The effect of loading rate on pile bearing capacity of saturated sand

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The effect of loading rate on pile bearing capacity in saturated sand

Master of Science Thesis
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The findings, interpretations and conclusions expressed in this study do neither necessarily reflect the views of the UNESCO-IHE Institute for Water Education, nor of the individual members of the MSc committee, nor of their respective employers.
I would like to thank my mother and father, Jareerat and Udom, for their endless love to me.

This work is dedicated to them.
Abstract

Pile load tests are commonly used by engineers to determine its bearing capacity. At present, there are three methods of pile load tests: the static, the dynamic and the quasi-static test. The static pile load test is done by applying an axial load on the pile with a long duration. The dynamic and quasi-static tests are done with an impact load on pile head of very short duration. However, the required force pulse in the quasi-static test is longer than in the dynamic test. This research focuses on the comparison between quasi-static and static tests. An important aspect in order to verify the results of quasi-static application with respect to more widely used static loading.

The results of quasi-static tests have both static and dynamic components. Then, in order to convert the results of a quasi-static test to static pile bearing capacity, the dynamic component (inertial and damping effects) in the soil responses have to be understood. The effect of generates pore water pressure and its dissipation during pile penetration are unclear and can limit the interpretation of the results of a quasi-static test.

This research investigates the effect of loading rate on pile bearing capacity and on generation of excess pore water pressure. Scale tests have been done on a model pile in saturated sand. A steel bar with a piezometer cone at the toe is used as a model pile.

In order to study the loading rate effect, the tests are carried out with three different penetration speeds. In the CPT and the static loading test, the pile is pushed into sand by a hydraulic actuator with controlled speeds of 20 and 1 mm/s, respectively. In the quasi-static test, the model pile is driven into soil by a dropping mass. At the cone tip, the tip resistance, sleeve fiction, and pore water pressure are measured as a function of time. At the pile head, the force, displacement and acceleration are measured as a function of time.

The test results show that for the type of sand used in this study the pile resistance for different loading rates are similar. From the quasi-static tests, it is found that the inertia force is important in the dynamic resistance. It is about 40 % of the static resistance.

The tests also provide information about the generated excess pore water pressure caused by pile penetration. The magnitude of pore water pressure depends very much on the rate of pile penetration. The higher the speed, the higher the pore water pressure will be measured. However, the magnitude of the pore water pressure is relatively small in comparison to the magnitude of the cone resistance. Therefore, it does not significantly influence the bearing capacity of the pile.

Keywords: Quasi- static pile load test, model pile, pile load test, saturated sand, excess pore water pressure, axial load pile
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\( \sigma'_{h} \) horizontal effective stress (N/m\(^2\))

\( A \) the area of the cone rod (m\(^2\))

\( c \) the velocity of wave propagation (m/s)

\( s_p \) the permanent deformation of soil (m)

\( s \) the displacement of the toe of pile (m)

\( D_r \) relative density (-)

\( E \) the modulus of elasticity of material (N/m\(^2\))

\( e \) void ratio (-)

\( F_m \) force measured at the top of pile (N)

\( F_u \) ultimate force (N)

\( J \) the damping coefficient (Ns/m)

\( J_c \) The Case Damping constant

\( k_m \) stiffness of spring attached with the ram (N/m)

\( k_s \) stiffness of soil (N/m)

\( m \) mass (kg)

\( q_c \) cone tip resistance (N/m\(^2\))

\( q_f \) cone sleeve resistance (N/m\(^2\))

\( R_D \) a diameter ratio between calibration chamber and cone (-)

\( R_s \) static soil resistance (N)

\( R_T \) total static and dynamic soil resistance (N)

\( v \) the velocity of pile (m/s)

\( Z \) pile impedance (Ns/m)
1 Introduction

1.1 Background

Recent trend in foundation practice is to use deep foundations for the structures. The deep foundation is a solution for the engineer to bring down the load exerted by structure on the ground surface to the deeper soil layer when the upper soil layers are not sufficient to support the design load. Piles are mostly used to serve that purpose. Nowadays, the structures become heavier and higher, and piles have to be larger and have higher capacity.

To use piles as a foundation for the structures, engineers have to know the pile bearing capacity. A pile load test is normally executed in order to know that value. The pile test must be done to ensure the pile ultimate capacity. The test must ascertain the pile load deflection behaviour in order to avoid a differential settlement problem.

In last few decades the static pile load test is common used by practical engineers in order to attain the soil pile bearing capacity. In a test, a pile is loaded by an axial force on the top of it. Force will be increased step by step during the test and after each load increment the settlement of pile is measured. The relation between two variables is plotted in a graph to determine the interaction between pile and soil.

It is not an easy matter to perform the static test. Rather than balance a load directly on top of a pile – which may become unstable and dangerous – kentledge is placed over the pile, and the load is applied to it by jacking from the kentledge. During the test, which takes several days, the burden load will be increased until the ultimate strength of pile is reached. There are two methods to define the ultimate resistance. The first one is plotting the load-settlement curve, and another is plotting time versus displacement. The ultimate force is decided at the point where a large amount of settlement is occurring.

For the reason that the static method is a cumbersome procedure and time consuming, the dynamic method is introduced to be a solution. It can be a solution of the static load test by the way of lower cost and more mobility. However, it has some disadvantages compared to static load testing.

- The stress-wave, which occurs during the test can generate tension force and crack or break the pile.
- The pile can be damaged by the bending moment from applied load, which can be eccentric.
- The test results include the effect of stress waves; they need high experience engineers and computer programs to interpret.
Therefore, during the past few years new developments intends to overcome the disadvantages of dynamic load testing. The quasi-static load test was first introduced in 1988. It was proposed to be a new alternative, which combines the advantages of both conventional static and high strain dynamic load tests. In principle, the quasi-static test has been designed to have longer applied force duration than the dynamic test. This force can keep the pile under constant pressure and prevent tension stress occurring in the pile. In the quasi-static test, every part of the pile moves in the same direction and basically with the same velocity. The pile has more static behaviour and can be simplified as a rigid body. In the dynamic test, the pile has variations in velocities and displacements between different levels. This causes tension stress developing in the pile. Figure 1.2 shows the diagram of forces, displacements and velocities along the pile shaft. Figure 1.3 shows the comparisons of force and time relation in the three load tests.

Figure 1.2: Example of pile behaviour for different types of load tests
(Source: Middendorp, P., Bermingham, P., and Kupier, B.)
The quasi-static load tests are executed in two ways. The first method is called pseudostatic and another one is called statnamic. Figure 1.4 shows the concept of pseudostatic test. In this method a massive load is dropped on a pile. Several springs have been attached at the bottom of the dropping mass. The spring stiffness is used to lengthen the force-time duration of the impact load.

Figure 1.5 shows the principle of quasi-static load test. The load on the pile head is obtained by launching a reaction mass with an explosion of gas in a pressure chamber. The mass moves upward due to gas expansion, and the reaction pushes the pile into the soil. The duration and loading rate can be controlled by mass volume and chamber size. During these two tests the displacements are measured both in the pile itself and in the load on the pile head. Due to the data measured, the ultimate pile bearing capacity is obtained from the calculations.
The quasi-static load test has potential to determine static load behaviour without calibration with static load tests. The results of one quasi-static test can be compared to the behaviour of pile under a quickly performed static load test without cyclic loading (Horvath, B. 1990). The effects of cyclic loading can be taken into account by performing successive quasi-static load test on one pile.

1.2 Statement of the problem

Although the quasi-static load test can be considered more static than the dynamic load test, the quasi-static test still has of a dynamic component. Therefore, more research is needed to understand the interaction between pile and soil behaviour during pile loading. The method for analysing results also needs to further develop knowledge.

Fundamentally, the most important parameter that influences the pile bearing capacity between the static, quasi-static and dynamic testing methods is the loading rate. With the derived combination of other soil parameters, such as permeability, graded size distribution, density, void ratio, dilatancy parameters, and others, they can affect the test results. Some researches have been already done on the effect of pile loading rate on soil, as the study with soft soil by Brown (2002, 2004) and with unsaturated sand by Dijkstra (2004). Studies with saturated sand have still not been carried out. In the saturated sand, the rate of pile loading can have influence on pore water pressure in soil. So the question is “Does the effect of loading rate and induced excess pore water pressure have an effect on the measured pile resistance?”

1.3 Scope of work

This research has the purpose to study the effect of loading rate on pile bearing capacity of a pile in saturated sand. Series of tests will be done with a model pile in a calibration chamber. A piezometer cone, “the model pile”, is pushed into soil with different loading rates. The pile resistance parameters and pore water pressure from different tests are measured, and then examined to find their correlations with each other. To achieve the purpose of study, these following parameters are set for doing the tests.
1.3.1 Rate of cone penetration
The rate of penetration is one of the main parameters focus of this study. It is not clear that when the loading rate increases, whether the soil pile bearing changes or remains the same.

1.3.2 Pore water pressure
The excess pore pressure is one of the most interesting factors in this study. The main objective of the research is to determine how the excess pore pressure affects the pile bearing capacity of soil when loading rates acting on soil are different.

1.3.3 Soil in-situ density
The soil density is a control parameter to do the test. The interaction between soil, pore water pressure and pile penetration mostly depends on soil density and the corresponding void ratio.

The other soil parameters, for example, soil structures and grain size distribution along the penetration path, are not controlled and it is assumed that they are in the same condition throughout this research.

1.4 Objectives of the study
The main objective is: to find the effect of loading rate on excess pore water pressure and pile pile bearing capacity of saturated sand.

A secondary objective is: to find the effect of different soil densities and void ratio on excess pore water pressure and pile bearing capacity during pile penetration at different velocities.

1.5 Overview of the study
Chapter 2 presents a literature study which is concerned with previous work done on the effect of pile penetration rate on pile resistance in granular material, and induced pore water pressure caused by pile penetration. In this chapter, the general aspects about behaviour of saturated sand during pile loading and boundary effects in a calibration chamber are also included. Finally, the rheological model proposed by Smith to analyse the results from quasi-static load test is presented.

Chapter 3 presents the testing procedures and test equipment used in this study.

Chapter 4 shows the pictures of the equipment.

Chapter 5 presents the results from a calibration test, which is done with unsaturated sand in order to compare its results to the ones from a previous study done (Dijkstra, 2004). In this chapter, explanations about the test results of three different tests are also given.

Chapter 6 begins with the presentation of test results from CPT and the static load tests.
Then, an analysis of test results from CPT and the static test is presented. In this chapter, the effect of quasi-static test on soil is also examined by study of the differences in pile resistance obtained from static load tests, which are done before and after the quasi-static test. Finally, the test results from different vibration time samples are compared in order to see how different soil density values affect the pile bearing capacity.

Chapter 7 presents the result obtained from samples compacted with of quasi-static load tests. The main analysis is done in order to know how the penetration rates affect pile resistance by comparing the results of quasi-static to static load tests.

In Chapter 8, the influence of excess pore water pressure on pile resistance is investigated. The pile resistances from the static and quasi-static tests are compared. The influence of inertia component on pile resistance in the quasi-static test is studied by using the rheological Smith model.

Chapter 9 presents the test discussion and the limitations.

Finally, conclusions of the study are presented and recommendations are given in Chapter 10.
2 Literature study

In this chapter a summarise is given of the literatures from previous researches, related to behaviour of model pile testing in saturated sand with different loading rates. First, a general discussion about behaviour of saturated sands during pile penetration, Cone Penetration Testing (CPT) and a calibration chamber is presented. Second, the studies about the effect of pile penetration rate on pile bearing capacity and pore water pressure are given. Then, the concept of Smith model is described.

2.1 General discussion

2.1.1 Behaviour of saturated sand during pile loading

Generally, the behaviour of saturated sand during pile penetration can be divided into two types – a contractive and a dilative behaviour, as shown in Figure 2.1. Sand response in a contractive or dilative way depends on its density. If sand is loose, it will respond to undrained loading with a contractive behaviour. In the contractive behaviour, sand grains can be moved by applied force. The soil structure will be reformed and soil grains will move into void spaces between them. Due to the compression of the soil structure, soil becomes denser; pore water pressure increases in positive value and

reduces the soil effective stress. In dense soil, the response of a highly dense saturated soil in undrained condition shows a dilative behaviour. When the soil with a low void ratio is sheared, its grains cannot move into the void space between particles. They are forced to move up and over the adjacent soil grains. That change causes having more spaces among soil particles. As a consequence of that, pore water pressure in soil becomes smaller, and effective stress value becomes larger. Theoretically, dilatancy occurs around a cone tip where the penetration rate of cone is high and the permeability of soil is low.
2.1.2 Cone and Piezometer Penetration Testing (CPT and CUPT)

Cone penetration testing (CPT) has been widely used to evaluate soil properties. It gains more benefit in cost and method to evaluate soil properties than obtaining undisturbed sample for laboratory testing. Cones have many different standards such as in diameter or conical tip area and can have load cells or strain gauges at the tip and sleeve locations. In recent years there has been additional sensors such as pore pressure transducers, inclinometers and accelerometers installed in the cone in order to provide additional soil information.

![Figure 2.2: The Piezometer cone profile](image)

The CPT has been used to determine static loading capacity and settlement analysis. It allows engineer to determine allowable soil pile bearing capacity and to estimate soils density and friction angle. While the Piezometer Cone Testing (CUPT), Figure 2.2, has been introduced to measure excess pore pressures together with static cone penetration tests (Vlasblom, 1972) and later used to detect soil strata (Smits, 1982).

From the literature, Marsland and Quartermann (1982) studied factors affecting the measurements and interpretations of quasi-static penetration tests. They concluded that the cone resistance can be affected by many factors, for example, the cone shape, the stress- strain relationship and stress paths, the rate of penetration, the existing in-situ stresses and past stress history, pore water pressure induced by inserting the cone, the permeability of soil and soil fabric features.

