

Rotary Press-in Piling in Hard Ground

by

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Fourth-year undergraduate project in
Group D, 2012/2013

I hereby declare that, except where specifically indicated, the work submitted herein is my own original work.

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Technical Abstract

By exploiting negative shaft friction of previously installed piles to provide a reaction force, the relatively new technology of *rotary press-in piling* can offer significant advantages over conventional piling techniques. Press-in piles boast superior stiffness and capacity to equivalently-sized *non-displacement* piles. When compared to driven piles, the method is also relatively noise and vibration free, applicable at marginal or constrained sites, and does not require temporary works, making it increasingly popular in urban environments. A prediction of the static capacity of the pile can also be made from the final installation load reducing some of the design uncertainty. However, the well-recognized limitation of press-in technology is a risk of insufficient capacity of the machine at hard ground sites. This has been partially overcome by applying rotary motion to the pile simultaneously to the axial displacement, but further measures are necessary to expand the use of the Silent Piler to hard ground.

A series of 21 field tests were performed in 2012 in Kochi, Japan, on a site of dense sand and gravel. The piles were all installed by means of water-aided rotary surging with varying loading rates and water injection conditions. Four different open-ended tubular piles were used to investigate a number of techniques aimed at reducing ground resistance. This included the effects of nozzle orientation, flow rate, surface projections, 'teeth' design, and loading rates on installation. In this report, surface projections refer to M20 bolts welded onto the bottom part of the outer shaft of one of the test piles. Due to the complex cyclical nature of the data, the number of loading cycles required for 0.1m pile penetration was used as a measure of total ground resistance.

Tests with different water injection conditions were compared to determine the effects of nozzle orientation and flow rate on ground resistance. Increasing the flow rate was found to significantly reduce the total ground resistance, with additional advantages of a reduced effect of friction fatigue and lower installation times. This was attributed to a more extensive disturbance and loosening/erosion of the soil at the pile toe, generation of positive excess pore water pressures at the base, and an improved flow of fines around pile shaft. This beneficial effect of higher flow rate fits well with previous research. Nozzle orientation was found to have no effect on ground resistance for the test conditions described in this report. These results, however, are inconclusive and further tests are required.

Surface projections were observed to have little effect in softer ground, but proved to impede pile penetration in harder soil strata due to increased shaft resistance. This was explained by means of elevated outer shaft 'roughness' impeding the flow of soil

around the pile. Other disadvantages of surface projections included negative impact on installation speed, and enhanced effect of friction fatigue on pile capacity. Both a new 'teeth' design and welding the 'teeth' along the central circumference of the pile base resulted in a modest reduction in ground resistance.

Installations with different install/extract rates were then compared to investigate the relationship between the loading rates and ground resistance. It was shown that an increase in the loading rates results in a greater total ground resistance. Since more cycles were required to drive the pile, higher rates had marginal effect on the overall installation speed. The greater ground resistance was attributed to the increased inertial effects or generation of negative excess pore water pressures around the pile shaft. Further research is required to separate the effects of install and extract rates.

Based on the data analysis, a recommendation for a reduced upstroke length was made which would notably reduce a total installation time. A brief analysis into the shaft friction angle showed a significant discrepancy between the angle determined from the ratio of velocities of a point on the pile shaft, and from the ratio of the corresponding stresses. A difference in the value of vertical and horizontal pile roughness was suggested as a possible explanation. A simplified linear correlation between installation speed, IFR (Incremental Filling Ratio), and penetration depth was proposed. Factors contributing to the early plug formation were identified as reduction in the flow rate and lower extract rate. A number of recommendations were made for productive areas for further research to validate mechanisms proposed in this report, in particular the effect of the loading rates on ground resistance.

Acknowledgements

This project and the experimental work were carried out with the support of Giken Seisakusho Ltd., as a part of an ongoing collaboration initiated by Mr Kitamura and Professor Malcolm Bolton. I am very grateful to Mr Kitamura for his generosity. Particular thanks must go to Dr Stuart Haigh for teaching and advising as my project supervisor. The assistance of Giken engineers Mr Y. Ishihara, Ms M. Nishigawa, Ms N. Ogawa and Mr K. Okada was greatly appreciated as was the technical assistance of Giken operators on-site. I am also thankful to the Giken engineers for the results of field tests performed subsequent to our departure from Japan. Gongyan Gao's meticulous notes and help in Japan have been invaluable, as were the explanations and advice given by Professor Malcolm Bolton, Mr Winstanley, Mr Shepley and Ms Burali d'Arezzo during my two years as a Giken scholar. Final acknowledgements must go to Mark for Matlab assistance throughout this project and proofreading this report.

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Nomenclature

mBGL	metres Below Ground Level	v_a	axial velocity
CPT	Cone Penetration Test	v_r	rotational velocity
SPT	Standard Penetration Test	v_{up}	extract rate
UWA	University of Western Australia	v_{down}	install rate
API	American Petroleum Institute	p	pitch (v_r/v_a)
ISL	Inner Soil Length	δ	soil-pile friction angle
IFR	Incremental Filling Ratio	ϕ_{base}	base friction angle
EP	Earth Pressure	α	shaft friction angle
PWP	Pore Water Pressure	α_u	derived from velocities
D	pile diameter	α_s	derived from stresses
R	pile radius = $D/2$	α'	normalized friction angle α_u/α_s
t	pile base thickness	Q	ground resistance (also GR)
h	pile embedment depth	Q_b	base resistance
a, b, c	UWA design method coefficients	Q_s	shaft resistance, $Q_s = Q - Q_b$
A_r	area ratio	T	resistive torque
$q_{b,lim}$	base limiting stress	n_r	number of cycles
$\tau_{s,lim}$	shaft limiting stress	σ'_{v0}	in-situ vertical effective stress
N_q	bearing capacity factor	σ'_h	horizontal effective stress
κ	horizontal stress coefficient	q_b	unit base resistance
q_c	CPT cone resistance	q_{bf}	unit base resistance at failure
f_s	CPT sleeve friction	τ_s	unit shaft resistance (also q_s)
F	axial load	τ_{sf}	unit shaft resistance at failure
G	shear modulus	σ_b	base normal stress
ω	axial displacement	τ_b	base shear stress
d_u	upstroke length	τ_{sv}	vertical shaft friction
v	average installation speed	τ_{sh}	horizontal shaft friction
		max	maximum value per cycle
		min	minimum value per cycle

1 Introduction

1.1 Motivation

Rotary press-in piling technology is a relatively new method of driving *displacement* piles, the underlying principle of which is illustrated in Fig. 1-1. It is well documented that *displacement* piles offer increased pile stiffness and axial capacity compared to *non-displacement* piles of an equivalent size, e.g. bored cast-in-place piles (Deeks et al., 2005; Deeks, 2008). However, the most common methods for installing *displacement* piles result in excessive construction pollution of vibration, noise and dust which is problematic and often unacceptable in urban areas. In *press-in piling*, piles are installed by means of a static load derived from hydraulic rams. By gripping onto the previously installed piles, the Silent Piler exploits their negative shaft friction and the interlocking extraction resistance to press subsequent piles into the ground. The additional rotary motion is applied simultaneously with the axial displacement by means of torque produced by the hydraulic rams. Rotation of the pile allows mobilisation of the shaft friction at an angle, thus reducing the required axial force, which is crucial in hard ground where depth of penetration might be limited by the capacity of the piler (Bond, 2011).

Unlike machines relying solely on their self-weight as a reaction, the use of negative shaft friction from the reaction piles allows a small and compact rig. Application of a static load permits a good level of control of the penetration direction and consequently complex and accurate pile alignments. It also ensures consistent quality and load carrying capacity of the installed piles. The installation is relatively vibration and noise free, reducing construction pollution to a minimum, and thus minimizing the impact pile driving might have on the surrounding buildings and infrastructure (Rockhill et al., 2003; White and Deeks, 2007). Furthermore, while temporary works such as piling mats are often required by the traditional piling techniques, making them slow and expensive, the Silent Piler can install piles at a much higher pace as these are not needed. The 'self-walking' system also permits all the processes involved in the pile installation to take place from above the pile head, thus allowing installation at marginal or constrained sites (White and Deeks, 2007).

As a result of the reduced number of loading cycles and pre-loading of the pile in the final jacking stroke, both capacity and stiffness of the press-in piles compare favourably to the use of driven or bored deep foundations. A prediction of the static capacity of the pile can also be made from the final installation load, which is of particular relevance in sandy soils where the obtained capacity is valid for both undrained (short-term) and drained (long-term) loading due to quick dissipation of excess pore water pressures. This prediction allows reduction of some of the uncertainty in the axial capacity encountered, particularly in case of bored piles.

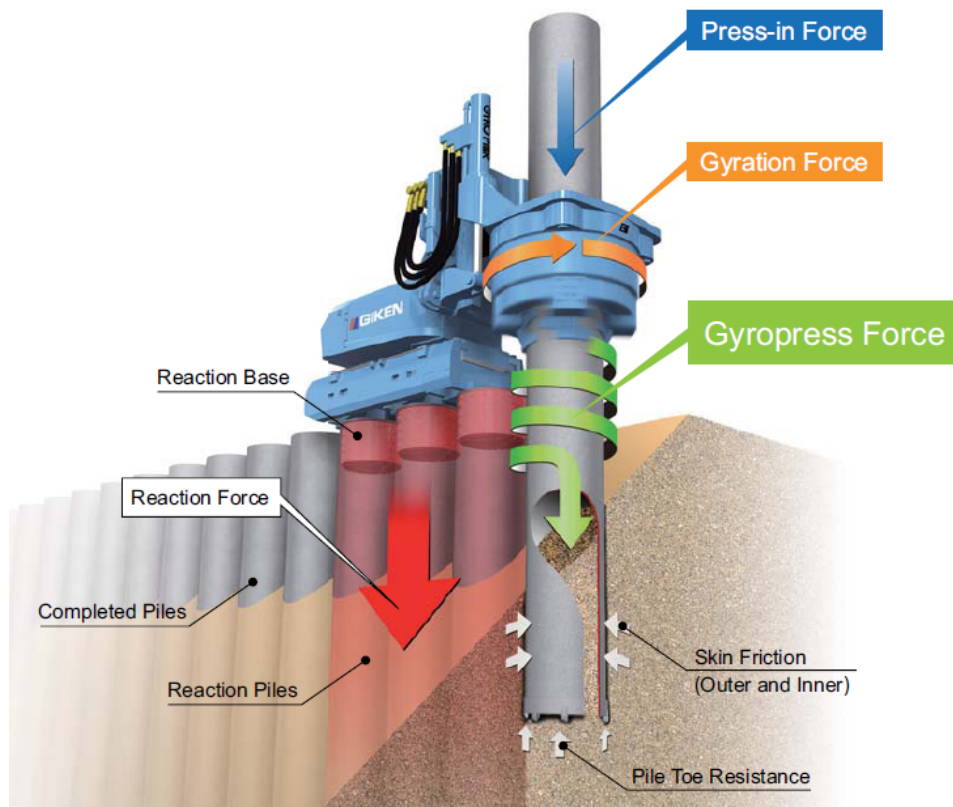


Fig. 1-1: *Diagram illustrating the underlying principle of Giken Technology: extraction resistance of previously installed piles provides the reaction force for further installation; torque applied in addition to the axial force allows mobilising shaft friction at an angle and thus reduces required axial load (image courtesy of Giken).*

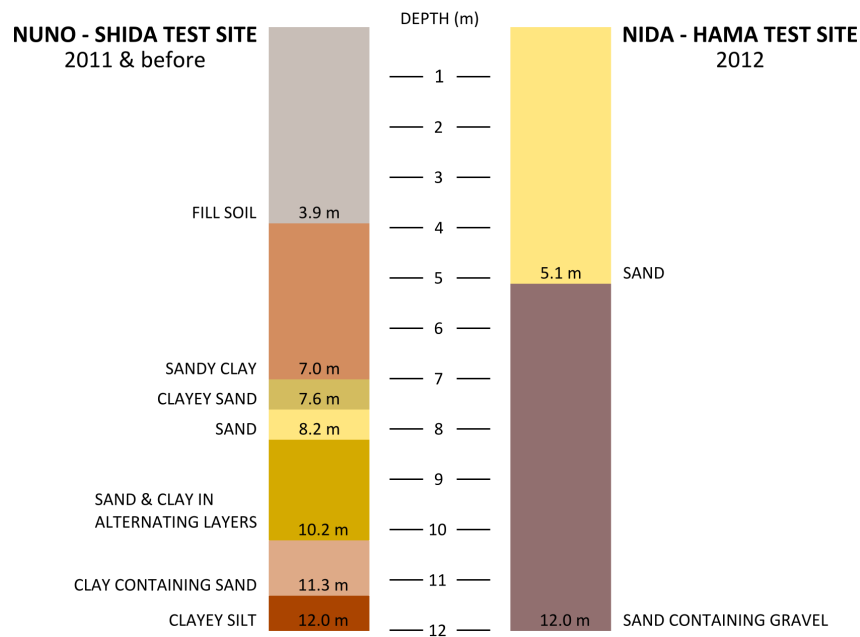


Fig. 1-2: Comparison of the soil profiles between the Nuno-Shida (2011 & before, clayey and silty soil) and the Nida-Hama test sites (2012, sandy soil).

A well-recognized limitation of the press-in technology is the risk of insufficient capacity of the machine to drive the pile to the required depth at hard ground sites (White et al., 2010). The Silent Piler was initially designed for, and extensively used in, the soft, clayey ground which tends to dominate in South-East Asia. Therefore, during the past few years of the ongoing collaboration between Cambridge University and Giken Seisakusho Ltd., the annual summer tests were carried out on the Nuno-Shida test site, which is rich in clayey and silty soil (Fig. 1-2). The research focused on overcoming the limited capacity of the piler and involved for example investigation into the plugging of open-ended tubular piles (Shepley, 2009), the effect of rotary jacking (Bond, 2011), or the significance of pore water pressures on press-in piles (Winstanley, 2012).

The advantages of press-in technology make it desirable to expand its use to piling in hard ground. Therefore, the purpose of this piece of research is to investigate a number of techniques aimed at reducing the required installation loads in dense sandy soils, thus overcoming the limitation of the piler's capacity. Hence, summer testing of 2012 was conducted at the Nida-Hama test site characterised by dense sand and gravel (Fig. 2-7).

1.2 Aims

The purpose of this project is to investigate a number of methods aimed at reducing both shaft and base resistance during installation, thus allowing expansion of the use of the Silent Piler to hard ground. More specifically, it will aim to:

- explore the effect of water injection on ground resistance. This involves two different nozzle orientations and the effect of the flow rate;
- determine the effect of surface projections on pile shaft resistance during installation;
- briefly assess the relationship between the 'teeth' design and their orientation and ground resistance;
- investigate the effect of the loading rates on ground resistance;
- briefly discuss any changes in pile plugging behaviour due to the differences in water injection, 'teeth' design, and loading rates. Explore a potential relationship between pile plugging and installation speed;
- help to identify productive areas for future research.

The long-term objective of this piece of research is to produce a set of guidelines for press-in piling in hard ground, thus enabling the use of the Silent Piler to be expanded to more problematic soils, thereby bringing positive implications for the construction industry.

1.3 Definitions

It is important to clearly define a number of terms used throughout this report:

Rotary Surging This installation procedure was applied in all 2012 tests. The method involves rotating the pile continuously in the same direction, while the pile proceeds down either by a fixed amount of displacement or until the ground resistance reaches the limit set on the piler (downstroke); at this point the pile recedes upwards by a fixed amount of displacement (upstroke), the process is repeated until final depth is reached.

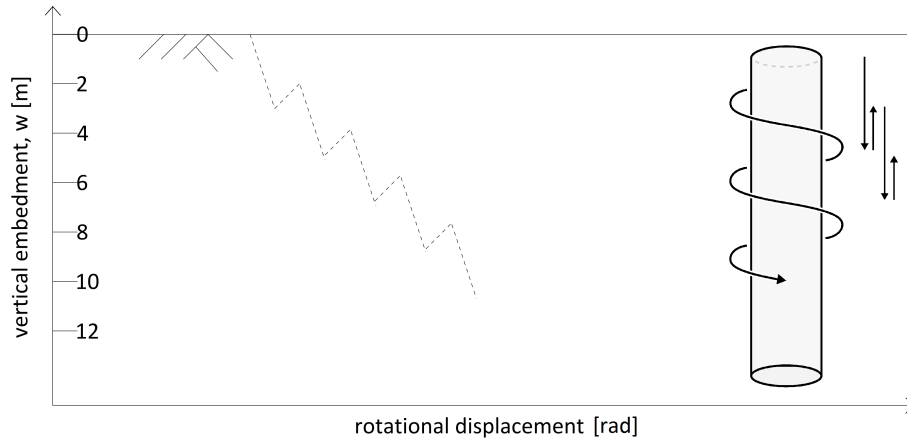


Fig. 1-3: Schematic diagram of the rotary surging installation procedure for all of the 2012, Nida-Hama test cases. The displacements and number of strokes are only indicative.

