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Evaluation of installation effects on the set-up of field displacement piles in sand

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Abstract

The paper presents results from a new series of tests on displacement piles in sand, involving different installation modes, and combines these with results from previous tests at the same site as well with test data at two other well investigated sand sites to provide fresh insights into factors affecting ‘short term’ capacity and set-up of shaft friction. It is shown that the shaft capacity measured shortly after installation reduces systematically with the logarithm of the number of impact blows or jacking increments per unit shaft area imparted during installation. However, the degree of set-up of shaft friction for piles increases with an increase in the number of blows, and piles installed using a large number of blows can attain highest ‘long term’ shaft capacities, despite having the lowest ‘short term’ capacity. The tests indicated that the driving impact frequency had a relatively small influence on shaft friction, while piles installed by vibration attain ‘short term’ capacities comparable to driven impact piles but showed negative set-up.

Keywords

Installation method, time effects, capacity, pile foundations, sand

Introduction

Piled foundations are an integral, but an expensive, component of the ever-increasing global construction market. As such, cost savings associated with the inclusion of the benefits of the post-installation increase in pile capacity with time are immense. This increase, commonly known as set-up, is a widely accepted phenomenon for driven piles in clays and is incorporated in their design as the primary governing mechanism involving dissipation of excess pore pressures is understood. Tavenas and Audy (1972) reported one of the first cases of pile set-up in sand, measuring 70% increase in capacity of concrete piles in the first 20 days after driving. Since then, evidence of a positive gain in capacity with time emerged from well-documented pile testing case histories in sand (e.g., Axelsson 2000; Chow *et al.* 1998; Gavin *et al.* 2013; Jardine *et al.* 2006; Karlsrud *et al.* 2014; Skov and Denver 1988; Åstedt *et al.* 1992). Even so, despite the growing body of empirical evidence showing that set-up for driven piles in sand can be even more significant than in clays, substantial variability in the observed levels of capacity gain with time and a general lack of understanding of the basic mechanisms are such that set-up in sand is usually ignored in standard design practice.

An example of the scatter in observed set-up is provided in Figure 1a, which presents results from a database of pile tests in sand compiled by Chow *et al.* (1998) and plots the variation with time of the pile capacity normalized by a measured or inferred capacity after 1 day. These results show that there is an average increase in capacity of about 50% per log cycle of time. Lim and Lehane (2015a) show that uncertainties associated with (a) combining static and dynamic load tests in the same database, (b) multiple testing of the same piles, (c) the inference of shaft friction from compression tests and (d) the estimation of the 1 day capacity contribute to the degree of scatter shown in Figure 1a. The same authors also showed that the interpreted trend for an increase in capacity with time depends critically on the reference time

(which is 1 day in Figure 1a) and that normalizing measured shaft capacities with the capacity calculated using one of the existing design methods provides more consistent trends.

In the light of the uncertainties incorporated in Figure 1a, a series of full scale tests involving first-time tension loading of driven pipe piles was undertaken at three European test bed sites, namely Dunkirk (Jardine *et al.* 2006), Blessington (Gavin *et al.* 2013) and Larvik (Karlsrud *et al.* 2014). The capacities measured at various times after pile driving are normalized by the capacity calculated using the UWA-05 method (Lehane *et al.* 2005) and plotted against the set-up time in Figure 1b. The results indicate a consistent trend with tension capacity reaching a typical design calculated capacity after about 20 days but increasing to about 2.5 times this value after about a year. Such a high level of set-up, which was reasonably consistent across the three sites, prompted Lehane *et al.* (2017) to suggest application of the following time correction factor (F_{time}), which is also plotted in Figure 1b, to the capacity calculated using any of the American Petroleum Institute (API) Cone Penetration Test (CPT) methods (API 2006) :

$$F_{time} = \frac{1}{\exp(-0.1 \times t^{0.68}) + 0.45} \quad (1)$$

where t is the time in days after installation.

The function described in Equation (1) is a design recommendation, erring on the conservative side of the data points. Jardine *et al.* (2006) present a similar correction factor, which is a piecewise function, referred to as intact ageing characteristic (IAC). It is clear that application of ageing trends, as represented by Equation (1) or the IAC, would lead to significant economies in practice. However evidence such as that provided by Lim and Lehane (2015b) and Carroll *et al.* (2017), indicating that little or no set-up occurred in certain circumstances, suggests caution and prompted the investigation presented in this paper which

involved first time tension loading of pipe piles installed using three impact/vibration installation modes. The tests were performed at the Shenton Park sand site where there has been a considerable number of previous pile tests with different installation modes (Lehane 2008; Lim 2013; Schneider 2007; Xu 2007). This paper presents a comparison of the trends from all Shenton Park tests and first-time tension tests from other sand sites, enabling conclusions to be drawn regarding the relationship between shaft capacity, installation mode and set-up time. The experimental tests are described after a brief overview of a range of perspectives relating to the causes for set-up of shaft friction in sand.