Smits (1982) suggested that piezometer cones must have a rigid and fined graded filter for measuring dynamic pore pressure. A sintered stainless steel is selected to be a filter because it produces little noise when measuring pore pressure in dense sand.

2.1.3 Calibration Chamber

The cone and calibration chamber has been used in Soil Mechanics Engineering to evaluate a correlation between soil properties and the CPT testing parameters under controlled conditions. The accuracy in correlations founded from a testing in calibration chamber is widely accepted for estimating the parameters in the field. Moreover, it has to be kept in mind that to avoid the test results from the boundary effect, the calibration chamber has to be designed as large as practical. However, even the chambers are designed to the largest practical dimensions, they are still finite. That means the test results from a model pile still cannot be a real representative of those from in-situ tests, unless the cone reading are recorded with an accepted reasonably free boundary conditions. In general, the test chamber has a cylindrical shape and is made of steel.
Parkin and Lunne (1982) studied the boundary effect in the laboratory calibration chamber on a cone penetration test for sand. They used two sizes of the Fugro cone of diameter 25.2 and 37.7 mm. and 60° tip angle. The chambers used have two sizes, 0.762 m diameter and 0.941 m height, and 1.219 m. diameter and 1.50 m. height. The cones are pushed into sand sample by hydraulic actuator with velocity of 20 mm/sec. From the results shown in Figure 2.3, Parkin and Lunne concluded that for loose sand (Dr ≈ 15-30%) the cone resistance is independence from boundary effects, but for the dense sample (Dr ≈ 90%) the desirable diameter ratio, R_d, for normally and over consolidated sand is 50 and 100, respectively.

Houlsby and Hitchman (1988) performed a series of tests using a 36 mm diameter cone penetrated in different dry sand densities in a large calibration chamber. The cone is pushed by a hydraulic ram with velocity 20 mm/s to a depth of 0.80 m. The chamber is 0.9 m diameter (tank to cone diameter ratio of 40) and 1.0 m height. They concluded that the cone tip resistance only depends on horizontal effective stress, not on the vertical one. They also suggested that the ratio between cone resistance and horizontal effective stress depends on internal friction angle of soil.

### 2.2 The effect of pile penetration rates on soil pile bearing capacity

The soil pile bearing capacity in saturated sand condition depends on pore water pressure effect, while the excess pore water pressure depends on the rate of pile penetration. From that correlation, there are two ways to study the effect of pile penetration rate on the soil pile bearing capacity in saturated sand. One way to do that is to vary the rate of penetration and observe the measured point resistance values. This
kind of tests the pile driving velocities must be controlled to produce the drained and undrained conditions, which can produce positive or negative pore water pressure in soil. The other way is studying the behaviour of pore water pressure while pile penetrating into soil. The excess pore pressures are measured by using piezometer in the cone or water pressure transducers imbedded in the study area. Therefore, the study review about a pore water pressure development and its effect on pile resistance during pile penetrating are divided by the investigated method.

2.2.1 The tests done to measure pile resistance values by varied the rate of pile penetration

Kamp (1982) studied the influence of the rate of penetration on the cone resistance. He studied the tests done by Fugro cone. The tests are done at the seabed with the penetrations rates 2 and 20 mm/s. The results are shown in Figure 2.4. It can be seen that the cone resistance value measured from the lower rate of penetration is less than ones recorded from the higher penetration rate. This conclusion is confirmed by another test done in another location with the penetration rates of 0.033 and about 16 mm/s as shown in Figure 2.5.

Figure 2.4: Cone Penetration Test results on the North Sea

Figure 2.5: Cone Penetration Test results on the North Sea
Another series of tests is carried out in order to study the rate of penetration effect on the cone resistance. The 7.95 mm diameter cone is driven in a 0.25 m diameter “Rowe” consolidation cell, $R_D = 31$, with different rates 0.2, 2 and 20 mm/s. The investigations indicated that there are no significant different measured cone resistances from the different rates of penetration as shown in Figure 2.6.

Rahardjo, Brandon and Clough (1995) performed a series of tests to study the cone penetration resistance in silty sand in a calibration chamber. They used a standard cone, which has 1,000 and 15,000 mm$^2$ tip and cone friction sleeve area, and a minicone, which has 420 mm$^2$ and 6,300 mm$^2$ tip and cone friction sleeve area, as model piles. The model piles are pushed into silty sand in a 1.5 m diameter chamber by hydraulic piston. From test results, they concluded that the cone tip resistance values correlate to void ratio. When the void ratio is increasing but less than 0.46, the tip resistance is gently decreasing. However, the resistance can be decreasing significantly if the void ratio is greater than that limit as shown in Figure 2.7.
Lu and Impe (1996) studied model pile behaviour during driving in saturated and dry uniform Mol sand in a calibration chamber with diameter of 800 mm and height of 900 mm. The 36.4 mm diameter of piezocone is used as a model pile, and the pile tip is fixed at the depth 0.55 m below the soil surface. The dynamic test is done by using a drop hammer of 17.53 kg as a driving load and the using drop heights are various from 0.1 to 0.8 m. The static and cone penetration tests are carried out by driving the pile with hydraulic penetration equipment.

From their study, it can be seen as in Figure 2.8 that the test results from static and dynamic test have the same cone tip resistance value.

![Figure 2.8: Pile tip resistance and displacement](image)

They explained that for the dynamic test, the inertia forces in the soil is large and leads to stiffness interaction mode, dominating the process as long as the acceleration is high, the deformation being more like punching out a solid soil block beneath pile tip. As later on the pile movement slowing down, plastic failure zones develops and then same ultimate resistance as in static load test can be reached.

They also found that the pile tip resistance value is depending on the weight drop height as shown in Figure 2.9.

Eiksund and Nordal (1996) performed a model pile test to study the pile resistance and pore pressure response to dynamic load driving in a F-75 Ottawa sand and Lebanon silt. The 63.5 mm diameter model pile is driven into a pressure chamber by an actuator and drop hammer loading system. The penetration velocities in the test are ranged between 0.8 and 800 mm/s. The result showed that there is almost no effect from different penetration velocities on pile resistance in dense sand material as shown in Figure 2.10.
Figure 2.9: Dynamic pile tip resistance from various drop heights driving test

Figure 2.10: Pile resistance from different rates of penetration
2.2.2 The tests done to studied the excess pore water pressure during pile penetration

Since 1974, the effect of excess pore pressure on the soil pile bearing capacity in the Cone Penetration Testing (CPT) has been pointed out. Schmertmann (1974) noted that the negative or positive pore pressure could increase or decrease the pile resistance values. The study of penetration rate and the excess pore pressure in soil mass has been investigated since then. The wide range penetration velocities have been used in order to establish the drained and undrained conditions.

Caillemer (1975) studied the pore pressure distribution around the penetration cone in a loose and dense sand condition. Imbedded water pressure transducers and a 10-cm² Fugro cone with a transducer at its shoulder are used in the test. The result from his study revealed that in a loose sample value of water pressure increased very little. But in the dense sample the water pressure could be read negative when the cone is far from transducer but turned to positive value as the cone approached imbedded transducer.

Lheur (1976) performed a cone penetration test in dry and saturated fine sand to evaluate pore pressure development around a cone. The pore pressures are recorded from several imbedded transducers in different levels and at cone shoulder. He concluded that the saturation of sand has no effect on penetration resistance value. The test result also revealed a small positive excess pore pressure at the cone and in the soil mass during penetration in loose sands, but negative values could be recorded from denser samples.

Bunnell (1978) performed a cone penetration test with a Wissa cone. The pore pressure transducers are imbedded in different level and in different laterally distances from a cone. The results showed the positive pore pressure both at the cone tip and imbedded lateral transducers in loose sands. Whereas in dense sand the positive pore pressures are only recorded at the penetration tip and the negative values are observed away from the cone.

Schmertmann (1978) performed a test with the Wissa-Probe into easily liquefiable mine tailing sand. The test is carried out at a site in Florida. The probe is penetrating to a depth about 4.5 m in saturated fine sand. He concluded that the magnitude of pore pressure recorded from the tip of cone depends on the effective stress, dilatancy behaviour, permeability and the rate of cone penetration.

Tumay et al (1981) performed a series of field test in fine-grained sandy soil with different cone shapes and sizes to evaluate the effect of them on recorded pore pressure. They concluded that the magnitude of pore pressure during pile penetration is depended on the shape of cone; the lowest angled tip cone generated highest penetration resistance but lowest induced pore pressure.

Bruzzi and Battaglio (1987) performed field penetration test in different soils with different cone penetrometers to evaluate the effect of cone model, transducers location, and filter material. They noted that the degree of saturation in filter both in the penetration cone or imbedded transducer has an influence on the magnitude and response time of recorded pore pressure. They also suggested that the values read from unsaturated cone are not accurate. They further identified that a worn and clogged filter may result in a reduction of the filter permeability, which can affect the recorded data.
The test results also revealed that the magnitude of the pore pressure measurement is greatest at the cone tip and reduced as the measurement location moves toward the friction sleeve.

Chandra and Hossain (1993) studied the pore pressure response to pile driving in the field. The test is carried on pile driven in clay at the depth to 15 m. The pile is a 0.6 x 0.26 x 21.00 I-section pre-stressed concrete. The excess pore pressures are recorded from 18 piezometers surrounded the pile. In addition, 6 dutch cones and 6 vane shear test and 2 boreholes are done at the place. They concluded that the influence of excess pore pressure due to pile driving is very small after 1 m and can be negligible after 2 m from pile axis. They also concluded that shear modulus has many influences on the pore pressure, and soil permeability can affect the rate of dissipation of excess pore pressure as well as the starting time of dissipation.

Robertson, Woeller and Gillespie (1989) evaluated the excess pore pressure and drainage conditions around driven piles using the cone penetration with pore pressure measurements. They studied large diameter steel piles driven in a field near Vancouver. The piles penetrated through marine clayey silt. A cone penetration test with pore pressure measurement (CPTU) is performed shortly after pile is driven. The results from CPTU are used to compare with those recorded from multipoint piezometer installed near the pile group. Figure 2.11 shows a location of CPTU and multipoint piezometer. Figure 2.12 shows that pore water pressure values recorded from CPTU and multipoint piezometer are decreasing with a radial distance from pile.

![Figure 2.11: Location of CPTU and piezometer relative to pile](image)
Hoelscher and Barends (1996) studied the soil motion near the toe of dynamically loaded pile in-situ. The studied pile is driven to the sand layer at 15 m, and the final depth is 18.2 m from the surface. The set of four water pressure transducers is imbedded around the final pile level with others kind of transducer. The results showed that while the pile is driving, the water pressure at the transducer far away the pile toe first compression occurred and followed by decompression, but the recorded water pressure at the transducer near by is opposite. The study also compared the water pressure occurred between the quasi-static test and the last blow of pile driving. The result are shown in Figure 2.13, it shows that the water pressure occurred during the quasi-static test is larger than one from the pile driving.
Lu and Impe (1996) apart from studied the pile resistance during pile driving in saturated uniform Mol sand; they also studied the behaviour of excess pore water pressure during pile driving. The results are shown in Figure 2.14. From the figure, the excess pore pressure in the beginning goes slightly to the negative then suddenly increases to a large positive value. They explained that due to the dilatancy of sand the excess pore pressure around the pile tip decreases, and even reaches to the negative value. But after the dilatancy, the pore water flows into a shear zone and then the pore pressure value is rising. Furthermore, the maximum positive value is related to the peak pile acceleration of pile driving as it shown in Figure 2.15. The ratio of peak excess pore pressure behind the pile tip to the pore pressure at rest improves sharply when the pile acceleration crosses a threshold.

![Image](image1.png)

Figure 2.14: Excess pore water pressure behind pile tip

![Image](image2.png)

Figure 2.15: Maximum excess pore water pressure behind pile tip against pile peak acceleration and hammer drop height
Eiksund and Nordal (1996) performed a series of tests to study the pore pressure response due to different pile loading rates. They studied with saturated sand and Lebanon silt in a calibration chamber as already mentioned in 2.3.1. The penetration velocities in the test are ranged between 0.8 and 800 mm/s. The results showed that initially the pore water pressure is positive but then followed by a large negative value due to dilatancy near the pile toe. The positive pore pressure response is highly dependent of penetration velocity. The same trend can be seen in the magnitude of the negative pore pressure as it shown in Figure 2.17. The negative pore water pressure is increasing from 1 kPa at 0.8 mm/s to 33 kPa at 800 mm/s. From the figure, it can be seen also that when the penetration stops, the pile rebounds and some of the soil dilation is reversed and consequently an excess amount of pore water causing the positive pore pressure response. Due to drainage time for the excess pore pressure this effect is visible in the tests at 8 mm/s to 80 mm/s.

2.3 A one-dimensional soil model for analysing axial pile response under dynamic and static loading

Analysis of the axial force response of pile needs an appropriate soil model to simulate the transfer of force on pile shaft and the force at base of pile. The one-dimensional soil model has been widely accepted to analyse the axial response of pile for static and dynamic loading conditions. It must be recognised that for a one-dimensional soil model, the soil surrounding a pile is a continuum. Along the pile shaft, it is assumed that the interactions between pile and each layer of soil are independently from each other, no interaction between neighbouring layers. The assumption is also used for the interaction at the pile base; the interaction at the far away soil element can be neglected.

