M.Sc. Thesis

On the possibility of simulating pile set-up in sand by means of centrifuge model testing

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Summary

An increase over time in bearing capacity of displacement piles is often found in the field. This phenomenon is called pile set-up. The increase is predominantly caused by an increase in shaft resistance and is observed even until 1000 days after installation. In the Dutch situation, a proposed tightening of the design standard for pile foundations makes the need to understand this phenomenon relevant. It is thought that pile set-up and group effects can contribute to the bearing capacity of piles and it would be worthwhile to incorporate them into the design standards. Although there are some ideas on the mechanisms that play a role there is no quantitative model to describe this mechanism. From literature the bearing capacity seems to increase linearly with the logarithm of time, however, the results depict a large scatter. It is assumed that a lot of factors influence pile set-up. This thesis depicts the results of a study on the possibility of simulating pile set-up in sand by means of centrifuge model testing. Conditions of this type of physical models can be controlled and makes them very useful for parameter studies.

This study presents relevant literature about pile set-up. The presumed mechanisms behind the phenomenon and influencing factors are discussed. Because set-up is only observed for displacement piles, installation effects are also investigated. It is believed that the governing mechanism behind pile set-up is to some extent rearrangement of sand grains. Some literature indicates that long-term set-up is already present minutes and hours after installation. This allows investigating the set-up mechanisms under controlled conditions in a centrifuge. A pilot test is performed with the geotechnical centrifuge of Deltares. Two test series are run to investigate the set-up of a single pile and a group pile until 1000 minutes after installation. Two instrumented piles are tested several times during the run. Special attention is given to the installation and the surface roughness of the piles. Two different installation methods are applied: monotonically jacking and pseudo-driving.

From the test results, it cannot be concluded if it is possible to simulate pile set-up in sand, observed in the field, by means of centrifuge model testing. Increases in shaft friction are observed for the jacked pile, as well as the cyclically installed pile. However, it is unlikely that these changes are only due to the factor of time. It is believed that the imposed pile displacements (test boundary conditions) disturb the results to a great extent. Clear influences of the installation of neighbouring piles are observed. It is recommended to perform following tests force-controlled. Perhaps longer lasting tests are needed. It is advisable to vary with soil homogeneity, soil uniformity and pile diameter in further model investigations. All of these parameters can influence the degree of set-up. Future research should also focus on field studies.
### List of symbols

- $A$: long-term set-up factor
- $B$: model width
- $C_u$: coefficient of uniformity
- $D$: pile diameter
- $D_{eq}$: equivalent pile diameter
- $D_r$: relative soil density
- $d_{50}$: median grain size
- $e$: void ratio
- $F$: force
- $f_s$: ratio between two cone resistances
- $G$: shear stiffness
- $g$: gravitational acceleration
- $h$: distance from pile tip
- $h_m$: model height
- $K$: lateral earth pressure coefficient
- $k$: hydraulic conductivity
- $m$: mass
- $n$: porosity or scale factor
- $Q$: pile capacity
- $q_{base}$: base resistance
- $q_c$: cone resistance
- $R$: pile radius
- $R_a$: average roughness
- $R_{max}$: absolute roughness
- $R_n$: normalized roughness
- $r$: arm length of the centrifuge
- $t$: time
- $v$: speed
- $\Delta h$: boundary displacement
- $\delta_f$: ultimate interface shear friction angle
- $K$: intrinsic permeability
- $\mu$: fluid viscosity
- $\rho$: density
- $\sigma'_v$: vertical effective stress
- $\sigma'_r$: radial effective stress
- $\tau_f$: ultimate shear stress
- $\omega$: angular velocity
1 Introduction

1.1 Background

In the Netherlands, the method of Koppejan is commonly used for calculating the bearing capacity of pile foundations. In this method pile factors are applied to account for pile type and installation method. Earlier study (Van Tol et al., 2010) stated that this method overestimates the base capacity of prefab concrete piles and closed steel pipe piles, based on ca. 25 high quality load tests. The measured base capacities prove on average to be only 67% of the predicted values. Therefore, the Dutch norm committee has decided to reduce the pile factors, unless pile suppliers can prove the opposite. They have to test their pile type by load testing before 2016.

However, from foundation practice hardly serious problems are published, i.e. failures of structures or excessive settlements. It cannot be excluded that occasionally greater than expected settlements occur, resulting in (slight) cracking, but generally this is not the case. So, it is more likely that current design methods are incomplete, disregarding aspects that provide for additional pile capacity or reduced pile loading. From economic perspective it is attractive to investigate those ‘hidden’ safeties and incorporate them in the design standards - avoiding oversized pile foundations in future. The mechanisms that govern the pile behaviour have to be understood. Stoevelaar et al. (2012) investigated within a literature study seven mechanisms that could explain this hidden safety. They concluded that time-dependent behaviour and pile group effects can be regarded as the most promising ones.

Often the bearing capacity of displacement piles increases with time after installation. This is called pile set-up and cannot be attributed only to the dissipation of the excess pore pressures. Long-term set-up of driven piles is primarily related to the increase of the shaft resistance and is stress level dependent (Axelsson, 2000). Soil creep and ageing are mentioned to be the main causes of this increase; however, not all literature is unequivocal. Set-up is not observed on bored piles in sand. So, a hypothesis is that some aspect of the installation process is a necessary condition for set-up to occur (White and Zhao, 2006). At the moment no time correction formula is implemented in the calculation of the capacity of piles installed in sand. One of the goals of the future is to develop a time function, which relates capacity with time. The most conventional relation for pile set-up is a linear relation between bearing capacity and the logarithm of time. This relation is confirmed by many studies, but no accordance about the parameters is present yet. Repeated tests on open steel pipe piles showed a reduction of bearing capacity and tests with relative low cyclic loadings showed an acceleration of set-up (Jardine et al. 2006). For application in practice, conservative parameters for pile set-up have to be found.

Important pile group effects are densification and changes of stresses in the surrounding soil due to the installation of a neighbouring displacement pile. Also, different time-dependent behaviour between a single pile and a group pile are mentioned (Axelsson, 2000). These effects depend on the relative density of the sand, the distance between piles and the pile configuration. In the Netherlands, the CUR-committee has published a calculation rule for the dimensioning of tension piles (CUR-publication 2001-4), which takes into account the installation effects of a pile group. For compression piles, such a method does not exist yet.

Based on in situ load tests and laboratory tests, it is possible to describe pile set-up and pile group effects quantitatively. When those phenomena can be simulated within scaled model tests, a lot of effort can be saved (lower costs
compared with in-situ tests). Another advantage of laboratory testing is the possibility to achieve specific conditions. Therefore it is proposed to run a pilot test within the geotechnical centrifuge of Deltares. In a geotechnical centrifuge it can be expected that if the mechanical behaviour of the soil is strongly dependent on stress level, then such behaviour should be correctly reproduced in the model. Extrapolation of the results to full-scale conditions is limited by the uncertainties surrounding scale and size effects. The possibility of simulating time-dependent effects and group effects on the bearing capacity of driven piles in sand within a geotechnical centrifuge will be investigated in the current thesis. It will be tried to gather realistic values for pile set-up by ‘static load tests’ on the model piles.

1.2 Objectives and limitations

The main objective of the current investigation is to demonstrate the possibility of obtaining realistic parameter values for time-dependent and group effects on the capacity of displacement piles in sand by testing model piles within a geotechnical centrifuge. This objective is subdivided into the next questions:

- What are the influences of the installation method, i.e. what are the differences between the set-up of a monotonically jacked and a ‘driven’ pile?
- What are the consequences of several load tests and cyclic loading for the time-dependent behaviour of the pile capacity?
- Is there a difference between the set-up for a single pile and for a pile group?
- What is the influence of the order of installation?
- Is it possible to get sufficiently reliable values for the parameters of pile set-up and densification of the surrounding soil? Are these values comparable with those from literature?

This study is limited to displacement piles in sand. Dynamic effects of pile driving are not taken into account. Out of scope are the influences of the relative density of the sand, pile shaft roughness, grain size and distribution of the sand, penetration depth, pile diameter, etc.

1.3 Outline

The current thesis can be subdivided into:

- An overview of aspects, relevant to the current investigation, which influence the bearing capacity of displacement piles in sand (chapter 2);
- The test setup to investigate time and group effects, having in mind the possibilities and limitations of centrifuge model tests (chapter 3);
- The analysis of the test results and recommendations for future research (chapter 4 and 5).
2 On the bearing capacity of driven piles in sand

2.1 Bearing capacity of pile foundations

In many areas of the Netherlands, especially in the Western part of the country, the soil consists of layers of soft soil (clay and peat), of 10-20 meter thickness, on a stiff sand layer, of pleistocene origin. Similar situations occur in several other coastal areas, dominated by a delta of rivers. The sand layer below the soft soil owes its bearing capacity for a large part to its deep location, with the soft layers acting as a surcharge. The properties of the sand itself, a relatively high density and a high friction angle, also help to give this sand layer a good bearing capacity. The system of soft soils and a deeper stiff sand layer is very suitable for a pile foundation (Verruijt, 2001). Piles are used where a structure cannot be supported satisfactorily on a shallow foundation. A single pile can be defined as a long slender structural member used to transmit loads applied at its top to the ground at lower levels. A compression loaded pile carries the applied load via shear stress mobilised on the surface of the pile shaft, called skin friction, and bearing capacity at the pile base, called end bearing. Piles are mainly classified into two types: (1) driven or displacement piles – which are usually fabricated before being driven, jacked, screwed or hammered into the ground – and (2) bored or replacement piles – which first require a hole to be bored into which the pile is then formed, usually of reinforced concrete. Driven piles are either wood, reinforced concrete, or steel.

2.1.1 Prediction methods

The magnitude of the pile bearing capacity depends on pile and soil characteristics. The ultimate bearing capacity of a pile foundation is mainly governed by the soil properties (strength and stiffness response of the soil) and the pile-soil interface properties (Dijkstra, 2009). For the determination of the bearing capacity of a foundation pile it is possible to use a theoretical limit analysis (indirect method). In this analysis the basic parameters are the shear strength of the sand layer, characterized by its friction angle, and the effective soil stresses. But the capacity of driven piles cannot be entirely predicted by theoretical methods: stresses acting against the pile and the disturbed pile-soil contact zone are not known. In engineering practice a simpler, more practical and more reliable (direct) method has been developed, on the basis of a cone penetration test (CPT), considering this as a model test (Verruijt, 2001). It is assumed that the failure mode of the soil beneath a pile base is similar to the failure mode under a CPT-cone. In The Netherlands the formula of Koppejan, also known as the 'Dutch method', is often used for the prediction of the pile end bearing. This method takes into account the different cone resistances below (up to a depth of 0.7 to 4 times the pile diameter) and above (8D) the level of the pile point in such a way that the failure mode will prefer the weakest path. (Cone penetration tests in sands often show considerable variation in penetration resistance with depth.) In practice this method has proven to be successful for single piles (Dijkstra, 2009). The Dutch method corrects for the pile type (displacement or non-displacement piles) and the shape of the pile base using several pile class factors. The key parameters controlling cone resistance are the relative density Dr, the effective stress level σv, and the compressibility (Bolton et al, 1999). Soil compressibility can be related to grain crushing and rearrangement.

A pile may also have a bearing capacity due to friction along the length of the pile, what is the case for piles in sand layers. The maximum value of the skin friction can be determined using a friction cone. Because of the local values are often very small, the measured data is not very accurate. For sandy soils the friction therefore is correlated to the cone resistance in the Dutch method. Field test performed by
Imperial College London indicate that the local ultimate shear stress \( \tau_f \) can be related to the radial effective stress at failure \( \sigma'_{rf} \) by the simple Coulomb failure criterion:

\[
\tau_f = (\sigma'_{rf}) \cdot \tan(\delta_f)
\]

(2.1)

where \( \delta_f \) is the ultimate interface shear friction angle (Gavin and Lehane, 2002). The horizontal stresses in a soil mass are usually unknown. It is convenient in pile design to express the radial stress as the product of the initial vertical effective stress \( \sigma'_v \) and a coefficient of lateral earth pressure \( K \). But finding appropriate values for \( K \) is rather difficult, a fact reflected in the wide range of values for \( K \) proposed in the literature (Loukidis and Salgado, 2008). \( K \) varies with density and stress level. Both compressibility and dilatancy properties of granular materials tend to have also certain influence on \( K \) (Guo, 2010). However in literature some discrepancy is found in the shape of the shaft friction distribution along the pile, the maximum shaft friction occurs just above the pile base.

Pile design must be accompanied by in situ load testing. Eurocode 7 emphasises that pile design must be based on static load tests or on calculations that have been validated by these tests.

### 2.1.2 Pile load testing

The most reliable method to determine the maximum bearing capacity of a pile is to perform a static load test: the pile is loaded with a test load after driving to the design depth. This is expensive and time consuming. The usual procedure is to drive several piles in a group and use the adjacent piles for reactions to apply the load. A rigid beam spans across the test pile and is attached to the reaction piles. A large-capacity jack is placed between the reaction beam and the top of the test pile to produce the test load increments. The ultimate pile load is commonly taken as the load where the load-settlement curve approaches a vertical asymptote or as the load corresponding to some amount of pile head settlement (Bowles, 1996). It should be noted that there are many procedures for static pile load testing. Due to different loading paths, a pile will exhibit different load-settlement response influencing the conclusions. The results may also be influenced by group effects and pre-shearing effects. If a pile has been tested more than one time, the effects of previous loading tests may influence the capacity (Augustesen, 2006).

