No load tests reported.
UCD4	Steel-circular	73	4.7	Installed in 100 mm jacking strokes to a final depth of 4.7 m. Load tested 30 days after installation at a rate of 13 mm/s.
TCD1	Concrete-square	250	6	Installed by driving through fill and penetrated sleech under self-weight of pile and hammer. Static maintained load test in tension 99 days after installation
TCD2	Concrete-square	250	6	Installed by driving through fill and penetrated sleech under self-weight of pile and hammer. Static maintained load test in tension 85 days after installation

entered the lower sleech (at a depth of between 1 and 3 m bgl) to 0.25–0.5 at the end of installation. The average shear stress mobilized by the pile TCD1 during installation was estimated based on the self-weight of the pile and the hammer. The τ_{av} values measured on the model steel piles (UCD1 and UCD4), which were installed using 100 mm jacking and therefore experienced 28 and 33 jacking strokes (N), respectively, during installation were similar in the lower sleech (~ 5.5 kPa). The highest τ_{av} values were measured on the model pile installed using variable jacking stroke lengths (UCD2), which experienced 18 jacking strokes and the two monotonically installed piles (UCD3

and TCD1), which were installed with effectively only one jacking stroke ($N = 1$).

Radial total stress

The radial total stresses (σ_r) measured during the installation of pile UCD4 are shown in Fig. 7a. The σ_r values increased rapidly with increasing pile penetration, until the pile tip entered the lower sleech at 2 m bgl. Here, the rate of increase slowed, and the effect of friction fatigue is evident with a clear trend for σ_r in a given horizon to reduce as h/D increased is clear. The effect of the jacking stroke length on the (σ_r) values developed during installation was

Fig. 6. Average shear stress during installation.

investigated during the installation of pile UCD2 by increasing the jacking stroke length from 100 to 200, and 400 mm (See Fig. 7b). The radial stress measured at all levels on the pile was seen to increase when the jacking stroke length increased. The normalized radial total stress (σ_r/q_c) data from all pile installations at the site are compared in Fig. 7c, where a clear trend for the normalized radial total stress (σ_r/q_c) measured at a given h/D level to increase with jacking stroke length is observed.

The combined effect of N and h/D on the normalized radial stress developed during installation is considered in Fig. 7d, where it is clear that σ_r/q_c decreased as the number of loading cycles increased, although the σ_r/q_c value at a given N value, also decreased as h/D increased. This trend is clear in Fig. 7 for the piles installed using a single jacking stroke (i.e., UCD3 and TCD1).

Pore-water pressure measured during pause periods between jacking strokes

The peak excess pore pressures (Δu = maximum pore pressure – hydrostatic pore pressure) generated in pause periods between jacking strokes (i.e., when the pile was stationary), during the installation of pile UCD4, is shown in Fig. 8. Values of Δu were relatively low in the more permeable upper silt and increased significantly as the sensors entered the lower silt. A distinct h/D trend is evident in Fig. 8, with the Δu values at a given depth decreasing as h/D increased such that values at h/D of 5.5 and 10.5 were 8% and 18%, respectively, lower than the value at $h/D = 1.5$. Jacking stroke length was found to influence Δu values in a manner similar to the σ_r values when the jacking stroke length was varied during the installation of the pile UCD2.

Gibson and Anderson (1961) show that the excess pore-water pressure developed at the pile–soil interface during undrained installation can be estimated from the shear modulus, G , and s_u using the cavity expansion method (CEM) approach where

$$[5] \quad \Delta u = s_u \ln \frac{G}{s_u}$$

Although G is a strain dependent parameter, Randolph (2003) suggests that for lightly overconsolidated clay, the maximum pore-water pressure at the pile–soil interface is in the range 4–6 s_u . Given that the silt has a relatively constant s_u value with depth, this ratio equates to an operational G value in the range 1 to 10 MPa. This corresponds to G measured at triaxial shear strains in the range 0.003%–0.3% (Fig. 4), or G/s_u in the range 50–500. The Δu values measured at $h/D = 1.5$ are seen in Fig. 8 to approach the lower bound CEM prediction after the pile had penetrated a relatively short distance into the lower silt. However, in a given horizon, Δu values at $h/D = 5.5$ and 10.5 approached the lower bound CEM prediction, but only after relatively large pile penetration. It should be noted however, that the CEM method assumes ideal undrained penetration and does not include for any partial consolidation at points remote from the pile tip, because of the time taken to install the pile.