2.3.1 The Smith model

Smith (1960) proposed the traditional soil model for pile driving analysis. He represented the forces exerted in the pile-soil interface by an elastro-plastic spring, a combination of spring and plastic slider, and a dashpot. The elastro-plastic spring is used to model a static response, while a dashpot is used to represent viscous and inertia effects. Smith also assumed that the soil mass beyond the slip layer is infinitely rigid. Thus, energy transmitted to the deforming and moving soil is included in the losses represented by spring and dashpot. The Smith model is shown in Figure 2.16.
Figure 2.17: Pore pressure response in saturated sand from different penetration velocities
In Smith numerical algorithm, the static pile bearing capacity of a pile can be related to its dynamic behaviour as the following equations:

\[ R_T = (s_p - s) k_s (1+Jv) \]

or

\[ R_T = (R_s (1+Jv) \]

- \( R_T \) total static and dynamic pile resistance
- \( R_s \) static pile resistance
- \( s \) the displacement of the toe of pile (m)
- \( s_p \) the permanent deformation of soil (m)
- \( k_s \) the spring coefficient of soil (kN/m)
- \( v \) the velocity of the toe of pile (m/s)
- \( J \) the damping coefficient (taken in the range 0.05-0.5 s/m)

It can be understood from the algorithm that the static soil resistance, \( R_s \), is a function of relative displacement of the pile to the soil. It is assumed to be present both during static and dynamic tests. The damping resistance is not present under static loading. Its value is proportional to a pile velocity. The total resistance, \( R_T \), is the sum of static and damping resistance. That means the \( R_T \) and \( R_s \) are equal to each other when the damping resistance is zero.

The expression of the limiting value of the maximum elastic deformation of soil, \( s_p - s \), is defined by Smith as a term of a quake, \( q \). The quake is a functional of pile diameter.

Gibson and Coyle (1968) published results of triaxial tests at the Texas A&M University, which compared the total dynamic resistance to the static values at various velocities. They concluded that

\[ R_T = R_s + R_s J_c v^N \]

The experiments indicated exponents of \( N = 0.18 \) for clay and 0.20 for sand

Goble and Rausche (1976) included the non-dimensional Case damping approach in the WEAP program. This approach has earlier been used for Case Method and CAPWAP capacity calculations (Rausche, Moses and Goble 1972). The soil resistance calculation is simplified to

\[ R_T = R_s + J_c Z v \]

- \( Z \) pile impedance (s/m)
- \( J_c \) The Case Damping constant

Therefore, the damping coefficient can be derived as

\[ J_c = J_c \frac{EA}{c} \]
2.4 Summary

The study is focused on the pile resistance values due to the different rates of pile penetration in saturated cohesionless soil. Some conclusions are summarised in the as following:

- The pile bearing capacities from the static and dynamic penetration velocity ranges are not significant different.

- For the dynamic pile load test, the excess pore water pressure in saturated dense sand caused by pile penetration at the pile tip can be negative due to the dilatant behaviour of sand.

- The induced excess pore water pressure depends on rate of pile penetration.

- The excess pore water pressure is a dependency of compression and dilatancy behaviour in soil.
3 Test set-up

To understand the quasi-static pile load test, the best way is doing a field test study. However, to study pile and soil behaviour in a field is not as easy as to do in a laboratory as a scale model. In the field, the test set-up is complicated and the instrumentation is hard and expensive to do. In order to achieve the highest quality of data, the field study must be very expensive to be carried out. Moreover, the results from field study must be influenced by soil variation inevitably, which are not an idea to be used for rigorous research purposes.

Those restrictions can be overcome by carrying out the tests on an instrumented model pile. The pile is embedded in sand in a calibration chamber. In this case sand properties are known and can be considered as a homogeneous sample. The instrumental set up can be considering cheap and a series of tests can be done regardless about cost. That means the model test allows the examination be repeated done than in a field.

This chapter presents the sand, the equipment used in the study, for example, the calibration chamber, loading system and measuring tools.

3.1 The calibration chamber

In this study, the testing chamber consists of rigid thick steel wall, of 1900 mm diameter and 3200 mm height, with a taper shape bottom. On the top of the chamber there are two steel beams to be a support frame for the pile driving system. The diameter ratio, $R_D$, is 52, which obtained from the chamber diameter of 1,900 mm and the piezometer cone diameter of 36 mm. This $R_D$ value is greater than 50, which is recommended by Parkin and Lunne (1982) and it is judged as a satisfactory.

![Diagram](image)

**Figure 3.1: The test locations in the calibration chamber**

The series of tests have been done only at the centre point of a chamber (location ii, Figure 3.1) in each soil sample preparation. It is clear that if the test is done in other locations, the $R_D$ value cannot be equal as 52 in all directions and the measured cone resistance values must be certainly influenced from the lateral boundary effects. Moreover, from the previous study done by Dijkstra (2004) the tests are done in three
different locations (Figure 3.1). The results shows that the data measured at the off-centre positions have a tendency being scatter over time, while the results measured at the centre position are more reliable than ones at the other locations.

3.2 The sand

The sand is already existed and available from other previous studies. The sand used in this study is moderately coarse sand. It has a median grain size, $D_{50}$, of 0.27 mm and the permeability varied between $4 \times 10^{-4}$ – $9 \times 10^{-4}$ m/s, the relative density of 60%. The internal friction is 25 degree. Figure 3.2 shows the grain size distribution of sand used in this study.

![Grain size distribution curve](image)

Figure 3.2: Grain size distribution of sand in the calibration chamber

3.3 The fluidization and vibration system

The equipment, which is used in order to set up soil sample in the previous works, is still used in this study. The equipment is consisted of a fluidization system and a vibration system, as shown in Figure 3.3. Apart from that, there are two water storage tanks and a water pump. The two storage tanks and a pump are designed to provide enough water to fill in the system about 1.50 hours. With that amount of time, the fluidization could make the soil sample become homogenise sufficiently. Therefore, in this study the 1.50 hours is chosen to be duration to fluidize the sample and it is done after each series of tests have been finished.

To do a series of tests, the soil density is assigned to be a criterion. Therefore, in order to achieve different levels of soil density the vibration system is used for that purpose. After the soil has been fluidized for 1.50 hours, it is vibrated by two vibration machines, which are attached at the side of the calibration chamber. The time for vibrating the sample is set as it as in the previous study - 5, 10 and 15 minutes. With the same vibration time, the same soil densities should be attained, and then the test results between two tests can be compared. However, the vibration duration is changed later to be 30 minutes in order to make soil has higher density. Generally, when the last test is
done in each day, the soil is vibrated, and then left at least 18 hours before the next series of test would begin in the following day.

Figure 3.3: The calibration chamber, the fluidisation and vibration system  
(Broere, W., “Tunnel Face Stability and New CPT Application”, 2001)

### 3.4 Loading mechanism

The thesis has an aim to study how saturated sand response to different pile penetration velocities. Three different speeds of pile loading, 2, 20 mm/s and speed in the quasi-static loading range, are set to test that query. The driving tool for the CPT and the static load test is different from one used for the quasi-static test. The hydraulic actuator is used to push the model pile into sand in the first two kinds of test, while drop weight is used to hit the pile and drive it into soil for the latter case.

One series of test is consisted of one CPT, one quasi-static test and two static tests. The series of test began with the CPT. After the CPT have been done, the static load test I is carried out, and then followed with the quasi-static test. The static test II is the last test
done in each of test series.

The type of static load test used in this study is a constant rate of penetration test. Generally, the penetration rate should be between 0.5 and 2.0 mm/min. But that speed is too low to do with the available hydraulic actuator in TU laboratory. The slowest penetration rate, which can be done, is about 1 mm/s and it is used in this study. To do the static test, the model pile is pushed by the actuator with the velocity in the range between 1-2 mm/s. With that rate of penetration, it is determined that the test results could be taken into account as representatives of results of the real static tests. The hydraulic actuator is controlled manually, and this resulted in some minor variation in the rate of penetration during the tests. However this variation is small, and the rate of penetration could be considered as constant for the all the static tests.

As same as done in the static test, in the CPT the pile is pushed into soil by hydraulics, but with faster velocity. The velocity used for this case is the standard velocity designed for the CPT, about 20 mm/s. The speed could be considering equal and constant in every CPT, because there is a special level arm designed for pushing the pile at that speed.

The hydraulic actuator and its installation are shown in Figure 4.3. It compriss a loading frame, hydraulics and its speed control. The loading frame is fixed with two steel beams. The beams are designed to provide the reaction force when the actuator pushed the model pile into sand. To control the pile driven speed, there are two level arms; one for the standard velocity used for the CPT and another one for other slower speed.

In the case of the quasi-static load test, the loading mechanism is more dynamic than the others. It is composed of a drop mass, an aluminium guiding tube, a set of springs and a trigger bar, Figure 4.4. The drop mass have a weight of 63.9 kg. It is used as a ram to hit a pile head and push a model pile into soil. The aluminium tube is used to guide the ram move inside it and hit a pile head. The trigger mechanism, Figure 4.5, consists of an aluminium bar and an aluminium rod. The rod is used as a trigger to hold the ram and to launch it to hit a pile, while the bar is used as an arm for the trigger system.

3.5 Measuring tools

In this study, there is a set of devices to measure the interesting parameters. The piezometer cone is used to do the test. It can measure the cone tip resistance, the sleeve friction and the induced excess pore water pressure caused by cone penetration.

The equipment used in CPT is only the piezometer cone, a personal computer, software and a set of amplifiers. However, in the static and the quasi-static load test there is other electronic equipment uses to measure others parameters; namely, force at pile head, acceleration and displacement value. The equipment used to measure those parameters is a strain gauge (for measuring force at pile head), an acceleration transducer and a displacement gauge. The installation of the tools is shown in Figure 3.4.

3.5.1 The piezometer

In this study, the piezometer cone is used to measure the induced excess pore water pressure during cone penetration. In general, there are two alternatives to measure the pore water pressure in soil.
First alternative is using an embedded transducer to measure the pore water pressure. This method is normally used when pore water pressure and its changes in space and time are wanted to study. The benefit of using embedded transducers is that the transducers can be located separately at any locations where the information is wanted to measure. The value of excess pore water pressure can be recorded either in vertical or horizontal direction away from the cone. Second alternative is using the piezometer cone measure the pore water pressure value. The piezometer has a pore pressure sensor installed inside; therefore, it can measure the pore pressure directly. However, the pore water pressure, which is measured, is only the information near the cone tip. The changes of pore water pressure in vertical direction and only along penetration path of the cone can be examined.

The piezometer is selected to use in this study, because only the induced excess pore pressure near the cone tip is interested to investigate whether how it can influence on pile resistance. A standard type of piezometer has a pore pressure sensor located between the tip and friction sleeve.

3.5.2 The strain gauge

The strain gauge is used to measure the force at pile head value in the static and the quasi-static load test. Its function is measuring the strain changed in the object, in this case pile rod, and then transferring that measured data into force signal. The below equation can show well how the function is. The gauge have a bandwidth of 20 kHz in order to measure any value in a very small time step; in this case the time step is 0.05 ms. Therefore, the maximum force, which can be record from the setting tool is:

\[
F_{\text{max}} = \xi E A_p = 100 \times 210 \text{ Gpa} \times 7.63 \times 10^{-4} \text{ m}^2
\]

while \( \xi = 100 \times 10^{-6} \)

then \( F = 16 \text{ kN} \) (for one strain gauge)

3.5.3 The acceleration transducer

The acceleration transducer is used only in the quasi-static load test. It is needed, because it would provide the measured acceleration value during cone moving in soil. Moreover, by integrating the measured acceleration the velocity could be obtained. Both values are used to analyse the effect of dynamic resistance in soil, which will be mentioned in the next chapters. Due to working with a small time step in the quasi-static range, 0.05 ms, that means the working frequency of acceleration transducer is 20 kHz, and then the accompanied adapter must have this range of frequency, too.

The acceleration transducer has to be installed outside the cone rod on a mounting steel plate. The transducer could not be installed inside the rod, because the rod is used to be a passageway for a cable, which connect the cone and the amplifier.

3.5.4 The displacement gauge

The displacement gauge is a linear stroke potentiometer. It is used to measure the movement of pile rod during pile penetration both in the static and the quasi-static load test. Its stand is mounted on the steel beam, which can be counted as a fix boundary. The measuring pinpoint of gauge is placed on the mounting steel plate, which is used as
a base of the acceleration transducer. When the steel plate is moving downward, the measuring pinpoint is moving in the same direction. With this mechanism, the displacement could be measured. Moreover, it can be said that the measured displacement and velocity data did not have time differences, because the measuring point of them share the same support.

![Figure 3.4: The calibration chamber and measuring tool](image-url)

The displacement gauge

The acceleration transducer

The strain gauge

Steel rod

The piezometer cone
4 Pictures

Figure 4.1: The calibration chamber and water storage tanks

Figure 4.2: The fluidization and vibration process

a) First water at the top of the tank
b) Some moments later
c) Vibration (interference in water surface) 
d) Drained situation
Figure 4.3: The hydraulic actuator

Figure 4.4: The drop mass, springs and aluminium guiding tube
Figure 4.5: The trigger bar for launching the drop mass in the quasi-static test

Figure 4.6: The displacement gauge and the amplifier
Figure 4.7: The acceleration transducer
5 Verification of testing procedures and presentation of test results

This chapter begins with the results from a verification test. Then, the definition of the ultimate resistance, which is used in this study, is discussed. Finally, the explanations of pile soil behaviour during pile penetration in CPT, the static and quasi-static test are presented.

5.1 Verification of testing procedures

Before the study of pile load tests with saturated sand starts, there has been one set of tests done with unsaturated sand. The test is done in order to calibrate the test procedures with the previous study carried out by Dijkstra (2004). The results from the calibration test shows a good agreement with the results from the previous research.