Piles in granular soils are often tested one day to a month after driving. In sand layers time lapse of one day is usually sufficient for excess pore pressures to dissipate, however, capacity gains were measured if load tests are made later on. In any soil sufficient time should elapse before testing to allow dissipation of residual stresses (Bowles, 1996). These residual stresses in the pile are caused by equilibrium between shaft resistance on the upper part of the pile that prevents the pile from moving upwards and shaft resistance on the lower part and the base resistance facing upward trying to push the pile upward (elastic rebound of the soil). As a result the pile is not fully unloaded. Neglecting this stresses while interpreting leads to erroneous interpretation of the instrument readings during a static pile load test (Dijkstra, 2009). Both the base and shaft resistance are fully mobilised for displacements equal to approximately 10% of the equivalent diameter \( D_{eq} \) for piles in sandy soils (Augustesen, 2006). The Dutch code NEN6745 includes guidelines for displacement-controlled load tests. The rate has to be larger than 0.005 \( D_{eq} \)/min and smaller than 2.5mm/min. One has to stop testing when the force on the pile is no longer increasing or when the pile head displacement becomes more than 20% of \( D_{eq} \).
2.1.3 Soil strength characteristics

Strength and stiffness are continuum descriptions of soil behaviour. In non-cohesive soil the continuum soil properties originate from the grain properties as well as the contact forces between the grains in the grain assembly. Sand, therefore, exhibits stress related soil properties, i.e. a higher stress state – what means larger contact forces – yields higher strength and stiffness. Strength and stiffness depend on the density and stress of the soil, called soil state (Dijkstra, 2009). At the surface sand will slip easily through the fingers, but under a certain compressive stress it gains an ever increasing stiffness and strength. This property can be seen in daily life in the packaging of coffee.

Another fundamental property of a granular soil, which defines its behaviour under stress and the strength appearing in specific conditions, is dilatancy (Sobolevski). Shear deformations of soils often are accompanied by volume changes. Loose sand has a tendency to contract to a smaller volume, and densely packed sand can practically deform only when the volume expands somewhat, making the sand looser. This is called dilatancy. The space between the particles increases when they shear over each other (Verruijt, 2001). The magnitude of dilation depends strongly on the density of the soil, with denser samples expanding more rapidly (Houlsby, 1991). When there is no resistance to dilatant movement of the soil, there is free dilatancy. Otherwise there is constrained dilatancy (difficult for quantitative evaluation). The angle of contact friction is higher for free dilatancy, but the shear resistance is higher when dilatancy is constrained (Sobolevski). In deep soil layers, failure mechanisms develop in condition of constrained dilatancy. The conditions around a pile impose kinematic constraint on soil movements, therefore high stresses will develop in a dilatant soil (Houlsby, 1991).

The deformations of a soil often depend upon time, even under a constant load. This is called creep. Although it is often assumed that sands show practically no creep, sand creeps at very high stress levels (Verruijt, 2001). Stress relaxation, the inverse phenomenon, is a drop in stresses over time at constant deformation. Creep can lead to rearrangement of soil particles into more stable configurations. The rate of sliding of two particles relative to each other depends on the ratio between shear and normal force at their contact. The magnitude of creep is thought to be a function of confining stress, stress ratio, relative density and angularity whilst log-linear and log-log relationships have been found for the time-dependent volumetric and shear strain response (Bowman and Soga, 2002).

Particle breakage during creep can also contribute to time dependent deformations of sands. During loading, the protrusions of grains can be ground off, producing fines. These fines fill the voids between larger particles. Crushed particles rearrange themselves progressively with time (Soga, 2004). Particle crushing can occur at high pressures, encountered in application such as pile driving. There are close relations between time effects, energy input and crushing. The stress-strain response of sand can be considered as a combination of frictional sliding and particle crushing. At high stresses, particle crushing dominates the deformations. Since particle crushing is a time dependent phenomenon, described as static fatigue, a close relation between time effects and crushing in granular materials has been established. (Karimpour and Lade, 2010).
2.2 Installation effects

The strength and stiffness of a pile foundation is influenced by the installation method. The axial stiffness of a jacked pile may differ from conventional driven or bored piles. Jacked piles have a high base stiffness due to the preloading of the soil below the base during installation, and the presence of residual base load (Deeks et al., 2005). Tension or unloading cycles may eliminate or reduce residual base load and reduce the resulting head stiffness. Establishing the stress conditions around piles driven in sand is crucial to understanding processes such as group interaction or capacity growth with time. Driving involves extreme stresses and strains, particle breakage and load cycling, creep and localised interface shear processes (Jardine et al., 2013). Since pile set-up is not observed on bored piles in sand, some aspect of the installation process of displacement piles is a necessary condition for set-up to occur (White and Zhao, 2006).

2.2.1 Pile driving

Almost all of the prefabricated piles are driven into the ground with an impact hammer. The kinetic energy from each blow of the ram displaces the pile deeper into the soil. Figure 2.1 depicts a typical pile head displacement versus time during one stroke. The maximum initial downward motion is the sum of the elastic and plastic response of the pile-soil interaction. The upward elastic motion of the pile during pile driving is called pile rebound. The final value of the pile head displacement is the permanent penetration for a blow, called set. Therefore, rebound is the difference between the maximum initial downward motion and the final set. Other deep foundations are typically put in place by excavation and drilling. Driving piles, as opposed to drilling shafts, is advantageous because the soil displaced by driving compresses the surrounding soil, causing greater load-bearing capacity.

![Figure 2.1: Pile head displacement during one hammer blow (Van Tol and Everts, 2007)](image)

2.2.2 Disturbance of the soil (state) close to the pile

Lehane et al. (1993) performed field load tests on instrumented piles that were installed in loose to medium dense sand using jacking. The radial (horizontal) stress acting on the shaft at a specific level showed a tendency to decrease as a pile is jacked into the ground. In other words, the radial stress decreases with increasing \( h/R \), where \( h \) is the distance from the pile toe and \( R \) is the pile radius. This phenomenon is called friction fatigue. Test results from Axelsson (2000) suggest also that the increase in average horizontal effective stress on the shaft due to loading up to failure is very small at the end of driving, also at great depths. These results from earth pressure cells on the pile shaft indicated that a distinct soil arch developed as a result of pile driving in sand, even at considerable depths. For better understanding of the influences of pile installation on the surrounding soil state, results from several model tests are discussed below.
Robinsky & Morrison (1964)
By means of radiography techniques, the displacements and compaction of sand around instrumented model piles had been studied by Robinsky and Morrison (1964). It has been found that the width of the soil displacement envelope varies, increasing with pile diameter, roughness and sand density. Within the envelope, displacement and compaction is evident. The main compaction zone, approximating the shape of a cone, is found beneath the pile point. At the zone limits, vertical expansion begins to take place, what is caused by the downward movement of the soil below the point away from the previously compressed soil. This results in the increase of the void ratio of the sand in the immediate vicinity of the pile, so creating a thin sleeve of loose sand and thereby setting up ideal conditions for a soil arch. It is suggested that a cylinder of dense soil, originally compacted by the pile point, encircles the loosened sand and prevents, by arching, the development of off full lateral earth pressure on the pile (Figure 2.2). The angle of internal friction of the loosened sand is considerably lower: little load transfer can occur by friction to the surrounding ring of dense sand. The effect of the high degree of compaction achieved below the pile point is thus partially lost by the formation of a sleeve of loose sand around the pile. Little, if any, compaction can occur along the pile as it progresses downward. It has to be noted that the tests were done in 1g model conditions using a relatively small test chamber. These conditions lead to a strong dilatant behaviour of the soil near the pile and a shallow failure mechanism in which failure planes reaching the surface occur. This is not realistic for deep foundations.

Figure 2.2: Schematic illustration of zones created during driving (a) and relative density and arching mechanisms in the surrounding soil due to driving (Augustesen, 2006).

Leung et al. (2001)
The centrifuge model test data from Leung et al. (2001) reveal that progressive redistribution (reduction) of the soil stresses around the pile tip and along the lower pile shaft is a cause for the reduction in pile penetration resistance during an installation lapse. The magnitude of the reduction is noticeable even for short lapse duration (40s) and increases with the duration (the rate of reduction attenuates rapidly with time). The longest lapse lasted 20 min. According to the authors, the stress redistribution may be associated with some overall densification of the soil matrix. The results suggest that the stress reduction close the pile tip is dominant in the lateral direction. A smooth pile was jacked (10 mm/s) in dry sand.

Kobayashi and Fukagawa (2003) used X-ray techniques for the characterization of soil displacements around a penetrating model pile. The authors embedded lead shot
in the sand around the pile and high soil stresses were reproduced. They concluded that the shape of the displacement field is influenced by the initial density of the sand. For loose conditions the displacement field is sideways and downward. For the dense conditions a more upward mechanism was found. Contraction in radial direction was found within the region around the edge; expansion in radial direction directly below the tip. Expansive deformation toward a vertical direction was observed near the edge and contraction right below the tip. A bulb-shaped core was formed below the tip of the cone and moved along with the cone. It has to be noted that the markers could influence the soil behaviour.

*White & Bolton (2004)*

White and Bolton (2004) used a image-based deformation measurement system to observe the soil deformation around a model pile. Tests were conducted in a plane-strain calibration chamber and a stress of 50kN/m² was applied to the surface. Two kinds of sands were used in the tests: Dog's Bay carbonate sand (DBS) and Leighton Buzzard silica sand (LBS). They found that the base resistance increased as the initial relative density increased. During LBS tests, a significantly higher base resistance was observed than for the DBS. This distinction can be attributed mainly to the much more significant tendency for crushing of particles in the carbonate DBS. A region of highly crushed soil below the pile tip was observed during tests. This was observed even in the LBS, which has high crushing strength. Slips of soil were observed to slide out from the nose cone and flow around the shaft of the pile ('coating' it as it penetrates). A central core of the nose cone is stationary relative to the pile tip, but the shoulders of the zone are not. Volumetric strains reveal the variation of density with offset from the pile shaft. Adjacent to the pile shaft, the soil has become denser, following irrecoverable volume change in the nose cone. In the LBS, the dilation close to the pile shoulder caused a local loose zone.

It was found that the deformation consists primarily of downward movement below the pile, moreover, horizontal displacement of 25μm are detected in the far field, at a distance of 5D from the pile centreline. The authors tracked the full displacement trajectories of soil elements. Figure 2.3 shows an example observed from a LBS test. As the pile approaches, the movement is generally downwards, with the soil element trajectory curving towards the horizontal as the pile passes. The final part of the element trajectory is upwards. After the pile tip has passed the soil element, the soil relaxes back towards the pile shaft. The 'tail' of the trajectories revealed the behaviour of the soil flow adjacent to pile shaft. It was also found that the zone of downward displacement is concentrated closely around the pile shaft, whereas the horizontal displacement decays slowly with radial distance from the pile. Furthermore, the strain level adjacent to the pile is lower than below it. It has to be noted that in a plane strain test all soil movements are enhanced.

![Figure 2.3: Soil element trajectory](image-url)
Dijkstra (2009)

Dijkstra (2009) investigated in a series of geotechnical centrifuge tests the change of soil density close to the pile by measuring change in electrical resistance of saturated Baskarp sand. The results indicate that for initial porosities between 0.386 and 0.439 a decrease of soil density is measured during pile installation (monotonic pile jacking). The magnitude of the porosity change during installation was largest for the densest sample. The loosening of the soil near the pile is most probably caused by dilatant soil behaviour during initial monotonic shearing. During the initial unloading, where the pile moves slightly upwards, the change in direction brings the soil locally to a denser state. During following pile load tests no significant porosity changes were observed. When the pile was unloaded again at the end of the pile test a further densification was observed. Cyclic loading with small amplitude contributed strongly to densification. The later larger amplitude cycles led to a limited densification only. During the cyclic tests an increase in shaft resistance was found in the small cycles, whereas a reduction was found in the large displacement amplitudes. According to the author, the densification near the pile shaft correlates well with the observed shaft resistance reduction. The stress build up near the pile shaft and the corresponding loosening of the soil during the pile installation stage is destructed by densification and loss of shaft resistance during the subsequent cyclic tests at large displacement amplitude (>0.5 mm). It has to be noted that the local initial porosity close to the pile could be higher than prepared for the complete sample. Or maybe the soil is densified during spin up of the centrifuge.

Yang et al. (2010) & Jardine et al. (2013)

Calibration chamber experiments were performed, investigating the evolution of stresses around instrumented model piles during simulated driving into a heavily instrumented sand mass. The soil stresses are shown to vary spatially relative to the pile tip location. As well as showing considerable radial variation, the stresses developed at any given depth build sharply as the tip approaches, and reduce rapidly as it passes. Clear decreases are evident between the behaviours seen close to the shaft during alternate penetration and pause periods. Load-cycling effects are most significant close to the shaft, where the local stress paths indicate a tendency for constrained dilatant behaviour, with radial stresses increasing, during loading. In contrast, markedly contractant radial stress reductions are evident on unloading (Jardine et al, 2013).