Background

A number of mechanisms to explain set-up of displacement piles in sand have been hypothesized but it is clear that the phenomenon is not yet fully understood and is difficult to quantify with any level of confidence. Chow *et al.* (1998), for example, proposed that set-up may be due to (i) increases in radial effective stress (σ'_r) on piles after installation, (ii) corrosion or bonding of sand particles to the pile shaft increasing friction and (iii) increases in the strength and stiffness of the sand mass around the pile. The authors disregarded corrosion as a major cause since reports in the literature confirm capacity gains in steel, timber and concrete piles, both above and below the corrosion zone. Laboratory model pile tests reported by Robinsky and Morrison (1964), Allersma (1988) and Chong (1988) suggested that compaction and particle breakage at the pile tip during installation creates a thin zone of loose sand around the pile shaft. As a result, Chow *et al.* (1998) argue that hoop stresses develop by arching and shield the pile from a region of high radial stresses some distance from the pile wall, but that creep induced relaxation of the arching subsequently allows gains in radial effective stresses close to the pile. However, recent laboratory model tests reported by Jardine *et al.* (2013) and Rimoy *et al.* (2015) confirmed that while arching did occur, it did

not provide an explanation for the low levels of set-up observed in their series of tests on jacked piles.

Mitchell (2008) discusses the ‘enigma of sand ageing’, providing many examples of its effects on stiffness, penetration resistance and liquefaction potential. No clear trends emerge and, given that the rate of property improvement varied considerably across the case histories examined, it was concluded that ageing is site and sand type specific. Laboratory element testing (e.g., Chow 1997; Joshi *et al.* 1995) also display a dependence on sand type but consistently show that prolonged periods of ageing lead to a more dilatant response on subsequent shearing. This characteristic is considered by Bowman and Soga (2005) to be a significant contributor to pile set-up because of the restraint to volume change of the sand adjacent to the pile shaft; such increases in lateral stress were recorded on the precast concrete piles tested by Axelsson (2000), although much of the σ'_r data recorded were obtained in re-tests. Lim and Lehane (2015b) recorded little change in radial stress increase ($\Delta\sigma'_r$) on jacked piles, which was consistent with the virtual absence of set-up shown by these piles. This contrasting response with the large set-up of other displacement piles, such as indicated in Figure 1, and the absence of set-up for bored piles in sand, led Lim and Lehane (2014) to propose expressions relating expected levels of set-up to the disturbance imparted by a given installation mode.

Recent discrete element modelling of pile set-up in sand (e.g., Zhang and Wang 2016) has been successful in replicating the laboratory-observed effects of increased dilatancy and particle breakage and suggests that ageing is likely to be caused by small particle movements and rotations occurring after some disturbance (such as the installation of a pile). These movements and rotations do not cause a significant change in void ratio but lead to a stiffer soil fabric and a more diffuse arrangement of load chains with less heavily loaded contacts. It therefore follows that set-up may indeed be largely due to hypothesis (iii) suggested by Chow

et al. (1998), mentioned above, and that the varying levels of observed pile set-up reflect differences in values of $\Delta\sigma'_r$ during pile loading because of different installation modes and their relative influence on different sand types and states. Gavin *et al.* (2015) provide a useful summary of current perspectives on set-up of pile shaft friction.

Studies of set-up conducted at small scale in laboratory testing chambers such as those of Jardine *et al.* (2013), Lim and Lehane (2014a) and Rimoy *et al.* (2015) have indicated set-up trends that are often different and less pronounced than those seen in the field. Several factors are likely to contribute to such differences, including the use of freshly deposited sand in testing chambers, scale effects, boundary conditions, and stress state. Therefore, to eliminate any uncertainties regarding laboratory experiments, this paper only considers trends indicated in field tests and excludes chamber tests from the database under consideration.

Experimental Programme

Site Conditions

The new series of pile tests described in this paper were conducted at the University of Western Australia (UWA) Shenton Park sand site, which has been comprehensively investigated using a range of in situ and laboratory tests (e.g., Lehane *et al.* 2004; Schneider *et al.* 2008) as well as being used for a number of field experiments including driven piles (Schneider 2007; Xu 2007). The stratigraphy at the site comprises a 5 m to 7 m thick deposit of aeolian siliceous sand overlying weakly cemented limestone. Sand grains vary from sub-angular to sub-rounded with D_{50} , D_{60} and D_{10} values of 0.42 ± 0.02 mm, 0.47 ± 0.02 mm and 0.21 ± 0.01 mm respectively and a fines content of less than 5%. Maximum and minimum void ratios are 0.81 and 0.45 respectively.

Cone Penetration Tests (CPTs) were performed in the vicinity of the new series of pile load tests, as indicated in Figure 2. The first set of CPTs (CPT1 to CPT3) were performed in April

2017 one week prior to the start of pile testing and the second set (CPT4 to CPT6) took place in August 2017 at the completion of the testing series. The CPT end resistances (q_c) traces and the corresponding profile of the coefficient of variation are plotted in Figure 3. It is evident that variability is typically 20% in the upper 1.5 m of the deposit where q_c ranges from 5 to 10 MPa. The variability in q_c below this depth to the toe level of the piles is very small, with the mean trend increasing from 4 MPa at 1.7 m to 5.5 MPa at 4 m. Sand replacement density tests at 2m depth indicated that the sand had a bulk density of 1670 kg/m³, water content of 3.5% and a degree of saturation of 14%. These indices indicate a relative density of 45% at this depth.