Pore-water pressure during a jacking stroke

During each jacking stroke, the pore-water pressure measured at the pile–soil interface changed continuously. The variation of the pore-water pressure response during a typical installation jacking stroke of the pile UCD4 is considered in Fig. 9. The response can be considered in three phases:

- (1) an initial phase where pile head settlement is low (<3 mm) and the pore-water pressure at all h/D levels remained largely unchanged
- (2) when the pile head settlement exceeded 3 mm ($\sim 5\%$ of the pile diameter D), the pore-water pressure values decreased rapidly, reaching minima when the pile head displacement was less than 10 mm
- (3) for displacement above 10 mm, the pore-water pressure values recovered at all levels and increased sharply when the pile head load was removed at the end of the jacking stroke

Similar changes in pore-water pressure response during jacking strokes were noted during installation of the ICP in both normally and overconsolidated clay deposits. Lehane and Jardine (1994) state that the phenomenon occurs because shear induced pore pressure changes are developed in a narrow shear zone adjacent to the pile shaft. These shear induced pore pressures are generally lower than the excess pore-water pressure in the soil mass outside the shear zone, which result from the increased mean stress due to undrained penetration of the pile.

Radial effective stresses

Although total radial stress typically increased slightly during each jacking stroke (Fig. 7), the continuous variation of the pore-water pressure at the pile–soil interface resulted in significant variations of the radial effective stress σ'_r , mobilized over each jacking stroke. The variation in σ'_r and total pile head load resistance measured during a complete jacking stroke are shown in Fig. 10. Since changes in σ'_r result largely from pore-water pressure changes, they therefore should be considered in the same manner:

Fig. 7. (a) Radial total stress during installation of UCD4. (b) Radial total stress during installation of UCD2. (c) Radial total stress during installation for all piles. (d) Combined effect of number of cycles and h/D on σ_r/q_c .

- (1) During the initial phase of loading, when the pile head displacement was less than 3 mm, the radial total stress increased slightly and the pore-water pressure did not change, thus leading to increases in radial effective stress.
- (2) At intermediate pile head displacement (between 3 and 10 mm), the pore-water pressures reached their minima. This was accompanied by further increases in radial total stress and consequent large increases in radial effective stress and a peak load resistance was developed.
- (3) As the pile head displacement continued, the gradual increase in both radial total stress and more significant increase in pore-water pressure led to continuous changes in radial effective stress along the pile shaft, with peak

radial effective stresses being attained, followed by reductions leading to a brittle load–displacement response.

Equalization

Dissipation of excess pore-water pressure

The variation of excess pore-water pressure measured on pile UCD2 during a 9.5 day equalization period is shown in Fig. 11. The dissipation of pore-water pressure is considered by plotting U_d against time, where

$$[6] \quad U_d = \frac{(u - u_0)}{(u_{\max} - u_0)}$$

where u , u_0 , and $u_{\max}$ are pore-water pressure at any time,

Fig. 8. Stationary pore-water pressure developed during installation of pile UCD4.

Fig. 9. Variation of pore-water pressure measured during a typical 100 mm jacking stroke.

the hydrostatic pore-water pressure, and the maximum pore-water pressure reached during installation, respectively.