Water Injection In all tests, water was injected at the base of the pile to facilitate the press-in process. Three different modes were employed: (1) circumferential inside and radial outside, (2) double circumferential inside, and (3) circumferential and upward inside water injection (Fig. 1-4). Modes (1) and (2) were investigated in detail.

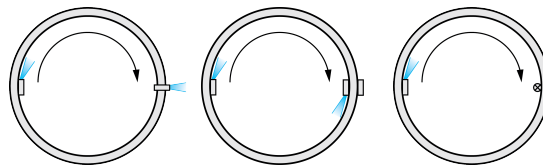


Fig. 1-4: Water injection modes used, as seen from pile toe. From left to right: mode (1), mode (2), and mode (3). The arrow indicates rotation direction of the pile.

Pitch A dimensionless measure of the amount of rotation during installation, ratio of the circumferential speed of a point on the wall of the pile to the axial speed of the pile;

$$p = v_r / v_a.$$

IFR Incremental Filling Ratio (Fig. 1-5). A measure of the internal soil column of an open-ended tubular pile. If the top of the soil column moves upwards at the same rate as the downward procession of the pile, the installation is unplugged. This is defined as *coring*, $IFR = 1$. If the soil column moves downwards at the rate of the pile press-in, this is defined as *plugging*, $IFR = 0$. The pile can then be analysed as if it was closed-ended (Shepley, 2009). For $0 < IFR < 1$ installation is classified as *partially plugged*.

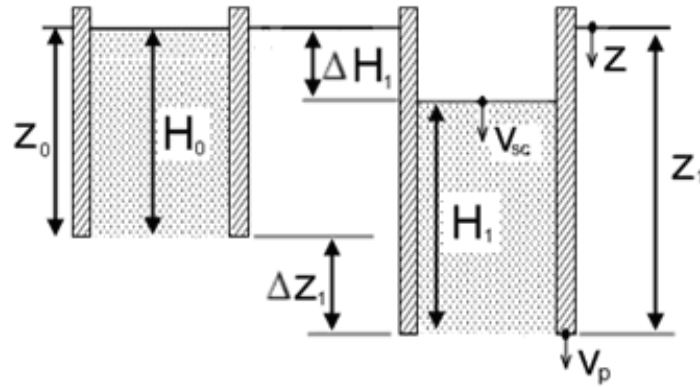


Fig. 1-5: A schematic view of plugging of an open-ended tubular pile. The Incremental Filling Ratio is defined as $IFR = \frac{\Delta z_1 - \Delta H_1}{\Delta z_1}$ (Paikowsky et al., 1989).

Surface projections M20 bolts with their heads covered in hard protective material, were welded onto the outer shaft of the pile up to a height of $2D = 1.6m$ from the pile toe; applicable to *Pile II* only (Fig.1-6).

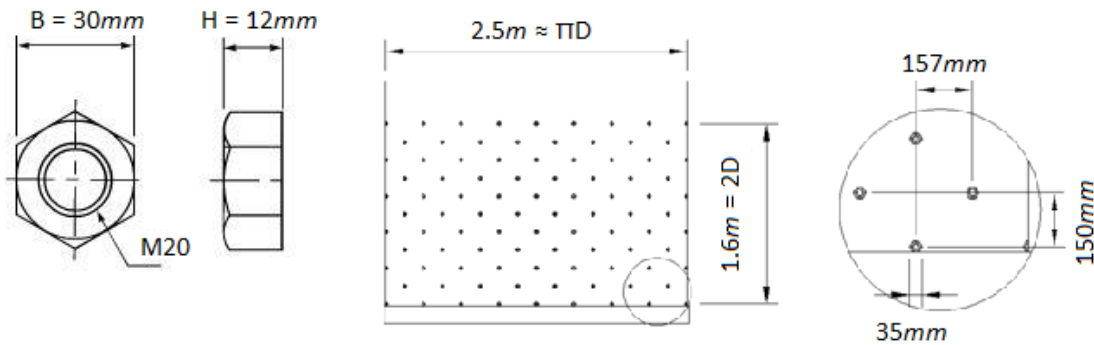


Fig. 1-6: Schematic view of surface projections on Pile II (image courtesy of Giken).

1.4 Background Theory

1.4.1 Stresses around the pile

Fig. 1-7 illustrates key stages in the stress history of a soil element adjacent to a pile jacked in sand in undrained conditions. During installation, when the pile toe approaches a soil element, stress on the pile increases to a level comparable to the CPT cone resistance, q_c while the soil element is pushed radially outwards (B). The stresses then drop as the element passes the pile toe (C). An estimate of this new stress level can be made from the CPT sleeve friction, f_s . Stage D involving friction fatigue is explained in section 1.4.2 and a stress path representing stress changes during installation is given in Fig. 1-8.

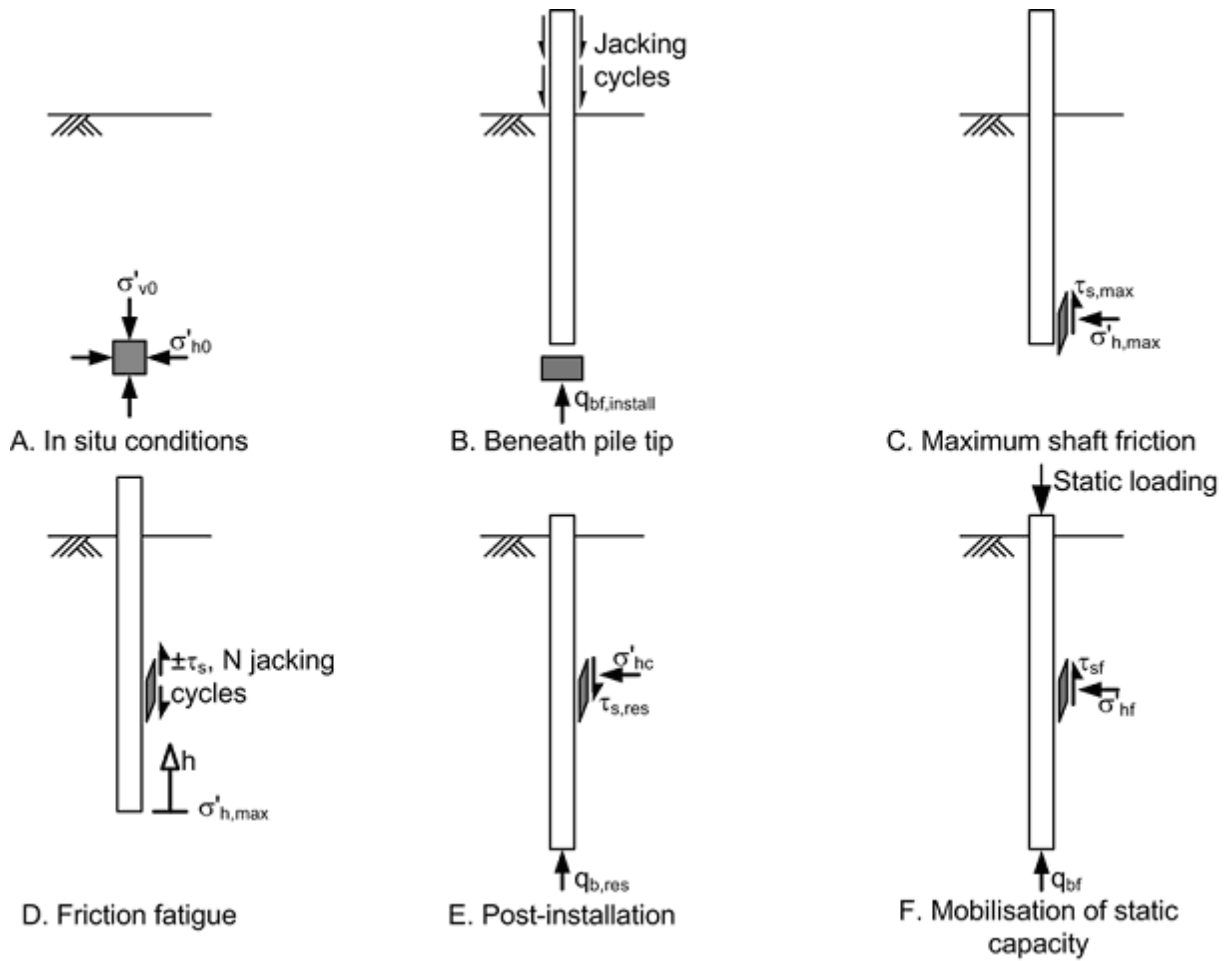


Fig. 1-7: Stages in the loading life of a displacement pile in sand (from White and Deeks (2007)).

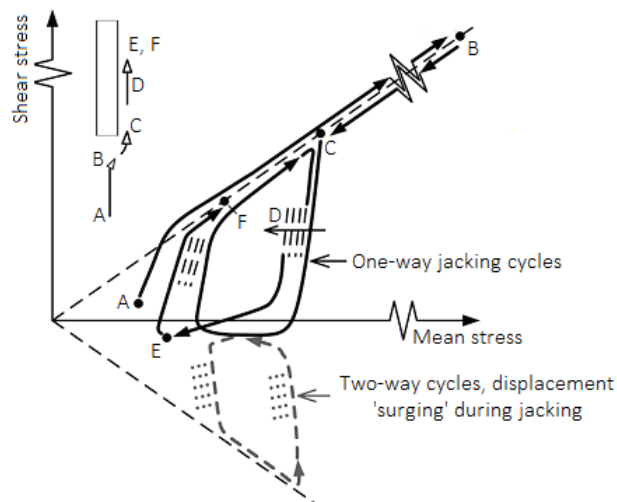


Fig. 1-8: Schematic stress history of a soil element adjacent to a pile jacked in sand (edited from White and Deeks (2007)).

At completion of the installation, the head load drops to zero, with potential residual load locked-in at a stress-free state (E). The final stage of the stress path around a *displacement* pile in sand involves a small increase in σ'_h as τ_{sf} is mobilised during loading (F), see Fig. 1-7 & 1-8.

1.4.2 Friction Fatigue

Although *displacement* piles offer higher capacity due to compaction and stiffening of the surrounding soil during installation as compared to the stress relief often experienced by *non displacement* piles, Bond and Jardine (1991) found that their unit shaft resistance decreases with increasing distance above the pile toe, an event particularly relevant in sand. This phenomenon, defined as 'friction fatigue' results from cyclic shearing of a soil element adjacent to the pile shaft, leading to contraction of the sand and thus reduction in effective horizontal stresses (White and Lehane, 2004). Lehane et al. (1993) explored mechanisms of the shaft friction in sand and linked the 'h/R effect' to the CPT cone resistance, q_c . Bond (1989) and Randolph (1994) modelled this as an exponential decay in shaft capacity with distance from the pile toe. The extent of this effect depends on both the number of loading cycles and their magnitude. Hence, reducing number of cycles required to install the pile may result in an improved pile capacity. Fig. 1-9 illustrates a friction fatigue mechanism proposed by White and Bolton (2002). This involves a combination of local volume reduction resulting from cyclical shearing and subsequent unloading of stress in the far-field soil.

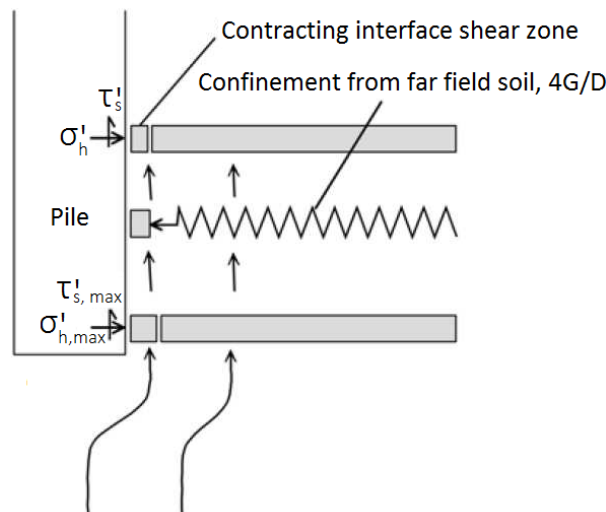


Fig. 1-9: A schematic illustration of the friction fatigue mechanism where the friction τ'_s reduces with the distance from the pile toe (edited from White and Bolton (2004)).

1.4.3 Surging

Including short extraction strokes between longer downward penetration strokes, defined as surging, is sometimes exploited to reduce installation loads for jacked piles. At the cost of a reduced pile capacity due to friction fatigue, surging can decrease both shaft and base resistance during installation, allowing pile jacking in harder soil strata. Drops in q_s and q_b are achieved by means of friction fatigue and generation of positive excess pore water pressures at the pile toe, respectively (Delano, 2010). Since this report deals with drained soil only, the latter effect was not exploited. Conversely, Delano (2010), identifies an increase in the base resistance in the sandy layer due to the compaction of the soil. However, in the 2012 summer testing, piles were equipped with a set of 'teeth' at their base. Hence, surging combined with continuous rotation of the pile was expected to result in loosening of the soil and thus, reduced base resistance.

1.4.4 Rotation

Rotary jacking aims at mobilising shaft friction at an angle thus reducing the required axial installation load, but entailing a need for an additional head torque. In open-ended tubular piles, rotation leads to a reduction in both outer and inner shaft resistance with little effect on the base resistance. Cone formation at a pile toe was identified as a reason for large q_b by Bond (2011), but will not apply to the cohesionless soils that are analysed in this report. The potential indirect effect of combined rotation and surging on base resistance was discussed in section 1.4.3. The strength and stiffness of rotation-aided jacked piles in sand is further explored by Deeks (2008).

1.4.5 Water Injection

Water injection, also known as water jetting when applied at very high pressures and flow rates, is a popular method of reducing installation loads when press-in piling is used in hard ground.

The effect of water jetting can be explained by means of erosion of the soil matrix and the resulting scour mechanism forming around the pile, or soil fluidisation. Movement of soil particles was also observed during water-aided installations by means of particle uplift and subsequent flow of fines upwards to the surface and then away from the pile (Denton, 2004). Two different mechanisms of particle transportation were offered by Shepley (2013) via soil fluidisation and resultant rearrangement of particles around the pile, or a suffusion process. As noted by Shepley (2013) most authors link the flow of fines around the pile shaft at the pile/soil interface to the elimination/reduction of shaft resistance during installation. It is

also believed that disruption of the soil structure caused by water injection at the pile base can reduce its base resistance. A number of studies on water jetting have been undertaken in the past. Carter (1996) explored the effect of the nozzle size on water-aided penetration by installing sheet piles at a sandy gravel site. Centrifuge tests investigating base and shaft response of water-jetted tubular piles in dry sand, saturated sand, and saturated silica flour were carried out by Schneider et al. (2008) at University of Western Australia.

In water injection jacking, at lower pressures, water is used as a complementary technique in reducing installation loads at flow rates of $\sim 10\%$ of that suggested by Tsinker (1988) for pile jetting in dense sand (Shepley, 2013). Ground conditions that require water injection are specified by Giken as sand or sand mixed with gravel, with N values between 20 and 50. N is defined as an average number of blows needed to progress an SPT sampler tube by 75mm at that depth. Recommended injection pressures and flow rates were given as $4\text{--}14\text{MPa}$ and $100\text{--}325\text{l/min}$, respectively (Shepley, 2013). The use of a lower flow rate aims at minimising the disruption of the soil matrix to that required to install the displacement pile only. Shepley (2013) refers to Carter (1996), Nagayama (2009), and Ishihara and Ogawa (2009) for the results of full scale tests, where water injection was utilised to double the installation speed of jacked piles.

In this report three modes of water injection were exploited to reduce installation loads of full-scale tubular piles, out of which two are analysed in detail. The total applied flow rates ranged from $90\text{--}120\text{l/min}$ (shared between two nozzles). The extensive flow of fines around the pile shaft was consistently observed during installations. The effect of both nozzle orientation and flow rate was briefly assessed in this report.