An interesting characteristic of the Shenton Park site is that the saturation level in areas which have trees varies slightly from the end of the wet season to the end of the dry season and this affects the in situ stress state and stiffness due to suction (Lehane *et al.* 2004). Byrne and Randolph (2003) reported results of dynamic and static testing on small pipe piles and found that pile capacities were also affected by seasonal variations. No trees are present in the area of the test site described here and therefore such seasonal effects are not expected. The similarity between CPTs 1 to 3 conducted prior to the start of the wet season and CPTs 4 to 6 conducted following 480 mm of rain, 4 months later, confirm the absence of a seasonal effect.

Pile testing details

Details of the pile testing programme are summarized in Table 1. Eight 165 mm diameter open-ended pipe piles, labelled SP26 to SP33, were installed to an embedment (L_{emb}) of 4 m in the test area. The piles were galvanized mild steel tubes, which were sandblasted to a centerline average roughness of about 15 μ m. A steel headpiece was used as an anvil to minimize damage to the pile body during impact driving and to facilitate attachments for the execution of tension tests. A special headpiece was also manufactured for piles SP32 and

SP33, which included a 200×200×25 mm plate to allow the vibrator to clamp to the pile. Three different installation methods were employed: (i) impact driving using a conventional drop weight on piles SP26 to SP28, (ii) impact driving with high frequency using an air hammer on piles SP29 to SP31 and (iii) vibration. The specification details of the installation equipment are provided in Table 2.

After pitching of piles SP26 to SP28 using an air hammer, impact driving of these piles was performed with the drop weight on a winch system attached to a cross bar at the top of scaffolding. A quick release pin was operated manually from the scaffolding at certain levels, allowing the operator to release the weight and attach it again after resting on the pile head. Pile SP27 was installed using a 1.5 m drop height, but this led to some damage at the pile head and therefore the height was reduced to 0.8 m for SP26 and SP28. These piles took approximately 4.5 hours to install because of the manual nature of the process. Piles SP30 and SP31 were installed over a period of 1-2 minutes with no disruptions at a frequency of around 6 blows/s, while a lower frequency of 5 blows/s was used for SP29.

Blow counts were recorded for each 100 mm driving increment and these are plotted in Figure 4 for piles SP26 to SP31. It is interesting to note that, on average, the number of blows for installation with the impact hammer are similar to those of the air hammer, despite the latter imparting less energy per impact. Pile SP29 required nearly twice as many blows, possibly due to several interruptions that occurred during installation. Measurements of the soil plug length during pile installation was not generally possible due to the experimental set-up, but final plug length ratios (*PLRs*) were measured at the end of installation.

Static load tests

Static tension load tests were performed on the piles to evaluate their ‘short-term’ capacity after about 6 days and their longer term capacity after 4 months (~117 days). Each pile was

subjected to a single test and therefore uncertainties raised by Gavin *et al.* (2015) and others, regarding the lower level of set-up in re-tests compared to first-time loading were avoided.

The arrangement employed for tension testing is shown in Figure 5. A high strength steel frame with a pyramidal arrangement and 0.5 m wide footings was established around each pile, providing the reaction force to pull the pile out of the ground. The footings were located a clear distance of in excess of 5 pile diameters either side of the test piles, ensuring minimal interaction. Loading was applied using a hydraulic jack reacting against a nut placed at the end of a high tensile strength threaded rod. This rod was connected to the head piece at the pile top and passed through the loading frame, hollow load cell and the jack. A standard reference beam arrangement, as indicated in Figure 5, was employed to allow pile head displacements to be measured with displacement transducers (DTs). The uplift force was applied in (small) increments of 5 kN, each of which was maintained for 10 minutes to allow for stabilization of displacement before the next load increment was applied. The data logger recorded readings from the load cell and displacement transducers at 5-second intervals.

Average shaft shear stress-displacement responses measured during all first-time load tests conducted at specific ageing periods on three types of pile are shown in Figure 6. Chin's hyperbolic extrapolation (Chin 1970) was applied to determine ultimate shaft shear stresses (provided in Table 3) at 10% of the pile diameter ($=16.5$ mm). In contrast to the trends indicated in Figure 1, it is seen that there is generally little difference in the short-term load-displacement responses and capacities (measured at 6-7 days) with those measured after a set-up period of 117 ± 2 days. In addition, it is evident that the installation mode has a small influence on the capacities developed. The capacities of the impact driven and air hammer driven piles show modest increases from 18 kPa to 20 kPa and from 23 kPa to 26 kPa respectively over the set-up period. However, the shaft capacity of the vibrated pile actually

reduces from 26 kPa to 22 kPa over this period and the longer-term load-displacement response appears more brittle.