In the verification test, the CPT cone is used to do the test. The test is done with unsaturated sand. The sand have been fluidized for 1.50 hours, and then vibrats for 10 minutes. The pile penetration tests are done only at the centre of the calibration chamber due to avoiding boundary effects, as already mentioned in 3.1. The results from the verification test and from the previous study are shown in A 1. The soil in these tests have been fluidized for 1.50 hours and vibrated for 10 minutes.

From A 1, it is clear that the test results from the previous research are similar as ones from the verification test. Most of all measured values from the calibration test are in the range of test results done in the previous study. Although the sleeve friction in the static load test I is failed to measure, it can be seen from the other tests that measured sleeve friction have only insignificant differences. Therefore, the results did prove the testing procedures.

A 1: The results from the previous study and the calibration test with unsaturated sand

<table>
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<tr>
<th>date</th>
<th>CPT</th>
<th>static load test I</th>
<th>quasi-static load test</th>
<th>static load test II</th>
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<td>sleeve MPa</td>
<td>force kN</td>
<td>tip MPa</td>
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<td>verification</td>
<td>17.5</td>
<td>0.10</td>
<td>22.3</td>
<td>15.6</td>
</tr>
</tbody>
</table>

5.2 The ultimate resistance in the static load tests

Normally, in the in-situ static pile load test the pile can be determined fail and the ultimate pile capacity is reached when pile displacement is more than 10 % of pile diameter. But in this study the ultimate cone resistance in the static load test is defined
at the point when a model pile is moved about 20 mm or 50 % of pile diameter.

From the force and displacement curve shown in Figure 5.1, if the 10 % of pile diameter displacement is set to be a criterion for an ultimate force (Fu) value, it can be seen that Fu is very difficult to identify. The Fu value read from the graph can be fall in a wide range between 10-12 kN. Therefore, the 10 % of pile diameter displacement condition is not favourable. To get rid of that uncertainty in reading graph, the new criteria is set to define the ultimate value point. From the same figure, if the Fu is read at the point B, its value can be easier to obtain from the graph reading. The Fu is falling in a small range, and therefore more exact value can be attained. However, the Fu values from both definitions are compared and presented in the Appendix 1. They are only 12 % different in their values.

5.3 Presentation of test results from the CPT

In the CPT, the piezometer is pushed into soil to measure 3 different parameters – tip resistance, sleeve friction and pore water pressure. The piezometer is pushed at the velocity of 20 mm/s from the depth about 0.60 m to the depth at 1.00 m below soil surface. This test procedure took time about 20 seconds in each CPT.

Figure 5.2 to Figure 5.4 show the measured parameters during pile penetration in the CPT. The pile tip resistance and skin friction values are increasing according to penetration distance until the pile is stopped. After the pile is stopped, tip resistance decreases, while sleeve resistance increases. That changes happen only in a second in a relaxation period. After the relaxation period, both of the resistance values are decreased and become zero in the end. The relaxation behaviour cannot be seen in excess pore water pressure. The pore water pressure is increasing due to penetration of the pile. The induced pore water pressure can be generated up to 8 kPa before it decreases after the pile is stopped moving.
Figure 5.2: A graph plotted between pile tip resistance and time

Figure 5.3: A graph plotted between sleeve resistance and time

Figure 5.4: A graph plotted between pore water pressure and time
5.4 Presentation of test results from the static load tests

In the static load tests, force at pile head and displacement values are other two data, which are measured apart from tip resistance, sleeve friction and pore water pressure. The cone is pushed with very slow velocity at 1 mm/s for the distance of 20 mm. The test took time about 20 seconds in each test. The force at pile head is measured by the strain gauge and the displacement is measured by the potentiometer.

The results of static pile load test can be seen from Figure 5.5 to Figure 5.8. The measured force at pile head and the pile tip resistance have the similar development. The values are increasing fast at the beginning in the first ten seconds. After that the increase rate is slow down. They reach their maximum point at the time when the pile is stopped and begin decreasing afterward.

Figure 5.5: A graph plotted between force at pile head and time

Figure 5.6: A graph plotted between pile tip resistance and time
Figure 5.7: A graph plotted between sleeve resistance and time

Figure 5.8: A graph plotted between pore water pressure and time

Figure 5.9: A graph plotted between force at pile head and displacement
The behaviour of sleeve friction and excess pore water pressure are different from the behaviour of force at pile head and pile tip resistance. The skin friction and pore water pressure values start increasing and reach their maximum point rapidly only in a second. Then, the values fall down a little before keep constant. After pile is stopped, the skin friction and pore water pressure start decreasing.

From Figure 5.9, the force at pile head increases slowly in the first 2 mm of pile displacement. After the displacement is more than 2 mm, the rate of change between force and displacement becomes higher and higher until the pile displacement is about 10 mm. Then, the rate of change is more constant.

### 5.5 Presentation of test results from the quasi-static pile tests

In the quasi-static load test, the pile is hit by a dropping mass. The mass is drop from the certain of height, 0.15 m, in an aluminium guiding tube. The potential energy from the height of 0.15 m converts to the kinetic energy before the ram hit a head of the pile. The approaching velocity is known by calculation and it is about 1.7 m/s. After the pile have been hit, all parameters are measured. In the quasi-static test, the acceleration is one parameter have to be measured. It is used directly as raw data to analyse the pile resistance, and also indirectly to obtain the velocity values by integration. Both of them are used for calculation the dynamic pile resistance, the acceleration is used to calculate inertia resistance, while the integrated velocity is used to calculate the damping force.

The measured parameters from the quasi-static test are shown in the Figure 5.10 to Figure 5.15.

![Figure 5.10: A graph plotted between force at pile head and time](image)
Figure 5.11: A graph plotted between pile tip resistance and time

Figure 5.12: A graph plotted between pile sleeve resistance and time

Figure 5.13: A graph plotted between total pile resistance and time
From Figure 5.10, the force at pile head and displacement are plotted against time. It can be seen that the force value increases when the pile displacement is not more than 2 mm. When the displacement is over than 2 mm the force starts falling down. However, the value increases again when the pile is hit after the rebound.

Figure 5.11 shows the tip resistance value, it increases immediately after the pile displacement occurs and keeps increasing until the displacement is 10 mm. When pile rebounds and moves upward, the tip resistance decreases quickly. Then it goes up again when the pile is pushed back into soil.

Figure 5.12, the sleeve resistance reaches its maximum point when the displacement is about 2 mm. Then it slowly goes down. The sleeve resistance becomes zero during the rebound. There is no sleeve friction measured in the tension behaviour. Then, the value increases again due to a small distance of pile penetration.

Figure 5.13 and Figure 5.14 show pile resistance increases only in the first 2 mm pile penetration.
displacement. After then, the value is constant during the pile penetration into sand. And when the rebound happens, the value falls. It can be seen from Figure 5.14 that the total resistance is constant during pile penetration while many of changes are seen in the force at pile head values.

Figure 5.15 shows that pore water pressures is initially positive during the pile penetration. Then, the pore water pressure becomes a negative value caused by the dilatant behaviour of the sand near the pile tip. The pore water pressure becomes positive again when the pile rebounds, and then follows with negative caused by a next penetration. This phenomenon continues until pile stops moving.

Figure 5.16 can describe the behaviour of soil and its behaviour well. As the pile is pushed into sand, the sand begins to contract, resulting in an increase pore water pressure. The increase pore water pressure still continues according to the pile penetration until sand cannot contract any longer. Then, the dilatant behaviour begins. The soil granular slides and moves over the others. Consequence, there is more space among sand particles, the pore water pressure decreases and becomes negative value. The dilantant phase stops when the pile rebounds. The sand grains move back to the stable condition. Therefore, the pore water pressure increases again and becomes positive.

<table>
<thead>
<tr>
<th></th>
<th>Δ Volume</th>
<th>ΔU</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td>0</td>
<td>Initial Condition</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Zero Load - Zero Displacement</td>
</tr>
<tr>
<td>2</td>
<td>▼</td>
<td>▲</td>
<td>Pile pushed into soil causing loose soil to densify</td>
</tr>
<tr>
<td>3</td>
<td>▲</td>
<td>▼</td>
<td>Continued displacement of pile into soil - Soil changes from contractive to dilative in behavior</td>
</tr>
<tr>
<td>4</td>
<td>▼</td>
<td>▲</td>
<td>Unloading of pile - Soil particles settle and move into more stable configuration</td>
</tr>
<tr>
<td>5</td>
<td>▼</td>
<td>▼</td>
<td>Loading of pile stops - Soil continues to settle as pore pressures dissipate at rate dependent on hydraulic conductivity of soil until steady state reached or until pile loaded and cycle repeats</td>
</tr>
</tbody>
</table>

Figure 5.16: Excess pore water pressure during one load cycle and explanation of soil behaviour.

(Δ Volume = change in volume, ΔU = change in pore water pressure)
(Source: Ashford, S. A., Weaver, T. J. and Rollins, K.M. “Pore Pressure Response of Liquefied Sand in Full-Scale Pile Load Tests)
However, to explain the pile-soil interaction behaviour in the quasi-static load test, the simplification model is drawn to illustrate the mechanism as shown in Figure 5.17. This simplification model is composed of two masses and two springs. The first set of mass and spring is used to be the representative of the ram and the set of springs attach to it. Another mass is a mass of ram, while another spring is a representative of soil stiffness. In this simplification model, the effect of dynamics resistance is not included in it. The dynamic resistance, which is consisted of an inertia resistance and a damping force, will be discussed later in Chapter 8.

![Figure 5.17: The simplification model for pile-soil resistance](image)

\[
\begin{align*}
\text{m}_{1} & = \text{ram mass} \\
\text{m}_{2} & = \text{pile mass} \\
\text{k}_{m} & = \text{stiffness of spring attached with the ram} \\
\text{k}_{s} & = \text{stiffness of soil}
\end{align*}
\]

The mechanism starts from:

a. The ram (and springs) is released from a set height in a guide tube over the pile head. It accelerates due to gravity until it strikes the head of the pile.

The impacted force shows a spike in all measured values.

b. The ram hits the head of the pile. It generates a force at the pile head as it impacts upon it. The springs are shortening, due to the acting force. The ram speed is decreasing and the duration of contact is lengthening due to spring behaviour. The pile starts moving downward with acceleration. Soil has displacement with elastic behaviour at the beginning but the displacement returns to plastic behaviour later on.

The force at pile head, pile tip resistance and sleeve friction are increasing their values.

c. Springs reach their shortest length. The ram starts moving upward due to the
reaction force from pile head pass through springs. The pile keeps moving downward with deceleration. Soil displacement is going on with its plastic behaviour and reaches its maximum displacement at point f. Force acting at pile head through the springs is decreasing due to the movement upward of ram and downward of pile.

The force at pile head, and the total pile resistance reach their maximum values at point c. After point c., although pile sleeve resistance is decreasing but its value is compensated by the increasing value of pile tip resistance. The summation of two values is constant during c and d, while the force at pile head is decreasing in this duration.

e. Soil has a rebound with elastic behaviour and pushes the pile upward. The velocity upward of pile is more than the velocity of ram. Therefore, the head of pile hit the ram again at point f.

All parameter values are increasing again due to a rebound of soil and reach their peak values in this range.

g. The ram is moving upward. Finally it reaches its highest point and starts going down and the same process starts again.
6 Test results analysis for the CPT and the static test in saturated sand

This chapter presents and analyses the results of the CPT and the static load tests. First, the test results of both kinds of tests are presented. Then, the results between the static load test I and II are compared to each other. It is compared in order to investigate whether the quasi-static loading has any effects on soil properties. Next, the test results from the CPT and the static tests are compared to study the influence of penetration speed on pore water pressure the pile resistance. Finally, the investigation between soil density and the pile resistance is analysed and discussed.

6.1 The presentation of test results from the CPT and the static tests

The set of test results from the CPT and the static load tests done with the f-1.5 v05 and f-1.5 v15 soil samples are shown as following charts in Figure 6.1. The different soil density samples are identified according to the vibration time. For example, the sample, which is vibrated 5 minutes, has its abbreviation as f-1.5 v05.

![Tip resistance](image)

a) pile tip resistance from f-1.5 v05

![Tip resistance](image)

b) pile tip resistance from f-1.5 v15

![Sleeve resistance](image)

c) sleeve resistance from f-1.5 v05

![Sleeve resistance](image)

d) sleeve resistance from f-1.5 v15
From chart a and b, it is clear that the pile tip resistance values from the CPT and the static tests are very quite similar in the both samples. The values are between 9 and 14 MPa in the sample f-1.5 v05 and between 8 and 12 in the sample f-.5 v15. Normally, the tip resistances from the CPT are higher than ones from the static test. The results still show that the tip resistance values from the static test I and II in each of test series are not considerably different.

From chart c and d, the sleeve resistance values from CPT and the static tests are similar. And no difference of sleeve resistance can be seen between the result from static test I and II.

From chart e and f, the excess pore water pressures from the CPT are normally higher than ones from the static load tests. The pore water pressures generate during pile driving should depend on the pile loading velocity.

Chart g and h show that the force at pile head values from the static test I and II are quite matching. Although the values from sample f-1.5 v05 look higher and more scatter when compares to the values from sample f-.5 v15, but their ranges are between 12 and 16 kN.

Figure 6.1 shows that the test results from the static test I and II are quite comparable in many cases. Normally, the tip and sleeve resistance from the static tests are not very
different from the CPT, but the measured pore water pressures of them can be seen lower than ones from the CPT. The different pile penetration velocities can influence on the pore water pressure but not on pile tip and sleeve resistance.

### 6.2 The effect of quasi-static loading on soil properties

The results from the static load tests I and II are shown in A 2. The values and their ratios are used to study the effect of quasi-static pile loading on soil properties. The effect can be determined by comparing the test results in the static test I and II. If the differences between two tests are small, then it can be concluded that the effect of pile loading on soil is too small or not exist.

