Chapter 1

Introduction and related concepts

1.1 Introduction

'Scientific approaches to pile design have advanced enormously in recent decades and yet, still, the most fundamental aspect of pile design -that of estimating the axial capacity- relies heavily upon empirical correlations'

M.F.Randolph (2003)

Technology and development nowadays move at a vertiginous rate. High-rise buildings, bridges that communicate countries, increasing offshore construction, are just few examples that corroborate it. More and more there is an increasing necessity and use of deep piled foundations with higher capacity. Perhaps even more relevant, more and more there is an increasing necessity of evaluation and interpretation methods to optimize pile design, more cost-effective but equally reliable solutions are desired. This concept is the one moving this thesis.

Much of the design of pile foundations is still dominated by determination of axial capacity by means of empirical design piles. This research has been carried out aiming to be a little but hopefully useful contribution to the topic. Randolph [1] noted the existing dichotomy between (a) empirical correlations to quantitatively achieve a proper design and (b) the conceptual and analytical frameworks for estimating pile capacity. Ideally, any ‘empirical’ trend should be supported by ‘science’ in the form of models. Nevertheless, uncertainty and knowledge gaps appear in-between. This explains the scope of the current research: not only experimental testing has been performed, also, and more relevant for this concrete report, an attempt to develop a suitable analytical model has been made, as well as a numerical model. The purpose is to link all three approaches, to propose coherent and consistent conclusions and solutions from both technical and scientific point of view.

This report focuses on estimation of axial capacity for driven piles in saturated sand. More particularly, what attracts the interest is the methodology. The new pseudostatic test combines the reliability of the widespread static test with the efficiency of the dynamic one. It can be directly correlated to the static without need for calibration; nevertheless, it keeps the costs down. Bermingham (2004) compared the prices for US market: 100$/ton for the static test versus 10$/ton for the pseudostatic one.
1.2 Related concepts

However, it is still a dynamic test and this could be a source of complications. The main objective is to investigate the accuracy and applicability of this pseudostatic test in saturated sand. In this case, two factors may be critical: loading rate and generation of excess pore pressure. In dry sand condition, the first one was previously studied experimentally by Dijkstra (2004). Now the generation of excess pore pressures, how they can affect the effective stresses and the bearing capacity must be studied. A new series of tests equivalent to those of Dijkstra but in saturated sand have been performed in cooperation with IHE-Unesco student E.Archeewa. Besides also the scientific insight has been questioned. An extensive literature study will present the potential and shortcomings of the available scientific models for the pseudostatic case. With the literature as background, an analytical model for the pseudostatic case will be adapted. A numerical model by means of Finite Element Method with PLAXIS will also be presented. This two models will be compared with the experimental results. The intention is to merge 'science' and 'empirism'. '...we must incorporate such science in our teaching and our practice, using empirical approaches to validate and calibrate them, but not replace, scientific theory.' M.F.Randolph (2003)

1.2 Related concepts

To be able to carry a meaningful research and achieve consistent results or conclusions, a proper understanding of the issue and its related theoretical background is crucial. Dealing with a problem as an isolated question, without enough comprehension of its context, there is no way it can be properly solved. It is necessary not only to know which is the problem but also why this problem arises and which is its significance. This is the aim of this section.

1.2.1 Problem analysis

In this subsection the tools for understanding and defining the problem are provided. First some related theoretical concepts related to the topic are explained \(^1\), further on the problem itself can be stated and the reasons and significance of its solution. In following sections, objectives and limitations will be defined in order to get a clear idea of the boundaries, potential and shortcomings of the current research. With this frame in mind the next step will be to define the research to solve the problem hereby presented.

Introduction of the subject

Concepts on pile technology A pile foundation is a relatively long and slender element that is bored or driven into the soil or casted in-situ. They are mainly used to transmit the structural load to a firmer, less compressible soil or rock at greater depths. Other possible uses include:

- Sustain horizontal forces, like those from bridge abutments or retaining walls.
- Increase the stability of tall buildings
- Carry uplift forces
- Avoid scour damage
- Compact loose sands

Piles can be classified according to the type of material they are made of, the mode of load transfer, the method of installation or the degree of ground displacement during pile installation. According to that we can find piles made of:

\(^1\)theoretical concepts extracted from Simons and Menzies [2] [3] books on foundation and piling engineering
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Figure 1.1: Pile uses

- Timber
- Concrete
- Steel
- Combinations of diverse materials

Piles transfer load to the soil in two ways, by friction or directly to the soil below the tip:

\[ Q_{\text{pile}} = Q_{\text{friction}} + Q_{\text{tip}} \]  

(1.1)

The relative proportions of load carried by side-resistance or end-resistance depends on the shear strength and stiffness of the soil and on the installation method of the pile. It is interesting to note that the vertical movement of the pile required to mobilize full end-resistance is much greater than that required to mobilize full frictional resistance. Depending on the dominant mode, one can distinguish between:

- **End-bearing piles**: they derive their resistance (from now on called capacity) mainly by axial transmission of the load to an underlying hard, impenetrable layer of soil or rock.
• **Friction piles**: if the pile cannot be driven to such a hard stratum, the load is then primarily borne by skin friction or adhesion between the shaft surface of the pile and the adjacent soil.