Pile installation in pressurised (dense) sand involves particle breakage and shear band formation. Breakage commences beneath the pile tip. The thickening with h/R of the annulus zone of highly compacted soil suggest additional breakage and abrasion linked to accumulating effects of shearing and cycling applied during installation, which was further augmented by later large-displacement cyclic loading. Sharp stress reductions leave the sand in a heavily overconsolidated and therefore dilatant state at point above the pile tip (leading to strong local dilation when the pile is loaded statically). The reduction of σ', with increasing h/R has been ascribed to local contraction resulting from extreme cyclic loading and shear zone densification (or growth) due to particle breakage. Material that has experienced extreme shearing imposed around the pile tip can be expected to have strongly time-dependent mechanical behaviour. Porosity may vary radially: the crushed material is displaced radially as the pile tip advances, developing concentric zones around the pile shaft involving different degrees of particle crushing (Yang et al, 2010).

In some cases the soil around the pile shaft loosened. This loosening near the pile shaft is probably dilatant behaviour of the soil (large shear deformation). This failure mechanism can be observed especially at low horizontal stresses. In higher stress conditions contractive behaviour near the pile shaft (at several particle diameters from the pile) has been observed, that resulted in a far field loosening of the soil. It can be stated that the density and stress state of the soil in the vicinity of the shaft change during pile installation from their original in situ values and the soil density
and the stress state after pile installation are not uniform, varying predominantly in the radial direction. In short, ideal conditions to trigger creep and stress relaxation.

The mechanism of interface contraction provides the initial conditions for set-up of displacement piles in sand, where ‘set-up’ refers to a time-related increase in shaft capacity, not related to pore pressure change. Immediately after passing the pile tip, the distribution of radial stress is as shown by the curve OA in Figure 2.4, created as the soil is pushed outward during flow around the pile tip. There is a sharp reduction in radial stress if the interface zone contracts due to cyclic shearing (friction fatigue). As a result, the radial stress acting on the pile shaft is lower than beyond the zone influenced by the interface contraction (B). Over time, the high gradients in the stress field around the pile relax, which features an increase in the radial stress acting on the pile shaft (C). This is similar to the mechanism in which high circumferential stresses, which initially ‘arch’ around the pile shaft, relax onto the pile surface over time, leading to an increase in shaft friction. (White and Bolton, 2004)

![Figure 2.4: Radial stress distribution due to interface contraction (White and Bolton, 2004)](Image)

### 2.2.3 Effects on/of neighbouring piles

Ekström (1989) showed, according to Axelsson (2000), from a summary of several studies that the zone of soil compaction, due to pile driving, normally takes place within a distance of approximately 6D from the pile shaft. No increase in horizontal stress on the shaft was observed as a result of driving an adjacent pile at a distance further than 9D. A relatively large increase in the horizontal stress was registered when a neighbouring pile was driven, although the driving was easy, indicating that the soil arch surrounding the pile was unstable and that a slight disturbance was enough to break down the arch. Axelsson (2000) mentions a direct increase in horizontal stress by approximately 10kPa on average due to the installation of a pile at a distance of 2.8m (12D). He also mentions smaller stress relaxation for a pile which is driven into soil previously disturbed by the driving of reaction piles. This would imply a less positive influence on possible time-dependent capacity increase.

The effect of interaction between two steel displacement piles (D=102mm, L=6m) is investigated by Chow (1995). During the installation of the second pile at 4.5D centre-to-centre to the last pile, a large increase in the effective radial stress around the first pile is found. The increase is largest when the base of the second pile is at the instrument level of the first pile. After the base had passed, a gradual decrease in effective radial stress is found. The end value of the effective radial stress is twice the initial value after installation of the first pile. Negative shear
stresses during the approach of the pile base indicate a downward movement of the soil around the first pile. When the pile base of the second pile has passed the instrument level of the first pile, the shear stresses become positive, indicating an upward movement of the soil. The instrument that did not face the second pile directly registered only a slight increase in negative shear stress after the pile base of the second pile is passed, indicating a different stress regime at the other side of the pile. After installation of the second pile the first pile gained 51% in shaft capacity and lost 43% in base capacity. The overall capacity increased by 19%; the reduced base capacity indicates an uplift of the pile. The second pile yielded the same total capacity as the first pile after the installation of the second pile. From these tests can be concluded that in pile groups the load distribution along the pile will be different from single piles. The pile will react more stiﬀ if the shaft resistance is relatively large compared with the base resistance. (The mobilization of the shaft resistance requires less pile-soil displacement than the mobilization of the base resistance.)

**Densification factor**

In the Netherlands, the CUR has published a calculation rule for the dimensioning of tension piles, taking into account the effect of installing neighbouring piles (CUR-publication 2001-4). The difference in bearing capacity of a single pile and a pile in a pile group is attributed to densification of the soil due to the installation of the displacement piles and a decrease in vertical effective stress during pulling. The effect of installing a group of soil displacing piles is described with an increase of the cone resistance. The densification influence is calculated using a cylindrical area around the pile, with a radius of $6D_{eq}$. Simpliﬁcations are a decrease in pore volume equal to the volume of the installed pile and a linear decrease from the edge of the pile to a distance of $6D$ from the centre of the pile. The ratio of the cone resistance after and before installation is called $f_1$. This is expressed in an increase in the relative density $D_r$. The factor $f_1$ can be determined using Jamiołkowskii’s relationship (1988) between cone resistance, soil stress and relative density:

$$f_1 = \frac{q_{cx1}}{q_{cx0}} = e^{2.93 \cdot D_r}$$

(2.2)

For the determination of $\Delta D_r$

$$\Delta e = \frac{1+e_0}{48}$$

(2.3)

can be used, with $e_0$ is the initial void ratio. By using only the factor $f_1$, the effect of increasing stresses in the soil on the shaft resistance is not taken into account. According Stoevelaar et al. (2012) the group effect will be strongly inﬂuenced by the initial soil density. Due to installation of a displacement piles in sands with low relative densities, predominantly densiﬁcation will occur within a zone around the pile and soil stresses will change hardly. In sands with high relative densities, the stresses increases, what causes, rather than densiﬁcation occurs. The increase of horizontal stresses will be signiﬁcant and lead to a higher shaft resistance.
2.3 Changing of bearing capacity with time

2.3.1 Pile set-up

The bearing capacity of displacement piles is often observed to increase with time, even after dissipation of installation-induced excess pore pressure. This phenomenon is known as pile set-up. In sand the dissipation of excess pore pressures can be expected to take place within a few hours (Axelsson, 2000). Set-up of driven piles in non-cohesive soils can be substantial, as several studies in recent years have shown. Set-up rates of 20%-170% per log cycle of time and increases in capacity by factors of 5 or more have been reported (Chow et al, 1998; Bullock et al, 2005). Set-up is a phenomenon closely related to the pile shaft, usually along the lower part of the pile. Little or no set-up is observed occurring at the toe (Axelsson, 2000). Nevertheless, the underlying mechanisms of set-up are not clearly understood. Long-term set-up, the increase in bearing capacity that takes place after the dissipation of excess pore pressures, in non-cohesive soil can roughly be divided into three main time-dependent causes, based on the hypotheses presented by Schmertmann (1991) and Chow et al. (1998):

1. Stress relaxation (horizontal creep) in the surrounding soil arch, which leads to an increase in horizontal effective stress on the shaft.
2. Soil ageing, which leads to an increase in stiffness and dilatancy of the soil, which implies large horizontal effective stresses acting against the shaft during loading. (Rearrangement of particles, leading to an interlocking between grains and the surface roughness of the pile.)
3. Chemical processes which may cause bonding between the sand particles and/or between sand particles and the pile surface.

It is suggested that the first two mechanisms start directly after pile installation and are also a part of the short-term set-up that takes place during the dissipation of excess pore pressures. But there are also delays observed in the commencement of set-up, beyond a period when pore pressures would be expected to dissipate (Axelsson, 2000). Jardine et al. (2006) supposed that the beneficial ageing processes commence within a few days after driving, although “this early period” requires further investigation. It is, however, unclear which one of the mechanisms is predominant under different conditions, and also how long this process continues. Bullock et al. (2005b), for instance, argue that the time-related increases are due either to a more dilatant response to loading or to gains in interface friction angle $\delta$. Pile set-up is found above and below the water line, with concrete, steel and timber piles, what suggests chemical processes have little influence (Bowman and Soga, 2005).

The magnitude of set-up is affected by many factors, e.g. density and stiffness of the soil; grain size distribution; grain strength; grain shape and structure; moisture content of the soil; concentration of salt, silica etc.; pile diameter; pile penetration depth, surface roughness and installation method (Axelsson, 2000; Alawneh et al., 2009). For instance silty soils, as well as well-graded soils, have a greater potential for interacting with the surface roughness of a pile, and angular shaped particles interact better than rounded particles, giving greater potential for large set-up. Results from Axelsson (2000) show that the increase in capacity is strongly dependent on the applied horizontal pressure, indicating that set-up is stress-level dependent and a function of depth. It is also suggested that ageing effects were related to the energy input during densification (Baxter, 1999). More violent soil disturbance results in greater ageing or capacity increase (Bowman and Soga, 2005). According to Chow et al. (1998), a few cases are reported where the
capacities of piles in sand reduced with time. One example is a closely spaced pile group.

Skov and Denver (1988) proposed a method to estimate the long-term pile capacity \( Q_t \) in cohesive and cohesionless soils from the short-term pile capacity \( Q_0 \) using the following correlation:

\[
Q_t = Q_0 \left( 1 + A \cdot \log \frac{t}{t_0} \right)
\]  

(2.4)

where:

- \( t \) = time after the end of initial driving.
- \( t_0 \) = initial reference time elapsed since end of driving (=0.5 day).
- \( Q_0 \) = pile capacity at time \( t_0 \).
- \( Q_t \) = pile capacity at time \( t \).

It is recommended to use \( A = 0.2 \) for piles in cohesionless soils. Chow et al. (1998) reported that, based on data collected from the work of 14 researchers, values of \( A \) vary from 0.25 to 0.75. Axelsson (1998) reported \( A \)-values from 0.2 to 0.8. The proposed correlation is very similar to the equation for the increase of the small strain shear modulus of sands with time (Baxter, 1999). Despite that Equation (2.4) was based on limited case histories in cohesionless soils, it remains the most commonly used method to estimate the increase in pile capacity with time. Several case histories presented confirm the equation, at least up to a month from the end of pile driving, see Figure 2.5. In almost all case histories the piles were dynamically tested. Due to the many affecting factors, Axelsson (2000) states that the set-up factor \( A \) should be considered to be site specific.

![Figure 2.5: Case histories of pile set-up where \( t_0 \) is between 0.5-4 days after EOD (Axelsson, 2000)](image-url)
2.3.2 Supposed mechanisms behind pile set-up

Lehane et al. (1993) conclude that the local shear stresses, $\tau_f$, acting against the pile shaft at failure follow the simple Coulomb failure criterion:

$$\tau_f = (\sigma_{rc} + \Delta\sigma_{rd}) \cdot \tan(\delta_f)$$  \hspace{1cm} (2.5)

where $\sigma_{rc}$ is the radial effective stress after installation; $\Delta\sigma_{rd}$ is the dilatant increase in local effective stress during pile loading; and $\delta_f$ is the ultimate interface shear friction angle. Using cavity expansion theory, the radial stress change resulting from a boundary displacement of $\Delta h$ applied to an elastic soil mass with a shear stiffness $G$ is given by:

$$\Delta\sigma'_{rd} = 2\Delta h \frac{G}{R}$$  \hspace{1cm} (2.6)

When all other factors are held constant, this expression suggests that $\Delta\sigma'_{rd}$ decreases with an increasing pile radius. Data from Lehane et al. (1993) reinforce this analysis and serve to illustrate the significant contribution of dilation to shaft capacity of small-diameter piles. It follows that particular care is needed when extrapolating from small-scale test to full-scale conditions. Setup may be considered the summation of two processes related to the above equations. The first directly involves soil creep under at-rest conditions, resulting in an increase in radial stress around the pile ($\sigma_{rc}$) with time. The second involves soil ageing, which is manifested as an increase in soil stiffness with time - greater than that accounted for by a decrease in soil volume - resulting in an increase in the change of radial stress during loading. The boundary displacement $\Delta h$ is a function of the surface roughness: the increase in stress is directly proportional to surface roughness. No greater effect of setup for smaller diameter piles, as is predicted by cavity expansion theory, is expected. The installation of larger piles will shear the soil to a greater extent. Therefore, greater recovery of stiffness is to be expected.

When a pile is loaded vertically, the behaviour of the soil surrounding the pile could be idealised as a thin soil band close to the pile which deforms in simple shear. As the soil shears, it dilates and the soil outside this zone is pushed outwards. The more the soil dilates, the greater the normal stress becomes. The more the soil crushes, the more the normal stress reduces. For a relatively smooth pile it may be that the failure will occur solely at the pile-soil interface and the dilatant properties of the soil may be much less important. This depends on the shear zone thickness relative to the diameter of the pile. If the zone remains the same thickness, this has implications for the use of small scale tests (Houlsby, 1991).