**A 2 the test results from the static test I and II**

<table>
<thead>
<tr>
<th>soil</th>
<th>date</th>
<th>static I</th>
<th>static II</th>
<th>Pile resistance ratio (I/II)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Force (kN)</td>
<td>Tip (MPa)</td>
<td>Sleeve (MPa)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>f-1.5 v-5</td>
<td>20-Jan-05</td>
<td>13.80</td>
<td>10.00</td>
<td>0.040</td>
</tr>
<tr>
<td></td>
<td>21-Jan-05</td>
<td>13.10</td>
<td>10.04</td>
<td>0.044</td>
</tr>
<tr>
<td></td>
<td>24-Jan-05</td>
<td>18.40</td>
<td>14.00</td>
<td>0.048</td>
</tr>
<tr>
<td></td>
<td>25-Jan-05</td>
<td>12.70</td>
<td>9.50</td>
<td>0.040</td>
</tr>
<tr>
<td></td>
<td>26-Jan-05</td>
<td>16.00</td>
<td>12.50</td>
<td>0.060</td>
</tr>
<tr>
<td></td>
<td>27-Jan-05</td>
<td>14.00</td>
<td>10.70</td>
<td>0.040</td>
</tr>
<tr>
<td></td>
<td>4-Feb-05</td>
<td>15.30</td>
<td>12.44</td>
<td>0.047</td>
</tr>
<tr>
<td></td>
<td>7-Feb-05</td>
<td>16.20</td>
<td>12.78</td>
<td>0.054</td>
</tr>
<tr>
<td>average</td>
<td></td>
<td>13.00</td>
<td>10.04</td>
<td>0.030</td>
</tr>
<tr>
<td></td>
<td>14-Jan-05</td>
<td>12.30</td>
<td>7.90</td>
<td>0.037</td>
</tr>
<tr>
<td></td>
<td>17-Jan-05</td>
<td>13.00</td>
<td>10.17</td>
<td>0.023</td>
</tr>
<tr>
<td></td>
<td>18-Jan-05</td>
<td>11.90</td>
<td>8.50</td>
<td>0.040</td>
</tr>
<tr>
<td></td>
<td>19-Jan-05</td>
<td>14.80</td>
<td>12.00</td>
<td>0.036</td>
</tr>
<tr>
<td></td>
<td>2-Feb-05</td>
<td>15.00</td>
<td>12.00</td>
<td>0.043</td>
</tr>
<tr>
<td></td>
<td>3-Feb-05</td>
<td>14.10</td>
<td>10.00</td>
<td>0.023</td>
</tr>
<tr>
<td>average</td>
<td></td>
<td>13.00</td>
<td>10.04</td>
<td>0.030</td>
</tr>
</tbody>
</table>

- x means the parameter is failed to measure in that test

The comparison of pile resistance ratio, the summation of tip and sleeve resistances, show that the test results from the static test I and II are similar. Their ratios are in each soil sample are not different than 10 %. Then, the quasi-static test does not change the soil properties. The test can be done repeatable and the test result cannot be changed.

Due to the quite compatible in the test results of the static test I and II. Therefore, the next investigation is done to examine whether the results from both tests can be considered as the same set or not. If the analysis shows they can be considering as the same data, one of them will be used as a representative of group to compare with the test results from the CPT or the quasi-static load tests.

The t-test statistic method is widely used to check the two groups of data, which have
possibly different distributions, and the question is whether they are really different or
they are only difference by chance. The t-test method is applied to the set of test results
data, one from the static test I and another from the static test II. Since those set of two
data are originated from the same soil preparation, they can be classified as a pair group
sample.

The analysis is done under an assumption that both dataset have a realization of a
normal distribution. In order to prove that assumption, the data are plotted and
compared to a cumulative normal distribution as show in the Appendix 5. The curves
show that they are fitted with the normal distribution curve.

The t-test model has the null hypothesis; the parameters from the results of static pile
load test I are equal to the results of static pile load test II.

\[
H_o : X = Y
\]

\[
H_1 : X \neq Y \quad \text{at significant level } \alpha = 0.05
\]

**X** = data set from the static test I

**Y** = data set from the static test II

Then:

\[
T_p = \frac{\bar{X}_n - \bar{Y}_m}{S_p}
\]

The pool variance:

\[
S_p^2 = \frac{S_X^2}{n} + \frac{S_Y^2}{m}
\]

Where

\[
S_X^2 = \left( \frac{1}{n-1} \right) \sum_{j=1}^{n} (X_j - \bar{X}_n)^2
\]

is variance of \(X\)

\[
S_Y^2 = \left( \frac{1}{m-1} \right) \sum_{j=1}^{m} (Y_j - \bar{Y}_m)^2
\]

is variance of \(Y\)

\[
\bar{X}_n, \bar{Y}_m = \text{mean value of the data set } X \text{ and } Y
\]

\(n, m\) = number of sample in data set \(X\) and \(Y\)

The condition is that hypothesis null - the parameters from the results of static pile load
test I and II are equal- is true when \(T_p < T_{n+m-1,0.05}\) and is not true when \(T_p > T_{n+m-1,0.05}\).

It can be seen from the A 3 that the hypotheses are true for all parameters. The results of
the static test I are not different with the ones of the static test II. For that reason, to
analyse the static pile load test the data can be chosen from either of the static pile load
test I or II. From now on, test results analysis uses the results of static pile load test I as
a representative of the group. The results from calculation are shown in the A 3.

A 3: The t-test statistical properties of two data sets of the static load tests
6.3 The effect of pile penetration from the CPT on pore water pressure and pile resistance

The pile penetration speed can have effects on excess pore water pressure and pile resistance values. In order to see whether the rate of pile penetration influence on those parameters, the test results of different pile driven velocities are compared to each other. In the comparison study, the results from static load test are used as benchmarks. The values of pile tip resistance, sleeve friction and pore water pressure from the CPT are compared to ones from the static load tests. The ratios of those three parameters are shown in the A 4.

A 4: The ratio of tip resistance, sleeve friction and pore water pressure values between the CPT and the static load tests

<table>
<thead>
<tr>
<th>Sample conditions</th>
<th>Type of parameters</th>
<th>$T_p$</th>
<th>$T_{n+m-1,0.05}$</th>
<th>$H_0$ accepted</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.50 hrs fluidisation 5 minutes vibration</td>
<td>Force at pile head</td>
<td>0.694</td>
<td>2.145</td>
<td>True</td>
</tr>
<tr>
<td></td>
<td>Pile tip resistance</td>
<td>0.293</td>
<td>2.145</td>
<td>True</td>
</tr>
<tr>
<td></td>
<td>Sleeve friction</td>
<td>0.316</td>
<td>2.145</td>
<td>True</td>
</tr>
<tr>
<td></td>
<td>Pore water pressure</td>
<td>1.568</td>
<td>2.145</td>
<td>True</td>
</tr>
<tr>
<td>1.50 hrs fluidisation 15 minutes vibration</td>
<td>Force at pile head</td>
<td>0.276</td>
<td>2.681</td>
<td>True</td>
</tr>
<tr>
<td></td>
<td>Pile tip resistance</td>
<td>0.004</td>
<td>2.681</td>
<td>True</td>
</tr>
<tr>
<td></td>
<td>Sleeve friction</td>
<td>1.238</td>
<td>2.681</td>
<td>True</td>
</tr>
<tr>
<td></td>
<td>Pore water pressure</td>
<td>1.621</td>
<td>2.681</td>
<td>True</td>
</tr>
</tbody>
</table>

### Sample conditions

<table>
<thead>
<tr>
<th>Tip resistance</th>
<th>ratio</th>
<th>Sleeve friction</th>
<th>ratio</th>
<th>Pore water pressure</th>
<th>ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample condition</td>
<td>CPT</td>
<td>Static</td>
<td>CPT</td>
<td>Static</td>
<td>CPT</td>
</tr>
<tr>
<td>1.50 hrs fluidized 5 minutes vibration</td>
<td>10.70</td>
<td>10.00</td>
<td>1.07</td>
<td>0.042</td>
<td>0.040</td>
</tr>
<tr>
<td></td>
<td>10.30</td>
<td>10.04</td>
<td>1.03</td>
<td>0.043</td>
<td>0.044</td>
</tr>
<tr>
<td></td>
<td>13.65</td>
<td>14.00</td>
<td>0.97</td>
<td>0.050</td>
<td>0.048</td>
</tr>
<tr>
<td></td>
<td>11.37</td>
<td>9.50</td>
<td>1.20</td>
<td>0.037</td>
<td>0.040</td>
</tr>
<tr>
<td></td>
<td>11.98</td>
<td>12.50</td>
<td>0.96</td>
<td>0.048</td>
<td>0.060</td>
</tr>
<tr>
<td></td>
<td>10.00</td>
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<td>0.93</td>
<td>0.040</td>
<td>0.040</td>
</tr>
<tr>
<td></td>
<td>12.90</td>
<td>12.44</td>
<td>1.04</td>
<td>0.043</td>
<td>0.047</td>
</tr>
<tr>
<td></td>
<td>13.11</td>
<td>12.78</td>
<td>1.03</td>
<td>0.050</td>
<td>0.054</td>
</tr>
<tr>
<td>Mean value</td>
<td>11.75</td>
<td>11.49</td>
<td>1.03</td>
<td>0.044</td>
<td>0.046</td>
</tr>
<tr>
<td>Deviation</td>
<td>1.374</td>
<td>1.639</td>
<td>0.08</td>
<td>0.005</td>
<td>0.007</td>
</tr>
<tr>
<td>1.50 hrs fluidized 15 minutes vibration</td>
<td>11.00</td>
<td>10.04</td>
<td>1.10</td>
<td>0.051</td>
<td>0.030</td>
</tr>
<tr>
<td></td>
<td>8.90</td>
<td>7.90</td>
<td>1.13</td>
<td>0.040</td>
<td>0.037</td>
</tr>
<tr>
<td></td>
<td>9.00</td>
<td>10.17</td>
<td>0.88</td>
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<tr>
<td></td>
<td>9.00</td>
<td>8.50</td>
<td>1.06</td>
<td>0.037</td>
<td>0.040</td>
</tr>
<tr>
<td></td>
<td>12.20</td>
<td>12.00</td>
<td>1.02</td>
<td>0.043</td>
<td>0.036</td>
</tr>
<tr>
<td></td>
<td>12.04</td>
<td>12.00</td>
<td>1.00</td>
<td>0.043</td>
<td>0.043</td>
</tr>
<tr>
<td></td>
<td>10.00</td>
<td>10.00</td>
<td>1.00</td>
<td>0.040</td>
<td>0.023</td>
</tr>
<tr>
<td>Mean value</td>
<td>10.31</td>
<td>10.08</td>
<td>1.03</td>
<td>0.042</td>
<td>0.033</td>
</tr>
<tr>
<td>Deviation</td>
<td>1.447</td>
<td>1.562</td>
<td>0.08</td>
<td>0.004</td>
<td>0.008</td>
</tr>
</tbody>
</table>

- x means the parameter is failed to measure in that test
From the A 4, the tip resistance values from the CPT and from the static load tests are the same, although the results from the CPT are higher than from the static test but the differences are insignificant.

For the sleeve friction, the same conclusion can be drawn. The ratios from both soil samples can be considered in range of 1 although the higher in variance values can be noticeable.

The measured pore water pressures are different from the tip and sleeve resistance. The values of pore water pressure from the CPT are higher than ones from the static load test. The higher rate of pile penetration can generate higher excess pore water pressure.

It can be concluded that the excess water pressure can be influences from pile penetration speed. The CPT, which has a rate of pile penetration of 20.0 mm/s can generate higher excess pore water pressure than the static tests. However, it has no influence on pile resistance.

### 6.4 The influence of soil densities on pile resistance and pore water pressure during pile penetration

Normally, when soil density changes, the pile resistance from different soil densities should be different. It can be seen from A 4 that the measured pore water pressure, tip and sleeve resistance are quite similar from the two soil samples, f-1.5 v05 and f-1.5 v15. Then, the test results from two samples are studied to investigate whether the similar values of them are really similar or small different.

The t-test is used to analyse two sets of data. It has the null hypothesis that values of all parameters from soil sample f-1.5 v05 are equal to f-1.5 v15.

\[
H_0 : \bar{X} = \bar{Y}
\]

and

\[
H_1 : \bar{X} \neq \bar{Y}
\]

at level \( \alpha = 0.05 \)

\( X \) = data set from the static load test done in a soil sample f1.5 v05

\( Y \) = data set from the static load test done in a soil sample f1.5 v15

The condition is that hypothesis null - the values of all parameters from soil sample f-1.5 v05 are equal to f-1.5 v15 - is true when \( T_p < T_{n+m-1,0.05} \) and is not true when \( T_p > T_{n+m-1,0.05} \).