In function of the installation method, two more classes exist:

• **Bored pile**: Made from reinforced concrete cast in a pre-drilled hole in the ground.

• **Driven pile**: Made of steel, concrete or timber; they are hammered into the soil.

However, it is the last classification, according to the degree of ground displacement during installation, the most suitable one. Piles either fit in one of the three categories:

• Small displacement pile

• Large displacement pile

• No displacement pile

This research deals with *closed-end steel tubular displacement piles*. They are characterized by a high bending and buckling resistance, and have favorable energy absorbing characteristics for impact loading. They are not susceptible to damage caused by tensile stresses during driving and can withstand hard driving. All steel piles considered in this study are driven ones. The study soil is *sand* and the pile will be axially loaded. Piles in granular soils mainly act as *end-bearing piles*.

**PILE CAPACITY AND FAILURE** The concept of *pile capacity*, as the ability of the pile to pursue its service, to meet the loading requirements, has already been introduced. The pile capacity may be evaluated either considering the supporting strength of the soil, which is the perspective that interests the geotechnical engineer.

The *ultimate bearing capacity* is the limit state for which larger loads cause the pile to fail. For end-bearing piles it is directly linked, and may be identified, to the ultimate base capacity. The load causing ultimate failure of pile material or the load at which the bearing resistance of the soil is fully mobilized is known as *failure load*. But, from an engineering approach, failure of the structure may have occurred long before the ultimate load is reached, mainly with criteria related to serviceability state. There are different criteria depending on the different construction codes.

**LOAD-SETTLEMENT CURVE** The relationship pile vertical displacement versus applied axial load on top is a very useful tool. At early stages of the loading the settlement is very small and mostly due to elastic behavior. If the load were to be removed in such a point as A (see figure 2.2), the head of the pile would rebound almost to the original level. The largest portion of load is carried by shaft resistance on the upper part of the shaft. In a point as B exists some permanent settlement, indicating that plasticity has started to occur. Although still most load is laterally supported by friction, some of it is being carried at the pile tip. When point C is reached, settlement increases rapidly with little further load increase. The ultimate load has been reached and is mainly carried by end-bearing. The determination of this ultimate load on beforehand is then very important to the engineer.

**PILE LOAD TESTS** We have seen how important it is to know the capacity of a pile, even better, to know it’s full load-displacement curve. Load tests are performed on-site on test-piles to determine these properties. Normally piles are initially designed according to analytical or experimental methods based on soil characteristics or estimated loads. During design stage pile load tests are performed to:

1. Determine settlement under working load
2. Determine ultimate bearing capacity

3. As a proof of acceptability.

Traditionally *static* load testing has been used as it provides reliable guaranty. However, its main shortcomings are:

- High costs
- Long time required

An alternative is provided by means of the *dynamic* test. Its main advantage versus the static one is that it doesn’t require devices to obtain the reaction force because this is a result of a change of momentum, leading to less time required and minor costs. This reaction force can be assimilated to the ultimate pile bearing capacity, but it poses a
1.2 Related concepts

Figure 1.3: Kentledge reaction stack for static pile test

major handicap: this measured pile capacity is a combination of both static and dynamic components, which implies that calibration with a static test is required. Besides, other remarkable disadvantages of the dynamic test are:

- Stress-wave phenomena introduces tension waves that can damage the pile
- Eccentric loading can introduce bending stresses, also risking to damage the pile
- The obtained results require cumbersome manipulation by means of signal-matching
- In some cases it may be difficult to mobilize the full capacity of the pile

To satisfy Industry’s demand for cost-effective and accurate means of testing deep foundations, the **pseudostatic** test has been developed. Still considered a dynamic test, it combines high loads with low pile velocities and accelerations like in the static test, but with the quickness of the dynamic one. The required reaction mass is only 5-10 per cent of the one for static test and, apart from that, its long duration when compared to the dynamic test emphasizes the static component, allowing the operator to be able to establish in a straightforward manner the static load behavior without need for calibration. Hence, a test can be considered pseudostatic or quasistatic if the duration for which the load is above 50 per cent of the maximum load fulfills the condition:

\[ t_{50} \gg \frac{2l}{c} \]  \hspace{1cm} (1.2)

where \( c \) is the wave celerity in the pile material. This relatively long duration keeps the pile always under compression, avoiding the possible development of tension stresses. Hence, it can be assumed that the pile reacts as a rigid body and, soil displacement and pile top displacement are equal. Moreover it guaranties the central location of the dropping mass, doesn’t introduce eccentricity, it’s fully axial loading.