Shenton Park Field Test Database

Figure 6 indicates trends that are apparently in marked contrast with the high levels of set-up shown in re-tests on other impact driven piles at the same site, reported by Schneider (2007). In all, a total number of 33 pile tests have been performed at Shenton Park between 2005 and 2017 with a typical embedment of 4 m. Of these, 31 tests are on displacement piles with diameters ranging from 42 mm to 165 mm, while the two other tests are on bored piles with diameters of 225 mm and 340 mm. A summary of all Shenton Park pile test data is provided in Table 3.

In keeping with the contention discussed previously and proposed by Lim and Lehane (2014b) that the disturbance caused by installation is the primary factor giving rise to ageing and pile set-up, a range of different factors were examined for the database of Shenton Park tests. The most consistent trend emerged for the ‘short term’ capacity (taken as the capacity developed between 1 day and 7 days) when this disturbance was quantified as the number of blows or jacking cycles per unit external area of pile shaft (N_{bA}). This trend is shown in Figure 7a (excluding the vibrated pile) and shows a clear tendency for an approximate linear reduction in ultimate shaft friction with the logarithm of N_{bA} , with τ_{av} being largest for the bored piles (with no cycles) and lowest for the pile SP8 with $N_{bA}=3888$.

As discussed with reference to Figure 1b, to allow for different pile lengths, pile diameters and modest changes in the CPT q_c profiles, all of the measured short term capacities of standard impact driven or jacked displacement piles at Shenton Park are normalized by the capacity calculated using the UWA-05 method (τ_{UWA-05}) and plotted against N_{bA} in Figure 7b. A similar pattern to that observed in Figure 7a emerges and it is evident that the applied N_{bA}

value has a profound effect on the short term capacity. This trend is also consistent with the tendency for reduced shaft friction with number of installation cycles shown by White and Lehane (2004) in centrifuge tests, which could be explained by higher levels of contraction of sand at the shaft interface for greater levels of cycling.

The Shenton Park database includes measured shaft capacities with set-up periods of between about 40 days and 376 days (with a mean of about 205 days). These are referred to here as ‘long term’ capacities, although it is acknowledged that the set-up periods are variable and those with only 40 days set-up (which are jacked piles) would be better described as ‘medium term’ capacities. It is also noted that some of the data include re-tests on piles, which may have capacities less than those if they were loaded for the first-time (Jardine *et al.* 2006).

The combined set of normalized ‘short term’ and ‘long term’ capacities at Shenton Park is plotted against N_{bA} in Figure 8a and against the time after installation in Figure 8b. Data from the field study by Schneider (2007) include re-tests on driven piles (piles 1-9 in Figure 8a), while jacked piles (Lim and Lehane 2015b) and driven piles from the current (2017) study are first-time tests (note that pile ‘pairs’ in Figure 8a are: 10-12, 13-16, 17-19, 20-21, 22-23, 26-27, 29-30). The trend observed is essentially the same as that shown by Lim and Lehane (2014) where the ratio of ‘long term’ to ‘short term’ capacity increases with an increased level of disturbance, where the level of disturbance, in this case, is quantified by N_{bA} . It is of interest to note that, when plotted within this framework, the relatively small set-up shown by the recent test on the piles installed by drop hammer impact (SP27) is reasonably consistent with the general set-up in view of its relatively low N_{bA} value. It is also apparent that, despite the very low post-installation capacities of piles with high N_{bA} values, their ‘long term’ capacities can exceed those of piles with high ‘short term’ capacities (and low N_{bA} values).

Figure 8b presents the effect of time on measured shaft capacities. Jacked piles, which experience the lowest level cycling (SP10-22), have the highest short-term capacity ratios but experience a minor change with time. The heavily cycled piles (SP1-9) indicate relatively low capacity ratios 3 to 5 days after installation but much higher capacities after about a year. The new SP piles (SP26 and SP29), which have a moderate level of cycling, show lower levels of set-up which, as for the jacked piles, corresponds with relatively high ratios of shaft capacity measured shortly after installation.

Vibrated vs air hammer vs drop hammer installation

The air hammer driven piles at Shenton Park (SP29 and SP30) involved about 160 low impact blows per meter penetration and it appears from the similarity of their capacities to the vibrated piles (SP32 and SP33) that the mode of installation led to a stress state around the piles comparable to that of the vibrated piles (which were installed at a frequency about five times faster). It is noteworthy, however, that the capacity of the vibrated piles reduced by 15% between 7 days and 117 days; this characteristic has also been observed by Borel *et al.* (2006) and Doherty (2015) and suggests that vibration leads to a different sand structure to that created by standard impact driving. The number of blows required to install the air hammer and standard drop hammer piles is broadly similar (Figure 4), although the former were installed at a much high frequency and have 30% higher capacity; the proportional gain in capacity with time for both pile types is comparable.

Comparison with other test bed sites

Short term capacity

As for the Shenton Park site, the dense marine sand at Dunkirk, France, and the very dense heavily over-consolidated sand at Blessington, Ireland, have been used for extensive field testing of piles and to assist with studies of pile set-up. Table 3 includes all relevant details

concerning these pile tests and associated references, which includes 8 piles at Dunkirk (labelled DK1 to DK8) and 13 piles at Blessington (BL1 to BL13). Recent tests results on short 50 to 60 mm diameter piles at these sites (DK7-8, BL11-13) are discussed separately.