If the hypothesis null is accepted, it means that the soil densities from the different time vibration are the same. The provided vibration system could not change soil density, and therefore the pile resistances from different samples have the same values. The results of t-test analysis are shown in A 5.
A 5: The t-test statistical properties of data set from 5 and 15 minutes vibration time

<table>
<thead>
<tr>
<th>Load test method</th>
<th>Type of parameters</th>
<th>$T_p$</th>
<th>$T_{n+m-1,0.05}$</th>
<th>$H_0$ accepted</th>
</tr>
</thead>
<tbody>
<tr>
<td>CPT</td>
<td>Pile tip resistance</td>
<td>1.977</td>
<td>2.145</td>
<td>True</td>
</tr>
<tr>
<td></td>
<td>Sleeve friction</td>
<td>0.740</td>
<td>2.145</td>
<td>True</td>
</tr>
<tr>
<td></td>
<td>Pore water pressure</td>
<td>2.689</td>
<td>2.145</td>
<td>False</td>
</tr>
<tr>
<td>Static</td>
<td>Force at pile head</td>
<td>1.836</td>
<td>2.145</td>
<td>True</td>
</tr>
<tr>
<td></td>
<td>Pile tip resistance</td>
<td>1.703</td>
<td>2.145</td>
<td>True</td>
</tr>
<tr>
<td></td>
<td>Sleeve friction</td>
<td>3.378</td>
<td>2.145</td>
<td>False</td>
</tr>
<tr>
<td></td>
<td>Pore water pressure</td>
<td>1.425</td>
<td>2.145</td>
<td>True</td>
</tr>
</tbody>
</table>

From A 5, the t-test statistical values of all parameters showed that the null hypothesis is accepted. The soil densities from the two samples are not different. Only for the pore water pressure in the CPT and the sleeve friction in the static pile load test rejects the null hypothesis. However, if the pore water pressure and sleeve friction values are carefully considered, it can be seen that their values and their standard deviation are very small. To reject the Ho in these two cases can be a type I error. This type I error comes from the considering about the measured small values; the error in measuring system can be quite large, and has an effect in statistical analysis.

With the same principle, the t-test method is also examined the test results from sample f-1.5 v30. The sample f-1.5 v30 is introduced in order to achieve more soil density value than the sample f-1.5 v15. The test result data of f-1.5 v30 are analysed and compared to f-1.5 v05 and f-1.5 v15, which are combined together and used as the same data group. The t-test has the null hypothesis that values of all parameters from soil sample f-1.5 v30 are equal to the other two samples.

$$H_0 : \bar{X} = \bar{Y}$$

and

$$H_1 : \bar{X} \neq \bar{Y}$$

at level $\alpha = 0.05$

$X$ = data set from the static load test done in a soil sample f-1.5 v30

$Y$ = data set from the static load test done in a soil sample f-1.5 v05 and f-1.5 v15

The condition is that hypothesis null - the values of all parameters from soil sample f-1.5 v30 are equal to f-1.5 v05 and f-1.5 v15 - is true when $T_p < T_{n+m-1,0.05}$ and is not true when $T_p > T_{n+m-1,0.05}$.

The results of statistical analysis are shown in A 6.

From A 6, it can be concluded that the density of soil sample f-1.5 v30 is not very different from the soil densities in sample f-1.5 v05 and f-1.5 v15. The available vibration system cannot change the densities of soil in the calibration chamber. However, this comparison is done only between the test results from the CPT and the static tests. This conclusion must be confirmed with the quasi-static load test.

A 6: The t-test statistical properties of data set from 30 minutes and other vibration time
<table>
<thead>
<tr>
<th>Load test method</th>
<th>Type of parameters</th>
<th>$T_p$</th>
<th>$T_{n+m-1,0.05}$</th>
<th>$H_0$ accepted</th>
</tr>
</thead>
<tbody>
<tr>
<td>CPT</td>
<td>Pile tip resistance</td>
<td>0.799</td>
<td>2.101</td>
<td>True</td>
</tr>
<tr>
<td></td>
<td>Sleeve friction</td>
<td>2.715</td>
<td>2.101</td>
<td>False</td>
</tr>
<tr>
<td></td>
<td>Pore water pressure</td>
<td>0.898</td>
<td>2.101</td>
<td>True</td>
</tr>
<tr>
<td>Static</td>
<td>Force at pile head</td>
<td>0.447</td>
<td>2.101</td>
<td>True</td>
</tr>
<tr>
<td></td>
<td>Pile tip resistance</td>
<td>0.612</td>
<td>2.101</td>
<td>True</td>
</tr>
<tr>
<td></td>
<td>Sleeve friction</td>
<td>0.604</td>
<td>2.101</td>
<td>True</td>
</tr>
<tr>
<td></td>
<td>Pore water pressure</td>
<td>0.175</td>
<td>2.101</td>
<td>True</td>
</tr>
</tbody>
</table>
7 Test results analysis for the quasi-static test in saturated sand

This chapter presents the test results from the quasi-static load tests. First, the test results are presented, and then compared to the results from the static load tests. This comparison shows how the velocities of quasi-static pile loading influence on pore water pressure and the pile resistance. Finally, the influences of different soil densities on pile resistance are discussed.

7.1 The presentation of test results from the quasi-static tests

The results of quasi-static load tests are presented in this topic. The tests have been carried out with different vibrated-time samples, mainly are the f-1.5 v05 and the f-1.5 v15. The test results of both samples are shown in the Figure 7.1.

![Graphs showing test results](image)

a) tip resistance from f-1.5 v05  
b) tip resistance from f-1.5 v15

![Graphs showing test results](image)

c) sleeve resistance from f-1.5 v05  
d) sleeve resistance from f-1.5 v15
From the Figure 7.1, chart a and b show that the tip resistances from the quasi-static test are between 10 and 12 MPa. And, nearly the same values can be read from different soil samples.

Chart c and d, the sleeve resistances read from the charts have the lowest value of 0.04 MPa and the highest one of 0.06 MPa. As the same as the tip resistance values, the ranges of sleeve resistance from both samples are very alike.

Chart e and f show the excess pore water pressure values during pile penetration. The variation of pore water pressure values can be seen clearly. The lowest value is 0.028 MPa and the highest one is 0.065 MPa. The measured values from sample f-1.5 v15 are found more scatter than the sample f-1.5 v05.

From chart g and h, the forces at pile head from both samples are in the same range, from 16 to 21 kN. The drop heights, which are used to drive the model pile, have some variations during the experiments.

The statistical values of test results from quasi-static tests are shown in A 7, while the test results can be seen in Appendix 4.
A 7: The statistical values of test results from the quasi-static load tests

<table>
<thead>
<tr>
<th>Sample conditions</th>
<th>Type of parameters</th>
<th>mean value</th>
<th>standard deviation value</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.50 hrs fluidized 5 minutes vibration</td>
<td>Force at pile head</td>
<td>19.92</td>
<td>1.13</td>
</tr>
<tr>
<td></td>
<td>Pile tip resistance</td>
<td>11.36</td>
<td>1.13</td>
</tr>
<tr>
<td></td>
<td>Sleeve friction</td>
<td>0.051</td>
<td>0.006</td>
</tr>
<tr>
<td></td>
<td>Pore water pressure</td>
<td>0.043</td>
<td>0.007</td>
</tr>
<tr>
<td>1.50 hrs fluidized 15 minutes vibration</td>
<td>Force at pile head</td>
<td>19.43</td>
<td>1.62</td>
</tr>
<tr>
<td></td>
<td>Pile tip resistance</td>
<td>10.50</td>
<td>0.83</td>
</tr>
<tr>
<td></td>
<td>Sleeve friction</td>
<td>0.048</td>
<td>0.005</td>
</tr>
<tr>
<td></td>
<td>Pore water pressure</td>
<td>0.049</td>
<td>0.013</td>
</tr>
</tbody>
</table>

The statistical analysis of test results from the quasi-static test confirms the conclusion from the charts in Figure 7.1. Considering from the mean and standard deviation values of all parameters, the measured values from both samples are similar.

**7.2 The effect of pile penetration from the quasi-static test on pore water pressure and pile resistance**

In this study, the highest pile penetration rate is in the quasi-static test. The test results from the quasi-static load test are compared to ones from the static load test in order to examine the effect of pile loading rate on pore water pressure and pile resistance values. The comparisons are shown in Appendix 4. As same as done in the study of the loading effects of CPT on soil pile bearing capacity, the test results from the static test are set as references.

The study shows that the tip resistances from the quasi-static test are almost the same values as ones from the static tests, only 5% in mean values and with small deviation.

The same trend is also found in sleeve resistance. Although the ratios between values of sleeve resistances from the quasi-static and the static test have the range between 0.70 and 1.50, but that comes from the Type I error in the sleeve friction data from the static test.

Then, the total pile resistance from the quasi-static and static tests are calculated to find their ratios. The calculation shows that the ratios of total resistance between two kinds of tests are between 0.88 and 1.22 and have average value of 1.10.

Figure 7.2 shows the comparison between total pile resistance from the static and quasi-static test. It can be seen from the graph that the ultimate pile bearing resistance from two kinds of penetration loading is very comparable.
The study reveals that the excess pore water pressure measured in the quasi-static tests are higher than ones in the static tests about 9 times. The maximum measured excess pore water pressure in the quasi-static is 0.067 MPa and the minimum is 0.031 MPa.

The analysis shows that the rate of pile penetration in quasi-static test can generate high excess pore water pressure values. However, they do not effect on pile resistance. The total pile resistance values are constant.

7.3 The influence of soil densities on pile resistance and pore water pressure during pile penetration

The investigation about the effect of soil density on pile resistance and pore water pressure during pile penetration is still carried on with the quasi-static tests. In the CPT and the static load tests the study shows that measured values from soil f-1.5 v05 and f-1.5 v15 are not significantly different. Then, it leads to a conclusion that the vibration system cannot change sample density. However, to confirm that conclusion the same investigation has to be done on the results of the quasi-static load tests. First, the results from sample f-1.5 v05 and f-1.5 v15 are examined. If the result of the study shows a good agreement, then the investigation on sample f-1.5 v30 will be continued.

The t-test is used to analyse two sets of data. It has the null hypothesis that values of all parameters from soil sample f-1.5 v05 are equal to f-1.5 v15.

\[ H_0 : \bar{X} = \bar{Y} \]

and

\[ H_1 : \bar{X} \neq \bar{Y} \] at level \( \alpha = 0.05 \)

\( X \) = data set from the quasi-static load test done in a soil sample f-1.5 v05

\( Y \) = data set from the quasi-static load test done in a soil sample f-1.5 v15
The condition is that hypothesis null - the values of all parameters from soil sample f-1.5 v05 are equal to f-1.5 v15 - is true when $T_p < T_{n+m-1,0.05}$ and is not true when $T_p > T_{n+m-1,0.05}$.

A 8: The t-test statistical properties of data set from 5 and 15 minutes vibration time

<table>
<thead>
<tr>
<th>Load test method</th>
<th>Type of parameters</th>
<th>$T_p$</th>
<th>$T_{n+m-1,0.05}$</th>
<th>$H_0$ accepted</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quasi-static</td>
<td>Force at pile head</td>
<td>0.622</td>
<td>2.145</td>
<td>True</td>
</tr>
<tr>
<td></td>
<td>Pile tip resistance</td>
<td>1.692</td>
<td>2.145</td>
<td>True</td>
</tr>
<tr>
<td></td>
<td>Sleeve friction</td>
<td>1.135</td>
<td>2.145</td>
<td>True</td>
</tr>
<tr>
<td></td>
<td>Pore water pressure</td>
<td>1.075</td>
<td>2.145</td>
<td>True</td>
</tr>
</tbody>
</table>

From the results of t-test analysis are shown in A 8, it can be seen the hypothesis null is accepted. This proves that the tests are done in the same soil density. The provided vibration system is not efficient to change soil density, and therefore the pile resistances from different samples have the same values.

The investigation is also done with the soil sample f-1.5 v30. The study gives the same conclusion as shown in A 9.

A 9: The t-test statistical properties of data set from 30 minutes and other vibration time

<table>
<thead>
<tr>
<th>Load test method</th>
<th>Type of parameters</th>
<th>$T_p$</th>
<th>$T_{n+m-1,0.05}$</th>
<th>$H_0$ accepted</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quasi-static</td>
<td>Force at pile head</td>
<td>0.708</td>
<td>2.101</td>
<td>True</td>
</tr>
<tr>
<td></td>
<td>Pile tip resistance</td>
<td>0.003</td>
<td>2.101</td>
<td>True</td>
</tr>
<tr>
<td></td>
<td>Sleeve friction</td>
<td>0.588</td>
<td>2.101</td>
<td>True</td>
</tr>
<tr>
<td></td>
<td>Pore water pressure</td>
<td>1.685</td>
<td>2.101</td>
<td>True</td>
</tr>
</tbody>
</table>
8 The influence of excess pore water pressure and dynamic component on pile resistance in the quasi-static test

From the results of the quasi-static tests, one can be noticed that the excess pore water pressure generated during pile penetration is very high. Another is that the force at pile head and total pile resistance are not equal as in the CPT or the static tests. There are two parameters have to be investigated how they play roles in the test results of quasi-static load tests in the saturated sand. The first investigated parameter is the excess pore water pressure and the second one is a dynamic component. The excess pore water effect on pile resistance is analysed by considering how the test results from unsaturated sand are and then compares them to the test results form saturated samples. After the effect of excess pore pressure is known, then the effect of it can be separated from the effect of rate pile penetration in the analysis.

8.1 The influence of excess pore water pressure on pile resistance

From Chapter 7, the results from the quasi-static tests show that the measured excess pore water pressure are higher than ones from the static test at least 5 times. Their values can be raised up to 0.067 MPa. To investigate the effect of excess pore pressure on soil pile bearing capacity, the test results from unsaturated and saturated sand are analysed.

From the summary of section 6.4 and 7.3, it can be taken into account that the study has done only with one density of sand. Therefore, the test results from all different vibration time are combined and used as one dataset. The average values of measured parameters from the tests done with unsaturated and saturated sand are shown in A 10.