2 Field Tests

A series of 17 tests were performed on the Nida-Hama test site in Kochi, Japan in July and August 2012. 7 of these tests took place in August, subsequent to our departure from Japan. An additional 4 tests were carried out on the same site in September 2012 although the corresponding results were not made available until March 2013.

Thus, a total of 21 tests was completed using four different tubular open-ended piles and an SP7 Giken Silent Piler which can generate up to 2.6MN axial force and 830kNm of torque from its cylinders (Giken, 2011). An ECO400S Piler was used to drive sheet piles prior to the testing that acted as reaction piles for the first test piles. All subsequent tests and reaction piles were installed using the piler 'self-walking' system. The tests aimed at driving each pile until its toe reached 11.5m installation depth. This was achieved with varying degrees of success. The tests took between $1\text{h}20\text{min}$ and 7h to complete, depending mostly on the pile type being driven and on the ground condition.

Piles were installed in a straight line and the separation between the piles was kept at ~ 1.3 pile diameter centre to centre, (Fig. 2-2). Although such close spacing between the piles does not guarantee the testing to be completed without interference from the neighbouring piles, it reflects the actual construction process. The condition was also kept identical from test to test and hence is assumed not to influence the results of this report.

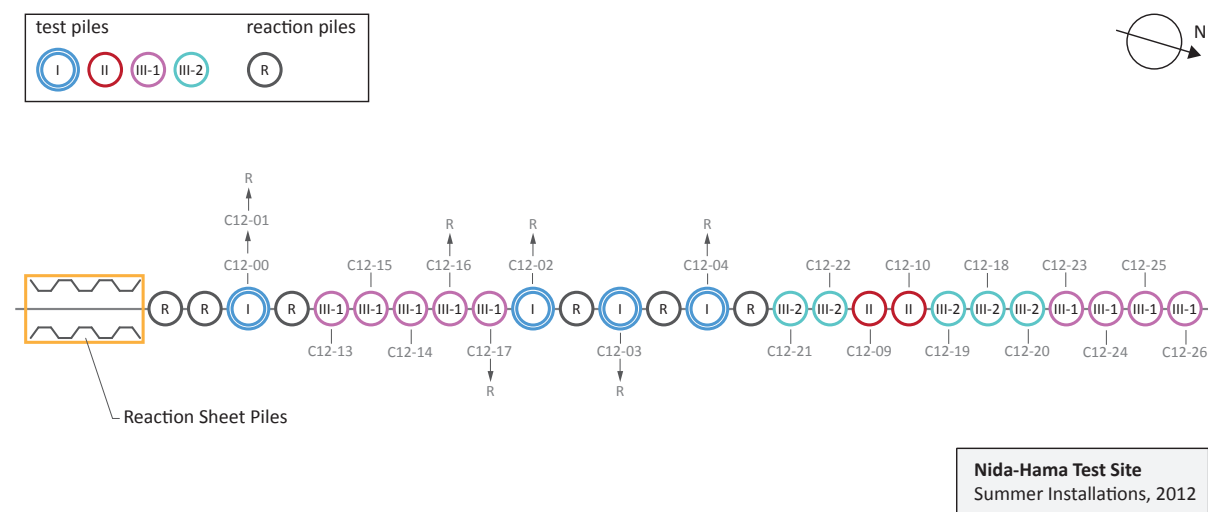


Fig. 2-1: Site layout for the 2012 testing. The image is not to scale and all distances are for indicative purposes only. Pile I and Piles II, III-1, III-2, and reaction piles, R were of 1000mm and 800mm outer diameter, respectively.

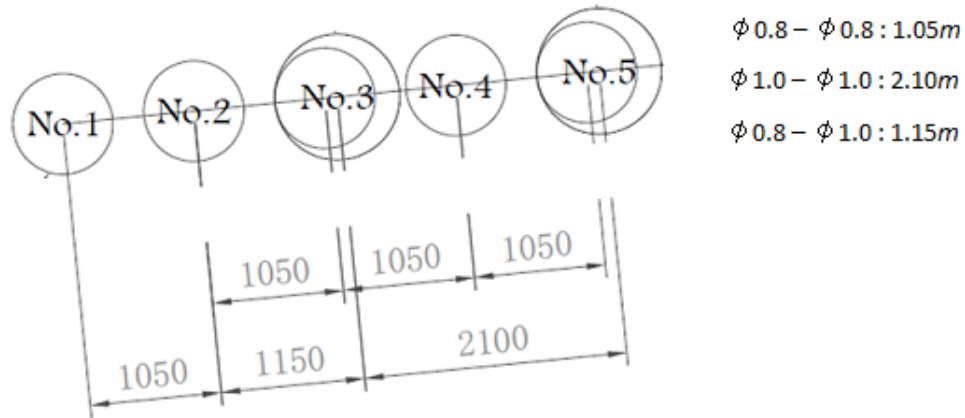


Fig. 2-2: Details on the separation between the piles (image courtesy of Giken).



Fig. 2-3: The piler and indicative distances between the test piles.

2.1 Test Piles

2.1.1 Pile I

Pile I was a double wall pile constructed using two open-ended tubular piles of 1000mm and 800mm diameters, 13.5m length, and 9mm thickness connected with a welded base plate at the pile toe. The 100mm gap between the piles allowed locating four Pore Water Pressure (PWP) sensors and two Earth Pressure (EP) sensors on both the inside and the outside of the pile. The location of the sensors and their numbering system is given in Fig. 2-4. The pile was also equipped with a set of six 'teeth' at the pile base and two nozzles at its toe to allow water injection.

PILE I

(double-wall pile)

**Pore Water Pressure Sensors**

Located at 0.25m, 1.5m, 4.5m, and 7.5m
(inside and outside)

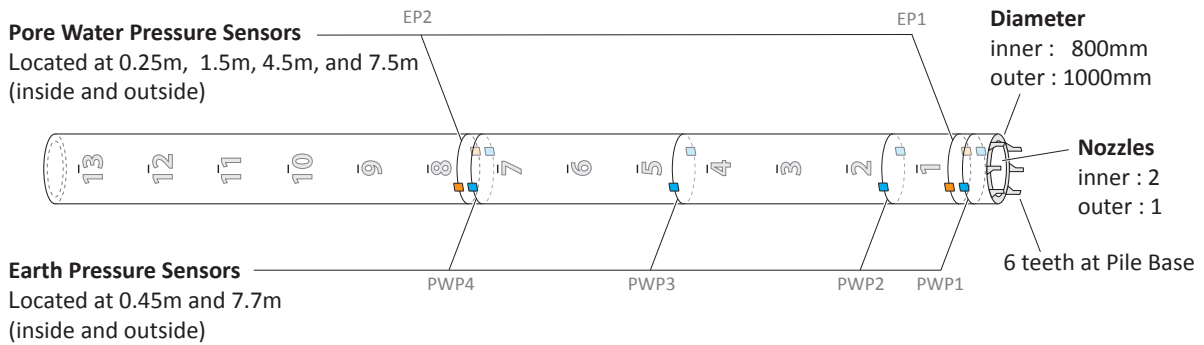
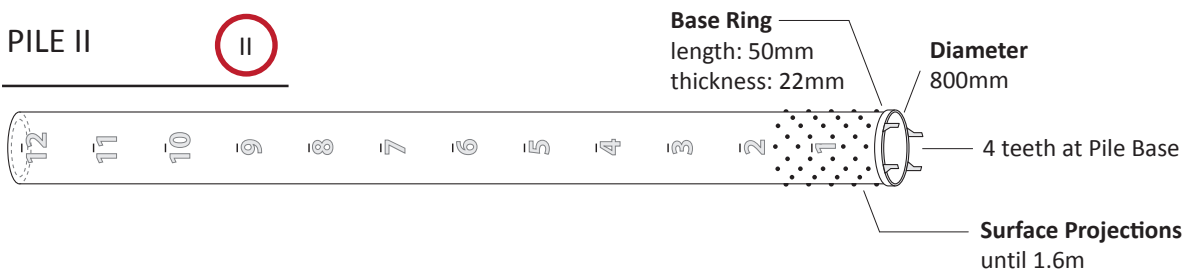
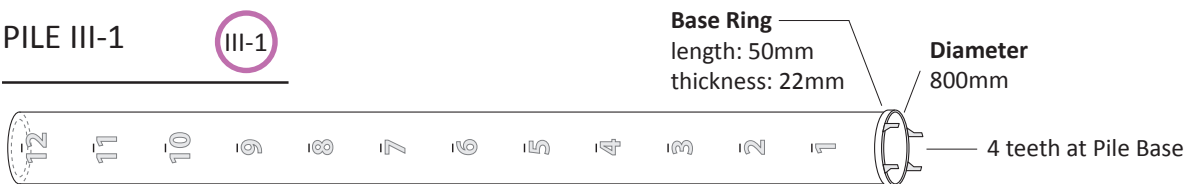
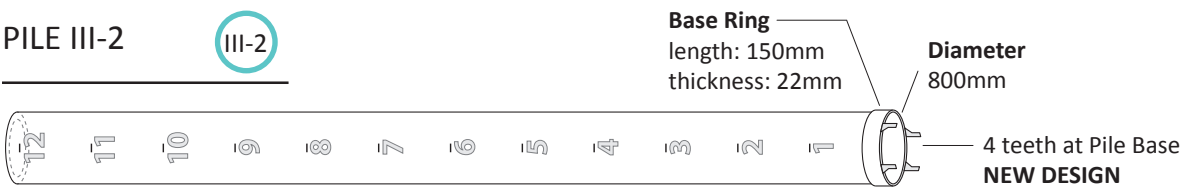
**PILE II****PILE III-1****PILE III-2**

Fig. 2-4: Schematic drawings on the four test piles used in installations of the summer testing, 2012, on the Nida-Hama test site.

2.1.2 Piles II, III-1, III-2

The three remaining piles used were single wall piles of 800mm diameter, 12.2m length, and 9mm thickness identical to typical piles used in construction, and were equipped with a set of four 'teeth' and two water nozzles at the pile toe with an additional base ring of 22mm thickness to increase the base stiffness. *Pile III-1* constituted a basic pile design. *Pile II* was identical to *Pile III-1* except for surface projections welded onto the outer pile wall from its base up to $2D = 1.6m$ from the pile toe. *Pile III-2* was a more recent version of the *Pile III-1* with a longer base ring and a marginally different 'teeth' design. This new design was introduced in a response to the extensive yielding of the *Pile III-1* during installation due to extremely hard ground conditions. The details of the single wall piles are given in Fig. 2-4.

2.2 Methodology

Before commencement of each test, a set of calibration strokes above the ground was carried out in order to measure the force required to lift the pile and the chuck with no ground resistance. Following calibration, the pile was installed to a depth of 11.5m by means of simultaneous surging strokes and rotation. The rotation rate was changed from mode 6 (3.39rpm) to mode 4 (2.42rpm) when the pile encountered the first 'hard' soil layer. The time of the rotation mode change was decided on manually by the piler operator.

As mentioned in section 1.3, the process was either displacement or force controlled. This means that either after the pile is installed one downstroke length the chuck de-clutches and is raised to grip the pile higher up, or once the ground resistance reaches its limit set on the piler, the upstroke follows without the chuck releasing the pile. Both axial force and torque are applied using pressurized cylinders. All installations were facilitated with water injection, where the modes were described in section 1.3. Water was injected through small tubular pipes welded along the inner shaft of the pile.

2.2.1 Depth Measurement

A coiled wire extensometer anchored at the ground level (stroke sensor) was used to measure the embedment of the pile. The wire was attached to a horizontal arm at the pile head which was free to rotate, but with two spring loaded drums of rope at the ground level ensuring the arm is kept directly above the sensor. A second stroke sensor was attached to the pile head with a counterweight, a sinker at the plug level in order to measure the inner soil length (ISL), (Fig. 2-5).

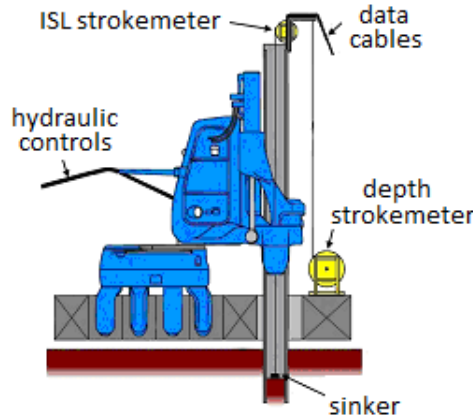


Fig. 2-5: Schematic diagram of the location of two stroke sensors (edited from Shepley (2009)).

2.2.2 Data Collection

The pressures from the hydraulic cylinders were measured with pressure sensors and transmitted through cables to one of the data acquisition systems (UCAM). Data from the stroke sensor and ISL sensor were collected in an analogous manner. Data from the PWP and EP sensors were transmitted through the cables into another, portable, wireless electronic data system (EDS), attached at the top of the installation pile. The EDS was detached from the pile at the completion of each test (or when its battery was running low, a major constraint to the tests) and the data was collected from a memory card. All data was sampled at a rate of $2Hz$.

2.3 Table of Tests

The tests were divided into four subgroups that correspond to the pile type being installed. The fifth, and final (Type III-3) subgroup corresponds to the tests that were carried out in September, 2012. These tests were not included in the initial testing schedule and the results from these tests were not made available for analysis until late March, 2013.

2.3.1 Type I

Type I comprises five installations, one with mode (1) water injection and the remaining four with mode (2) water injection. These tests aimed at investigating the effect of two different water injection methods on the PWPs and EPs both outside and inside of the pile. These tests also allowed to briefly look into the effect of the install/extract rates.

2.3.2 Type III-1

These tests served as a further investigation into the effect of the modes (1) and (2) for water injection and, at the same time, as a benchmark for the other single wall piles, since *Pile III-1* constitutes a basic pile design.

2.3.3 Type II

In the type II tests the effect of surface projections on the press-in condition was investigated. All type II tests were conducted with mode (2) water injection.

2.3.4 Type III-2

Due to the extremely hard ground condition on the Nida-Hama test site, a number of single wall piles experienced yielding and hence a substantial inward deformation of the base ring at the pile toe. Therefore, an additional pile design, *Pile III-2* was introduced. It should be mentioned that the new design of the 'teeth' adopted for *Pile III-2* was based on the Giken 2009 tests and their geometry will not be investigated in detail in this research project due to confidentiality issues. In addition to the investigation into the effect of the new 'teeth' and base ring design, type III-2 tests also allowed to proceed with the tests and verify the effects of the flow rate and different extract rates.

2.3.5 III-3

Type III-3 tests were carried out in September subsequent to our departure from Japan. These tests were conducted using the *Pile III-1* design, except for test C12-26 that had differently positioned 'teeth' at the pile base. These tests allowed to further explore the effect of install and extract rates. Additionally, type III-3 tests provided some data on the effect of 'teeth' location on the pile base on ground resistance.

Some of the tests were also finalized with a rotation at a constant, final depth until the resistive torque readings plateaued.

See table 2-1 for the summary of the tests.

Test Nomenclature	Pile	water injection mode	flow rate per nozzle (litre/min)	Final Depth (m)	Install Rate (mm/s)	Extract Rate (mm/s)	Surging stroke Down/Up (mm)	Rotation rate		Failure of the water system
								"fast" (mm/s)	"slow" (mm/s)	
C12-00	I	2	60	2.3	15	10.6	1300/40	177	127	
C12-01			60	7.8	15	10.6	1300/40	177	127	
C12-02			60	8.7	5	4	1300/40	177	127	
C12-03			60	10.4	15	10.6	1300/40	177	127	
C12-04	II	1	60	10.5	15/5	10.6	1300/40	177	127	
C12-09			60	11.0	5	4	1300/40	142	101	
C12-10			60	11.3	5	4	1300/40	142	101	
C12-13			60	11.2	5	4	1300/40	142	101	
C12-14	III - 1	2	60	11.3	5	4	1300/40	142	101	
C12-15			60	11.4	5	4	1300/40	142	101	
C12-16			60	9.6	5	4	1300/40	142	101	
C12-17			60	10.7	5	4	1300/40	142	101	
C12-18	III - 2	1	45	11.1	5	4	1300/40	142	101	
C12-19			45	10.0	5	4	1300/40	142	101	x
C12-20			60	10.9	5	18.5	1300/40	142	101	x
C12-21			60	10.8	5	4	1300/40	142	101	x
C12-22	III - 1	3 (inward/up)	60	11.1	5	4	1300/40	142	101	x
C12-23			60/30	11.5	8.2	6.1	1300/40	142	-	
C12-24			60/30	10.3	8.2	18.3	1300/40	142	101	
C12-25			60/30	11.3	9	21.1	1300/60	142	-	
C12-26*			60/30	11.5	9	21.1	1300/60	142	-	

Table 2-1: The summary of the test conditions for the summer testing, 2012, on the Nida-Hama test site.