The ‘short term’ capacity of the Dunkirk piles in the database corresponds to that measured at a mean set-up time of 5 days while the mean value time for the Blessington piles is 1 day. The ‘short term’ capacities at these sites are plotted together with the ‘short term’ Shenton Park tests in Figure 9 using the same format as Figure 7. Despite the range of pile configurations, sand densities and levels of over-consolidation, it is seen that the general trend for piles at all sites is the same with ‘short term’ capacities being comparable to the capacity calculated with the UWA-05 method at low N_{bA} values but reducing to about 60% of the UWA-05 capacity at $N_{bA}=1000$.

Influence of plugging

It is of interest to examine the relationship between N_{bA} and the development of the sand plug during driven installation of the pipe piles in the database. The database measurements of the plug length ratio with N_{bA} are plotted in Figure 10. It is seen that there is a general trend for relatively low N_{bA} values to correspond to a semi-plugged condition ($PLR \sim 0.4$) with progressively much higher N_{bA} required as the PLR increases towards a fully coring value. This trend is counter-intuitive as it may be expected that fully coring piles are easier to drive. However, N_{bA} is not related to the energy input and the plotted trend simply indicates that larger driving hammers that promote greater penetration per blow give rise to more plugged conditions. A comparison of Figure 7 with Figure 10 therefore suggests that greater ‘short term’ capacity can be achieved using a larger sized driving hammer.

Long term vs short term capacity

The median longer term capacities at Dunkirk and Blessington were measured at 230 days (with exception of piles reported by Chow *et al.* (1997) where it was 1990 days) and these are compared in Figure 11 with the respective ‘short term’ capacities at these sites as well as at Shenton Park. The ratio of the ‘long term’ capacities (which are acknowledged to be ‘medium term’ capacities in some instances) to the ‘short term’ capacities are plotted against N_{bA} and reflect the same trend seen in Figure 8. There is little gain in capacity with time for piles with relatively low N_{bA} values whereas, despite some scatter, long term capacities are typically 3 times the ‘short term’ capacity at $N_{bA} \sim 1000$. It may also be inferred from Figure 8 that piles driven with smaller sized hammers that are more prone to coring during installation have higher levels of set-up (as shown, for the example for largely coring piles included in Figure 1b), but clearly also have low ‘short term’ capacities.

Piles DK7-8, BL11-13

Carroll *et al.* (2017) report on recent tests conducted on short, small diameter pipe piles at Dunkirk (DK7 to 8) and Blessington (BL11 to 13). The Dunkirk piles were 51 mm in diameter, required 687 blows/m² to reach a final depth of 2 m while the Blessington piles were 60 mm in diameter, and required 64 blows/m² to reach a final embedment of 1.75 m, Carroll (2017). Unusually, the ‘short term’ capacity at 1 to 2 days of both piles was about 80% greater than the UWA-05 capacity and they are therefore omitted from Figures 9-11. Despite this feature, the Blessington piles showed no set-up, which is consistent with lack of set-up expected from Figure 9 for $N_{bA} = 64$. This relatively low N_{bA} value arose because of the use an over-sized hammer and a resulting installation that was more akin to installation by jacking. The Dunkirk piles setup by more than 70% at 85 days, which is also consistent with $N_{bA} = 687$.

Summary

In summary, it appears that the number of installation blows (or jacking increments) has a critical effect on the 'short term' capacity with greater 'short term' capacity being possible for piles which employ larger sized hammers. However, piles with a large number of installation blows evidently create significant disturbance to the sand around the pile shaft and the recovery from this disturbance (or ageing) can lead to 'long term' capacities that are greater than the 'short term' capacities of piles driven with a small number of blows. Care should therefore be exercised in application of the time correction factor given by Equation (1), which based on the trends shown in Figure 9 and Figure 10, is likely to only be applicable to larger diameter piles with high PLR and N_{bA} values.

It is also important to emphasize that the line in Figure 11 is indicative of a trend based on the observed field data and is not a design line.

Conclusions

The results from a new series of tests on displacement piles combined with results from previous tests at the same site as well as tests at two other well investigated test sites have revealed the following clear trends, which explain much of the perceived variability of set-up characteristics of piles in sand:

1. The 'short term' shaft capacity (i.e. capacity within a number of days of installation) of standard driven piles (or jacked piles) depends critically on the number of installation blows (or jacking increments) normalized by the external shaft area, N_{bA} . Average shaft frictions reduce approximately with logarithm of N_{bA} .
2. The 'long term' shaft capacity (measured after typically 40 to 350 days) is similar to the 'short term' capacity for lower N_{bA} values. However, the 'long term' capacity for

piles installed using many blows can exceed the ‘short term’ capacity despite having relatively low ‘short term’ capacities.

3. Although the value of N_{bA} has a strong influence on the ‘short term’ capacity of typical impact driven and jacked piles, vibrated piles and piles installed with high frequency impact (with an air hammer) have a slightly higher ‘short term’ shaft capacity than piles installed under impact at standard frequencies. These capacities were less than design capacities calculated using UWA-05 and the capacity of the vibrated piles actually fell by 15% over a four-month period after installation. These results highlight the sensitivity of shaft friction to the particular installation mode.
4. CPT design methods (e.g. UWA-05) can greatly over-predict the ‘short term’ capacity of driven piles in sand, but are likely to under-predict their ‘long term’ capacity. Time correction factors, such as Equation 1, need to be applied but should incorporate a measure of installation disturbance, such as the N_{bA} parameter suggested here.