According to Chow et al. (1998), changes in $\sigma_{rc}$ with time control the magnitude of set-up. The dominant process is thought to be gains in the radial effective stresses acting on the pile shafts resulting from the relaxation, through creep, of circumferential arching established around the pile shafts during installation. Sand creep also produces stronger dilation effects during loading (micro-rearrangement). Increases in sand shear strength and stiffness with time (ageing) involving the reorientation of sand grains and possible cementing or micro-interlocking processes were also potential contributing factors resulting in enhanced interface dilation and possibly interface friction angles. The relative contributions of these causes are uncertain, but the creep-induced stress redistribution process is thought to have the greatest influence. Pile surface corrosion was discounted as probable major cause.
Research by Axelsson (2000) has broadly reinforced the conclusions of Chow et al. (1998). Results from the earth pressure cells indicated that a distinct soil arch developed as a result of pile driving. Moreover, the arch deteriorated with time, leading to an increase in normal (horizontal) stress on the shaft. This increase is attributed to the effect of stress relaxation (creep) in the surrounding soil arch. The increase in horizontal stress continues for several months and is approximately linear with the logarithm of time. Furthermore, there was a clear tendency for the increase in normal stress to be strongly depth-dependent, indicating a stress level dependency. It was observed that a relatively large pile movement was needed to destroy the set-up. The results showed that constrained dilatancy (confined by the surrounding soil arch) can play a major role in the behaviour of friction piles in sand, resulting in an extensive increase in horizontal effective stress on the shaft during actual loading. Furthermore, the results also showed that this effect clearly increases over time. It is concluded that the observed set-up was primarily caused by an increase in confined dilatancy over time due to soil ageing.


1. The penetration of the pile toe during driving pushes the soil to the side. Furthermore, sideways whipping of the shaft and lateral stress waves gradually create a soil arch with high tangential stresses and low normal stresses acting on the shaft.
2. The arch deteriorates with time due to stress relaxation (creep), which leads to an increase in horizontal (normal) stress on the pile shaft. This process can, for most cases, be expected to continue for at least several months.
3. In conjunction with the creep process, the soil particles become increasingly interlocked with the pile surface. The rearrangement of soil particles leads to an increasing dilatant behaviour and stiffness over time.
4. During the subsequent loading of the pile, the effect of constrained dilatancy will generate large horizontal stresses on the shaft. The cavity expansion theory can be used for calculating the increases in horizontal stress, provided the dilation and soil stiffness can be quantified.

Figure 2.6: Conceptual model of pile set-up (Axelsson, 2000)

The basic conditions for the conceptual model are:
- The pile should be some type of driven (rammed) displacement pile (i.e. the soil disturbance during driving has to be significant in order to create a soil arch and the conditions for stress relaxation).
- The soil behaves in a clearly dilatant manner (non-cohesive soil).
- There is a pronounced interaction between soil and pile surface (i.e. the pile has to have a sufficient surface roughness, and further, the size and the surface characteristics of the grains have to enable interaction).
Axelsson (2000) also assessed pile set-up by torque testing on driven rods. The increase in residual torque was considerably smaller, indicating that the effect of set-up is almost completely destroyed as a result of the disturbance caused by the rotation. The rods showed an increasing set-up rate over time, as a result of the preloading. This is the opposite to the normal set-up behaviour of piles: the general experience, as well as the results for the piles in his study, is that the set-up rate decreases due to the disturbance caused by (repeated) load testing. This suggests that a form of disturbance coupled with shear stress application in the direction of subsequent loading plays an important role.

Bowman and Soga (2005) performed a series of triaxial creep tests, with the aim of shedding light on the mechanisms behind displacement pile setup in granular soils. A creep-driven hypothesis, involving kinematically restrained dilation of the soil close to the pile shaft and soil ageing, is proposed to explain pile setup. When a pile is driven or jacked into a granular material, the soil below and adjacent to the pile undergoes a high degree of shearing. At the end of driving, high shear stress ratios are maintained by the kinematic restraint caused by the presence of the pile. Initially, as the soil creeps to redistribute loads, the dense soil tends to contract, but with time, it tends to dilate. Under kinematic restraint, the soil body cannot expand; instead, the effective mean stress of the soil increases, causing the radial stress at rest to increase around the pile. Influences on the creep behaviour, and hence degree of setup, are found to be particle shape and strength, relative density, and rate of loading. The application of small cyclic perturbations during creep is found to accelerate the onset of volumetric dilation and hence is proposed to accelerate setup.

A model scale investigation into the set-up of displacement piles in sand is reported by White and Zhao (2006). Thirty-two model piles were installed into two test chambers, and repeatedly load tested. In one test chamber the water table was kept fixed, whilst in the other chamber the level of the water table was cycled. Due to the small diameter of model-scale piles, corrosion causes dramatic set-up that is unrepresentative of prototype conditions. Smooth model piles that do not rust experience modest set-up in the absence of changes in water table. Cycles of the water table level significantly increase the amount of set-up.

A programme of first-time loading and retest experiments has been performed on ten field-scale open-ended steel pipe piles driven in predominantly dense silica marine sand by Jardine et al. (2006). The piles developed substantial increases in their tension shaft capacities during the weeks and months after driving. Extreme loading cycles, including pretesting to failure and subsequent unloading, degraded shaft capacity and disrupted the growth of capacity with time. Whereas high-level cycling caused damage, low-level one-way load cycling (with load amplitude around 20% of the shaft capacity) accelerated the beneficial ageing processes.

It is believed that the governing mechanism behind pile set-up (soil ageing and stress relaxation) is to some extent rearrangement of sand grains. In order to change the sand properties (increasing dilatancy) the sand particles must be more compactly packed around the pile. This implies that the tangential hoop stresses must deteriorate with time, resulting in increase of horizontal effective stresses acting against the pile surface. The two processes leading to set-up are interrelated. In literature, the processes of rearrangement of sand particles and deterioration of hoop stresses with time are both called creep and stress relaxation, but in reality it is probably a combination of the phenomena (Augustesen, 2006). It should be mentioned that chemical processes (including corrosion) which might bond sand particles to the pile surface and increase the soil stiffness influence set-up.
Soil ageing
For almost every soil, the properties of it undergo changes with time. Lithification and chemical reactions can change sand into sandstone over geological time. Over engineering time, the soil behaviour can alter as stresses redistribute after construction. Field data suggest that recent disturbed soils gain strength and stiffness over time at constant effective stress. This phenomenon is called ageing (Soga, 2004). Soil ageing can be defined as an increase in strength, stiffness and dilatancy of the soil, which is not directly caused by a corresponding decrease in density.

Although a small decrease in density is inevitable during particle rearrangements, it is not sufficient to explain the changes in the mentioned soil properties. Schmertmann (1991) believes that, during ageing, small particle movements occur which lead to a more stable soil structure. Particles will tend to orient themselves depending on the direction of the major principal effective stress. These movements will also result in an increased micro interlocking at the particle contacts. Small particle movements or grain slippage could cause that two particles fit together in a more stable arrangement, thus providing an increased resistance to shear. During shear, this interlocking would manifest itself as an increase in stiffness and dilatancy. It is also believed that there may be a redistribution of stresses within the soil skeleton, which may decrease local shear stresses between particles and increase normal stresses between particles, resulting in an overall stiffer soil structure.

Chemical processes are a possible cause of ageing. Historically, the most widespread theory used to explain aging effects in sands has involved some type of chemical bonding between particles. Generally, this bonding mechanism has been thought of as cementation, which would increase the cohesion of a soil. However strong evidence of a chemical mechanism being responsible for some aging, there are several reported cases in which cementation is an unlikely mechanism of aging. For instance, ageing is observed in dry sand and cementation in dry sand is unlikely. It is hypothesized that the ageing effects are a result of a combination of rearrangement of particles and precipitation of the soluble fractions on the sand. If the mechanism is chemical, then aging effects would be a site specific phenomenon, dependent heavily on the chemistry of the soil and the groundwater. In contrast, if a mechanical mechanism is responsible, then the effects of aging on sands would be a more general phenomenon. Biological activity may also contribute to aging effects, but can not fully explain the wide range of conditions where aging effects have been observed. (Baxter, 1999; Soga, 2004)

Baxter (1999) suggests that controlled laboratory test programs may miss the true mechanism(s) responsible for ageing effects in the field. He stated that perhaps the inherent variability of natural soil deposits contributes to soil ageing. It is known that stresses are not uniformly distributed throughout granular soils. The concepts of load chains and internal stress arching suggest that there are stiffer zones within soils which carry greater stresses than adjacent weaker zones. By imparting energy the equilibrium stress distributions can be disturbed. Perhaps ageing effects in sands are due to the gradual redistribution of stresses, creating new stronger load chains.

Changes in sand fabric with time after load application in one dimensional compression are measured by Bowman and Soga (2003). The experimental evidence showed that large voids became larger, whereas small voids became smaller, and particles group or cluster together with time. Based on these particulate level findings, it appears that the movements of particles lead to interlocking zones of greater local density. The result, with time, is a stiffer, more efficient, load bearing structure, with areas of relatively large voids and neighbouring areas of tightly-packed particles. The increase in stiffness is achieved by shear connections obtained by the clustering.

2.3.4 Effects of cyclic loads
A acceleration of set-up via cycles of the water table was measured by White and Zhao (2006). From the initial stress conditions after installation, any creep-induced equalization of stresses will tend to increase the radial stress at the pile wall, and hence cause set-up. Creep of a granular material can be attributed to small
perturbations in loading causing sliding at points of contact. Deliberate small cycles of loading will accelerate this mechanism of creep by increasing the frequency that contacts slide and loads redistribute. These cycles of loading could be applied to the soil near the pile either by the pile itself, or by ambient changes in either the total stress or the pore pressure. However, high-level cycles of pile loading cause friction fatigue, reducing the shaft capacity (White & Lehane 2004). When the pile is subjected to cyclic axial loads, the pile shaft friction reduces due to a decrease in the normal stress caused by cumulative contraction of the sand within the shear zone close to the pile-soil interface. Bowman and Soga (2005) mention that cycles of too large amplitude will break down setup, by altering the contact force network. For more crushable materials, over-vigorous application of cycles may also break particles, leading to volumetric contraction, which would result in stress relaxation - the reverse of setup. It is also stated by Baxter (1999) that time-dependent increases in the shear modulus are very sensitive to disturbance. A dynamic disturbance can partially or completely destroy any previous time-dependent increase in the modulus. The magnitude of shear strain was found to be the governing factor in the magnitude of the decrease.

Meijers et al. (2009) give a qualitative description of the behaviour of drained sand under cyclic loading. A vibration causes in fact that the grains want to slip about each other and can be seen as a small shear deformation. When the shear strains are small, there is contraction. There will be much more contraction in loosely packed sands than in dense sands. At increasing shear strains, the sand becomes looser. This is significant in dense sands, but hardly anything happens in loose sands. The strain at the transition between contraction and dilatancy is called the phase transformation point.

An experiment was conducted by Jardine et al (2006) to assess whether low-level cyclic loading could accelerate the postulated arching creep processes and enhance capacity growth. 1000 tension cycles were applied over a 16h period, with cyclic load amplitude of around 20% of the shaft capacity. The pile-head displacement amplitude maintained constant at ±1.25mm under this relatively low-level cycling, and very little permanent displacement developed. A tension test to failure performed on the following day indicated a capacity of 53% greater than five months earlier. A parallel pair of static tests was performed on a similarly pre-failed pile over a comparable set of test dates, but without applying the cyclic loading. This pile developed a far more modest recovery (17%) in capacity.

A series of cyclically loaded axial pile tests was conducted in the beam centrifuge by Li et al. (2010). A dense sand sample is used. Under displacement-controlled cyclic axial loads, the pile head maximum force reduces with increasing number of axial load cycles, especially in the first cycles. It is believed that this reduction is caused by contraction and radial stress reduction of the sand within the shear zone close to the pile-soil interface: in each load cycle, although dilation occurs in the shear zone firstly, the shear band contracts significantly after reversal of the loading direction, leading to some net contraction per cycle. The decrement increases with increasing cyclic axial load amplitude.

Cyclic experiments have been performed with an instrumented model pile in a calibration chamber filled with dense sand by Foray et al. (2010). Tests involving one-way low-level (0 to ±30 or 60% of the tension shaft capacity) cycling, two-way high-level cycling and static tension testing to failure have been reported where the local stress paths were measured at the pile-soil interface. The local radial stresses decreased under both types of cyclic loading. During one way loading again a constant value is reached. During two way cyclic loading the horizontal stress reduced to zero. Gains in shaft capacity after low-level cycling were associated with strengthened interface dilation and ascribed to local densification and improved interlocking. High level cycling (contractive and dilatant responses) leads to reductions in local stresses and large losses in shaft capacity. From the results an capacity increase can be expected when the half amplitude normalised by $D$ is smaller than 0.01 (in the same circumstances).
3 Physical modelling of pile set-up

3.1 Introduction

The possibility of simulating realistic time-dependent and group effects on the bearing capacity of driven piles by “static load tests” on model piles within the geotechnical centrifuge of Deltares will be investigated. It is proposed to run a pilot test. When the effects can be simulated within scaled model tests, a lot of effort (lower costs compared with field tests) can be saved. Another advantage of laboratory testing is the possibility to achieve specific conditions, e.g. homogeneous soil characteristics. So, parameter studies can be done in future. The bearing capacity of displacement piles in sand will be measured and the influences of (1) different installation method (jacked and pseudo-driven), (2) different pile configuration (single and group pile) and (3) cyclic loading will be investigated in the proposed tests.