**Definition of the problem and previous findings**

The pseudostatic test combines the advantages of both static and dynamic tests, which turns it into a thrilling interesting new technology. It just has ‘one’, but determining, disadvantage: a lot of important questions about it have not been answered yet. TNO and Berminghammer developed the so-called statnamic device [4] and even proposed a simple model for it. Despite it, some question marks remain:
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- **Loading rate effect**: increasing loading rates can alter the ultimate bearing capacity. This concept was investigated by Dijkstra [5] in dry sand, finding not remarkable influence of the loading rate in the pseudostatic test. More references to Dijkstra’s work will be made along the report.

- **Excess pore pressures**: There is a research gap when considering saturated soils. Cohesive soils behave always undrained but for granular fine-grained soils like sand the effect of the rapid test in the water pressure is still unknown. Generally they are considered drained material but now, the larger loading rate can influence the soil behavior. As the excess pore pressures affect the effective stresses they may play a relevant role in the penetration resistance and thus the bearing capacity. Hence, is the pseudostatic method suitable to be used in saturated sands like the case of the Netherlands? Or may the test lead to erroneous results, compromising the design criteria of the foundation? Moreover, how reliable is the method when water is involved in the problem?

The second point is what defines the problem.

**Problem definition**: The applicability of the pseudostatic method in saturated sands is still unknown. To what extent due to higher loading rate excess pore pressures are generated and how they affect the bearing capacity needs further investigation. Correlation with the static values is necessary for the acceptance of the method.

And, besides, the one must keep in mind the results of Dijkstra’s [5] research in dry sand:
### 1.3 Previous researches

**STATIC TEST**

<table>
<thead>
<tr>
<th>Advantages</th>
<th>DYNAMIC TEST</th>
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<tr>
<td>-Reliability of results</td>
<td>-Short time required</td>
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<td>-Low cost</td>
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<tr>
<th>Shortcomings</th>
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<tr>
<td>-Long time required</td>
<td>-Eccentric loading possible → Possible pile damage</td>
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<td>-High costs</td>
<td>-Cumbersome result manipulation</td>
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<td>-Calibration with static required</td>
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<td>-Stree-wave phenomena → Tension waves</td>
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<td>-Difficult to mobilize full pile capacity</td>
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### PSEUDOSTATIC
(still dynamic but emphasizes static component)

**PSEUDOSTATIC vs STATIC**

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<th>Advantages</th>
<th>PSEUDOSTATIC vs DYNAMIC</th>
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<tr>
<td>-Short time required</td>
<td>-Longer time (but quick enough) → Emphasizes static component</td>
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<tr>
<td>-Required reaction mass: 5-10% of the static one</td>
<td>-No stress-wave introduced → No tension waves (pile always under compression)</td>
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<td>-Lower costs</td>
<td>-No need for calibration</td>
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<td>-Fully axial</td>
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<td>-Easy manipulation and model</td>
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**PSEUDOSTATIC vs DYNAMIC**

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PSEUDOSTATIC test still has to be better understood, especially in saturated granular soils there are 2 effects that need investigation:

1. **LOADING RATE** → J. Dijkstra’s thesis
2. **EXCESS PORE PRESSURES** → *Main purpose of this thesis*

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Dijkstra [5] designed the experimental set-up to be used in this research. He carried the same series of static and pseudostatic scaled tests as in the current case but in dry sand. He found that a velocity increase from 1 \( \text{mm/s} \) to 250 \( \text{mm/s} \), the difference static-pseudostatic, did not imply any significance increase in bearing capacity, only 4% for the tip and 6% for the shaft resistance. Therefore, no loading rate effect on the pile capacity was found.

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**Figure 1.5: Comparison among different load test methods**
1.3.1 Loading-rate effects

The only scientists who have explicitly modeled pseudostatic testing were Middendorp, Bermighammer and Kuipper [4] and they developed the so-called Unloading Point Method (UPM). Without attempting to detail the method here, it is interesting to point out that in this method the rate effects are the part that should be subtracted to the measured static load-displacement curve to obtain the static equivalent (if pore pressures could be neglected). It shows how remarkable the role of the loading rate can be. Besides, damping is found to be significant in pseudostatic testing, it can be directly responsible for a pile capacity increase of up to a 30 per cent.