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Notation

CoV	coefficient of variation
d_p	displacement to reach peak shaft friction
D	outer diameter
D_{10}, D_{50}, D_{60}	effective particle sizes
F_{time}	time correction factor
H	height
L_{emb}	pile embedment length
m	mass
N_{bA}	blows or jacking cycles per unit external area of pile shaft
PLR	plug length ratio
q_c	cone tip resistance
$q_{c,av}$	average cone tip resistance
Q_m	measured pile shaft capacity
Q_T	total pile capacity
Q_{T0}	initial (reference) pile capacity
Q_{UWA-05}	pile shaft capacity estimated using UWA-05 method
s_{av}	average set per blow
t	time after installation
t_w	pile wall thickness
$\Delta\sigma'_r$	radial stress increase
σ'_r	radial effective stress
τ_{av}	peak average shaft shear stress
τ_{UWA-05}	peak average shaft shear stress calculated using UWA-05 method

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Figure 1 Observations of pile ageing in sand: (a) Case histories of increase in total pile capacity with time (after Chow *et al.* 1998, with permission from ASCE) and (b) Set-up of full-scale driven piles in sand (after Lehane *et al.* 2017).

217x295mm (300 x 300 DPI)

Figure 2 Site plan showing relative testing locations for driven piles and cone penetration tests (CPTs) at Shenton Park.

114x79mm (300 x 300 DPI)

Figure 3 Measured CPT profiles and CoV at Shenton Park.

186x175mm (300 x 300 DPI)

Figure 4 Driving records showing: (a) blow count for drop weight installation and (b) blow count for air hammer installation.

196x187mm (300 x 300 DPI)

Figure 5 Schematic of tension test set-up – side view (not to scale).

56x19mm (300 x 300 DPI)

Draft

Figure 6 Average shaft shear stress-displacement curves from first-time tests: (a) drop weight installation; (b) air-hammer installation; (c) vibratory installation.

230x492mm (300 x 300 DPI)

Figure 7 Influence of the number of installation cycles on 'short term' shaft capacity of Shenton Park piles:
(a) average shear stress and (b) average shear stress normalized by UWA-05 capacity.

270x388mm (300 x 300 DPI)

Figure 8 'Short' and 'long' term shaft capacity of Shenton Park piles with respect to: (a) number of cycles per unit external area of pile shaft (N_{bt}) and (b) time after installation.

271x389mm (300 x 300 DPI)

Figure 9 Change of the 'short term' shaft capacity ratio with N_{bA} .

134x95mm (300 x 300 DPI)

Figure 10 Relationship between plug length ratio and N_{bA} .

134x96mm (300 x 300 DPI)

Figure 11 Variation of the degree of set-up with N_{bA} .

134x96mm (300 x 300 DPI)

Table 1 Pile details, testing programme and summary of installation data

<i>Pile ID</i>	<i>L (m)</i>	<i>D/t_w</i>	<i>Installation method</i>	<i>Installation date (2017)</i>	<i>Average driving frequency [blows/min]</i>	<i>Average drop height [m]</i>	<i>Total blow count [-]</i>	<i>PLR [%]</i>	<i>Loading date (2017)</i>	<i>Load at 10% D [kN]</i>
SP26	4	33	Drop weight	3 May	1.5	0.704	374	56	9 May	37.9
SP27	4	33	Drop weight	2 May	1	1.182	280	54	23 August	42.6
SP28	4	33	Drop weight	8 May	2	0.85	246	58	-	-
SP29	4	33	Air hammer	27 April	324	-	661	58	4 May	48.3
SP30	4	33	Air hammer	27 April	370	-	469	61	23 August	55.3
SP31	4	33	Air hammer	27 April	373	-	423	60	-	-
SP32	4	33	Vibration	27 April	1200	-	-	50	4 May	54
SP33	4	33	Vibration	27 April	1200	-	-	62	21 August	54.0

Table 2 Pile Installation Equipment

<i>Air hammer</i>	<i>Vibrator</i>	<i>Drop weight system</i>
NPK-PH400	NPK-HP7SX	solid steel cylinder, m=103.6 kg
0.6 MPa air pressure	1050 kg operating weight	4 m high aluminum scaffolding
200 mm stroke	max frequency 40 cycles/s	quick release pin
max impact frequency 9 blows/s	vibration force 104 kN	winch system

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Table 3 Characteristics of the test piles considered at three sites: Shenton Park, Blessington and Dunkirk