* Pile III-1 used in the installation C12-26 was equipped with 'teeth' that were located along the central circumference of the base, while for all the remaining single-wall pile tests 'teeth' were welded on its outer circumference.

2.4 Problems encountered

A number of problems were encountered during testing:

- Only few piles were successfully installed to the final depth of $11.5m$ due to the extremely hard ground condition (it was unfeasible to complete the test within one day at the chosen piler settings). Refer to the table 2-1 for the corresponding installation depths. Test C12-02 required temporarily applying manual settings at the depth of around $8mBGL$ and thus this data was treated with caution. Test C12-16 could not proceed to the final depth due to extensive yielding deformation of the pile base ring (see Fig. 2-6) which resulted in an extremely high base resistance beyond the capacity of the piler.



Fig. 2-6: *Undeformed (left) and yielded (right) pile base due to the extremely hard ground condition.*

- Test C12-00 failed at an installation depth of $2.3m$ due to continuous sliding between the piler's chuck and the pile (insufficient friction). A new grip for the chuck was provided the next day, but the data from the C12-00 test was discarded from the analysis.
- Extensive problems were encountered with the EDS remote data acquisition system for the type I tests. The initial lifetime of the EDS battery unit was insufficient to complete the test. This was dealt with by rescheduling the type I tests to the end of the testing period and producing a new triple battery unit.
- UCAM failed to record the data from the force pressure sensors (*i.e.* lack of the ground resistance data) for a number of tests: C12-09, C12-21, and C12-22.
- Water injection system malfunctioned for the tests C12-18, C12-19, and C12-20. The results for these tests were treated with caution. A few tests had to be paused due to the water level inside the pile reaching stroke sensor and the EDS.

2.5 Site investigation

2.5.1 Soil Profile

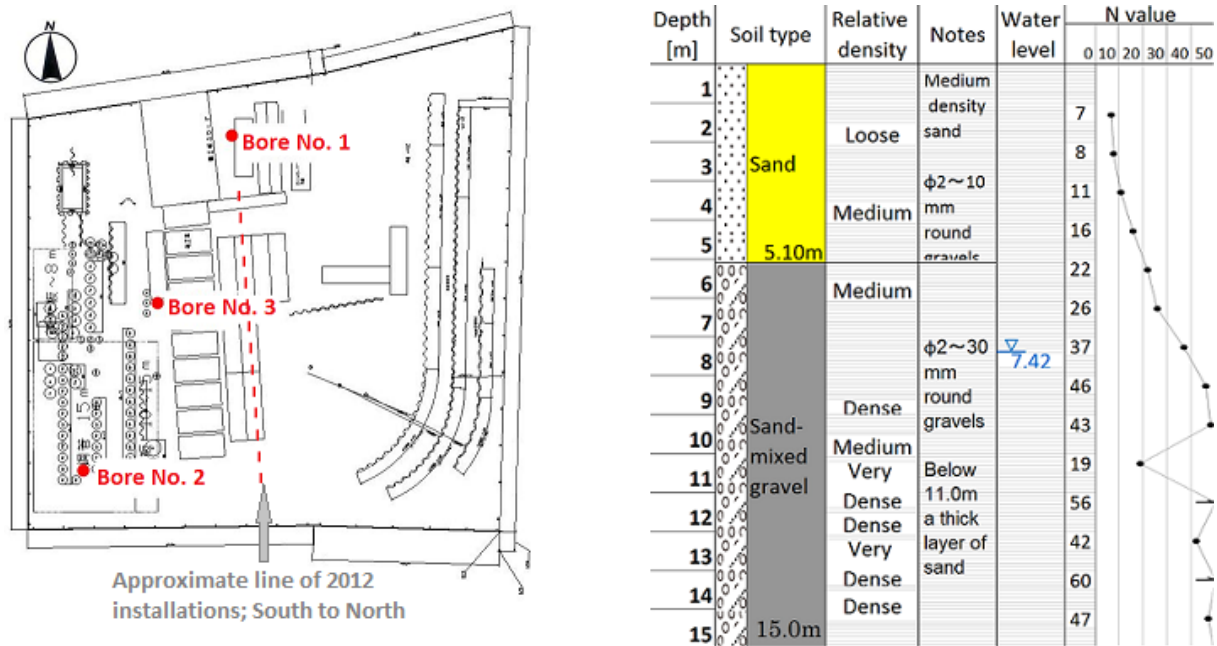


Fig. 2-7: Test site (left) and soil profile at a bore hole no. 2 (right). The SPT and bore hole data from location no. 3 is limited to the depth of 9mBGL and until that point reasonably consistent with the soil profile from bore hole no. 2. Hence, only the data for location no. 2 is included in this report (image courtesy of Giken).

Both, bore holes and an SPT were carried out at three locations at the a test site. The location of the bore hole and the corresponding ground profile that includes thick layer of sand and sand containing gravel is shown in Fig. 2-7. High N values indicate a thick layer of hard soil at around 7.5–10mBGL and another below 11.5mBGL. A layer of softer soil was observed at 10.5mBGL. The water table was measured to be at approximately 7.4mBGL, but due to the proximity of the sea and relatively permeable soil this is believed to undergo significant daily oscillations of approximately 0.7m on average, according to the available data from July to September, 2012. The experimental observations suggest fluctuation between 7–8mBGL (at the surface, during installation water level around the pile was observed to rapidly increase with no subsequent drop once the pile toe reached that depth).

2.5.2 Prediction of the installation loads

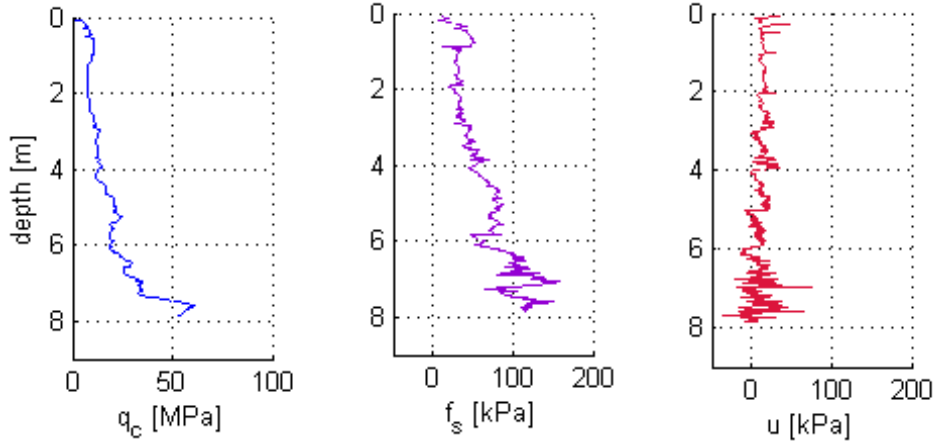


Fig. 2-8: CPT data for (from left to right) cone penetration resistance, sleeve friction and pore water pressure.

A CPT was also carried out at the site until the depth of $8m$, the results of which are presented in Fig. 2-8. The CPT data is consistent with the SPT results with the soil reaching a hard layer at approximately $7.5mBGL$.

Assuming an unplugged penetration, installation load necessary to continuously press a pile (*Pile III-1*, Fig. 2-4) into the ground was estimated from the CPT data as $\sim 5.1MN$ at $8mBGL$. As a comparison, the limit on the piler during single wall pile installation was set at $\sim 600kN$. This order of magnitude reduction in the required load was brought about by a combined effect of surging, rotation, water injection *etc.* thus allowing the use of the Silent Piler in hard ground conditions. This project commences the investigation into the additional installation techniques to reduce this required load even further.

2.5.3 Prediction of the pile capacity

The API (American Petroleum Institute) and UWA (University of Western Australia), (Lehane et al., 2005) design methods were used to estimate the ultimate capacity of a standard single wall pile at $8mBGL$, assuming plugged failure of the pile, water table at $7.5mBGL$ and soil-pile friction angle of $\delta \approx 33^\circ$ (dense sand and gravel). With unit weight of dense sand assumed as $\gamma \approx 26kNm^{-3}$ the API approach returned the ultimate pile capacity of $\sim 2.01MN$. The equations used in this calculation are given as follows:

$$\begin{aligned}\tau_{sf} &= \kappa \sigma'_{v0} \tan \delta \leq \tau_{s,lim} \\ q_{bf} &= N_q \sigma'_{v0} \leq q_{b,lim}\end{aligned}\tag{1}$$

A note should be taken that, although very popular in estimating capacity of pile foundations, the API method implies variation in δ with soil density and a constant K , while in fact, since the method was established instrumented pile tests have proven otherwise. Therefore, the API approach can be unreliable for determining shaft resistance when extrapolated beyond its original load tests' database.

Developed in 2005, the UWA method linking q_{bf} and τ_{sf} to the CPT cone resistance q_c is currently considered the most reliable design method for determining axial capacity of pile foundations in sand. The equations used are given below, for detailed explanation on the method and the indices used, refer to Lehane et al. (2005).

$$\begin{aligned}\tau_{sf} &= a q_c A_r^b \left(\max \left[\frac{h}{D}, 2 \right] \right)^c \tan \delta \\ q_{bf} &= (0.15 + 0.45 A_r) q_c\end{aligned}\tag{2}$$

By splitting the soil profile into three layers, the pile capacity estimated with the UWA method was given as $\sim 5.6 MN$. Jackson (2008) also proposed a modified UWA-05 method in order to predict installation loads for press-in piles.

3 Preliminary Data Processing

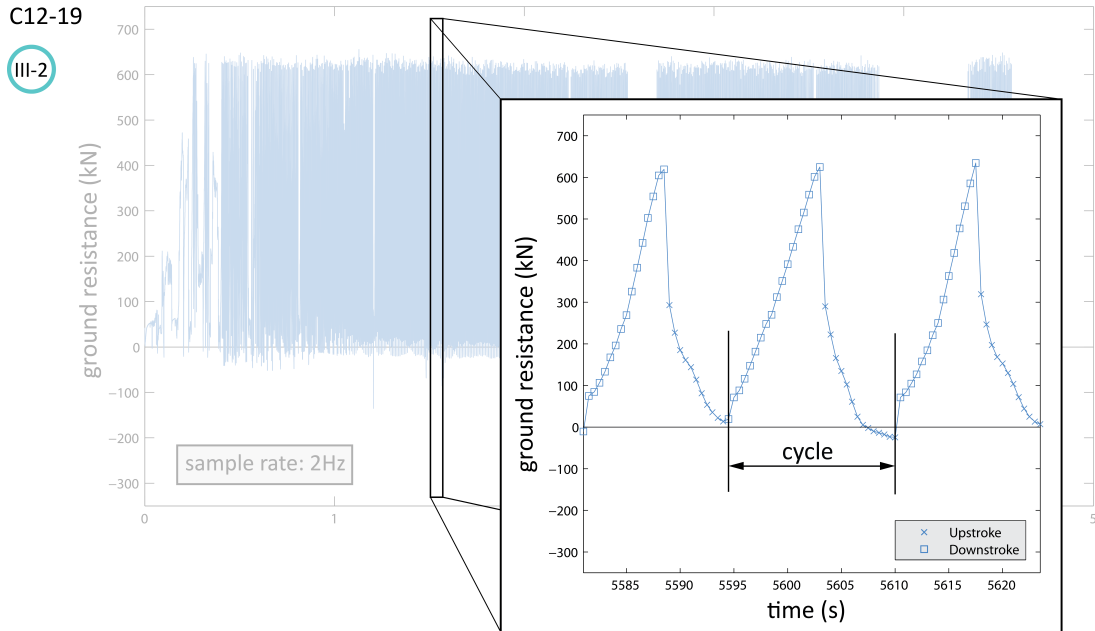


Fig. 3-1: An example of cyclical ground resistance data for a Pile III-2 test.

As previously explained, the installation process for all 2012 tests was either displacement- or force-controlled. First, rotary surging took place in alternating down and upstrokes, both of which were of fixed amount of displacement. Once the total ground resistance Q reached the force limit set on the piler, typically at a depth of $\sim 3.5mBGL$, the process became force-controlled. The length of a downstroke was then controlled by how fast Q reaches the force limit while the length of an upstroke remained unchanged. This resulted in a very complex set of cyclical data that posed some challenges to interpret since the measured force values were a direct result of the machine's settings rather than the actual ground resistance.

In order to interpret the results, the data was split into cycles according to ground resistance (Fig. 3-1). A cycle here is defined from the moment pile begins to advance downwards in a *downstroke*, where Q increases, until the moment it stops proceeding upwards in its subsequent *upstroke*. The soil was then divided into $0.1m$ layers where the number of cycles required for the pile toe to advance through each layer, defined as n_r is plotted against average layer depth in Fig. 3-2b along with the original ground resistance data in Fig. 3-2a. The use of n_r in the analysis has an additional advantage of translating onto the future shaft capacity of the pile, fewer cycles required corresponding to the reduced effect of friction fatigue. The n_r was treated as a measure of total ground resistance, Q in the subsequent analysis.

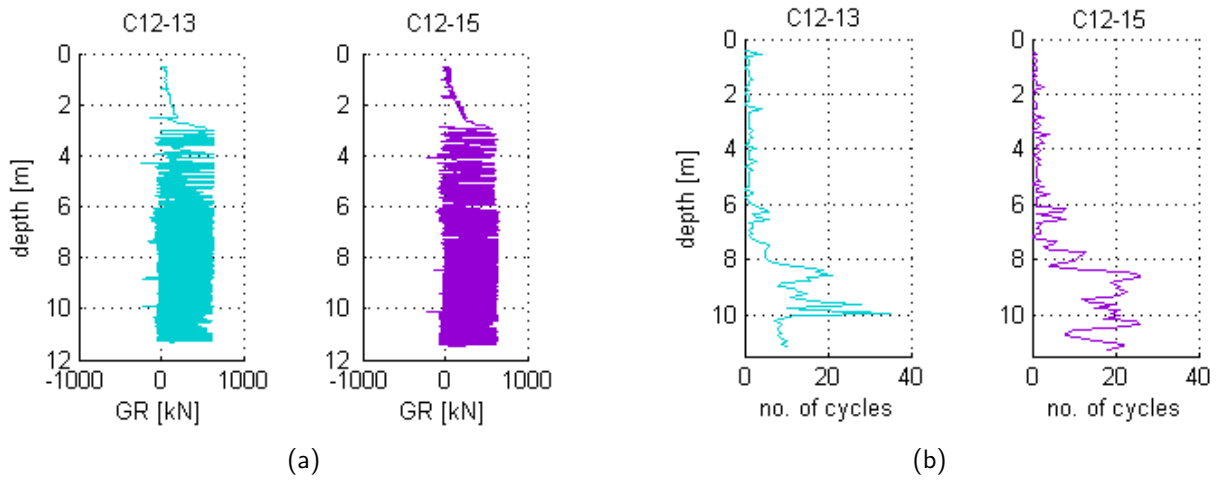


Fig. 3-2: (a) Measured ground resistance, GR vs depth for two installations and (b) number of cycles required for the pile to advance downwards through a layer of 0.1m.

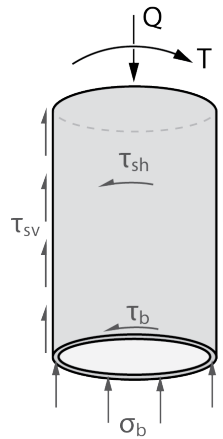


Fig. 3-3: Stresses acting on the pile during installation.

Initially, depth measurement was used to split the data into cycles. However, it was discovered that the time duration of depth cycles and ground resistance cycles did not match. Since ground resistance and torque cycles were consistent it was concluded that the depth measurement was at fault. The issue was investigated, but no conclusive reasons for the frequency mismatch were determined. This problem was solved by using ground resistance to split the data into cycles instead, as described above. The average value for depth per cycle was used in the subsequent data analysis.