The pore-water pressure increased during the initial stages of equalization as flow occurred towards the shear zone, and the maximum pore-water pressure u_{max} was developed within one minute of the end of the final jacking stroke. The rate at which pore-water pressure dissipation occurred was highest near the pile tip and it reduced as h/D increased, reflecting the more 3-D nature of drainage at the pile tip (Houlsby and Teh 1988). During installation of a pile with a 100 mm jacking stroke length (e.g., UCD4), the average time lag between the sensor at $h/D = 1.5$ reaching a given soil horizon and the sensors at $h/D = 5.5$ and 10.5 reaching the same depth, was approximately 10 and 20 min, respec-

Fig. 10. Variation of radial effective stress and load during a 100 mm jacking stroke.

tively. The pore-water pressure dissipation curve for the sensor at $h/D = 1.5$ (Fig. 11) reveals that the excess pore-water pressure at $h/D = 1.5$ decreased by 10%–15% over this time period. This suggests that the h/D effect (reduced pore-water pressure at large h/D values), evident in Fig. 8, was a result of partial pore-water pressure dissipation. The data in Fig. 11 were compared with theoretical pore pressure dissipation curves proposed by Randolph (2003) to estimate operational c_h values in the range 5 to 9 $m^2/year$. These compare reasonably with c_h values in the range 7 to 12 $m^2/year$ measured in piezocone dissipation tests.

Radial effective stress

The normalized radial effective stress measured at the end of equalization on piles installed using a range of installation methods (ranging from incremental jacking, installation using a single long jacking stroke, and installation under self-weight) are compared in Fig. 12. While it was clear that the installation procedure had a significant effect on the radial total stresses mobilized, the equalized radial effective stress (σ'_{rc}) values appear to be unaffected by the installation procedure (number or length of jacking strokes), with the σ'_{rc}/q_{cnet} ratio at a given h/D value being similar for all pile installations. A trend for the ratio σ'_{rc}/q_{cnet} to be highest near the pile toe is noted, although the ratio is relatively constant for h/D values above 5.5. This implies that the dominant effects of friction fatigue are confined to areas close to the pile base.

Static load tests

Both maintained load and constant penetration rate static load tests were performed on the piles. While multiple tests were performed on some of the piles (Gallagher 2006; Doherty and Gavin 2009), only the results of first-time loading

Fig. 11. Normalized pore pressure dissipation for pile UCD2.

Fig. 12. Normalized equalized radial effective stress on Kinnegar piles.

are discussed here. No load tests are reported for pile UCD3, as this pile formed part of a group of piles (installed after UCD3), which are discussed in detail by McCabe et al. (2008). The development of τ_{av} with pile head settlement (s) normalized by the pile diameter (D) during first-time static load test results are shown in Fig. 13a, where it is noted that:

- (1) A maintained load type static compression test was performed 20 h after the installation of test pile UCD1. The maximum τ_{av} value mobilized in the test of 4 kPa, which corresponds to an average value of $\alpha = 0.18$, was about 20% lower than the shear stress measured towards the end of installation of the pile. The normalized pore-water pressure dissipation curves (Fig. 11) suggest that

30%–50% of the excess pore-water pressure set-up during pile installation remained after 20 h (~1200 min). The high excess pore-water pressure would explain the low stiffness and resistance developed in the pile test.

- (2) A maintained load static compression test was performed 9.5 days after the installation of pile UCD2, when in excess of 90% of the excess pore-water pressure dissipation had occurred. The response was significantly stiffer than that evident from pile UCD1, and a peak shear resistance of 10.5 kPa ($\alpha = 0.48$) was achieved after a relatively large s/D of 0.13 (13% of the pile diameter). Surprisingly, this value was again about 20% lower than the shear stress mobilized during installation of the pile. Examination of the pore pressure sensors response during the static load test revealed minor changes with a small pore pressure increase of 5 to 10 kPa occurring during the load test. The pile was immediately retested in a constant rate of penetration compression test, where the pile was loaded at a rate of 0.44 mm/s. The pile behaviour during reloading was noticeably stiffer, and a brittle shear stress – displacement response was evident. A peak shear stress of 12.5 kPa, which is comparable to the installation resistance, was developed after a pile head displacement of 5 mm. This peak shear stress was associated with large reductions (~20 kPa) in pore-water pressure all along the pile shaft.
- (3) A constant rate of penetration compression load test was performed on pile UCD4, 30 days after installation, when the excess pore-water pressures had fully dissipated. The rate of loading was 13 mm/s. The mobilization of shaft resistance was similar to the other piles until the pile displacement approached $0.05D$ (5% of the pile diameter). Thereafter, the shaft resistance increased significantly until a displacement of $0.07D$, remained relatively constant until $s = 0.1D$, and then increased again. The average shaft resistance approached 22 kPa