Site	Pile ID	Installation method	Pile material	Pile end condition	D_0 [mm]	t_w [mm]	L_{emb} [m]	$q_{c,av}$ [MPa]	PLR	Σ cycles	N_{bA}	S_{av} [mm/blow]	Age [days]	τ_{av} [kPa]	τ_{UWA-05} [kPa]	d_p [mm]	References
Shenton Park	SP1	D	Steel	O	88.9	2.6	4	6.05	0.76	1642	1470	2.44	4	16	46	9.8	Schneider (2007)
													376	59	46	2.7	Schneider (2007)
Shenton Park	SP2	D	Steel	O	88.9	2.6	2.5	2.79	0.64	305	437	8.20	5	16	33	9.0	Schneider (2007)
													259	58	33	3.8	Schneider (2007)
Shenton Park	SP3	D	Steel	O	88.9	5	4	5.7	0.76	1840	1647	2.17	3	23	45	10.1	Schneider (2007)
													374	77	45	3.3	Schneider (2007)
Shenton Park	SP4	D	Steel	O	114.3	3.2	4	5.57	0.88	1830	1274	2.19	3	20	39	11.5	Schneider (2007)
													374	60	39	2.4	Schneider (2007)
Shenton Park	SP5	D	Steel	O	88.9	3.2	4	5.79	0.75	1486	1330	2.69	3	26	44	9.3	Schneider (2007)
													374	77	44	4.6	Schneider (2007)
Shenton Park	SP6	D	Steel	O	42.4	2.6	4	6.11	0.44	330	619	12.05	4	15	68	4.2	Schneider (2007)
													376	98	68	2.7	Schneider (2007)
Shenton Park	SP7	D	Steel	O	42.4	2.6	2.5	2.74	0.43	90	270	27.7	5	17	46	6.5	Schneider (2007)
													259	74	46	2.7	Schneider (2007)
Shenton Park	SP8	D	Steel	C	88.9	5	4	6.2	0	4344	3888	0.92	4	7	55	2.3	Schneider (2007)
													375	79	55	2.9	Schneider (2007)
Shenton Park	SP9	D	Steel	C	88.9	2.6	2.5	2.5	0	451	646	5.54	5	7	34	7.6	Schneider (2007)
													259	59	34	1.7	Schneider (2007)
Shenton Park	SP10	J	Steel	C	101.5	-	3.72	5	0	15	13	250	1	46	45	14.5	Lim & Lehane (2014)
Shenton Park	SP11	J	Steel	C	101.5	-	3.75	5	0	15	13	250	29	48	45	19.8	Lim & Lehane (2014)
Shenton Park	SP12	J	Steel	C	101.5	-	3.1	4.9	0	31	31	100	72	48	44	19.8	Lim & Lehane (2014)
Shenton Park	SP13	J	Steel	C	65	-	3.14	3.5	0	31	48	100	1	46	46	13.1	Lim & Lehane (2014)
Shenton Park	SP14	J	Steel	C	65	-	3.2	3.5	0	32	49	100	27	43	47	16.7	Lim & Lehane (2014)
Shenton Park	SP15	J	Steel	C	65	-	3.2	3.5	0	32	49	100	29	50	47	20.6	Lim & Lehane (2014)
Shenton Park	SP16	J	Steel	C	65	-	3.1	4.9	0	31	49	100	70	56	53	18.9	Lim & Lehane (2014)
Shenton Park	SP17	J	Steel	C	134.6	-	3.16	3.6	0	32	24	100	1	37	34	7.5	Lim & Lehane (2014)