Once the data was split into cycles, maximum and minimum values of ground resistance and torque per cycle were extracted to determine the resulting stresses acting on the pile (Fig. 3-3). The GR values were used to calculate normal base stress, $\sigma_b = \frac{Q_{max} + Q_{min}}{\pi D t}$ and vertical shaft friction, $\tau_{sv} = \frac{-Q_{min}}{2\pi D h}$ (Fig. 3-4). h is defined as the current embedment depth

of the pile. It should be noted that the value of Q_{min} is negative since the pile is in its upstroke. This calculation was based on the assumption that the values obtained when the pile is advancing upwards can be attributed solely to the shaft resistance (Ogawa et al., 2011).

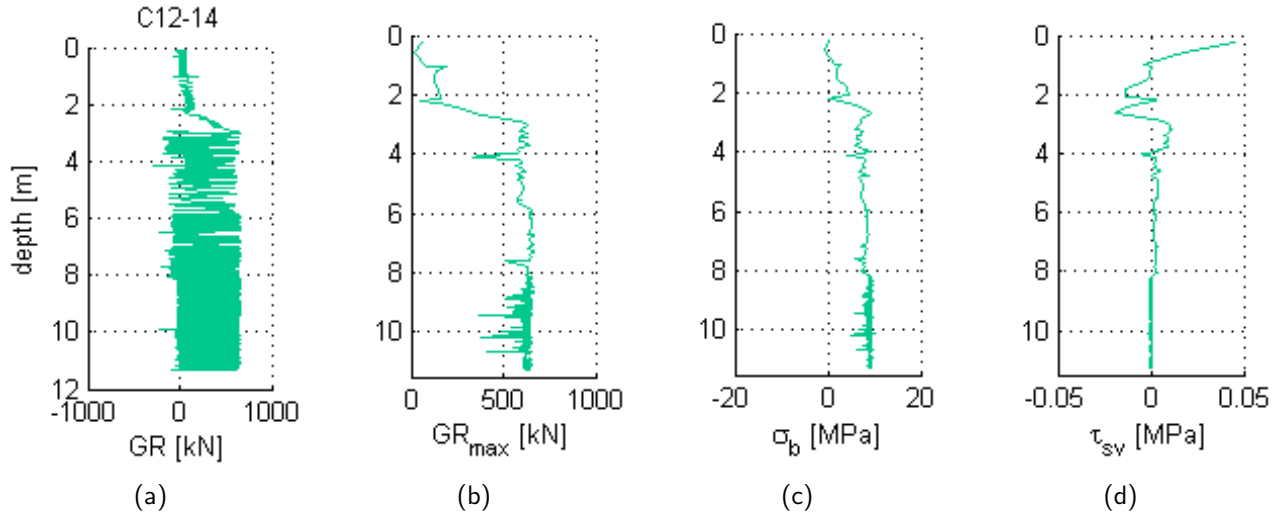


Fig. 3-4: Ground resistance and the corresponding stresses for C12-14: (a) Measured ground resistance, GR ; (b) maximum values of ground resistance per cycle, GR_{max} ; (c) the resulting normal base stress, σ_b ; (d) the resulting vertical shaft shear stress, τ_{sv} see Fig. 3-3. Plot of the minimum values of ground resistance per cycle were omitted here for clarity.

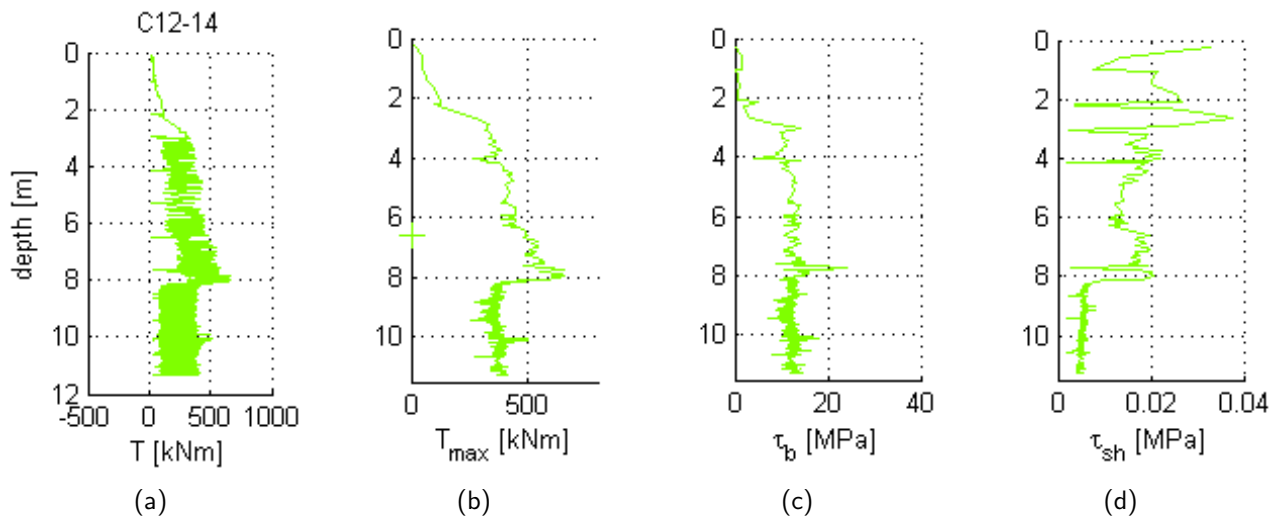


Fig. 3-5: Torque and the corresponding stresses for C12-14: (a) Measured resistive torque, T ; (b) maximum values of torque per cycle, T_{max} ; (c) the resulting shear base stress, τ_b ; (d) the resulting horizontal shaft shear stress, τ_{sh} see Fig. 3-3. The cross on the y-axis of plot (b) marks the time of the rotation mode change. Plot of the minimum values of torque per cycle were omitted here for clarity.

Analogously to the ground resistance, the process was repeated for torque to obtain the resulting base shear stress, $\tau_b = \frac{2(T_{max}-T_{min})}{\pi D^2 t}$, and the horizontal component of shaft friction, $\tau_{sh} = \frac{T_{min}}{\pi D^2 h}$, (Fig. 3-5). It was assumed that inner and outer shaft friction were equal (applies to both τ_{sh} and τ_{sv}). A note should be taken that stress values based on ground resistance, i.e. σ_b and τ_{sv} are not limiting values below approximately 3.5m (when Q reaches the force limit set on the piler).

The results of PWP's and EP's readings for *Pile 1* tests as well as pure rotation data at the final depth carried out for a selection of tests were briefly analysed, but no useful conclusions followed. The PWP's readings revealed a tendency for the occurrence of fast-decaying positive excess pore water pressures and insignificant negative excess pore water pressures during installation along the pile shaft.

Please note, that a majority of the experimental findings to follow are based on one or two installations only, and thus the conclusions presented in this report should be treated with caution.

4 Water Injection

Three different modes of water injection were used, where mode (1) and (2) were investigated in detail. The change in the water injection mode might affect both the shaft and base resistance of the pile as well as the base torque.

4.1 Comparison of the water injection modes

4.1.1 Pile I

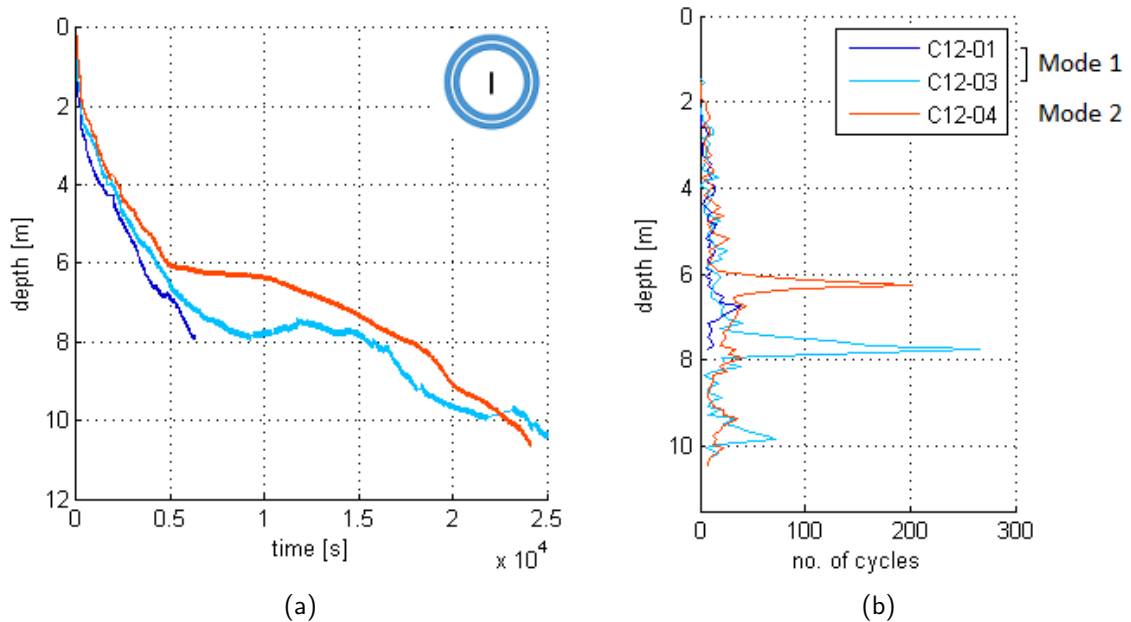


Fig. 4-1: Comparison of time taken to install the pile with depth (a) and a number of cycles required for the pile to proceed downwards, n_r , for Pile I (b). These tests aimed at comparing water injection modes (1) and (2).

The time taken to install the pile with depth, and the number of cycles, n_r , required for the pile to proceed downwards for *Pile I* (double wall pile, see Fig. 2-4) are plotted in Figs. 4-1 (a) and (b), respectively. These tests aimed at investigating the effect of water injection modes (1) and (2), all the other test conditions being unchanged from test to test. The tests C12-01 and C12-03 employed mode (2), double inside circumferential water injection, while test C12-04 used mode (1), inside circumferential and outside radial water injection (Fig. 1-4). The major peak in ground resistance was observed to occur at larger depths for tests located at the Southern end of the site (Fig. 2-1). Hence, this peak is not observed in Fig. 4-1 (b) for the test C12-01 which could not be completed due to the limited EDS battery, see section 2.4.

While the results of the tests C12-01 and C12-03 appear consistent, installation C12-04 proved to be slower between depths of 6–10mBGL, *i.e.* at harder soil strata and the location of increasing N value (Fig. 2-7). Installation C12-03 also records a $\sim 35\%$ increase in peak n_r compared to C12-04. The occurrence of the peak in n_r for the installation C12-04 at a smaller depth than for the installation C12-02 is a result of local soil variation. It could be also argued that when accounting for variation of the soil profile across the Nida-Hama test side, for direct comparison between the tests, the plot of depth against time for the installation C12-04 should undergo a shift of $\sim 1.4m$ along its initial slope, *i.e.* the distance between the peaks in n_r in Fig. 4-1 (b). Then, the overall installation speed of C12-04 could be considered higher than that of tests C12-01, and C12-03.

4.1.2 Pile III-1

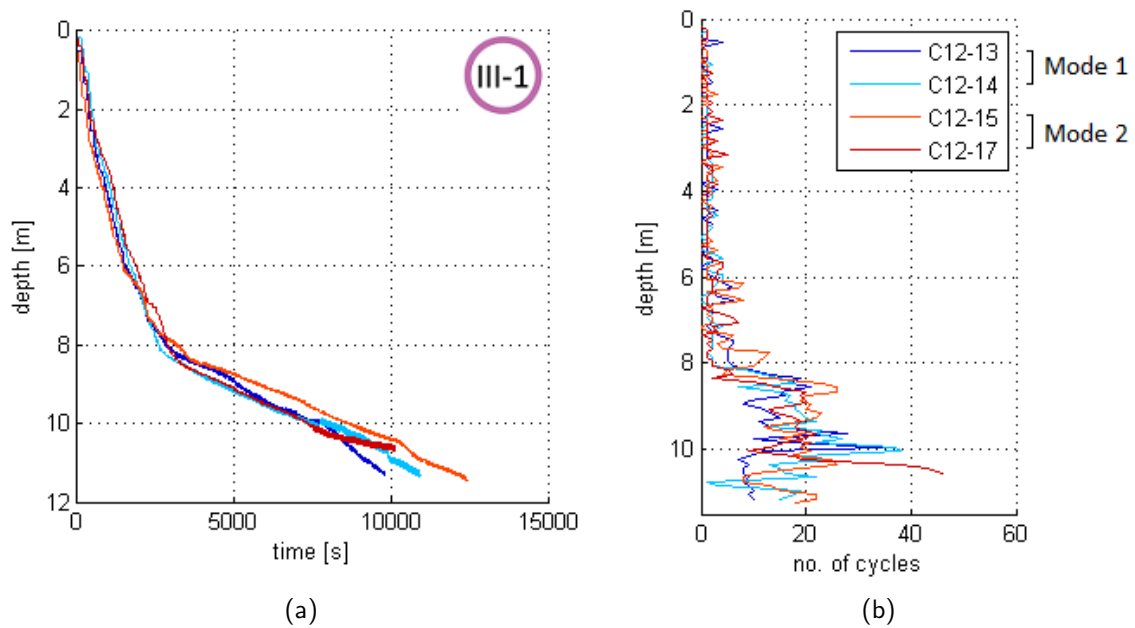


Fig. 4-2: Comparison of time taken to install the pile with depth (a) and a number of cycles required for the pile to proceed downwards, n_r , for Pile III-1 (b). These tests were carried out to further investigate the effect of different water injection modes.

Fig. 4-2 (a) and (b) present the results of the time taken to install the pile with depth, and number of cycles, n_r required for the pile to proceed downwards for *Pile III-1*. These tests aimed at verifying the results of *Pile I* tests on the water injection modes. Tests C12-13 and C12-14 exploited mode (2) water injection while installations C12-15 and C12-17 employed mode (1). The results of the installation C12-16 were not included on the plot due to the

extensive yielding of the pile's base ring, which started at $\sim 6mBGL$.

The installation of C12-17 could not be fully completed due to a sudden and significant reduction in install rate, see the slope of the plot in Fig. 4-2 (a), corresponding to the peak in ground resistance at $\sim 10mBGL$ (Fig. 4-2 (b)). No significant difference in the installation time or n_r was observed between the tests. It should, however, be noted that it were the installations that employed mode (1) water injection, C12-16, and C12-17 that encountered problems during jacking.

4.2 The effect of the flow rate

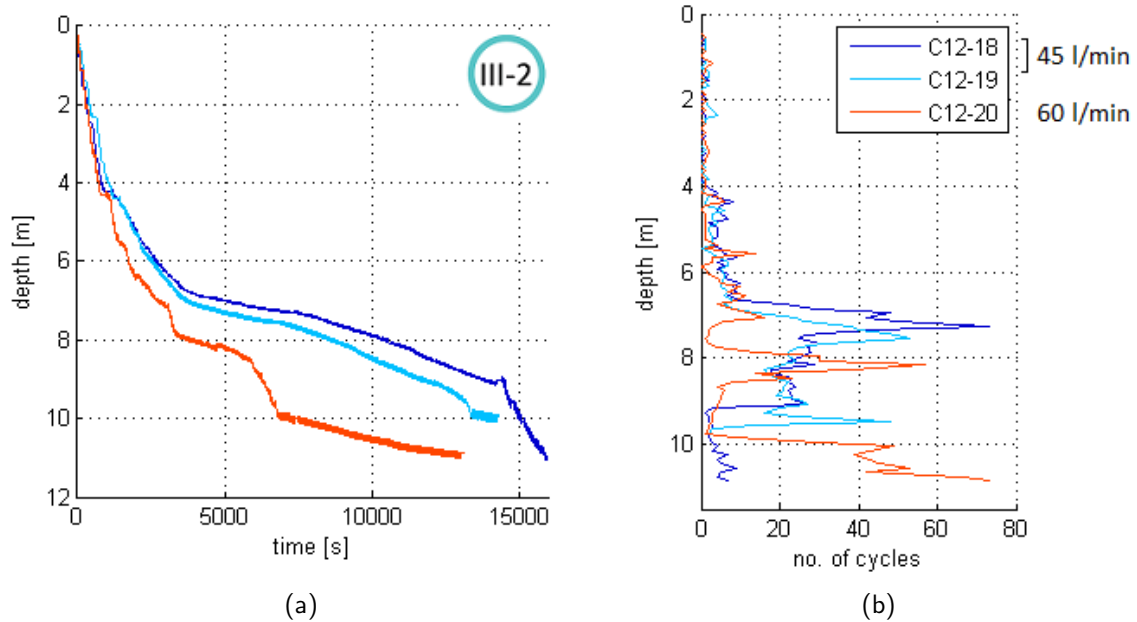


Fig. 4-3: Comparison of time taken to install the pile with depth (a) and a number of cycles required for the pile to proceed downwards, n_r , for Pile III-2 (b). These tests investigated the effect of the flow rate on the ease of installation.