For reliable results the soil state (stress and density) should be properly scaled in the model test. In the geotechnical centrifuge it can be expected that if the mechanical behaviour of the soil is strongly dependent on stress level, then such behaviour should be correctly reproduced in the model. Extrapolation of these results to full-scale conditions is limited by the uncertainties surrounding scale and size effects. Practical physical constraints of the physical model setup, like the size of the model container and the mean grain size, need to be chosen without negatively impacting the model tests. Although stress similitude could be achieved using the centrifuge, pile set-up is not necessarily accelerated at high g-levels in the same manner as consolidation. This means that long lasting centrifuge tests have to be performed. Results from Axelsson (2000) indicated that pile set-up, which take place during short-term, is mainly due to stress relaxation and soil ageing. According to Bullock (2005) the relation of Skov & Denver (1988) is also true for very short times. The test series will take approximately one day.

![Figure 3.1: The geotechnical centrifuge of Deltares.](image)
3.2 Geotechnical centrifuge modelling

3.2.1 Principle
The reason for using a centrifuge is to enable small scale models to feel the same stresses as a full scale prototype. The mechanical principle behind centrifuge modelling is rotating a mass body at constant radius with steady speed. To keep the mass in the circular orbit, it must be subjected to a constant radial centripetal acceleration \( \frac{v^2}{r} \) or \( r\omega^2 \), where \( \omega \) is the angular velocity. In order to produce this acceleration, the body must experience a radial force \( mr\omega^2 \) directed towards the axis. When the centripetal acceleration is normalized with \( g \), it can be stated that the body is being subjected to an acceleration of \( ng \), where \( n = \frac{r\omega^2}{g} \) (Muir Wood, 2004).

In a beam centrifuge a swinging platform is mounted on a horizontal beam. During spin-up of the centrifuge the base of the platform will move from the initial horizontal level to the vertical in-flight position. For the tests described below the geotechnical centrifuge of Deltares was used, see Figure 3.1. This beam centrifuge with 14 m span has a rather large functional internal height of 1.2 m, which resulted in large physical models at low acceleration level. This made the design of a model pile with all intended sensors achievable.

3.2.2 Similitude
If the prototype depth \( z_p \) is scaled with a scale factor \( n_z \) such that the model depth \( z_m \) becomes \( z_m = n_z z_p \), and similarly for the volumetric mass \( \rho_p \) and gravitational constant \( g \) by adding scaling factors \( n_p \) and \( n_g \) the scaled stress state yield:

\[
\sigma_{v,m} = n_p \rho_p n_g g_p n_z z_p
\]

where subscript \( m \) denotes model scale and subscript \( p \) prototype scale. \( n_p = 1 \), if the density of the grain material of the soil is not changed. The stress state is properly scaled if \( n_g n_z \) becomes 1. Therefore, the gravitational acceleration field should be increased. This is in particular important for problems involving stress gradients, like the stresses along a pile. An increase in stress state can also be simulated by applying a surcharge load. However, the problem then reduces to the study of situations where stress gradients are not of importance or neglected for other reasons. Not only the model depth, but all length measures in the model are linearly scaled. This includes the grain size and the pile dimensions. Relevant scaling factors (from prototype to model scale) are summarized in table 3.1.

<table>
<thead>
<tr>
<th>Table 3.1: Some scale factors for centrifuge modelling.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length</td>
</tr>
<tr>
<td>Mass density</td>
</tr>
<tr>
<td>Acceleration</td>
</tr>
<tr>
<td>Stress</td>
</tr>
<tr>
<td>Force</td>
</tr>
<tr>
<td>Strain</td>
</tr>
<tr>
<td>Displacement</td>
</tr>
<tr>
<td>Permeability (Darcy’s law)</td>
</tr>
<tr>
<td>Time (creep)</td>
</tr>
<tr>
<td>Velocity</td>
</tr>
</tbody>
</table>
Imperfections
The value of $n$ depends on the value of $r$. At any model that will be created, the value of $n$ will vary through the model. It is usually assumed that if the height of the model is less than about $0.1r$, then the variation in acceleration field is acceptable (Muir Wood, 2004). A minimum difference with a uniform acceleration field is achieved when $r$ is $1/3$ of the model height below the soil surface. The ratio of the height of the model to the effective centrifuge radius for the model determines the maximum ‘under’ and ‘over-stress’. The vertical stress in the model and prototype are now identical at a depth $z = 2/3 \ h_m$ (model height).

Further, it has to be noted that the acceleration field of $g$ is parallel at the scale of civil engineering (small dimensions relative to the earth’s radius) and the acceleration field within the centrifuge is radial. A free water surface will adopt a cylindrical profile in a centrifuge model. It might be proposed that this difference will be negligible when $B/r < 0.1$ (Muir Wood, 2004). In calculations can be counted with the actual variable and radial nature of the acceleration field.

Groundwater
The geotechnical centrifuge tests will be performed in fully saturated samples. In order to exclude consolidation effects the permeability should be sufficiently large. The soil deformation rate from installation of the displacement pile should not create a higher groundwater flux than the amount of outflow the soil structure can sustain. Large pore water pressure build up have to be avoided, which is governed by the soil’s hydraulic conductivity. The pile velocity should be much lower than the hydraulic conductivity. The hydraulic conductivity $k$ is defined by:

$$ k = \frac{K \rho_w \theta}{\mu} \quad (3.2) $$

where $K$ is the intrinsic permeability, $\mu$ the viscosity of the fluid, $\rho_w$ the pore water density and $g$ the gravitational acceleration. The model scale hydraulic conductivity $k_m$ can be derived as:

$$ k_m = \frac{n_k K p n_p \rho_w p n_g \theta_p}{n \mu_p} \quad (3.3) $$

When the pore water fluid is not scaled ($n_\mu = 1$ and $n_\rho_w = 1$), the soil is not substituted with another grain material and considering that the intrinsic permeability is relatively independent of soil and permeant ($n_K = 1$), the hydraulic conductivity will be scaled by $n_g$. In these experiments the pore fluid is deliberately not scaled, which results in a higher value for the hydraulic conductivity in the model tests than found in prototype conditions. Therefore, consolidation effects are less likely to occur.

3.2.3 Size and scale effects

Boundary effects
Due to the finite space in a geotechnical centrifuge, it is necessary to construct the model within the finite boundaries of a model container. It is recognised that these boundaries lead to slightly inaccurate simulation of field situations that are of ‘infinite’ lateral extent. In practice it is desirable that the boundaries should be sufficiently remote that they do not have significant impact on behaviour at the point of interest in the model. To minimize the boundary effects, the distance to the closest boundary should be taken larger than $20D$ in very dense sands and about $15D$ for loose sands (Schnaid and Houlsby, 1991)
**Grain size**
According to the scaling laws, it would seem that the soil particle size should be reduced by a factor of $n$. For instance, if the particle size is 0.2 mm in the prototype, the sand particle size should be 0.005mm for a 40g centrifuge test. So, in a centrifuge test clay might be considered as representing fine sand at equivalent prototype scale. However, clay has very different stress-strain behaviour compared to fine sand. Therefore it is common to use similar particle sizes, having realistic stress-strain behaviour.

Particle size effects are important for the interaction between soil and structure - a mechanism of formations of shear bands at the interface. Balachowski (2006) mentions that the thickness of a shear band is mainly related to the average size of grains. Observations of the shear zone have shown that the thickness of the shear bands typically approaches ten sand grain diameters. The ratio of grain size to pile diameter is often unusually large in laboratory studies (Loukidis & Salgado, 2008). This may affect the shaft resistance. The relative size of the pile compared to the grain size should not deviate from prototype scale due to stress scaling. If improperly scaled, the volumetric soil behaviour near the pile is unrealistic (Lehane et al., 2005). To avoid this effect for dense sands, the ratio $D/d_{50}$ should be larger than 100 (Garnier and König, 1998). However Balachowski (2006) states that the diameter of small piles should be larger than $200d_{50}$ to avoid scale effects, when the soil is highly dilative or contractive within the interface.

Gui et al. (1998) report cone penetration tests in sand in centrifuge models, considering a variety of effects including the ratio of the pile diameter to the average grain size $d_{50}$ of the sand. As this ratio falls below about 20 there is some evidence of effect on the tip resistance, with significant effect observed when the ratio drops to about 10.

**Pile roughness**
Two distinct types of soil behaviour within the interface between soil and structure can be identified: dilative and contractive behaviour during shearing. Differences in shaft surface roughness will influence the degree of the interaction between soil particles and the shaft. Surface roughness has a strong effect on the dilation of the soil. This has been observed in interface shear tests by Uesugi et al. (1988). It was shown that the zone, where dilatancy and particle reorientation took place, was approximately 5-8 mm thick ($5d_{50}$) for a rough interface. For a primarily smooth surface, slipping occurred at the interfaces and no shear zone developed. For a rough surface failure takes place in the soil at a distance from the shaft and the interface friction angle is close to the soil to soil friction angle. A significant scale effect for shaft friction can be expected for rough model piles. Strong scale effects are measured for small models and great initial normal stress (Balachowski, 2006).

The roughness of the interface depends on the roughness of the structure and the grain size of the soil. For the modelling of the interface roughness, the normalized roughness should be the same in the model and in the prototype. The normalized roughness is defined as the ratio of the absolute roughness $R_{\text{max}}$ to the $d_{50}$ of the sand $R_n = R_{\text{max}}/d_{50}$. $R_{\text{max}}$ is the maximum asperity height measured over a specific length $L_r$ (usually = $d_{50}$) along the frictional interface. See also Figure 3.2. The interface is perfectly rough if $R_n$ (high shear strength and dilatancy) is larger than 0.1 to 1. For small values of $R_n$ ($\leq 0.02$) the interface is smooth: the ratio between maximum shear stress and normal stress is low and no dilatancy occurs. In between is a zone of intermediate interfaces. For these interfaces, the ratio of shear stress to normal stress increases with increasing roughness. (Garnier and König, 1998).
According to Axelsson (2000) a pile with a rough surface would have greater potential for large pile set-up than pile with a smooth surface. Typical surface roughness’s of concrete piles of $R_{\text{max}} = 335.3\mu m$ and $R_a = 28.0\mu m$ (Pando, 2006) and $R_{\text{max}} = 72\mu m$ and $R_a = 7.6\mu m$ (Axelsson, 2000) were noted. $R_a$ is defined as the arithmetic average value of the profile departure from the mean line along the profile length.

**Pile dimensions**

The most model piles are relatively rigid. The pile length-diameter ratio is often small compared with a typical Dutch situation, where $L/D$ is around 50. The pile diameter has to be large enough in order to house all instrumentation, but should still have a reasonable $L/D$ ratio. The L/D ratio will also influence the ratio between base and shaft resistance.

**Mobilization of shaft friction**

Garnier and König (1998) mention the required displacement to mobilize ultimate shaft friction. For model piles, this is often comparable to the required displacement for full-scale piles (i.e. 2-10mm).
3.3 Test setup

The centrifuge tests focus primarily on the question whether the time dependency of pile capacity can be studied in the centrifuge. Because creep and relaxation processes do not scale with an increasing g-level, long lasting centrifuge tests have to be run. All piles will be installed in a single sample preparation in the container. Two piles are instrumented: pile 1, which can be considered as a single pile, and pile 2, a group pile in a group of 3 piles. Pile 3 and 4 are ‘dummy’ piles. The piles will be installed in flight. To study time-effects, displacement-controlled ‘load tests’ on the instrumented piles are planned at 1, 10, 100 and 1000 minutes after installation or after installation of the neighbouring piles. Two almost identical test series are planned, in order to demonstrate the repeatability and increase the reliability of the results. The centrifuge continues spinning from the start of the installation until the final load test. Vacuum will be applied within the centrifuge chamber during the whole test.

Important starting points for the design of the tests are (Axelsson, 2000):
- The pile should be some type of driven (rammed) displacement pile (i.e. the soil disturbance during driving has to be significant in order to create a soil arch and the conditions for stress relaxation).
- The soil behaves in a clearly dilatant manner (non-cohesive soil).
- There is a pronounced interaction between soil and pile surface (i.e. the pile has to have a sufficient surface roughness, and further, the size and the surface characteristics of the grains have to enable interaction).

3.3.1 Mechanical setup

The test will be performed at a gravity acceleration of 40g, so \( n \) is 40. The gravitation acceleration level is ‘correct’ at 110mm below the surface level (1/3 of the penetration depth). The total arm length \( r \) will be 5414mm.

The test setup is shown in Figure 3.3 and 3.6. The dimensions of the test setup are given in Table 3.2. The requirement to minimize the boundary effects is not fully fulfilled (the distance to the closest boundary should be taken larger than 20\( D \)). The \( L/D \) ratio of the model pile is 20. The model height is about 0.11 of the arm length \( r \), while the penetration depth is 0.06\( r \). The model width is 0.17\( r \), but the distance between the single pile and the pile group is only 0.06\( r \). So, it is expected that the variation in acceleration field is acceptable.