Site	Pile ID	Installation method	Pile material	Pile end condition	D_0 [mm]	t_w [mm]	L_{emb} [m]	$q_{c,av}$ [MPa]	PLR	Σ cycles	N_{bA}	S_{av} [mm/blow]	Age [days]	τ_{av} [kPa]	τ_{UWA-05} [kPa]	d_p [mm]	References
Shenton Park	SP18	J	Steel	C	134.6	-	3.16	3.6	0	32	24	100	29	39	34	21.9	Lim & Lehane (2014)
Shenton Park	SP19	J	Steel	C	134.6	-	3.1	4.9	0	31	24	100	71	43	41	10.1	Lim & Lehane (2014)
Shenton Park	SP20	J	Steel	C	65	-	3.49	5.3	0	35	49	100	1	52	71	17.1	Lim & Lehane (2014)
Shenton Park	SP21	J	Steel	C	65	-	3.57	5.3	0	36	49	100	38	53	71	18.4	Lim & Lehane (2014)
Shenton Park	SP22	J	Steel	C	134.6	-	2.73	5.1	0	27	23	100	1	43	50	10.1	Lim & Lehane (2014)
Shenton Park	SP23	J	Steel	C	134.6	-	2.77	5.2	0	28	24	100	41	43	51	17.4	Lim & Lehane (2014)
Shenton Park	SP24	B	CFA	-	225	-	4	5	0	-	1	-	14	60	-	18	Lehane (2008)
Shenton Park	SP25	B	CFA	-	340	-	4	5	0	-	1	-	14	57	-	15	Lehane (2008)
Shenton Park	SP26	D	Steel	O	165.1	5	4	5.8	0.555	374	288	6.68	6	18	26	16.5†	This study (2017)
Shenton Park	SP27	D	Steel	O	165.1	5	4	5.84	0.537	280	216	8.93	114	20	26	16.5†	This study (2017)
Shenton Park	SP28	D	Steel	O	165.1	5	4	5.8	0.575	246	190	10.16	-	-	26	16.5†	This study (2017)
Shenton Park	SP29	D	Steel	O	165.1	5	4	6.17	0.575	661	364	5.30	7	23	27	16.5†	This study (2017)
Shenton Park	SP30	D	Steel	O	165.1	5	4	6.17	0.605	469	258	7.46	119	26	27	16.5†	This study (2017)
Shenton Park	SP31	D	Steel	O	165.1	5	4	5.84	0.597	423	233	8.27	-	-	26	16.5†	This study (2017)
Shenton Park	SP32	V	Steel	O	165.1	5	4	5.51	0.5	-	-	-	7	26	26	16.5†	This study (2017)
Shenton Park	SP33	V	Steel	O	165.1	5	4	5.51	0.617	-	-	-	117	22	25	16.5†	This study (2017)
Dunkirk	DK1	D	Steel	O	457	13.5	19.31	21.8	0.604	2507	90	7.70	9	52	53	24	Jardine <i>et al.</i> (2006)
Dunkirk	DK2	D	Steel	O	457	13.5	18.9	20.64	0.479	2405	89	7.86	81	89	46	30	Jardine <i>et al.</i> (2006)
Dunkirk	DK3	D	Steel	O	457	13.5	18.85	22.49	0.466	2268	84	8.31	235	119	50	34	Jardine <i>et al.</i> (2006)
Dunkirk	DK4	D	Steel	O	324	19.1	11.1	13.83	0.49	721	64	15.4	188	34	50	12	Chow <i>et al.</i> (1997)
													1991	64	50	9.5	Chow <i>et al.</i> (1997)
Dunkirk	DK5	D	Steel	O	324	12.7	11.1	17.18	0.45	721	64	15.4	176	40	48	20	Chow <i>et al.</i> (1997)
													288	46	48	20	Chow <i>et al.</i> (1997)
Dunkirk	DK6	D	Steel	O	324	19.1	22	13.83	0.55	3040	136	7.24	188	77	52	N/A‡	Chow <i>et al.</i> (1997)
													1988	142	52	25	Chow <i>et al.</i> (1997)

Site	Pile ID	Installation method	Pile material	Pile end condition	D_0 [mm]	t_w [mm]	L_{emb} [m]	$q_{c,av}$ [MPa]	PLR	Σ cycles	N_{bA}	s_{av} [mm/blow]	Age [days]	τ_{av} [kPa]	τ_{UWA-05} [kPa]	d_p [mm]	References
Dunkirk	DK7	D	Steel	O	51	8	2	40	0.22	220	687	9.09	1	157	93	11.2	Carroll <i>et al.</i> (2017)
Dunkirk	DK8	D	Steel	O	51	8	2	40	0.22	220	687	9.09	85	273	93	9	Carroll <i>et al.</i> (2017)
Blessington	BL1	J	Steel	O	111	8.3	1.9	13.87	0.35	19	29	100.00	1	85	99	N/A‡	Gavin & Lehane (2003)
Blessington	BL2	D	Steel	O	114	6.8	1.8	13.87	0.85	480	745	3.75	1	50	65	4.6	Gavin & Lehane (2003)
Blessington	BL3	D	Steel	O	100	4.7	1.8	13.87	0.9	325	575	5.54	1	48	60	0.98	Gavin & Lehane (2003)
Blessington	BL4	D	Steel	O	75	3.8	1.4	13.87	0.93	376	1140	3.72	1	41	60	3.33	Gavin & Lehane (2003)
Blessington	BL5	D	Steel	O	340	14	7	15.58	0.693	695	93	10.07	1	46	64	9.8	Gavin <i>et al.</i> (2013)
Blessington	BL6	D	Steel	O	340	14	7	16.14	0.693	739	99	9.47	12	89	66	11.8	Gavin <i>et al.</i> (2013)
Blessington	BL7	D	Steel	O	340	14	7	15.73	0.719	529	71	13.23	30	51	66	10.1	Gavin <i>et al.</i> (2013)
Blessington	BL8	D	Steel	O	340	14	7	16.41	0.693	630	84	11.11	220	132	68	12.1	Gavin <i>et al.</i> (2013)
Blessington	BL9	J	Steel	C	73	-	2.7	17	0	54	87	50.00	1	109	124	20*	Gavin & O'Kelly (2007)
Blessington	BL10	J	Steel	C	73	-	2.49	17	0	3	5	830.00	1	136	127	19*	Gavin & O'Kelly (2007)
Blessington	BL11	D	Steel	O	60	4	1.75	15	0.36	15	64	83.33	1	165	84	9.8	Carroll <i>et al.</i> (2017)
Blessington	BL12	D	Steel	O	60	4	1.75	15	0.36	15	64	83.33	20	172	84	13.7	Carroll <i>et al.</i> (2017)
Blessington	BL13	D	Steel	O	60	4	1.75	15	0.36	15	64	83.33	80	154	84	14	Carroll <i>et al.</i> (2017)

†Failure load extrapolated at 10% D.

‡Not available.

*Displacement measured in the compression test.