Pile III-2 was used to investigate the effect of the flow rate on the ease of installation. The test C12-20 was carried out at the previously employed flow rate of $60l/min$ per nozzle, while installations C12-18 and C12-19 were conducted at a reduced $45l/min$, all other conditions remaining unchanged from test to test. The results of the time taken to install the pile and the number of strokes required are plotted in Fig. 4-3 (a) and (b), respectively.

It is clear from Fig. 4-3 (a) that the lower flow rate installations C12-18 and C12-19 resulted in a significantly slower process than the test C12-20. It can be also seen from Fig. 4-3 (b) that when integrated over depth, C12-18 and C12-19 required more strokes than the higher flow

rate test, C12-20. Similarly to the *Pile I* tests, a significant difference in the results between the two test conditions is observed mostly at high N values, $\sim 7-10mBGL$.

4.3 Discussion

From the results it can be concluded that in sand and gravel characterised by high N values, a higher flow rate of $60l/min$ reduces the number of cycles required for pile jacking n_r (analogous to the reduction in total ground resistance Q and thus the required installation load) as compared to the lower flow rate of $45l/min$. This has the additional benefit of reducing the effect of friction fatigue during installation, and thus an improved ultimate pile capacity. Higher flow rate also proves to have a positive global effect on the jacking, leading to a significantly lower installation time. For the tests discussed in this report, at a depth of $\sim 9.5mBGL$ the total installation time halves with a one third increase in the flow rate. The benefits of the higher flow rate might be attributed to a combination of a more extensive disturbance and loosening/erosion of the soil at the pile toe (and potentially generation of positive excess pore water pressures at the base) and thus reduced base resistance q_b , and a water stream at the base facilitating the flow of fines around the pile shaft, consequently reducing shaft friction, q_s . This is consistent with a number of previous studies that highlight the importance of maintaining sufficient flow rate during water jetting or injection to allow installation (Carter, 1996; Smith, 2003; Denton, 2004). Shepley (2013) also discusses the significance of the flow rate to the particle movement and soil fluidisation, though this effect being more apparent in water jetting applications. Tsinker (1988) also indicates that flow rate is of particular importance in cohesionless soils.

The results of the installations investigating two different water injection modes are inconclusive. *Pile I* tests indicate that mode (2), double inside circumferential water injection has a global positive impact on the installation time at harder soil strata, where N value increases with depth (though as discussed in sec. 4.1.1 this might be attributed to the local soil variation and then the effect would be in fact, reverse). Simultaneously, employing mode (2) appears to lead to a 35% increase in the peak n_r value. This might be a result of either reduced inertial effects or generation of negative excess pore water pressures at the base in water injection mode (1). The latter effect is considered unlikely since the limited measurements of excess pore water pressures for *Pile I* tests, on a pile shaft, $0.25m$ from pile toe, appear consistently positive. These results, however, are not confirmed by single wall pile, *Pile III-1* tests. Single wall pile tests show no or very little influence of the water injection mode on either time of installation or the amount of strokes required. Problems encountered by installations C12-16 and C12-17 suggest that mode (2) might be marginally more efficient at reducing base resis-

tance as compared to mode (1), thus preventing the base ring from yielding when the pile toe penetrates through harder soil strata. It should be also noted that due to the major differences in the geometry and the install/extract rates of *Pile I* and *Pile III-1* tests, these two types of installation might be governed by completely different mechanisms. The install/extract rates were equal to 15mm/s and 10.6mm/s , and 5mm/s and 4mm/s for *Pile I* and *Pile III-1*, respectively. *Pile III-1* is a pile typically used in construction and, therefore, these results are considered representative (*i.e.* nozzle orientation has no effect on the installation). However, further investigation into the effect of water injection mode at higher install/extract rates is necessary for the results to be conclusive.

5 Pile Design

Surface projections, as described in section 1.3, aimed at reducing outer shaft friction during rotary jacking of tubular piles, where ‘screwing’ motion allows the projections to efficiently cut through the soil. Two different ‘teeth’ designs were also investigated. The ‘teeth’ are believed to may have an effect on base resistance (both σ_b and τ_b , and the ratio $\frac{\sigma_b}{\tau_b}$), shaft resistance, and plugging behaviour of the pile.

5.1 Surface Projections

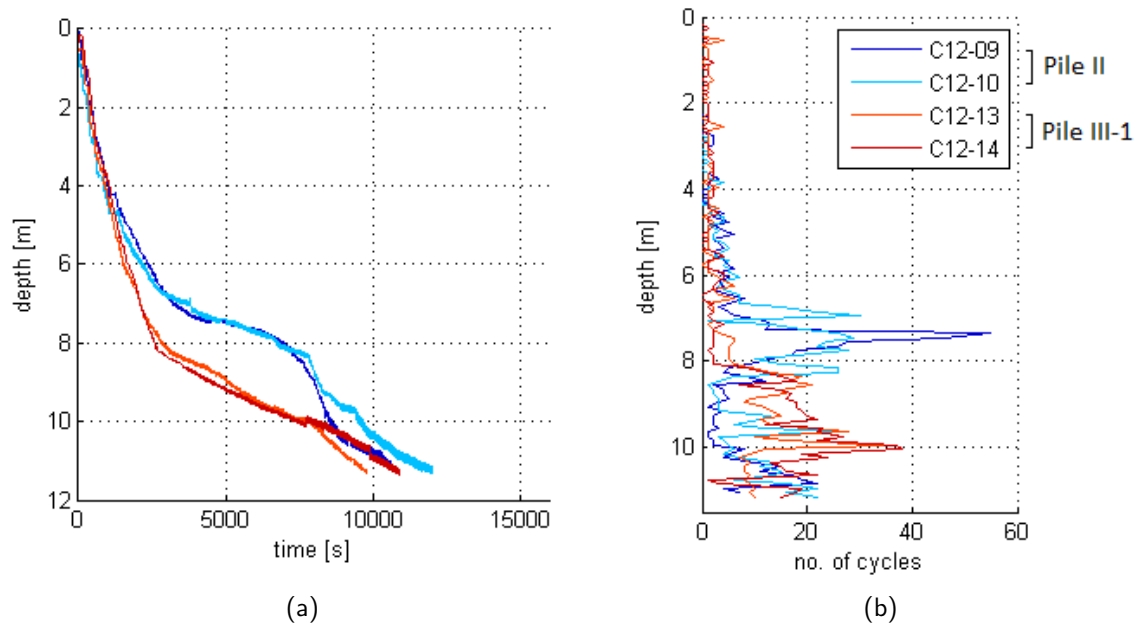


Fig. 5-1: Comparison of time taken to install the pile with depth (a) and a number of cycles required for the pile to proceed downwards, n_r , for Pile II (tests C12-09, and C12-10), and Pile III-1 (tests C12-13, and C12-14) (b). These tests aimed at investigating the effect of surface projections.

The results of time taken to install the pile with depth, and number of cycles, n_r , required for the pile to proceed downwards for Pile II and Pile III-1 are plotted in Fig. 5-1 (a) and (b), respectively. Pile II is identical to Pile III-1 except for surface projections welded on the outer shaft of the pile (Fig. 1-6 & Fig. 2-4). These tests aimed at investigating the effect of surface projections, all the other test conditions remaining unchanged from test to test. Similarly to Pile I tests in section 4.1.1 the family of peaks in ground resistance was observed to occur at larger depths for tests located at the Southern end of the site, C12-13, and C12-14, and at

lower depths for tests carried out further away, C12-09, and C12-10.

The results presented in Fig. 5-1 clearly show consistency between C12-09 and C12-10 tests, and between C12-13 and C12-14 installations. The *Pile II* tests prove to be slower between depths of $5mBGL$ and $10mBGL$, i.e. at harder soil strata and the location of increasing N value, see Fig. 2-7. The installation C12-10 also records a $\sim 50\%$ increase in peak n_r value as compared to *Pile III-1* tests. When integrated over depth, *Pile II* tests also prove to require a larger total number of cycles for the pile to penetrate through hard ground. The surface projections seem to have very little or no effect in softer soil, i.e. at smaller depths up to $\sim 5mBGL$, and at a low N soil layer at $\sim 11mBGL$. Similarly to section 4, the interpretation of the results of the depth measurement with time should be treated with caution due to soil variation across the test site.

5.2 Base ring and 'teeth' design

5.2.1 Length of the base ring and teeth design

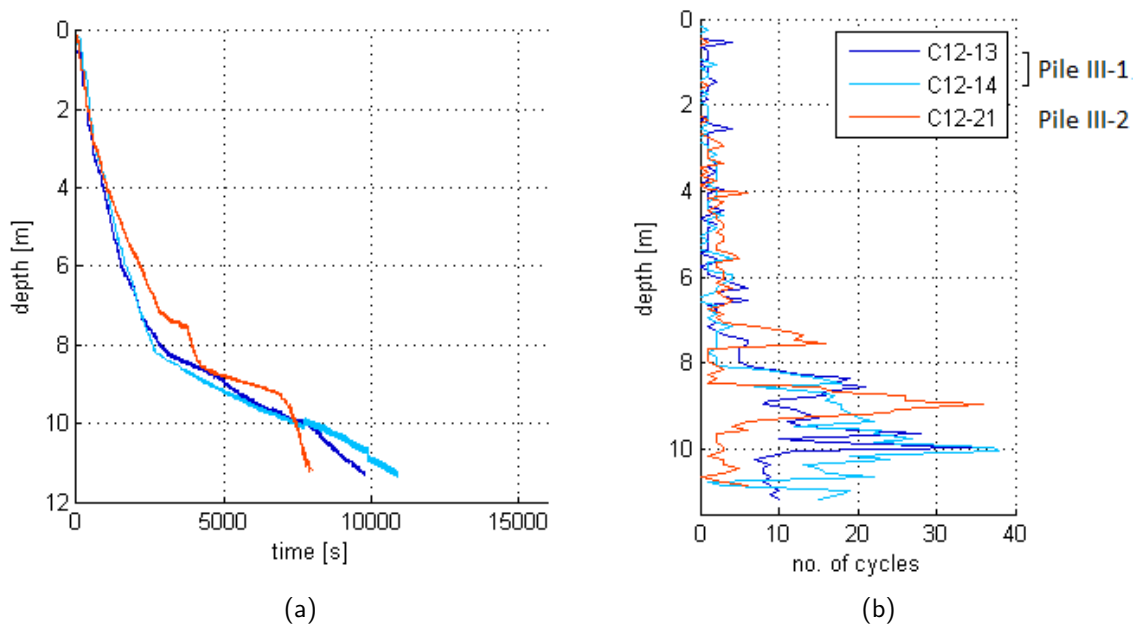


Fig. 5-2: Comparison of time taken to install the pile with depth (a) and a number of cycles required for the pile to proceed downwards, n_r , for Pile III-1 (tests C12-13, and C12-14), and Pile III-2 (test C12-21) (b). These tests aimed at investigating the effect of the length of the base ring and teeth design.

The use of a longer, $150mm$ instead of $50mm$, base ring was proposed during the summer testing, due to yielding experienced by *Pile III-1*. A brief insight into the data from installations

carried out at the Nida-Hama test site in 2009 also resulted in a decision to investigate a different 'teeth' design and thus *Pile III-2* was introduced. The 2009 test data indicated significantly lower installation times for similar pile geometry and test conditions except for the 'teeth' design. Due to a time constraint, the 'teeth' introduced for *Pile III-2* were not identical to those from 2009. The geometry and position of different 'teeth' designs, although considered an interesting subject of research, are not discussed in this report due to confidentiality issues. The results of the time taken to install the pile with depth, and number of strokes required for pile penetration are plotted in Fig. 5-2 (a) and (b), respectively.

The results of tests C12-13 and C12-14 appear consistent. The peak in ground resistance and thus in n_r for the C12-21 installation occurs at a smaller depth due to local soil variation. The C12-21 test is marginally slower than *Pile III-1* installations at soil strata where N value increases with depth (*i.e.* until $\sim 9mBGL$), but faster at a soft soil stratum between $\sim 9mBGL$ and $11mBGL$. Fig. 5-2 (b) indicates that this difference might be a result of soil variation, where in fact, *Pile III-2* results in reduced peak n_r values and the total number of cycles required for the pile toe to penetrate to the final depth.

5.2.2 Teeth location

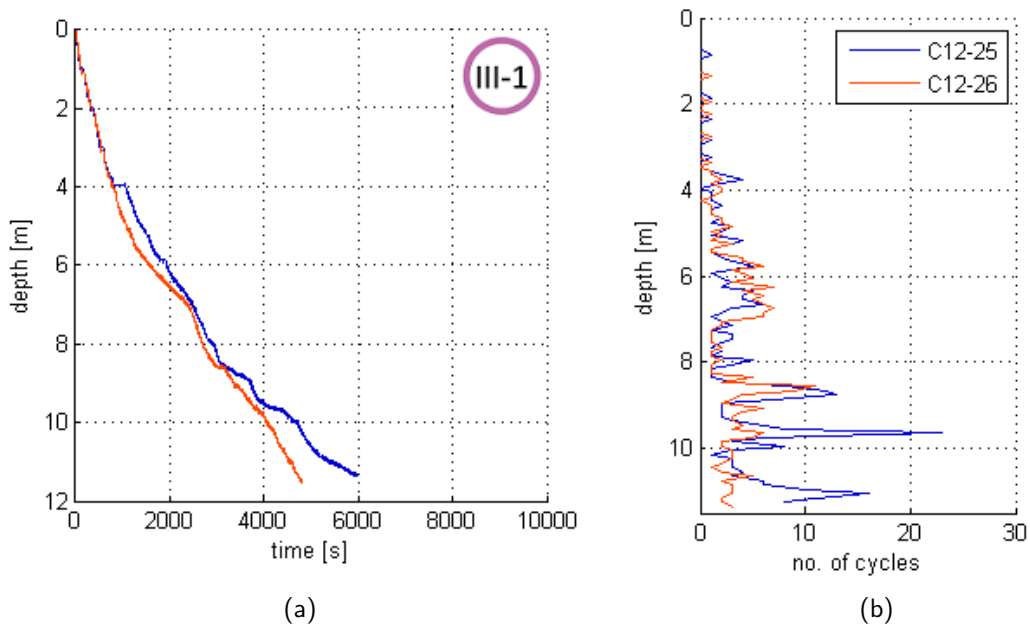


Fig. 5-3: Comparison of time taken to install the pile with depth (a) and a number of cycles required for the pile to proceed downwards, n_r for Pile III-1 with teeth located at the outer circumference of the base (C12-25) and at its central circumference (C12-26) (b). These tests aimed at investigating the effect of the location of the teeth on the pile base.

A number of additional tests with *Pile III-1* were carried out in September, subsequent to our departure from Japan. Two of these tests briefly investigated the effect of the location of the teeth at the pile base with C12-25 having teeth welded on the outer circumference of the base as all the remaining *Pile III-1* tests, and teeth located centrally for the installation C12-26. The install conditions were identical for both tests. The results of time taken to install the pile with depth, and number of cycles, n_r required for the pile to proceed downwards for these two installations are plotted in Fig. 5-3 (a) and (b), respectively.

The results of installations C12-25 and C12-26 are fairly consistent at most depths, except below $\sim 9.5mBGL$. The significant peaks in n_r experienced by the installation C12-25 suggest that this is presumably where the pile toe encounters a harder soil layer with a consistent increase in N value. As the pile toe penetrates through this layer, installation C12-26 requires significantly fewer cycles, with a decrease in a peak n_r value of $\sim 75\%$. The pile also exhibits a $\sim 20\%$ drop in the time required to install it to the final depth of $11.5mBGL$.