Fig. 13. (a) Mobilization of shear stress during load tests. (b) Pore pressure response during load test on UCD4. CRP, constant rate of penetration.

Fig. 14. Equalized radial effective stress on UCD, TCD, and IC piles.

($\alpha = 1$) at a pile head displacement of $0.2 D$ (20% of the pile diameter). The changes in pore-water pressure that occurred during this constant rate of penetration (CRP) test are shown in Fig. 13b. A large reduction of pore-water pressure (up to 35 kPa) that developed all along the pile shaft as s/D exceeded 5% caused the increase in shear stress noted in Fig. 13a. Positive rate effects for the sleeve had been noted from instrumented pile tests, triaxial compression tests, cyclic direct shear tests, and ring shear tests by McCabe (2002), Lehane et al. (2003) and Doherty and Gavin (2009).

(4) Static tension tests were performed on piles TCD1 and TCD2, using the maintained load procedure, 99 and 85 days after installation, respectively. The shear stress – displacement response was similar to that measured for the model piles at $s/D < 5\%$, with α approaching 0.5 at the end of the test.

Discussion

Both the radial total stress and pore-water pressure mobilized along the pile during installation were strongly affected by the mode of installation. Both total stresses and stationary pore-water pressure values measured at the pile shaft were highest near the toe and increased when the jacking stroke length increased, while the radial total stress was also affected by the number of load cycles experienced. During installation and some of the fast static load tests, the pore-water pressure at the pile shaft were reduced after a pile head displacement by approximately 5% of the pile diameter. This resulted in a temporary increase in radial effective stress at the pile–soil interface and an increase in the pile shaft resistance. This increased shaft resistance appeared to be rate dependent, with faster jacking rates leading to a larger reduction in pore-water pressure. This was in keeping with positive rate effects observed from direct shear tests and load tests on concrete piles (Lehane 2003), and ring shear tests and load tests on steel piles (Doherty and Gavin 2009) at the test site.

While the radial effective stresses that control the pile shaft resistance were strongly affected by mode of installation and load test procedure, the equalized radial effective stresses, which control the long-term shaft resistance, were similar for all four models and two full scale piles installed at Kinnear. For the piles installed at this site, a strong cor-

relation between σ'_{rc}/q_{cnet} and h/D is hereby suggested of the form

$$[7] \quad \sigma'_{rc} = 0.26 q_{cnet} h/D^{-0.2}$$

The effect of friction fatigue (expressed through the h/D term) is significantly weaker than that reported for piles in sand by Jardine et al. (2005) and White and Lehane (2004). Volume changes that occur during cyclic loading are believed to be a significant contributory factor to friction fatigue in sand. Given the short loading times and undrained response at the pile–soil interface in the low permeability soils described in this paper, such effects are likely to be constrained, and this factor may explain the reduced significance of friction fatigue for piles in clay.

When the piles installed in sleech were loaded using maintained static load tests (slow loading), the shear stress – displacement response of fully equalized test piles was similar (with α approaching 0.5). During static load testing of the piles, the development of positive excess pore-water pressure at relatively low pile head displacement ($<5\%$ D) results in a reduction of effective stress and thus some reduction factor, f_L , in the range 0.8–0.9, should be applied when calculating the local shear stress, q_s , developed during static loading

$$[8] \quad q_s = f_L 0.26 q_{cnet} h/D^{-0.2} \tan \delta$$

It is of interest therefore to compare the ratio of σ'_{rc} normalized by the net cone resistance (q_{cnet}) from this site with those from the IC test sites in Fig. 14. Although the data at all sites appear similar, it is likely that friction fatigue effects would depend on the initial soil state, and a higher h/D exponent may be required in heavily overconsolidated deposits.