![Figure 3.3: Top view test setup: model piles (D=16 mm) in a 900mm diameter container.](image_url)
Table 3.2: Dimensions of the test set-up

<table>
<thead>
<tr>
<th>Dimension</th>
<th>Measurement</th>
<th>Prototype</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pile diameter</td>
<td>1.6 cm ($A_{\text{base}} = 2 \text{cm}^2$)</td>
<td>64 cm</td>
</tr>
<tr>
<td>Penetration depth</td>
<td>$\pm 32 \text{cm} (20D)$</td>
<td>12.8 m</td>
</tr>
<tr>
<td>Height of soil body</td>
<td>600 mm</td>
<td></td>
</tr>
<tr>
<td>Diameter container</td>
<td>900 mm</td>
<td></td>
</tr>
<tr>
<td>Distance pile to wall</td>
<td>300 mm (18D)</td>
<td></td>
</tr>
<tr>
<td>Distance single pile to group</td>
<td>300 mm</td>
<td></td>
</tr>
<tr>
<td>Distance between piles in group</td>
<td>64 mm (4D)</td>
<td></td>
</tr>
</tbody>
</table>

3.3.2 Soil model

The strongbox will be filled with Baskarp sand. Fine sand is selected to reduce particle-scale effects. The Baskarp characteristics are depicted in Table 3.2 and Figure 3.4. The gradation of the particle size $C_u (d_{50}/d_{10})$ is 1.4, indicating a uniform distribution. Baskarp sand shows a strongly dilatant behaviour in a dense state (Dijkstra, 2009). The sand will be prepared by dynamic compaction of a fully saturated sample (Rietdijk et al., 2010). Relative densities, $D_r$, of 66.3% in test series 1 and 66.8% in test 2 were reached. During preparation care is taken to prevent excess vibration of the model container. Although the preparation method produces homogeneous samples, local density irregularities cannot be excluded. During the tests the sand sample is fully saturated. To prevent dehydration due to evaporation ca. 2 cm of water will be added after spinning up.

Table 3.2: Baskarp sand characteristics

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density grains</td>
<td>2.65 [kg/m³]</td>
</tr>
<tr>
<td>$d_{50}$</td>
<td>118 [μm]</td>
</tr>
<tr>
<td>$d_{50}/d_{10}$</td>
<td>1.4</td>
</tr>
<tr>
<td>$n_{\min}$</td>
<td>35.2%</td>
</tr>
<tr>
<td>$n_{\max}$</td>
<td>47.6%</td>
</tr>
</tbody>
</table>

Figure 3.4: Particle size distribution of the Baskarp sand from two different bags.
3.2.3 Model piles and instrumentation

The tests will be performed with rough pile surfaces, to have enough interaction between soil and pile surface. The suitable roughness will be obtained by a fine screw threat along the entire pile surface. See Figure 3.5. A mean maximum roughness ($R_{\text{max}}$) of 32.3μm and a mean average roughness ($R_{\alpha}$) of 8.47μm are reached. The interface behaviour is expected to be (total) rough: the normalized roughness $R_{n} = 0.27$. The $D/d_{50}$ ratio is 136. According to Balachowski (2006), scale effects for the shaft friction are expected.

![Image](image_url)

**Figure 3.5:** Detail of pile 1 after test series 2.

Only two piles are instrumented (pile 1 and 2). The pile instrumentation exists of measuring the pore pressure at the pile tip, the forces at the pile base (actually 15mm above the tip) and at 75mm above the pile tip (strain gauges). The friction force at the element between 15mm and 75mm from the pile tip can be determined by subtraction. The force at the pile head will be measured using a load cell. The total friction on the pile shaft can be determined by subtracting the measured force at the pile base from the measured force at the pile head. Piles 3 and 4 are 'dummy' piles. For all piles the displacements of its controlling plunger(s) will be measured. Also the temperature and the pore pressure at the bottom of the model will be measured during the whole test series. Data from all channels will be acquired at different sample frequencies during different test phases: 1200samples/s (installations), 400samples/s (cyclic loading) and 10samples/s (the rest).
Table 3.3: Observed maximum errors during calibration

<table>
<thead>
<tr>
<th>Quantity</th>
<th>Max. abs. error (kN/kPa)</th>
<th>Max. rel. error (% of total range)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_{\text{head}}$</td>
<td>0.008 kN</td>
<td>0.081</td>
</tr>
<tr>
<td>$F_{\text{base}}$</td>
<td>0.072 kN</td>
<td>0.725</td>
</tr>
<tr>
<td>$F_{\text{base+friction}}$</td>
<td>0.057 kN</td>
<td>0.570</td>
</tr>
<tr>
<td>Pore pressure</td>
<td>1.875 kPa</td>
<td>0.375</td>
</tr>
</tbody>
</table>

The instrumentation of the piles and the load cells are calibrated. The maximum errors are depicted in Table 3.3. It has to be noted that the maximum error for the base load of pile 2 is smaller: the absolute value is 0.011kN. The maximum relative error for the pile displacements is almost 0.5%. That means an absolute error of 0.22mm for the small plunger, which has a range of 50mm, and about 2.5mm for the large plungers. Vibration of the cylinders causes maximum displacements of 0.005mm for the small plunger and 0.05mm for the large plunger.

White and Zhao (2006) mention that if set-up is due to creep, and if creep arises from random perturbations of load, these random loads must be replicated within the centrifuge. Perhaps are the vibrations and the noise levels of the whole system sufficient to trigger creep of the sand.

Figure 3.6: Cross section of the test setup.
3.4 Test program

3.4.1 Installation of the piles

The installation of the model piles into the sand mass will be performed in a displacement controlled way. It is tried to simulate a ‘real’ pile-driving signal (hammer blows with rebound) to install pile 1. The soil disturbance during installation has to be significant in order to create a soil arch and the conditions for stress relaxation. Due to practical limitations, the other piles (pile group) will be jacked monotonically into the sand. The workspace and the amount of available plungers are limited. On the other hand, a comparison between two different installation methods can be made.

Earlier test results of model pile installation into Baskarp sand with a relative density of around 65%, which were performed within the centrifuge of Deltares, are depicted in Figure 3.7. From this results, a cone resistance between 12 and 18MPa is expected when the pile tip reaches the desired depth of 320mm ($\sigma'_v = 125kPa$). So, a maximum force on the pile base between 2.4 and 3.6kN is expected.

![Figure 3.7: Relation between $q_{base}$ and $\sigma'_v$ during pile installation in Baskarp sand ($D_r=65\%$) based on earlier performed centrifuge model tests performed at Deltares (at 35g, 40g and 80g levels and with $D_{pile}$ of 11.3 and 15mm)](image)

Starting point for the cyclic installation of pile 1 is a penetration rate of 25blows/25cm in the field, which means a progressive motion of 10mm/blow. Other assumptions are a pile-driving frequency of 50blows/min and hammer blow duration of 30ms. The penetration rate will not be scaled in order to minimize dynamic effects during installation. To have enough displacement relative to the grains, the final ‘set’ per installation cycle will be not scaled with a factor of 40. Chosen is a penetration rate of 1mm/s (1.2mm/cycle). The monotonically jacked piles will be installed also with a penetration rate of 1mm/s to the desired depth of 320mm.

Two plungers will be used for the cyclic installation of pile 1 (Figure 3.6). A large plunger will jack the pile to depth, while a small plunger will push downward and pull upwards. Two options have been explored to obtain a realistic pile-driving signal. A sinusoidal and a triangular signal are compared with each other. It seems that the sinusoidal signal is too evenly for simulating a hammer blow, therefore a triangular signal is chosen (see Figure 3.8).
Based on the load-settlement curve (NEN6743) a pile rebound of 1 to 2%\(D\) is assumed. The first part of the displacement is assumed to be an elastic soil deformation. In reality, the rebound can be larger and depends on the penetration depth (soil state). However, a rebound of 2%\(D\) (0.32mm) is chosen. It is expected that tension forces at the pile head are excluded for this ‘safe’ choice. In reality the head force is zero when the hammer loses contact with the pile. It is thought that the chosen upward movement of the pile during an installation cycle is sufficient for changing the direction of the interface soil shearing, which has to lead to soil contraction within the shear band. When no differences in installation method or no time effects are seen, the amount of rebound will be increased to 3% of the pile diameter in the second test series.

At Deltares, it is tried earlier to perform ‘cyclic’ installation of model piles within centrifuge tests (Stoevelaar et al, 2011). An upward movement of 1%\(D\) during an installation cycle was applied at gravitation acceleration levels of 40\(g\) and 80\(g\). The force on the pile head dropped to respectively ±40% and ±70% of the maximum load during an installation cycle. These results correspond with the expected higher rebounds at higher soil stress levels. A rebound of 5%\(D\) was also applied at a gravity acceleration level of 80\(g\) and. The force at the pile head dropped here to around zero. It is concluded that a rebound of 2%\(D\) seems a right and ‘safe’ choice at a level of 40\(g\).

### 3.4.2 Load tests and cyclic loading

The instrumented piles will be tested several times during the two runs. Before installing the dummy piles, the middle pile will be tested also a several times. So, a comparison between the behaviour of the instrumented piles based on the installation method can be made. Important are the waiting periods between the tests on the piles. Simulations of static load tests will be performed at \(t = 1, 10, 100\) and 1000min after installation of the piles itself or after installation of the neighbouring piles. These tests will be performed in a displacement-controlled way, with a penetration rate of 0.002mm/s up to a pile head displacement of 10%\(D\). Deltares performed earlier ‘static load test’ in centrifuge model tests at a rate of 0.00167mm/s. Each test will bring the pile 1.6mm deeper into the soil. After five pile tests the base level is lowered 8mm. Between the load tests, no force will act on the pile head. After a test the pile has to be pulled upwards manually in order to discharge the pile head.
After testing the pile at 1000min after installation, the pile head load will be reduced to 50% of the maximum load measured during the last test. Subsequently 50 cyclic displacements will be performed on the pile head during 1min. The pile movement will be 0.1mm, which is the difference between the highest and the lowest level during one cycle. After these cyclic displacements, the pile will be tested for the last time.

3.4.3 Test scheme

Because of the long operational time of a test series, the tests will be performed combined in one flight. Proposed is to start the program of pile 2 after the third test on the pile 1 (at $t = 100$ min after installation). Before applying the cyclic displacements on the single pile, the group pile has to be tested first to determine the pile behaviour at 1000min after installation. It is thought that the applied displacement cycles can influence the other pile behaviour. The ‘load’ tests on the piles take 14 min. This hinders the intended pile load test after 10 min. The second test will be performed 10min after the first test now. The third test will be performed at $t = 100$ min after installation again. Tables 3.5 and 3.6 depict a global overview of what will be done during the two test runs.

Table 3.4: Global test scheme for the first test series.

<table>
<thead>
<tr>
<th>Series 1</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Cyclic installation of pile 1</td>
<td>1mm/s*; desired amplitude of the small plunger: 1.52mm</td>
</tr>
<tr>
<td>1st test on pile 1 (&quot;1min&quot;)</td>
<td>0.002mm/s; 10% $D$; the pile is not unloaded in advance</td>
</tr>
<tr>
<td>2nd test on pile 1 (&quot;10min&quot;)</td>
<td>0.002mm/s; 10% $D$</td>
</tr>
<tr>
<td>3rd test on pile 1 (&quot;100min&quot;)</td>
<td>0.002mm/s; 10% $D$</td>
</tr>
<tr>
<td>Installation of pile 2</td>
<td>1mm/s</td>
</tr>
<tr>
<td>1st test on pile 2 (&quot;1min&quot;)</td>
<td>0.002mm/s; 10% $D$; the pile is not unloaded in advance</td>
</tr>
<tr>
<td>2nd test on pile 2 (&quot;10min&quot;)</td>
<td>0.002mm/s; 10% $D$</td>
</tr>
<tr>
<td>3rd test on pile 2 (&quot;100min&quot;)</td>
<td>0.002mm/s; 10% $D$</td>
</tr>
<tr>
<td>Installation of pile 3 &amp; 4</td>
<td>1mm/s</td>
</tr>
<tr>
<td>4th test on pile 2 (&quot;1min&quot;)</td>
<td>0.002mm/s; 10% $D$</td>
</tr>
<tr>
<td>5th test on pile 2 (&quot;10min&quot;)</td>
<td>0.002mm/s; 10% $D$</td>
</tr>
<tr>
<td>6th test on pile 2 (&quot;100min&quot;)</td>
<td>0.002mm/s; 10% $D$</td>
</tr>
<tr>
<td>4th test on pile 1 (&quot;1000min&quot;)</td>
<td>0.002mm/s; 10% $D$</td>
</tr>
<tr>
<td>7th test on pile 2 (&quot;1000min&quot;)</td>
<td>0.002mm/s; 10% $D$</td>
</tr>
<tr>
<td>Cyclic loading pile 1</td>
<td>50 cycles; sinusoidal movement with an amplitude of 0.05mm**</td>
</tr>
<tr>
<td>Cyclic loading pile 2</td>
<td>50 cycles; sinusoidal movement with an amplitude of 0.05mm</td>
</tr>
<tr>
<td>5th test on pile 1</td>
<td>0.002mm/s; 10% $D$</td>
</tr>
<tr>
<td>8th test on pile 2</td>
<td>0.002mm/s; 10% $D$</td>
</tr>
</tbody>
</table>

*) The maximum velocity of the pile during an installation cycle is circa 50mm/s.
Table 3.5: Global test scheme for the second test series.