5.3 Discussion

It has been shown from the results that surface projections have little effect in softer ground, but impede pile penetration in harder soil strata. A higher number of cycles required to jack the pile into the ground for *Pile II* tests suggests that presence of projections leads to an increase in total ground resistance Q . It is assumed that projections have no effect on base resistance of the pile and thus, this effect can be solely attributed to the shaft action. This might be explained by means of an elevated outer shaft 'roughness' impeding the flow of soil around the pile. The effect of pile shaft roughness on ground resistance has been explored by Sadrekarimi et al. (2009) in a series of 25 model pile tests in sand. The results of these tests strongly imply a tendency for ground resistance to increase with increasing surface roughness. This is further explained by a combination of elevated soil-pile friction angle and an enhanced tendency for sand to dilate (thus resulting in an increase in horizontal effective stresses acting on the pile). These arguments, however, cannot be translated directly onto the effect surface projections have on installation due to the ~ 2 orders of magnitude difference between the size of the projections and the outer shaft surface roughness. Further tests are required to explore the mechanisms governing the behaviour of soil around the pile in a presence of surface projections. The 2012 *Pile II* tests also indicate global negative effect that surface projections have on the installation time. Further, indirect disadvantage is brought by the enhanced effect of friction fatigue due to a larger number of cycles required to install the pile, thus resulting in a reduced ultimate pile shaft capacity.

The installations also appear to benefit from the new 'teeth' design and from welding the

'teeth' along the central circumference of the pile base. Both these modifications in pile design seem to result in a modest reduction in a number of cycles required for pile penetration. This might be attributed to a combined effect of new 'teeth' being more efficient at cutting through the soil, thus reducing base resistance, Q_b and potentially consecutively cutting through a larger annulus of soil reducing shaft resistance, Q_s . These results, however, are based on single test cases and, therefore, are inconclusive. Further investigation into the 'teeth' design though of major interest to the author, could not be undertaken due to confidentiality issues. A thicker base ring is believed to have no effect on ground resistance, but prevents deformation at the pile base due to its increased stiffness.

6 Rate Effects

6.1 Extract rate

6.1.1 Pile III-2

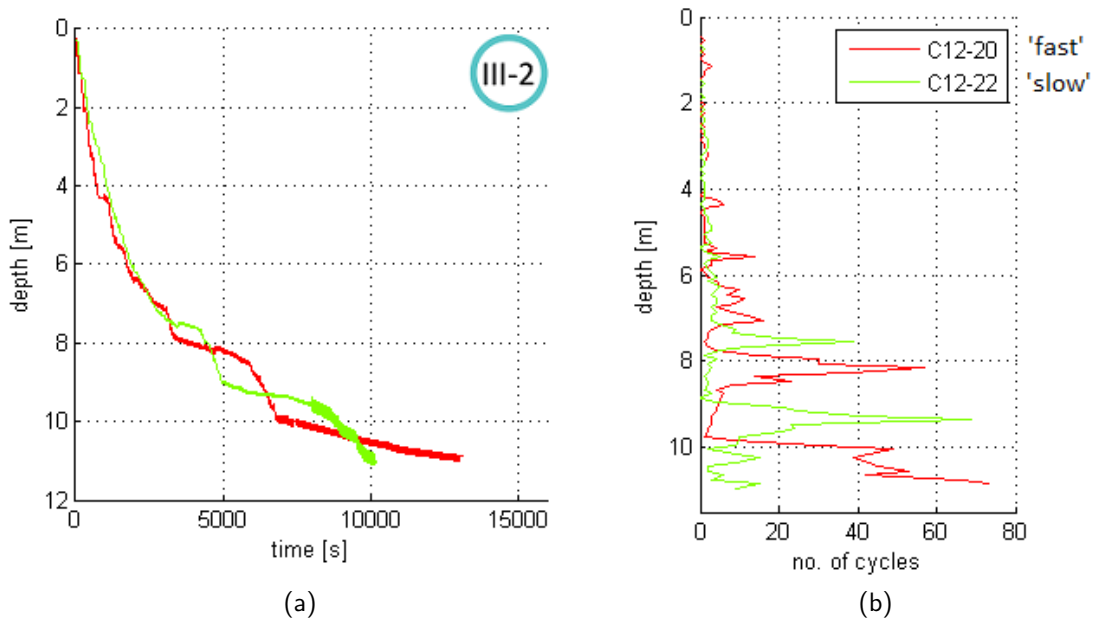


Fig. 6-1: Comparison of time taken to install the pile with depth (a) and a number of cycles required for the pile to proceed downwards, n_r , for Pile III-2 for extract rates of 18.5mm/s (C12-20) and 4mm/s (C12-22), both with an install rate of 5mm/s (b). These tests aimed at investigating the effect of the extract rate.

The results of the time taken to install the pile with depth, and the number of cycles, n_r , required for the pile to proceed downwards for Pile III-2 are plotted in Fig. 6-1 (a) and (b), respectively. These tests aimed at investigating the effect of the extract rate on the installation, with two distinct values of 18.5mm/s (C12-20), and 4mm/s (C12-22). The remaining test conditions were unchanged from test to test. As explained previously, since test C12-20 is closer to the Northern end of the site its peaks in n_r occur at lower depths than for the C12-22 installation.

Installation C12-22 required a total of 542 cycles to drive the pile to 11.1mBGL . With this number of cycles and an upstroke of 40mm , an increase in extract velocity from 4mm/s to 18.5mm/s would result in a 4250s ($\approx 70\text{min}$) reduction in total installation time. The results show, that despite its faster extract rate, test C12-20 results in a very similar installation speed to C12-22 (judging from the slope of Fig. 6-1 (a) with an attempt to account for the local soil

variation). The higher extract rate resulted in a total of 956 cycles required to install the pile to $10.9mBGL$ ($\sim 76\%$ increase). With peaks in n_r shifted to the larger depths, installation C12-20 also exhibits $\sim 40\%$ increase in n_r value at $\sim 8.2mBGL$. The same tendency can be observed at $\sim 11mBGL$ and in the occurrence of smaller peaks between $\sim 5.5-7.5mBGL$.

6.1.2 Pile III-1

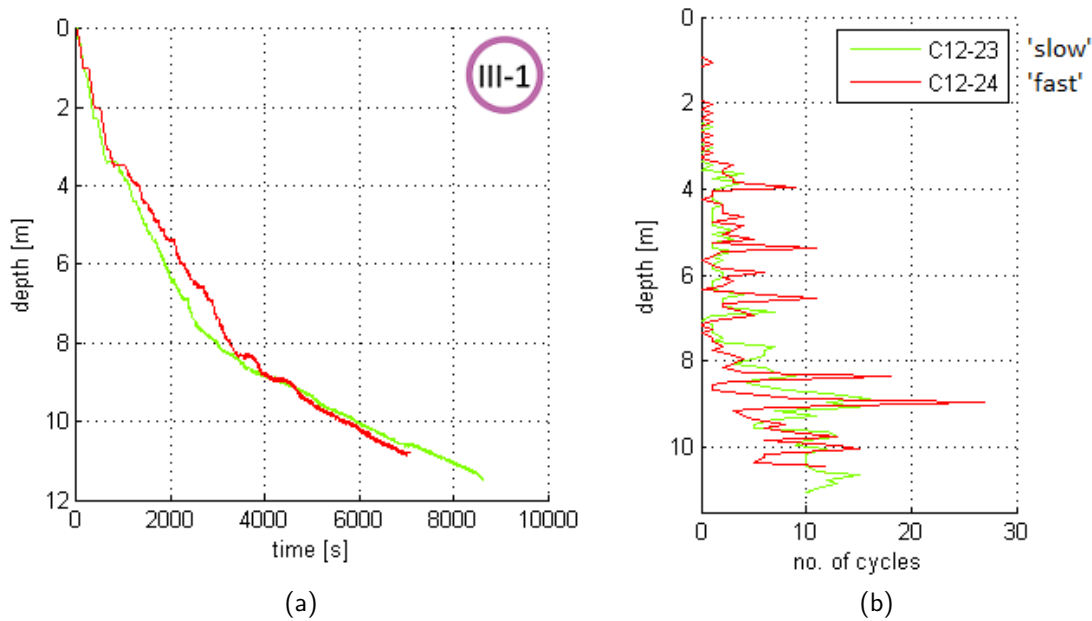


Fig. 6-2: Comparison of time taken to install the pile with depth (a) and a number of cycles required for the pile to proceed downwards, n_r , for Pile III-1 for extract rates of $6.1mm/s$ (C12-23) and $18.3mm/s$ (C12-24), both with an install rate of $8.2mm/s$ (b). These tests aimed at further investigating the effect of the extract rate.

Two of the additional tests conducted in September on the Nida-Hama test site aimed at confirming the results of section 6.1.1. The results of these tests are plotted in Fig. 6-2. C12-23 was carried out at an extract rate of $6.1mm/s$, while C12-24 employed a value of $18.3mm/s$. Another difference between the tests involves rotation rate. Most of the tests (except for C12-23, C12-24, C12-25, and C12-26) employ two rotation rates where the test commences with a 'fast' rotational speed, and subsequently the rotation rate is changed to 'slow' at $\sim 6mBGL$. In installation C12-24 this process is reversed and the test starts at a 'slow' rate. The three remaining tests that are the exception to the rule do not undergo this mode change and the entire installation employs the 'fast' rotation rate, see Table 2-1. Thus, the only difference in rotation rate between the two tests discussed here is between the ground level and $\sim 6mBGL$, where C12-23 is 'fast', and C12-24 'slow'.

There is very little notable difference in overall installation time between the tests. The installation characterised by a higher extract rate, C12-24, however, similarly to section 6.1.1 exhibits larger peaks in n_r , though almost regardless of the depth BGL.

6.2 Extract and Install rates

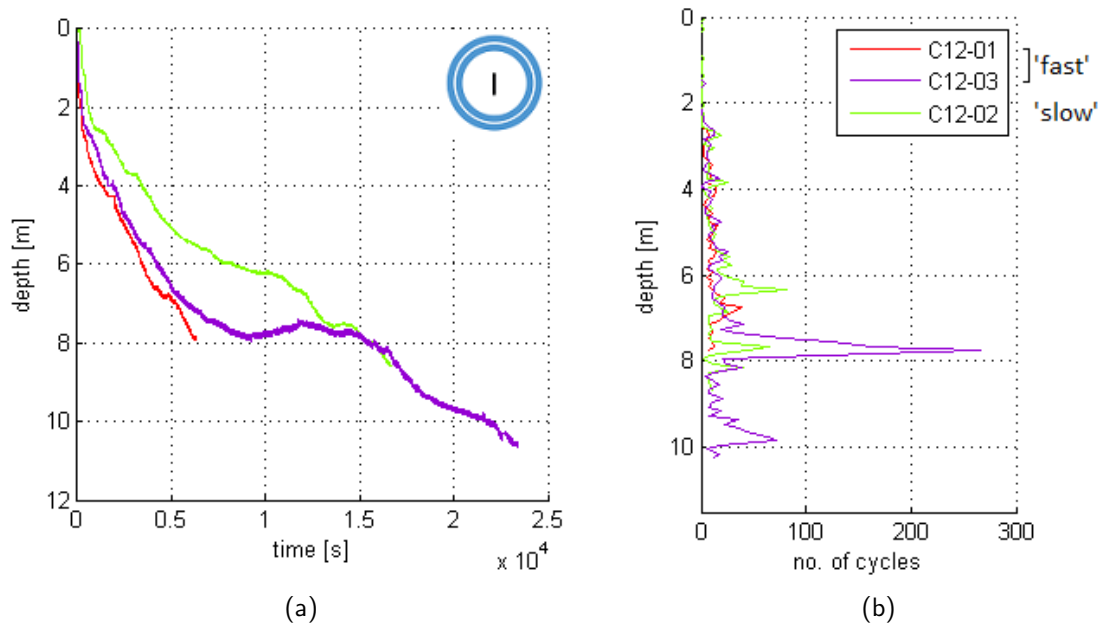


Fig. 6-3: Comparison of time taken to install the pile with depth (a) and a number of cycles required for the pile to proceed downwards, n_r for Pile I for install rates of 15mm/s (C12-01, and C12-03) and 5mm/s (C12-02), and extract rates of 10.6mm/s (C12-01, and C12-03) and 4mm/s (C12-02) (b). These tests aimed at investigating the effect of both install and extract rates.

The investigation into the combined effect of extract and install rates was also carried out using *Pile I* (double wall pile). The results of three tests conducted at two different test conditions are included below in Fig. 6-3. The tests C12-01, and C12-03 applied an install rate of 15mm/s and an extract rate of 10.6mm/s . The test C12-02 used install and extract rates of 5mm/s , and 4mm/s , respectively. As mentioned before, due to the soil profile variation across the site, the peak in n_r for C12-01 is assumed to have occurred at a larger depth than for tests C12-02 and C12-03 and thus was not captured on the plot.

Fig. 6-3 (a) indicates a different tendency than the single wall pile tests exploring variation in extract rate only. The test C12-02 clearly takes longer to install until the depth of $\sim 6.3\text{mBGL}$ (judging from the slope of the time plot) than C12-01, and C12-03. Between 6.3mBGL and

8.7mBGL (*i.e.* its final depth) overall speed of the C12-02 installation is approximately equal (or even marginally faster) than C12-01, and C12-03. Similarly to single wall piles, Fig. 6-3 (b) also shows that C12-02 requires significantly fewer cycles for penetration at greater depths than the remaining two installations. A reduction of $\sim 80\%$ in peak n_r is observed at $\sim 7.7mBGL$.

6.3 Discussion

From the results, it can be shown that an increase in extract rate results in a greater total ground resistance, Q . It is assumed that this effect is attributed solely to the shaft friction since only extract rate varies from test to test. An increase in a number of cycles required for the pile to penetrate downwards was observed for both *Pile III-1* and *Pile III-2*. Since more cycles are required, higher extract rate has a marginal effect on the overall installation speed. The greater ground resistance at higher extract rates might be attributed to the increased inertial effects or generation of negative excess pore water pressures around the pile shaft (dilation of heavily compacted sand when cyclically sheared). For the purposes of the analysis the soil profile will be considered in three layers, with dry sand between ground level and $\sim 4mBGL$, partially saturated sand between $\sim 4-6.5mBGL$, and saturated sand below $6.5mBGL$. Then the results for the first two layers are consistent with scaled model pile tests carried out by Dijkstra (2004). According to these tests at relatively low loading rates, in partially saturated sand an increase in resistance was observed at higher loading rates, while the influence of the loading rate was negligible in dry sand. This agrees with the trends observed in section 6.1, where no significant difference in n_r is noted until $\sim 4mBGL$ and small peaks were recorded for higher extract rate between $\sim 4-6.5mBGL$. It should be stated that Dijkstra (2004) indicates that his results for shaft resistance are inconclusive due to a large scatter of the data. Other studies (Huy et al., 2006; Archeewa, 2005) show that for model piles, loading rate has no significant effect on ground resistance in saturated sand. This conflicts with the results of Nida-Hama site tests where difference in peak n_r between installations with different extract rates is consistently observed below the water table. Archeewa (2005) also indicates the importance of inertial force in the analysis, and by analysing two extreme cases (*i.e.* static and dynamic load tests) Huy et al. (2006) shows that stiffness at the pile toe significantly increases due to dynamic effects that generate high negative excess pore water pressures leading to partially undrained behaviour of the pile.

The results of Gillard (2008) also indicate that at installation speeds observed in this report positive and negative excess pore water pressures that are generated during jacking are insignificant. This suggests that inertial effects are a dominating mechanism leading to

an increased ground resistance for higher loading rates. However, it might be argued that installations this report is based on are more likely to generate significant negative excess pore water pressures as a result of notably harder and denser sand that characterizes Nida-Hama test site as compared to Gillard (2008).

The tendency of the total ground resistance to increase with increased loading rates is also observed for *Pile I* tests. A much more significant increase in peak n_r for the higher loading rates test is recorded for the double wall pile. This might be attributed either to a the greater base resistance in addition to the increased shaft resistance (since both install and extract rates vary) or increased inertial effects. A different tendency in overall installation speed is observed for *Pile I* tests where lower both extract and install rates have negative global impact on C12-02 installation until the depth of $\sim 6.3mBGL$ (close to the water table) and none or positive effect below that depth. This is most likely a result of variation in install rate or different geometry of *Pile I* as compared to single wall piles.