<table>
<thead>
<tr>
<th>Type of test</th>
<th>Sample condition</th>
<th>Force at pile head, F (kN)</th>
<th>Tip resistance (MPa)</th>
<th>Sleeve friction (MPa)</th>
<th>Pore water pressure (MPa)</th>
<th>Total pile resistance, R (kN)</th>
<th>F/R ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Static test</td>
<td>unsaturated</td>
<td>24.67</td>
<td>17.25</td>
<td>0.104</td>
<td>n.a</td>
<td>23.07</td>
<td>1.07</td>
</tr>
<tr>
<td></td>
<td>saturated</td>
<td>14.49</td>
<td>10.97</td>
<td>0.041</td>
<td>0.005</td>
<td>13.26</td>
<td>1.09</td>
</tr>
</tbody>
</table>

It can be seen from the A 10 that in unsaturated sand, which have no excess pore water pressure generated during pile penetration, the force at pile head are higher than the total resistance about 7%. With almost the same percentage number, the different between force at pile head and the total pile resistance can be seen in the test done with saturated sand. Although the excess pore water pressure can be generated about 0.005 MPa in saturated sand, it does not change the ratio between force at pile head and total pile resistance. Therefore, the pile penetration speed of 1.0 mm/s can be used as a speed of the static test, no effect of excess pore water pressure on pile bearing capacity.

A 11, shows the excess pore pressures from the quasi-static test are higher than ones
from static test about 9 times, but the total pile resistances from both tests are alike. The
two values of total pile resistance are only 4% different.

A11: the pile resistance parameters from the static and quasi-static load test done with
saturated sand

<table>
<thead>
<tr>
<th>Type of sample</th>
<th>Type of test</th>
<th>Force at pile head, F (kN)</th>
<th>Tip resistance (MPa)</th>
<th>Sleeve friction (MPa)</th>
<th>Pore water pressure (MPa)</th>
<th>Total pile resistance, R (kN)</th>
<th>F/R ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>saturated</td>
<td>static</td>
<td>14.49</td>
<td>10.97</td>
<td>0.041</td>
<td>0.005</td>
<td>13.27</td>
<td>1.09</td>
</tr>
<tr>
<td></td>
<td>quasi-static</td>
<td>19.75</td>
<td>11.00</td>
<td>0.050</td>
<td>0.044</td>
<td>13.80</td>
<td>1.43</td>
</tr>
<tr>
<td>Ratio between</td>
<td>quasi-static and static</td>
<td>1.36</td>
<td>1.00</td>
<td>1.22</td>
<td>8.80</td>
<td>1.04</td>
<td></td>
</tr>
</tbody>
</table>

Therefore, it can be concluded that the induced excess pore water pressure caused by
pile penetration does not effect on pile resistance value. In other words, the pile
resistance values obtained either from the quasi-static or static test are not different.

8.2 The influence of excess pore water pressure on force at pile head

From 8.1 the investigation shows that the excess pore water pressure does not affect pile
resistance values. However, from A11 it can be seen that there is a change in the force
at pile head value. To investigate whether that change caused by excess pore water
pressure, the force at pile head and excess pore water pressure are plotted in the same
graph as shown in Figure 8.1 to see their correlation.

![Figure 8.1: A graph plotted between force at pile head and excess pore water pressure in the quasi-static tests.](image)

From Figure 8.1, there is no pattern to connect two measured values. They are very
scatter and do not go in the same direction. Then, in the quasi-static test the excess pore water pressure does not response to the force acting on pile head.

8.3 The influence of inertia component on force at pile head

From A 11, the force at pile head from the quasi-static test is higher than ones from the static test about 36 %, while the difference in total pile resistance is only 4 %. From the conclusion in 8.2 that the excess pore water pressure is not concerning to that difference in force at pile head value. Therefore, the difference must come from the dynamic resistance only. To analyse the dynamic resistance, the Smith model is used for the purpose, analysis of impact pile driving.

8.3.1 The model of quasi-static pile load test and its analysis approach

From the Smith model, there are three forces acting on pile to counter to the force at pile head. They are an inertia force, a damp resistance and a static resistance. The Smith model is shown in Figure 8.2.

![Figure 8.2: The Smith model](image)

According to the Smith model, the force at pile head in the quasi-static test is consisted of the tip resistance, the sleeve friction and a resistance from dynamic, while in the static test the force at pile head composes with only two first components. The different between force at pile head in the static and quasi-static load test can be written as the following equations:

\[
F_{\text{head, qs}}(t) = R_{\text{tip, qs}}(t) + R_{\text{sleeve, qs}}(t) + R_{\text{dynamic, qs}}(t) \quad \text{quasi-static} \quad (8.1)
\]

\[
F_{\text{head, st}}(t) = R_{\text{tip, st}}(t) + R_{\text{sleeve, st}}(t) \quad \text{static} \quad (8.2)
\]

The Smith model also mentions about the limit of static component in the dynamic response, represented by a combination of spring and plastic slider. That means the static resistance, which soil responses to force in the elastic behaviour, is limited by pile penetration distance, or quake. If the penetration distance is less than quake, the resistance has elastic response behaviour. The resistance force reaches its ultimate value, Ru, when the distance is equal or more than the quake. And the resistance becomes zero when the acting force disappears.
From Figure 8.3, it can be seen that at the point, where the force at pile head reach its maximum value, soil is still in the elastic behaviour. The total pile resistance is proportional to the pile displacement. The quake can be seen as value of 1.6 mm. When the pile penetration is over than the quake, the total resistance is almost constant, although the force at pile head value goes down.

Due to the pile resistance in the static test and quasi-static test have similar value. Then, the total static resistance, which is used in the analysis of impact pile driving in the quasi-static test, is the values from the static test.

From the static resistance in the quasi-static load test is equal to the resistance in the static load test; the equation 8.1 and 8.2 can be rewritten as following equations:

\[ R_{\text{tip, qs}}(t) = R_{\text{tip, st}}(t) \]  \hspace{1cm} (8.3)

And

\[ R_{\text{sleeve, qs}}(t) = R_{\text{sleeve, st}}(t) \]  \hspace{1cm} (8.4)

Then:

\[ F_{\text{head, qs}}(t) = R_{\text{tip, st}}(t) + R_{\text{sleeve, st}}(t) + R_{\text{dynamic, qs}}(t) \]  \hspace{1cm} (8.5)

Where

\[ R_{\text{dynamic, qs}}(t) = R_{a, \text{inertia}}(t) + R_{v, \text{damping}}(t) \]  \hspace{1cm} (8.6)

\[ R_{\text{tip, qs}}(t) = \text{tip resistance in the quasi-static test at time } t \text{ (N)} \]
\[ R_{\text{tip, st}}(t) = \text{tip resistance in the static test at time } t \text{ (N)} \]
\[ R_{\text{sleeve, qs}}(t) = \text{sleeve resistance in the quasi-static test at time } t \text{ (N)} \]
\[ R_{\text{sleeve, st}}(t) = \text{sleeve resistance in the static test at time } t \text{ (N)} \]
\[ R_{\text{dynamic, qs}}(t) = \text{dynamic resistance in the quasi-static test at time } t \text{ (N)} \]
\[ R_{a, \text{inertia}}(t) = \text{inertia force in the quasi-static test at time } t \text{ (N)} \]
\[ R_{v, \text{damping}}(t) = \text{damping force in the quasi-static test at time } t \text{ (N)} \]
\[ R_{\text{st}}(t) = \frac{R_u}{s(t)} \text{ when displacement, } s, \text{ is less than the quake, } q \]
\[ = R_u \text{ when displacement, } s, \text{ is more than the quake, } q \]
\[ R_u = \text{ultimate static pile resistance (N)} \]
\[ s(t) = \text{pile displacement at time } t \text{ (m)} \]
\[ q = \text{quake (m)} \]
\[ R_a(t) = \text{inertia force at time } t \text{ (N)} \]
\[ m = \text{mass (kg)} \]
\[ a(t) = \text{pile acceleration at time } t \text{ (m/s}^2) \]
\[ R_v(t) = \text{damping force at time } t \text{ (N)} \]
\[ v(t) = \text{pile velocity at time } t \text{ (m/s)} \]
\[ N = \text{a power factor (normally 0.2 for sand)} \]
\[ J = \text{the damping coefficient (N-s/m)} \]

Then the final equations are:

\[
F_{head,qs}(t) = R_{u,ct} \frac{s(t)}{q} + ma(t) + Jv^N(t) \quad \text{if } s(t) < q \quad (8.7)
\]

\[
F_{head,qs}(t) = R_{u,ct}(t) + ma(t) + Jv^N(t) \quad \text{if } s(t) > q \quad (8.8)
\]

Thus, for given values of mass, \( m \), quake, \( q \), total pile resistance, \( R_u \) and \( N \) together with measured values of the acceleration, \( a \), and displacement, \( s \), and with the integrated value of velocity, \( v \), the \( F_{head,qs} \) can be calculated. The calculated and measured force at pile head, \( F_{head,qs} \) are compared, and then the damping coefficient, \( J \), can be found from the smallest variance of the different values of them.

### 8.3.2 Data analysis of test results from the quasi-static tests

To analyse the test results from the quasi-static test, the acceleration and displacement are very important. The displacements data are used to determine the quake and then calculate the static resistance during elastic behaviour displacement in soil. The pile velocity values can be obtained by integration the acceleration data. Unfortunately, acceleration data are available only in six tests. The results of analysis are shown in A 12.

From using the method analysis as mentioned in section 8.3.1, the data analysis is done by:

- Obtain the total pile resistance from the results of static tests.
- Find a quake value in the quasi-static test
- Integrate the acceleration data to obtain velocity relates with time
- From the raw dataset, substitute the displacement, acceleration and velocity, which are related with time into equation 8.7 and 8.8.
- Then, force at pile head can be calculated, \( F_{cal} \), by try the damping coefficient value into the equations.
- The measured force at pile head, \( F_{meas} \), are used to compare with the calculated values, \( F_{cal} \), to find the minimum variance different values between both of them

In the data analysis, the mass, which is used to calculate the inertia resistance, is a total mass of ram and pile. In the statnamic test, the definition of mass is only from a mass of pile. However, the \( F_{cal} \) with the mass of pile definition cannot match with the \( F_{meas} \) in this study as seen in Figure 8.4.
From Figure 8.4, it can be seen that the $F_{\text{meas}}$ curve is strongly influenced by the springs. That strongly spring influence verifies that the ram and pile move together while pile penetrates into sand. Therefore, the inertia force is a result of a combination of the total mass between ram and pile. Unlike the situation in the statnamic test, it is no mass of ram to hit a pile; only a pressure from the expanding gas drives a pile into soil. The inertia force can be regarded only from pile mass.

The result of the data analysis is showed in Figure 8.5. The graph is plotted between $F_{\text{meas}}$ and $F_{\text{cal}}$. The $F_{\text{cal}}$ values are done with the best fit damping coefficient value, which is defined when the smallest different value between two curves is found. The mass used in the equation is a mass of pile, springs and pile.
A 12 shows the J value from data analysis. It can be seen that only the results from the test done on 16 December has an influence by damping force, with the damping coefficient of 0.07. With that damping coefficient value, the damping force exists in the system only 2.35 % of Force at pile head. In other tests, no influence of damping force is found.

It can be concluded that in the quasi-static test with a model pile, the inertia force is dominant in the dynamic resistance.

A 12: The component of soil resistance in the quasi-static load test and their coefficient values

<table>
<thead>
<tr>
<th>Day of testing</th>
<th>Force at pile head (measured) F meas</th>
<th>Total pile resistance (measured) Ru</th>
<th>Damping coefficient (best fit) J</th>
<th>Inertia force (calculated) Ra</th>
<th>F / R ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>16 December</td>
<td>17</td>
<td>12.3</td>
<td>0.07</td>
<td>4.3</td>
<td>1.38</td>
</tr>
<tr>
<td>14 January</td>
<td>19</td>
<td>12.5</td>
<td>0</td>
<td>6.5</td>
<td>1.52</td>
</tr>
<tr>
<td>17 January</td>
<td>20</td>
<td>13.0</td>
<td>0</td>
<td>7.0</td>
<td>1.54</td>
</tr>
<tr>
<td>18 January</td>
<td>20</td>
<td>12.5</td>
<td>0</td>
<td>7.5</td>
<td>1.60</td>
</tr>
<tr>
<td>20 January</td>
<td>21</td>
<td>13.8</td>
<td>0</td>
<td>7.2</td>
<td>1.52</td>
</tr>
<tr>
<td>21 January</td>
<td>21</td>
<td>13.1</td>
<td>0</td>
<td>7.9</td>
<td>1.60</td>
</tr>
</tbody>
</table>
9 Discussion

This chapter presents the overall idea of test results done in this research. The test results from three different pile penetration speeds are analysed together. The effect of loading rate on excess pore water pressure and pile bearing capacity are presented. Then, the influence of soil density on test results is discussed. Finally, the chapter presents the analysis of the dynamic component in the quasi-static test.

9.1 Effect of pile penetration speed on pore water pressure

The measured pore water pressure from three different tests show that its magnitudes depend very much on pile penetration speed as shown in Figure 9.1. The highest excess pore water pressure value can be measured in the quasi-static test, while the lowest one is measured in the static test. It can be seen that the pore water pressure values in the CPT and the static test are not very different.

Another aspect can be seen from the same graph. The graph represents the test results from different vibration-durations; however, it can be seen that the measured pore water pressure from the same pile penetration speed is not different. Then, it can be concluded that the soil density of unsaturated sand is not a parameter inducing excess pore pressure in this sand sample. In other words, the density of the sand does not change by the provided fluidization and vibration procedures used. This leaves some doubts.

![Figure 9.1: Measured pore water pressure in three different tests](image)

Further studies should be done to clarify this doubt. The first option is by changing or improving the fluidization and vibration. This would allow verifying whether the system works efficiently or not. If it works efficiently, the density of this sand sample will changed. Then, the test should confirm the conclusion that the soil density is not a
parameter to generate the excess pore water pressure. The second alternative is changing the test sand. This will determine whether other sand will respond to pile penetration speed and influence excess pore water pressure differently.