<table>
<thead>
<tr>
<th>Series 2</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Cyclic installation of pile 1</td>
<td>1mm/s; <em>desired amplitude of the small plunger: 1.65mm</em></td>
</tr>
<tr>
<td>1st test on pile 1 (&quot;1min&quot;)</td>
<td>0.002mm/s; 20%*D</td>
</tr>
<tr>
<td>Installation of pile 3</td>
<td>1mm/s</td>
</tr>
<tr>
<td>Installation of pile 2</td>
<td>1mm/s</td>
</tr>
<tr>
<td>1st test on pile 2 (&quot;1min&quot;)</td>
<td>0.002mm/s; 20%*D</td>
</tr>
<tr>
<td>2nd test on pile 1 (&quot;100min&quot;)</td>
<td>0.002mm/s; 20%*D</td>
</tr>
<tr>
<td>2nd test on pile 2 (&quot;100min&quot;)</td>
<td>0.002mm/s; 20%*D</td>
</tr>
<tr>
<td>3rd test on pile 2 (&quot;1000min&quot;)</td>
<td>0.002mm/s; 20%*D</td>
</tr>
<tr>
<td>3rd test on pile 1 (&quot;1000min&quot;)</td>
<td>0.002mm/s; 20%*D</td>
</tr>
<tr>
<td>Cyclic loading pile 1</td>
<td>50 cycles; sinusoidal movement with an amplitude of 0.05mm</td>
</tr>
<tr>
<td>4th test on pile 1</td>
<td>0.002mm/s; 20%*D</td>
</tr>
<tr>
<td>Installation of pile 4</td>
<td>1mm/s</td>
</tr>
<tr>
<td>4th test on pile 2 (&quot;1min&quot;)</td>
<td>0.002mm/s; 20%*D</td>
</tr>
<tr>
<td>5th test on pile 2 (&quot;100min&quot;)</td>
<td>0.002mm/s; 20%*D</td>
</tr>
</tbody>
</table>

The main differences between the two test series, which are decided in between both series, are:

- Larger amplitude of the small plunger is applied during the pseudo-driven installation of pile 1 in series 2. The pile head load will be smaller at the end of each cycle.
- Higher load is kept on the piles between the pile tests in series 2, in order to account for the own weight of the piles.
- Larger pile displacement is applied for testing (20%*D), in order to ensure that pile base and shaft friction will be fully mobilized.
- A different installation order of the piles of the pile group is applied. In the second test series, pile 3 is installed first.
- The piles are not tested 10min after installation. It is thought that these tests cause too much disturbance and give little information.
4 Test results

This chapter depicts the results of both test series. It is good to know in advance that the centrifuge stopped spinning during the second test series. It was intended to continue spinning the centrifuge one day longer, but there was an emergency situation: the centrifuge did not control its speed sufficiently. This is the reason why pile 2 is not tested at 1000min after installation of pile 4 and no cyclic loading is applied on this pile during the second series. During installation of pile 3 in the second test series, a problem with the data acquisition program occurred. Decided is to sample the data with a lower frequency (1sample/s) from that moment on. Pore pressures are only measured during the second test series. No distinction between shaft friction acting on the upper section of the pile and local shaft friction (acting on the friction element) is made for pile 2 in series 2: the provided values of $F_{\text{base+ local friction}}$ show an erratic pattern and are regarded as unreliable.

All presented values of $F_{\text{head}}$, $F_{\text{base}}$ and $F_{\text{base+ local friction}}$ are obtained by subtracting the initial values, measured just before installation, from the actual measured data values. In this way, the own weight of the piles is eliminated from the results. The influence of the increasing acceleration field with depth on the weight of the piles is negligible. The height difference between the situation before installation and after the final 'load' test causes an increase of 7% in the own weight (35N). On the other hand there is a buoyant force of around 25N on the pile, when the base reaches a depth of 320mm. The values of the pile base level (depth) are corrected in such a way that zero means that the pile base is at surface level.

Figure 4.1: Test setup
4.1 Installation of the model piles

Table 4.1: Initial levels of the pile base after installation

<table>
<thead>
<tr>
<th>Series</th>
<th>Series 1</th>
<th>Series 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pile 1</td>
<td>317.1mm</td>
<td>316.1mm</td>
</tr>
<tr>
<td>Pile 2</td>
<td>315.9mm</td>
<td>298.5mm</td>
</tr>
<tr>
<td>Pile 3</td>
<td>323.9mm</td>
<td>323.9mm</td>
</tr>
<tr>
<td>Pile 4</td>
<td>320.8mm</td>
<td>321.8mm</td>
</tr>
</tbody>
</table>

Table 4.1 depicts the reached levels of the pile base after installation. The base of pile 2 did not reach a depth of $20D$ (320mm). The measured axial forces on the pile head and base during the ‘cyclic’ installation of pile 1 are plotted in Figure 4.2. Only the maximum and minimum values during an installation cycle are given. The needed forces to reach depth are different for the two series. The maximum penetration forces in series 1 are almost linear with depth, while there can be observed a kink for the maximum values in series 2. This difference is dominated by the response of the pile base. It can be observed from the figure that there was more unloading of the pile base during the second installation. This difference is ascribed to the slightly larger displacement amplitude of the small plunger in test series 2, a difference is about 0.1mm (Figure 4.3). If it is known that $q_{base}$ is mainly a function of effective soil stresses and soil relative density, it is reasonable to explain the lower base resistance in series 2 by means of lower stresses around the pile base. Assumed is an approximately same density of both sand preparations. It is believed that the larger upward movement during an installation cycle causes more unloading of the soil around the base.

![Figure 4.2: Maximum and minimum forces during an installation cycle for the whole installation of pile 1.](image)

The intended movement of the small plunger for the ‘cyclic’ installation is not reached. This appears not to be a result from soil resistance: above the soil surface, the desired amplitude is not reached either. During series 1, the mean downward and upward displacement were approximately 1.1mm and 0.2mm. For series 2 these numbers are 1.2mm and 0.3mm. In series 1, the cyclic motion of the small plunger was stopped just before the desirable depth was reached. From there on, the pile was only jacked to depth by the large plunger. A stiffer response can be seen for the monotonically jacked part of the installation.
Figure 4.3: Movement of the pile head during two installation cycles of pile 1.

Figure 4.4 depicts the measured axial forces on the pile head and base during the monotonically jacked installation of pile 2 (both series). Compared with the cyclic installation, higher base resistances are observed during the jacked installation. This higher base resistance can be explained by the more building up of stresses around the pile point due to monotonically jacking. During the first series, this difference between both installation methods is small, indicating a more realistic pile driving simulation during the second series. Differences in measured forces acting on pile 2 during installation in both test series are mainly explained by the installation order of the pile group. Due to the prior installation of pile 3 the surrounding soil is densified, resulting in higher base and shaft resistances. The difference in shaft friction when the pile is at final depth is a factor of 1.25, while the predicted $f_r$-value is 1.10. It should be borne in mind that two samples were prepared.

Figure 4.5 depicts the shear stresses acting on the instrumented pile shafts during all installations. A distinction is made between the shaft friction acting on the friction element (local friction) and the friction acting on the part above this element. Also the mean shear stresses over the whole penetrated length are plotted. The figure gives only values for pile base depths of more than 100mm below surface level. When the pile base is at that level, the whole friction element is penetrated into the soil. From Figure 4.5 it is clear that more shaft resistance is mobilised during the ‘cyclic’ installations. This difference is especially visible for the friction acting on the upper section of the piles. Two way shearing provides more interaction with the surrounding soil. Higher rates during the cyclic installation might have a minor contribution to higher shaft friction (Al-Mhaeidib, 2006). The negative shear stresses for the upper section of pile 1 are difficult to interpret. It seems physically impossible. Although, perhaps it refers to soil movement along the pile shaft. The local shaft friction during installation seems to be mainly determined by the high soil stresses around the pile base. Lower values for the local shaft friction during the cyclic installation in the second series reinforce this presumption.
Figure 4.4: Head and base forces during installation of pile 2 in series 1 and 2.

Figure 4.5: Mean shear stresses during installation (increasing pile base level). A distinction is made between the shear stresses acting on the friction element (local friction) and the friction acting on the part above this element.
4.2 Pile tests

Figures 4.6 depicts the axial forces during the ‘load’ tests at several moments up to 1000 minutes after installation for both instrumented piles in both test series. Also the load-settlement behaviour after cyclic loading of the piles is plotted in these figures. Basically, no increases in overall pile capacity with time are observed. It should be borne in mind that each test will bring the pile deeper into the soil. Small increases are observed for the capacity of pile 2 after the installation of a neighbouring pile. In series 1 however, the capacity decreased initially due to the installation of the neighbouring piles.

It is thought that the applied pile unloading after a pile ‘load’ test, during the first test series, is not performed according its intention. If it is known that the own weight of the pile is almost 500N at 40g, the head load should not drop below 500N. The upward motion for the unloading appeared to be too large. This caused too much reduction of soil stresses around the pile base, or maybe a small cavity is formed below the tip, resulting in a decrease in base resistance and overall pile capacity. During the second test series, the force on the pile head is held greater than 500N. The pile capacity does not drop in this series. It has to be noted that the piles are not unloaded after installation in the first test series. During the second series was the final displacement of each ‘load’ test was twice as large compared with the first series (20%\(D\)).

Figure 4.6: Axial forces during all tests on pile 1 and 2 during both test runs.
4.2.1 Shaft resistance

Set-up is a phenomenon closely related to the pile shaft. Since it is the intention to investigate time dependent effects, the influence of increasing penetration depth with each test should be eliminated from the results. Figure 4.7 depicts corrected and normalized values of the mean shear stresses along the whole penetrated length during all pile tests. The shaft resistances are corrected for the surface area of the pile which is penetrated into the sand. The values are normalized for the mean effective vertical stress along the pile.

The lower shaft resistances measured for the jacked pile indicates more potential for pile set-up. Clear increases are observed for pile 1 between 100min and 1000min after installation and for pile 2 between 1min and 10min after installation, both in series 1. Just after installation of one or two neighbouring piles the shaft resistance is almost doubled! During following tests the initial increase of the shear stresses during loading reduces. However, still an increase remains relative to the measurements done before installation of the neighbouring piles. Soil densification and soil stress increase are obvious explanations for this increase in shaft friction. Other decreases are seen after cyclic loading for both piles. It is remarkable that the differences are smaller during series 2. The order of installation can have influenced the behaviour of pile 2 during the second series, but it is believed that the smaller upward ‘unloading’ movements play a major role. These movements can cause soil densification close to the shaft and breakdown of the presumably formed soil arch (creep). In order to clarify, the jacked pile underwent its first upward movement in series 1 after the first pile test and a clear increase in shaft resistance is observed during the following test.

Figure 4.7: Normalized and corrected shaft resistances during all pile load tests at different moments after installation.
4.2.2 Base resistance

To correct for the penetration depth, the base resistances can be normalized by:

$$q_{\text{base, norm}} = \frac{q_{\text{base}} - q_{\text{base, jacked}}}{q_{\text{base, jacked}}} + 1$$  \hspace{1cm} (4.1)

with:

$$q_{\text{base, jacked}} = 0.0582 + u + 1.2274$$  \hspace{1cm} (4.2)

based on the linear trend during the jacked installation of pile 2 in series 1, where \(u\) is the level of the pile base (model scale). These normalized base resistances are plotted in Figure 4.8. These values are independent of the penetration depth: they are a fraction of the (expected) value of \(q_{\text{base, jacked}}\) at the same depth.

Little or no increase in base resistance is expected. The results of series 2 meet this expectation. Clear decreases in base resistance can be seen after the installation of one or two neighbouring piles. It is thought that the soil beneath the base is pushed away from the pile or that soil relaxation occurs around the tip due to upward soil movement along the pile shaft during the installation of neighbouring piles (at 4\(D\)). It appears that the base capacity recovers after several tests, but considerable displacement is needed to regain the initial value. Other decreases are seen in series 1, especially between the first two tests on both piles. It turns out that the base resistance is susceptible to upward motions. (The pile was not unloaded before the first test in series 1.) In all cases an increase in base resistance is observed after the imposed cycles, what possibly refers to vibration-induced densification of the soil below the tip.

![Figure 4.8: Normalized base resistances of the instrumented piles during the load test in series 1 and 2.](image-url)
4.3 Other observations

4.3.1 Influences of cyclic loading
At the end of the test series small lateral pile movements are imposed. The piles were subjected to the cycles in an unloaded status in the first series, while there was no discharge of pile 1 before the cycles were applied in series 2. From Figure 4.9 it is clear that the vibration of the plungers is of the same order of magnitude as the imposed movements. Table 4.2 depicts the change between the measured resistances during the tests done before and after the cyclic loading. Small increases in the total capacity are observed after the imposed cyclic lateral movements. The increases are caused by higher base resistances, which possibly refer to vibration-induced soil densification below the tip. The shaft resistance is reduced after the applied cycles. In series 1, the ultimate shaft friction of pile 2 remains a factor of 1.11 larger than measured before the installation of the neighbouring piles. A factor of 1.14 is predicted.