Conclusions

A series of tests were carried out on model and prototype instrumented piles installed in Belfast soft sleech, and the following conclusions can be made:

- (1) Radial total stress, pore-water pressure, and therefore shear stresses developed during installation were strongly affected by the installation method adopted.
- (2) Equalized radial effective stresses, which in most cases control the long-term static pile resistance, did not appear to be affected by pile installation technique. Values of equalized radial effective stress developed on the piles in Belfast sleech were comparable with those measured in a variety of other clays, once stresses were normalized by the net cone resistance and the effects of pile geometry were considered.
- (3) The effect of friction fatigue was evident, with both radial total stress measured during installation, and radial effective stress measured after equalization, being highest near the toe, and reducing as h/D increased. After equalization, the degree of friction fatigue experienced did not appear to be affected by the installation procedure. The reduction of the shaft resistance due to friction fatigue effects was lower for piles installed in clay than for piles installed in sand.
- (4) The trend for pore-water pressures set-up during installation to exhibit a friction fatigue or h/D effect, could be

explained by partial dissipation of excess pore-water pressure, which occurred as points remote from the pile tip experienced unloading as the distance from the pile tip increased.

- (5) Pore pressure reductions that occurred at large pile head displacements ($>5\%$ D) during fast loading of the pile resulted in the development of significantly enhanced ultimate pile shaft resistance. The increase in pile capacity appeared from the limited data to be rate dependent. This suggests that comparisons between the long-term shaft capacity of piles and the resistance developed during undrained installation or rapid load tests performed after equalization, should be performed with caution.

The experimental data available suggest that the key parameters controlling the distribution of shaft resistance on a displacement pile in clay are the q_c value, the constant volume interface friction angle, and friction fatigue. The fact that the dominant parameter q_c can be directly correlated to the undrained strength, s_u , explains the generally good predictive performance of α -design approaches (such as eqs. [1] and [2]). However, refinement of these approaches through the inclusion of the friction fatigue effect and to allow for the variation of the interface friction angle, through correlations of the form suggested in eq. [7], should result in a significant improvement in the reliability of design methods for estimating the shaft resistance of displacement piles in clay.

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List of symbols

c_v	vertical coefficient of consolidation
c_h	horizontal coefficient of consolidation
D	pile external diameter
f_L	XXXX
G	shear modulus
G_{25}	small modulus at 25% of maximum shear stress
G_m	XXXX
G_{max}	small strain shear modulus
h	height above the pile tip
I_p	plasticity index
L	pile length
N	number of loading cycles
N_k	XXXX
N_{kt}	factor relating corrected cone end resistance and undrained shear strength
q_b	end bearing resistance of pile
q_c	end bearing resistance measured during cone penetration test
q_{cnet}	XXXX
q_{net}	cone resistance corrected for pore-water pressure, area and stress effects
q_s	local shear stress
q_t	cone resistance corrected for pore-water pressure effects and area effects
s	pile head displacement
s_u	undrained shear strength
s_{u-vane}	XXXX
U_d	XXXX
u	pore-water pressure
u_0	free field (hydrostatic) pore-water pressure
u_{max}	maximum pore-water pressure
α	empirical factor relating average shaft friction and undrained strength
Δu	excess pore-water pressure
$\delta_p, \delta_f, \delta_r$	interface friction angle at peak, failure, and residual, respectively
σ_r	radial total stress
σ'_r	radial effective stress
σ'_{re}	equalized radial effective
σ'_{rf}	radial effective stress at failure
σ_{ri}	XXXX
σ_v	vertical total stress
σ_{v0}	XXXX
σ'_{v0}	free field vertical effective stress
τ_{av}, τ_f	average and peak local shear stress, respectively, acting on the pile shaft