From Figure 9.2, it can be quite understandable that the excess pores water pressure in the quasi-static is very high, because it responds to compression wave propagation from pile toe. In the first one millimetre of displacement, it can be seen from the rate of penetration is still quite slow, and then the excess pore water can still flow out from the compression zone. However, when the penetration rate is about 350 mm/s then the pore water pressure value rises. Until the pile displacement reaches 6 mm, the water pressure becomes negative due to the local dilatancy of sand occurring. Then, the pile rebounds and water flows into the pile tip area. Then, pore water pressure starts increasing again. Similar results were also reported by Eiksund (1994)

The behaviour of the excess pore water pressure from this research is different than the results from research of Lu and Impe (1996). In their report, a small negative pore pressure happened first and then followed by the positive value due to the compression wave propagation from pile toe. They explained that a small dilative movement between the pile and the soil could occur before the soil is compressed as the stress wave arriving at the pile tip. But from Figure 9.2, it can be seen that the positive pore water pressure generated by the stress wave is very large and happens at the same time as the pile starts penetrating into soil. Then, the first negative pore water pressure cannot be noticeable.

Figure 9.2: A graph plotted between pore water pressure and time against pile displacement

In the CPT and the static test the pore water pressure value is not high like in the quasi-static test. The cone penetration speed in the CPT is about 20 mm/s and in the static test is about 1 mm/s. Comparing the rate of cone penetration in both kinds of tests and the
permeability value of this sand, 0.9 mm/s, it is clear that the rate of penetration cannot affect the pore water pressure. The water can flow out from the compression zone due to the high permeability of sand. Then, the pore water pressure cannot rise up very high.

### 9.2 Effect of excess pore water pressure on pile bearing capacity

Although the test results show the excess pore water pressure from the CPT and the static test are very different from the quasi-static test, the pile bearing capacity from the three tests are similar. Figure 9.3 shows that the measured excess pore water pressure can be divided into two groups. The lower values are the values from the CPT and the static test, while the higher ones are from the quasi-static test. From the graph, it is clear that the mean value of pile bearing in the two groups are similar. In the quasi-static test the excess pore water pressure can be measured at 0.07 MPa, however it is relatively small in comparison with the bearing capacity value. Then, it does not influence the pile bearing capacity.

![Graph](image.png)

**Figure 9.3:** A graph plotted between pile bearing resistance and excess pore water pressure

### 9.3 Effect of pile penetration speed on pile bearing capacity

In the quasi-static test, the test results include the static and dynamic component. As already mentioned in section 8.3.1, when the soil still has its elastic behaviour during pile penetration in the quasi-static loading, the pile resistance is equal to the resistance in the static test. Then, the pile resistance in this range is called the static resistance. When the pile penetration is above the quake thresholds, the soil will have plastic behaviour. The resistance in this range is called the dynamic resistance. Figure 9.4 presents the static resistance values from three kinds of tests. The static resistance is a total resistance of cone tip and sleeve resistance during the elastic behaviour of soil.
From the figure, it can be seen that the static pile resistance from the CPT, the static and the quasi-static test are very similar, although the excess pore water pressure as shown in Figure 9.1 are quite different. First, it means that the excess pore water pressure does not influence pile-bearing capacity of the model pile. Second, the rate of pile penetration does not affect pile-bearing capacity.

Eiksund (1996) also concluded, in the same direction, that the pile resistance is almost independent of velocity, and there should not be a need for any viscous component in a rheological model describing these results.

Figure 9.4: A chart plotted the static pile resistance and loading rate.

Figure 9.5 shows that the measured force at the pile head is higher than the static resistance value about 40% in average for the quasi-static test. In the CPT and static tests the differences between both values are not higher than 5%. These comparisons show that in the quasi-static test, the 40% difference between force at pile head and static pile resistance values is due to the dynamic resistance.

The dynamic resistance in the quasi-static test is composed of the inertia and damping force. The inertia is acceleration dependent, while the damping force is velocity dependent. The analysis has been done as mentioned in section 8.3. The results from the study show that the inertia is very dominant in the quasi-static test in this study, while the damping force is not significant.

For a better view, the analysis of this study shows that the inertia force is the result of the total mass hitting the pile head. The total mass consists of the mass of dropping weight, mass of springs and mass of pile. In the statnamic method it is recommended to use the mass of the pile only, because the action force comes from the expanding gas, which has no weight. On the contrary, in this study the action force comes from the momentum of ram. When the ram hits the pile head, the two masses must move together.
Therefore, the inertia should be considered as the result of all masses in the system.

Figure 9.5: Graph relating force at pile head and static pile resistance

Figure 9.6, the force at pile head, $F$, is plotted against the total of dynamic and static resistance, $R$, with different damping coefficient, $J$. $J$ values are varied from 0 to 0.20 which is done in order to investigate the effect of damping force on the $R$ value. It can be seen that the curve with $J$ value equal to zero, no damping force considered, is the most fitted to the measured forces at pile head.

Figure 9.6: A graph plotted between force at pile head, $F$, total of dynamic and static pile resistance, $R$, from different damping coefficient values, $J$, in the quasi-static test
Results obtained from resistance calculations in time using the Smith rheological model with different damping factors $J$, are compared with direct measurements. Figure 9.7 presents a comparison of the fit between calculations and observations showing that damping factor equal to 0 corresponds to the best representation of actual criterion.

Therefore, in this study the inertia is the main parameter in the dynamic resistance and the damping shows insignificant influence.

![Graph showing the relation the standard deviation of the difference between force at pile head and total pile resistance from different damping coefficient values](image)

**Figure 9.7:** Graph showing the relation the standard deviation of the difference between force at pile head and total pile resistance from different damping coefficient values

### 9.4 Limitations

#### 9.4.1 The limitation of changing soil density

In this study, the tests correspond basically to only one density of sand. Therefore, it is difficult to know how pile penetration rate affects pile-bearing capacity for different soil densities. The problem appears to be that the available fluidization and vibration do not change soil densities efficiently for this type of sand.

Therefore, to do the test a new method to compact sand should be considered. First, it can be done by installing a pressure cell inside the calibration chamber either at the top or bottom. Second, another option is to improve the fluidization procedure. Possible explanations for this shortcoming are to be found in the inadequacy of the procedure and equipment used for fluidization to achieve a good efficiency for the particular sand used.
9.4.2 The limitation of the rate of cone penetration in the static test

In the constant rate of penetration static load test, the pile loading speed is recommended to be between $8.3 \times 10^{-3}$ and $33 \times 10^{-3}$ mm/s. The pile loading speed in this test is about 1 mm/s. This means that the penetration speed has been taken into account in the pile resistance measurement.

The rate of pile penetration for the static test should be revised. However, to push a pile with very low velocity, $33 \times 10^{-3}$ mm/s, cannot be done manually. The automatic device to control the pile driving system could be a kind of a solution.
10 Conclusions and recommendations

This chapter presents the conclusions from the study. The recommendations for further study are also included.

10.1 Conclusions

- There is no loading rate effect in pile bearing capacity in the medium dense sand. For the cone diameter of 36 mm, the pile loading rate in a range of velocities 1 – 350 mm/s give the same pile bearing resistance.

- The dynamic pile resistance depends on the acceleration of the pile. From the result of Smith rheological model, the inertia force is very important in the pile resistance during the impact loading.

- The induced pore water pressure during pile driving is very dependent on pile penetration velocity. The magnitude of measured pore water pressure varies from 0.005 MPa at penetration rate of 1 mm/s to 0.045 MPa at penetration rate of 650 mm/s.

- The excess pore water pressure is a function of soil compressibility and dilatancy behaviour at the region near the cone tip.

- The excess pore water pressure does not influence the pile bearing capacity. The magnitude of pore water pressure is relatively small compared to the magnitude of pile resistance.

10.2 Recommendations

In order to achieve a more complete understanding of all factors, some additional investigations are recommended:

- One-scale pile

  This study has been done only with the piezometer cone as a model pile. The one-scale pile size should be studied and compared to the results from a model scale pile to investigate the scale effects.

- Cone shape

  Only the standard piezometer cone has been used in this research. Test using other cone shapes to investigate effect of cone angle and its size should be studied.

- Pile geometry

  The piezometer cone is used as a model pile in this research. With its round
shape and conical pile tip, the induced pore water pressure may not show any effects. In reality, many of pile shapes are used in a commercial market. The study to investigate how those variety shapes and the excess pore water pressure caused be different penetration rates influence on pile resistance should be done.

- Sand type

This research is done only with one sand sample. Test using different sands should be studied in order to extend the understanding about the loading effect on pile resistance.

- Rate of pile penetration

In this research the limitation of pile penetration velocity in the static test is about 1 mm/s. The lower velocity at $33 \times 10^{-3}$ mm/s should be investigated.

- The launching method in the quasi-static test

To study the influence of damping force in the quasi-static test, the strong influence of ram should be eliminated. The concept of expanding gas of the statnamic method should be considered.

- The soil density

In this study, the effect of soil density on pile bearing capacity is not clear. The density of sand could be a parameter for pile bearing capacity. Other compaction procedure should be improved in order to achieve the required densities. The test in different relative densities of sand should be carried on.
References


## Appendix 1

Table A1-1: The correlation between 10 and 50 % force at pile head in the static load test I and II

<table>
<thead>
<tr>
<th>date</th>
<th>force measured 10 % displacement (kN)</th>
<th>force measured 50 % displacement (kN)</th>
<th>Force ratio between 10 % and 50%</th>
<th>force measured 10 % displacement (kN)</th>
<th>force measured 50 % displacement (kN)</th>
<th>Force ratio between 10 % and 50%</th>
<th>Remarks</th>
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<td>13.00</td>
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<td>12.50</td>
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<td></td>
<td>0.88</td>
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Appendix 2

For the loading mechanism in the quasi-static test, a series of springs are implemented in order to achieve the longer wave pulse period. The spring used in the study is a disc spring type. It has a stiffness value of 7.5 kN/mm, an inside and outside diameter of 57 and 112 mm, respectively. The springs could be assembled as a combination to get a high or low stiffness value. There are two kinds for assemble the series of springs: in parallel and in series. The springs in parallel will make the stiffness value become higher. The same force apply the lower displacement occurred. But in this study, the high displacement value is needed. Therefore, the springs are assemble in a series order and the total stiffness can be expressed as:

\[ k_t = \frac{k}{n} \]

- \( n \) = number of springs
- \( k \) = spring stiffness
- \( k_t \) = spring stiffness of the series

In order to get the wave length of the dynamic pulse, the motion of pile head equation from Hoelscher(1995) is used.

\[
w(t) = \frac{2c_0\omega}{\alpha^2 + \beta^2} \left[ 1 - \left( \frac{\alpha + \omega}{2\omega} \right)^{\alpha - \omega \frac{2\pi}{\omega}} + \left( \frac{\alpha - \omega}{2\omega} \right)^{\alpha - \omega \frac{2\pi}{\omega}} \right]
\]

Then

\[
F(t) = Z_p w(t) = \frac{E_A}{c_w} \left[ 1 - \left( \frac{\alpha + \omega}{2\omega} \right)^{\alpha - \omega \frac{2\pi}{\omega}} + \left( \frac{\alpha - \omega}{2\omega} \right)^{\alpha - \omega \frac{2\pi}{\omega}} \right] \left[ \left( \frac{\alpha^2 - \omega^2}{2\omega} \right)^{\alpha - \omega \frac{2\pi}{\omega}} + \left( \frac{\alpha^2 - \omega^2}{2\omega} \right)^{\alpha - \omega \frac{2\pi}{\omega}} \right]
\]

While

\[
\alpha = \frac{k}{2c_v}
\]

\[
\beta = \frac{k}{2c_v} \sqrt{\frac{c_v^2}{mk} - 1}
\]

\[
Z_p = \frac{E_A}{c_w}
\]

\[
c = A_n \sqrt{E \rho_s}
\]

\[
v_o = \sqrt{\frac{2gh}{\rho_s}}
\]

and

\[
c_w = \sqrt{\frac{E_A}{\rho_s}}
\]

\[
m = \text{mass of ram} \quad 64.9 \quad \text{[kg]}
\]

\[
E_s = \text{young's modulus of steel} \quad 2.1 \times 10^{11} \quad \text{[N/m}^2\text{]}
\]

\[
A_p = \text{section surface} \quad 7.63 \times 10^{-4} \quad \text{[m}^2\text{]}
\]

\[
\rho_s = \text{volumetric mass of steel} \quad 7850 \quad \text{[kg/m}^3\text{]}
\]

\[
c_w = \text{wave velocity in pile} \quad 5172 \quad \text{[m/s]}
\]

\[
c = \text{damper coefficient} \quad 3.1 \times 10^4 \quad \text{[Ns/m]}
\]

\[
Z_p = \text{pile impedance} \quad 3.1 \times 10^9 \quad \text{[Ns/m]}
\]

\[
v_o = \text{initial velocity of the mass} \quad 1.25 \times 10^6 \quad \text{[kN/m]}
\]

\[
k = \text{spring stiffness}
\]
Then from the above equation, the period of pulse wave and Fmax can be obtained.

From analysis, the spring systems will give Fmax at 64.71 kN and period of 23.78 ms.
### Appendix 3

Table A3-1: Force at pile head and model pile resistance values from the loading test with saturated sand in a calibration chamber in this study.

<table>
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Average values are also provided for comparison.
## Appendix 4

Table A4-1: The values of force at pile head, pile tip resistance, sleeve friction and pore water pressure and their ratio values between the quasi-static and static pile load test

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<th>Tip resistance</th>
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Appendix 5