Table 4.2: Ultimate total force, normalized base resistance and normalized shaft resistance during testing after cyclic loading, all as a fraction of the value measured during the previous test.

<table>
<thead>
<tr>
<th></th>
<th>( n_{F_{\text{head}}} )</th>
<th>( n_{F_{\text{base;norm}}} )</th>
<th>( n_{F_{\text{shaft;norm}}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pile 1, series 1</td>
<td>1.03</td>
<td>1.05</td>
<td>0.96</td>
</tr>
<tr>
<td>Pile 1, series 2</td>
<td>1.03</td>
<td>1.04</td>
<td>0.98</td>
</tr>
<tr>
<td>Pile 2, series 1</td>
<td>1.05</td>
<td>1.08</td>
<td>0.95</td>
</tr>
</tbody>
</table>

Figure 4.9: Measured base resistance, shaft friction and lateral pile displacement during the imposed cyclic pile loading.
4.3.2 Measurements during installation of neighbouring piles

During the installation of pile 3 and 4 in series 1 and the installation of pile 4 in series 2 the base and head loads on pile 2 can be observed. For series 2 also the pore pressure at the pile base is measured. See Figure 4.10. When the neighbouring pile is halfway of its final depth, increases in shaft friction and decreases in the base loading are observed. This indicates upward soil movement along the pile shaft: the pile itself cannot move upwards due to the boundary conditions of the test (displacement controlled). This upward soil movement can cause relaxation of the soil around the pile point. It is also thought that the soil beneath the base can be pushed away by the installation of a neighbouring pile. The measured pore pressures during series 2 indicate soil dilation/loosening under the pile toe.

![Figure 4.10: Measured axial pile forces and pore pressure at the base of pile 2 during installation of pile 4 in series 2.](image)

4.3.3 Surface roughness abrasion and sand crushing

At some locations of the model piles the initial roughness is abraded after the test series. This is only seen for the instrumented piles. The instrumented piles exist of several parts. These parts are roughened separately. The diameter of the parts may slightly differ and the pile is maybe not aligned fully concentrically. The abrasion is observed after the second test series, which does not imply that pile surface not abraded during the first series. Among other locations, it is observed at the friction element. The abrasion of the roughness can cause lower local shaft friction: there will be less interaction between the pile and the sand. See Figure 4.11. The smoothened parts had a mean average roughness, $R_a$ of 1.2µm and a mean maximum roughness of 5.6µm.
Figure 4.11: Example of abrasion (friction element of pile 2) and the cemented cone.

After the first test series a light spot is observed within the sand mass, indicating locally a finer material. This was found at about the final location of the pile tip. The piles are pulled out of the sand mass during spinning at the end of the first series. The created holes are filled with surrounding sand (the sand mass is fully saturated). Grain size distribution of the finer sand indicates the presence of relative more fine-fraction particles. After the second series the holes are filled with (colo)red sand (before pulling, water is extracted from the sand mass). Beneath a depth of approximately 12D below surface level finer material along the pile shafts is found for all four locations. Finer material can be made visible, because it attracts more moisture. Also cemented sand in the shape of a cone is found below one of the monotonically jacked piles, see Figure 4.11 (left). This all refers to soil crushing beneath the pile base and along the shaft.

4.3.4 Local densities

The soil surface settlement was approximately 1mm after both series. Samples were taken in areas in which less or no disturbance was expected. The determined local densities indicate a consistent package. However, different mean values are determined for both series: 71.8% for series 1 and 65.4% for series 2. An error of 8% is assumed. Despite these results, it is believed that the second preparation was slightly denser than the first one. (The samples are taken by different people and some areas were highly disturbed, especially after the second series.) Possible differences in (local) soil density between both sample preparations cannot be excluded.

Figure 4.12: Remoulded surface around pile 1 after test series 2.
4.4 Result overview and discussion

Figure 4.13 depicts the normalized measured shaft resistances at 1, 10, 100 and 1000 min after installation as a fraction of its reference value, measured during testing at 1 min after installation. The measured resistances on pile 2 after installation of the neighbouring piles are also plotted (on the right). Suffix 'a' indicates that pile 4 was not installed yet and suffix 'b' means that pile 4 was installed already. From this figure a clear distinction between a single pile and a group pile development can be made. For a single pile, the shaft resistance increases or remains approximately the same during the following tests. In the second series, the group pile shows also little increase before the third pile is installed. A decrease with time is shown after the installation of one or two neighbouring piles, although the final shaft resistance is larger than before installation of pile 3 and 4. Chow et al. (1998) also mentioned a case of a closely spaced pile group where the capacity of a pile decreased with time. Remarkable are the clear increases in series 1, observed after the first test for the jacked pile and after the third test for the cyclic installed pile. These increases correspond with A-values of 0.15-0.25. This may indicate that more disturbance was needed for the breakdown the soil arch created by the cyclic installation. The results of series 2 are less exiting, what is believed to be the result of ‘correct’ upward movements for the unloading of the piles. The reduction of the high shaft resistances, due to the installation of neighbouring piles, after 100 min in series 2 is of the same degree as after 10 min in series 1. This indicates that the main part of stress relaxation of the initial high stresses occurs within the first 10 minutes.

A distinction between shear stresses acting on the upper part of the pile and those acting on the friction element (local stresses) can be made. Figure 4.14 depicts those values, which are first corrected for the penetration depth, in the same way as Figure 4.13. Remarkable is the large increase during series 1 between the shear stresses acting on upper part of the shaft of pile 2 during loading at t=1min and during the next test at 10 min after installation. This observation indicates densification of the soil close to the shaft between the two tests. Strikingly is also the decrease of the shear stresses acting on the friction element of pile 1 during tests at t=100 min and t=1000 min in series 2. Surface roughness abrasion is a possible explanation. Unexpected are the increases in local shaft friction, while the base resistance decreases (series 1).

Figure 4.13: Normalized and corrected maximum shaft resistances during tests at several moments after installation, divided by its reference value, measured at t=1 min after installation. A distinction is made between a single pile (left) and a pile in a group (on the right).
The same plots are made for the normalized base resistances (figure 4.15). Clear decreases are observed in series 1 and are ascribed to the large upward ‘unloading’ movements. Almost no changes were observed in series 2, what corresponds with literature. It is stated that no or little set-up occurs at the toe. The development of the base resistance after the installation of adjacent piles indicates the recovery of the prior lost resistance.

Basically, no increases in the overall pile capacity with time are observed. See Figure 4.16. The plotted values are not corrected for the fact that each test will bring the pile deeper into the soil, what should provide greater resistance. Possible explanations are:

- A delay in the commencement of pile set-up, which is also observed in the field. Maybe, a runtime of 1000 minutes is not sufficient to simulate the phenomenon.
- A negative influence of development of the base resistance. Decreases are even seen in series 1. There seems to be a complex correlation between the base and shaft capacity: increased shaft resistances are negated by decreased base resistances (and vice versa).
- The repeated pile testing. All imposed pile movements causes disturbance. This is especially observed after the upward movements during the first test series.
- Some conditions in natural deposits are not replicated in small-scale laboratory testing. For instance, cone penetration tests in sands often show considerable variation, or heterogeneity, in penetration resistance with depth.
- Scale effects, abrasion of the surface roughness and grain crushing can influence also the behaviour of the soil around the pile shaft, influencing the ultimate shaft resistance.
- The boundary conditions associated with penetration testing in rigid wall cylinders may prevent detection of time-dependent increases in penetration resistance that are measured under the free-field conditions in the field (Soga, 2004).

Figure: 4.16: Maximum head forces during pile tests at several moments after pile installation, divided by its reference value, measured at $t=1\text{min}$ after installation.
5 Conclusions and recommendations

5.1 Conclusions

It cannot be concluded from the test results if it is possible to simulate pile set-up in sand, observed in the field, by means of centrifuge model testing. Pile set-up is a phenomenon related to the shaft resistance. Increases in shaft friction are observed between successive tests. However, it is uncertain if the increases are only due to the factor of time, it seems to be possible to simulate pile set-up by means of centrifuge model tests.

- Little or no overall pile capacity increases are found within the current investigation. In some cases there was even a negative relation with time. Anyway, the observed increases are much lower than the (long-term) set-up A-values known from literature.

- Clear differences are observed between the two installation methods. However, set-up can occur for a jacked pile, as well as a pseudo-driven pile. Grain crushing is found along all piles and contributes to contraction. The lower measured shaft resistance at the end of pile jacking provides a higher potential for pile set-up.

- The base capacity is sensitive for upward movements of the piles. A slightly larger displacement during an installation cycle in the second series caused a significant difference in base resistance. Larger upward movements in the first series, to unload the piles after testing, caused decreases in base resistance. From previous centrifuge model tests it is also known that a small upward movement for the unloading of the piles can cause soil densification near the pile (Dijkstra, 2009). This might be the explanation for the increase in shaft resistance between following tests. Vibrations of the plungers during spinning can cause also the rearrangement of sand particles.

- There is interdependency between the base and shaft capacity: increased shaft resistances were negated by decreased base resistances and vice versa.

- Clear influences from the installation of neighbouring piles are observed and the order of installation is of importance. When a neighbouring pile is installed later than the pile in question, an increase in shaft resistance and a decrease of the base capacity is found, which is also observed in the field (Chow et al, 1995). The predicted factor for the increase in shaft resistance due to installation of neighbouring piles, $f_n$, corresponds with the results. Although higher shaft resistances are found for a pile in a group, it will reduce with time when a neighbouring pile is installed following the regarded pile. The installation of an adjacent pile may cause a sudden breakdown of the soil arch. No decreases are found when a neighbouring pile is installed before the regarded pile.

- Under the current test conditions, the overall pile capacity and the base resistance increased after the imposed cyclic displacements. Although the shaft friction reduced due to the cyclic loading, it remains higher than the measured friction during the first test after installation.
5.2 Recommendations

For better understanding of the results of the current investigation and possible following centrifuge model tests:

- Better understanding of the interdependency between base and shaft resistance is needed. The degree of redistribution is possibly influenced by the relatively small $L/D$ ratio of the model piles.

- Better understanding of scale and size effects is needed:
  - The installation of model piles will shear the soil a smaller extent than larger diameter piles. Jardine et al. (2006) mention that the ability to sustain arching is likely to increase with the level of prestressing imposed by pile driving.
  - The size of shear band formation in the soil-structure interface will influence the lateral friction (Balachowski, 2006). In general, the width of the shear band is assumed to be a function of the grain size.

For the design of following centrifuge model tests, investigating pile set-up:

- It is needed to perform the whole test program force-controlled. Anyway, the unloading of the piles should not be performed by upward pulling of the pile. (Maybe it is even better to keep force on the pile head after installation and during waiting.)

- A pile should not be tested more than once. Maybe it is better to install several piles in the same sand preparation and test them at different points in time after installation.

- Perhaps testing should be performed lasting even longer than 1000 minutes.

- It should be borne in mind that some conditions in natural deposits are not replicated in small-scale laboratory tests. Model tests are done using clean and homogeneous sand, whereas, in the field there will be impurities and heterogeneity of void ratio. Creep can also be expected to become more marked in sediments having high initial void ratios. Perhaps, a more heterogeneous sand sample or less uniform sand is needed.

For application in practice, to incorporate pile set-up in the design method:

- The set-up factor $A$, according to Skov and Denver (1988) should be considered to be site specific. Future research should also focus on field studies. Full-scale pile testing is necessary if the set-up is to be accurately determined.

- For predicting the size of the set-up at a specific site and its behaviour in the long-term (which would enable maximum use of the phenomenon), a deeper understanding is needed of the mechanisms involved and their effects under different conditions. Set-up characteristics can be determined by means of laboratory testing, where specific conditions can be achieved. Soil density and uniformity, homogeneity of the preparation, type of sand, pile diameter and roughness, penetration depth and centre-to-centre distance between...
group piles (the level of disturbance that is necessary to break down the effect of set-up) all can be varied by means of centrifuge model tests.

- Probably it is worthwhile to take a good look at all case histories again with the gained knowledge in mind.

- The end of initial driving (tension) shaft capacity may provide a lower bound to the shaft capacity of piles subjected to multiple static or cyclic tests to failure (Jardine, 2006).
Bibliography


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Appendix A: Local normalized shaft frictions

\[ \tau_{f: \text{norm}} = \frac{\tau_f}{\sigma_{\tau_{\text{mean}}}} \]  \hspace{1cm} (A1)

\[ \tau_f = \frac{F_{\text{shaft friction}}}{\text{surface area}} \]  \hspace{1cm} (A2)

\[ F_{\text{shaft: total}} = F_{\text{head}} - F_{\text{base}} \]  \hspace{1cm} (A3)

\[ F_{\text{shaft: local}} = F_{\text{base + local friction}} - F_{\text{base}} \]  \hspace{1cm} (A4)

\[ F_{\text{shaft: upper section}} = F_{\text{head}} - F_{\text{base + local friction}} \]  \hspace{1cm} (A5)
Appendix B: Technical drawing of the instrumented model piles
Appendix C: Extra pictures

Figure C1: The excavation of the sand model after test series 2.

Figure C2: A small band of finer material is observed along the piles.