7 Recommended Upstroke Length

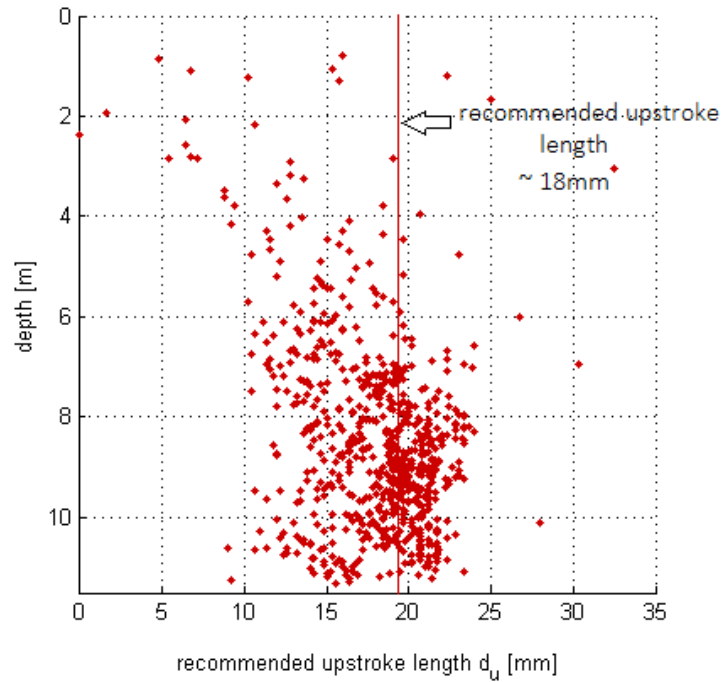


Fig. 7-1: Plot of a recommended upstroke length, d_u against depth for installations of single wall piles. Data from tests: C12-10, C12-13, C12-14, C12-15, C12-16, C12-17, C12-18, C12-19.

A brief investigation into the length of an upstroke, d_u was conducted. A d_u of 40mm was used across all tests, except for C12-25 and C12-26, two of the additional tests carried out in the fall. After splitting data into cycles, as described in section 3, each cycle was divided into a downstroke and an upstroke. From the plot of ground resistance against time, the time taken for ground resistance to drop to 5% of its peak value was determined. This time was multiplied with an upstroke velocity of 4mm/s to obtain the corresponding upward distance. The average was then taken across 10 cycles. The results of that distance are plotted in Fig. 7-1. The tests involved in the analysis are solely single-wall pile tests with extract velocity of 4mm/s (note that GR data for C12-09, C12-21, and C12-22 failed to be recorded).

The upstroke length of $\sim 18\text{mm}$ is required for ground resistance to drop to 5% of its peak value (*i.e.* a limit set on the piler). This is in a reasonable agreement with an estimate of the displacement required to mobilise full capacity of the pile determined by the rigid punch solution, $\omega = \frac{F}{5GR} \approx 15\text{mm}$ (at $\sim 600\text{kN}$, $G = 20\text{MPa}$). Assuming ~ 650 cycles needed to install the pile, reducing d_u from 40mm to 18mm would cut total installation time by 1h ($\sim 40\%$).

8 Friction Angles

8.1 Base friction angle

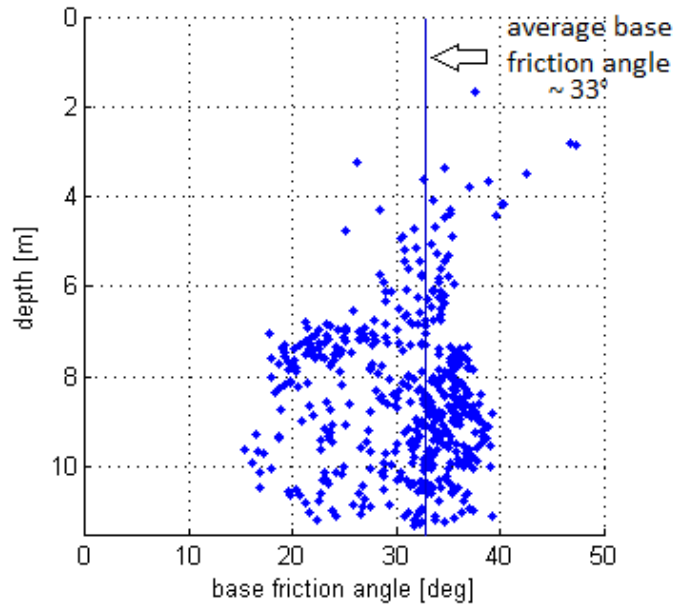


Fig. 8-1: Plot of the base friction angle against depth, where $\phi_{base} = \arctan(\sigma_b/\tau_b)$, for installations of single wall piles. Data from tests: C12-10, C12-13, C12-14, C12-15, C12-17, C12-18, and C12-19.

The variation of base friction angle with depth was determined from stresses acting on the pile base, $\phi_{base} = \arctan(\sigma_b/\tau_b)$, (section 3). From Fig. 8-1, ϕ_{base} lies between $\sim 20^\circ$ – 40° with an average of $\sim 33^\circ$, a reasonable estimate for the ϕ_{crit} value of the dense sand and gravel that dominate at the Nida-Hama test site. Note that the presence of 'teeth' was not accounted for in this calculation and, therefore, the values obtained are an approximation only.

8.2 Shaft friction angle

As explained in section 1.4.2, by rotating the pile the shaft friction is being mobilised at an angle, α , close to the horizontal in order to reduce the axial load required to install the pile. Cases where $\alpha = 0$ and $\alpha = 90^\circ$ correspond to the pure horizontal and vertical shaft friction, respectively (*i.e.* pure rotation/axial jacking). This angle can be determined either from the velocities of a point on a pile shaft, $\alpha_u = \arctan(v_{up}/R\dot{\theta})$, or from the ratio of the corresponding stresses, $\alpha_s = \arctan(\tau_{sv}/\tau_{sh})$. An interesting observation was made when comparing α_u and α_s . While these two values were expected to be in a reasonable agreement

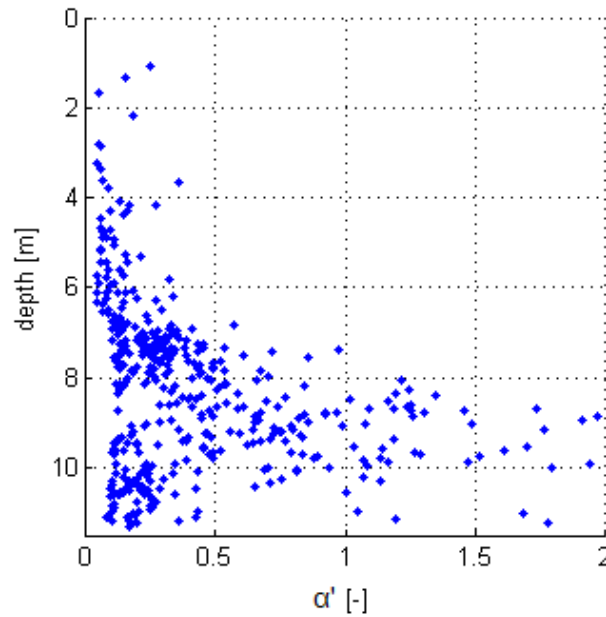


Fig. 8-2: Ratio of the shaft friction angle as obtained from the velocities of a point on the pile shaft, $\alpha_u = \arctan(v_{up}/R\dot{\theta})$ and as obtained from the stresses acting on that point, $\alpha_s = \arctan(\tau_{sv}/\tau_{sh})$. Defined as a normalized shaft friction angle, $\alpha' = \frac{\alpha_u}{\alpha_s}$. Data from tests: C12-10, C12-13, C12-14, C12-15, C12-17, C12-18, and C12-19.

(within the bounds of experimental error and accuracy of the piler), a clear tendency for $\alpha_s > \alpha_u$ was found. A variation of the ratio of the angles, here called a normalized shaft friction angle, $\alpha' = \frac{\alpha_u}{\alpha_s}$, with depth is plotted in Fig. 8-2. The ratio was found to fall within the range of ~ 0.05 to ~ 2 with an increase between $\sim 8mBGL$ and $\sim 10mBGL$. This deviation of α' from 1 could be explained by the difference in the pile roughness in the vertical and horizontal directions (where vertical $>$ horizontal) and could be analysed by means of cavity stability around the pile shaft. The variation of α' with depth might be explored further by means of the *relative* pile-soil roughness.

9 3D Correlation: installation speed, IFR, depth

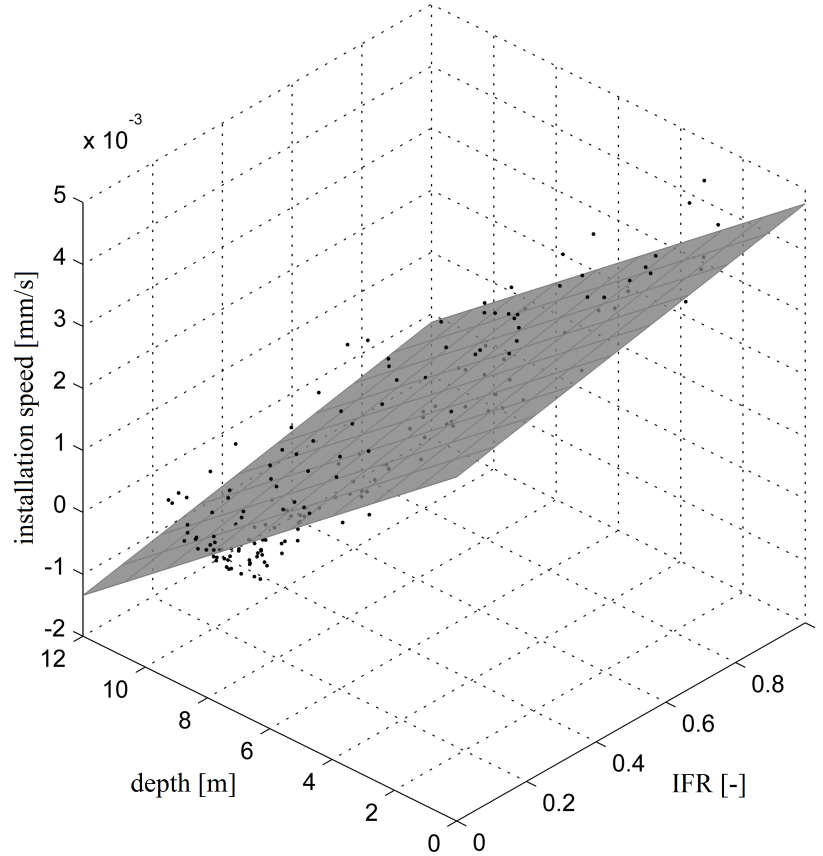


Fig. 9-1: 3D plot of installation speed (not to be confused with install rate), IFR, and depth BGL for all single wall pile tests. The approximate linear correlation was given as: $v \approx 1.2IFR - 0.4h + 3.6 [mm/s]$.

A correlation between the installation speed, IFR and penetration depth was also found. Installation speed is defined as the overall penetration speed of the pile toe, in contrast to the install rate which is defined as the velocity of the downstroke. All single-wall pile tests were used in the analysis and a simple linear plane was fitted through the data cloud (Fig. 9-1), to return an approximate correlation of $v \approx 1.2IFR - 0.4h + 3.6 [mm/s]$ for the Nida-Hama test site. The installation speed is expected to decrease with penetration depth due to increasing total ground resistance. The correlation between the installation speed and IFR might be considered consistent with Shepley (2009), where early plug formation was linked to slow installations. The mechanism of plug formation and the effect that plugging has on pile driveability are further discussed by White et al. (2004).

Other observations linked to pile plugging were made as follows:

- water injection mode has little effect on plug formation, with marginally lower IFR values for mode (2), for double circumferential water injection;
- a higher flow rate delays plug formation;
- surface projections might have an indirect effect on pile plugging. Increased shaft resistance results in a reduced installation speed which, in turn, leads to lower IFR values;
- a plug forms sooner at lower extract rates.

10 Future work

The suggestions for areas of further research into the rotary jacking in hard ground were made based on the results of the summer tests of 2012, on the Nida-Hama test site:

- Investigation of different water injection modes should be carried out in a controlled laboratory setting. Due to the similarity of the modes investigated in this report, variation in the soil profile across the Nida-Hama test site and limited accuracy of the piler, the arguments presented are inconclusive.
- The potential of a higher flow rate to reduce installation load required for pile jacking was clearly recognized in this report. Throughout analysis of previous studies combined with additional on-site testing could allow to arrive at an optimal flow rate that provides sufficient reduction in ground resistance while avoiding excessive disturbance to the surrounding soil structure, compromised base capacity, or hazardous site conditions that characterize water jetting applications.
- If the potential of surface projections in reducing shaft resistance is to be further explored, creating an analytical model that would investigate their distribution, size, and *relative* pile-soil roughness is suggested before any testing. It is suggested that surface projections is not the most productive area for further research. However, investigation into the effect of pile roughness on ground resistance is of interest.
- Further research into the rate effects would be of particular interest.

11 Conclusions

The purpose of this report was to investigate a number of methods aimed at reducing the required installation load for press-in piling in hard ground. To undertake this research a series of 21 tests on open-ended tubular piles were carried out at the Nida-Hama test site between July and September, 2012. The main conclusions drawn from the experimental data analysis in this report are as follows:

- Flow rate was found to significantly influence total ground resistance in dense sand and gravel, with higher flow rate reducing the number of loading cycles required to install the pile. Additional benefits of higher flow rate were identified as reduced effect of friction fatigue and a positive impact on installation speed. These improvements were explained by means of a more extensive disturbance and loosening/erosion of the soil at the pile toe and facilitating the flow of fines around the pile shaft.
- Nozzle orientation has no effect on ground resistance for test conditions described in this report. These findings, however, are inconclusive and further investigation into the effect of water injection modes, particularly for higher loading rates is required to confirm the results.
- Surface projections have little effect in softer ground, but prove to obstruct pile penetration in harder soil strata due to increased shaft resistance. This was explained by means of elevated outer shaft 'roughness' impeding the flow of soil around the pile which might be linked to a combination of an increased soil-pile friction angle and an enhanced tendency for sand to dilate, but further tests are required to validate this interpretation. Other disadvantages of surface projections include negative impact on installation speed, and enhanced effect of friction fatigue on pile capacity.
- Both the new 'teeth' design and welding the 'teeth' along the central circumference of the pile base, result in a modest reduction in ground resistance. It is suggested that the modified 'teeth' design is more efficient at cutting through the soil, allowing the pile to cut through a larger soil annulus during rotary jacking.
- It was shown that higher loading rates result in a greater total ground resistance. This was attributed to the increased inertial effects or generation of negative excess pore water pressures around the pile shaft due to dilation of cyclically sheared heavily compacted sand (resulting in a partially undrained penetration). Since more cycles are required, higher loading rates have marginal effect on the overall installation speed. Further tests are necessary to separate the effects of install and extract rates.

- An interesting observation was made on the shaft friction angle, with significant discrepancy between the angle determined from the ratio of velocities of a point on the pile shaft, and from the ratio of the corresponding stresses. Difference in the value of vertical and horizontal pile roughness was suggested as a possible explanation.
- It is believed that a recommended upstroke length of $18mm$ would result in $\sim 1h$ ($\sim 40\%$) reduction in the total installation time for single wall piles.
- A simplified linear correlation between installation speed, IFR, and penetration depth was proposed. Factors contributing to the early plug formation were identified as flow rate reduction, and lower extract rate.
- Further research is required to validate mechanisms proposed in this report. Investigation of the effect of the loading rates is of particular interest.

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A Health and Safety

As the site work for this project took place in the summer preceding the year, no risk assessment was submitted to the department. However, during the experimental work Japanese health and safety standards were rigorously adhered to and myself and all other visiting workers received full training prior to any activities on-site. Procedures were discussed each morning before commencement to ensure all members of the on-site team understood the work being undertaken. This was of particular importance to avoid any problems resulting from the language barrier whilst work was in progress. The craning operations were of a great concern during which a safe zone was designated not to be entered by the untrained workers until the risk had passed. The on-site work passed without any injuries.

Since the experimental work was completed on the southern island of Shikoku, Japan, in the coastal city of Kochi, on the Nida-Hama test site in proximity of the sea, risks were thought to be posed by earthquakes, tsunamis, and typhoons. None of these events occurred during the four week testing period, but plans were made to follow Giken or local authority procedures depending on when the event occurred.

On return to Cambridge health and safety considerations involved prolonged computer use. Regular breaks were scheduled to avoid repetitive strain injury and eye-strain. No health problems occurred as a result of the data processing and analysis stage of this project.