Other previous research stated that the bearing capacity of a pile (ultimate failure load) increases with increasing rate of loading. Al-Mhaidib [6] presented a relationship for sand as follows:

$$Q_u = C^* (LR)^n$$  \hspace{1cm} (1.3)

where $Q_u$ is the pile capacity corresponding to a loading rate $LR$, $C$ is a constant and $n$ is an exponent function of the sand density and the depth-to-diameter ratio. Especially the sand density affects the relationship between loading rate and bearing capacity, when compared to the depth-to-diameter ratio, that has much slighter effects.

Another proposal to model the rate effect is the one of Briaud and Garland [7]:

$$\frac{Q_{DP}}{Q_{SP}} = (\frac{t_D}{t_S})^n$$  \hspace{1cm} (1.4)

where $Q_{DP}$ is the dynamic capacity, $Q_{SP}$ the static one, $t_D$ and $t_S$ are respectively the loading times for dynamic and static tests, and $n$ is again an exponent function of the soil properties (0.01 for clean sand and 0.1 for soft, plastic clay).

Coyle and Gibson (1970) defined a power law:

$$R_t = R_s t (1 + J^* V^N)$$  \hspace{1cm} (1.5)

where $R_t$ is the static soil resistance, $J^*$ is the rate-effect factor, $V$ the penetration velocity and $N$ an experimental exponent. El Naggar and Novak [8] choose $N = 0.5$ and $J^* = 0.1 - 0.15$ for sand, for the base of the pile only as they assume that the rate-effects are negligible for the pile shaft.

Also the local side friction increases with larger rates of penetration (Te Kamp [9]). This can be relevant as piles derive their strength largely from skin friction and only some 17 per cent is derived from end-bearing (Jones, Bermingham, Horvath [10]).

This is clear for cohesive soils, the dynamic-to-static ratio is directly proportional to the logarithm of the penetration velocity ratio; the proportionality constant was named ‘soil viscosity coefficient’ by Dayal and Allen [11] and it’s inversely proportional to the soil strength. So on, we can affirm that the strength of clay increases significantly under dynamic loading, which could lead to overestimations of soil capacity. On the contrary, parallel research on granular soils showed that the effect of penetration velocity on cone and sleeve resistance is minimal, hence, dynamic effects are minimal for cohesionless soils. In dense sand there is almost no loading effect (Dayal and Allen [11], Eiksund and Nordal [12]). This means that, while pseudostatic may be too optimistic for clay, it should be suitable for sand (Brown [13]).

Related to the loading of the soil it must be noted that we are going to perform tests in the sequence: CPT-Static-Dynamic (Pseudostatic)-Static. This procedure, consisting in reloading cycles, can introduce some residual stresses that don’t affect the load of the pile but do affect the initial slope of the load-settlement curve, that gets stiffer, and the load distribution of the pile (Briaud and Tucker [14]). Precisely, the aim of the second
1.3 Previous researches

static test will be to evaluate these reloading effects and see whether they are significant or not.

The results of Dijkstra [5] are of special interest. He designed the test set-up to be used and performed the same series of tests but in dry sand. He found that a velocity increase from $1mm/s$ to $250mm/s$, the difference static-pseudostatic, did not imply any significance increase in bearing capacity, only 4% for the tip and 6% for the shaft resistance. Therefore, no loading rate effect on the pile capacity was found.

1.3.2 Excess pore pressure effects

During cone penetration or while driving a pile, the soil around the cone/pile is subjected to a combined compression and shear stress deformation and excess pore pressures are generated.

$$\bar{u} = \bar{u}_{\text{oct}} + \bar{u}_{\text{shear}}$$

Whether this excess pore pressures are detected by a piezometer on the shoulder of the cone will depend mainly on the permeability of the soil. Hence, Seed and Reese (1957) studied the phenomena and showed that pressures created by pile driving are transferred into the soil largely as an increase in pore water pressure. When undrained soil is loaded, the main part of the hydrostatic stresses is carried by the water because water can be considered nearly incompressible, so small volumetric changes lead to large hydrostatic stresses. A sand can be expected to display a fully undrained behavior for permeability values lower than $10^{-7}m/s$. It is difficult to predict these pore pressures as they are function of the pile driving energy transferred into the soil, the type of pile, the local drainage conditions, the stress history and the soil type and density (Eigenbrod and Issigoni [15]).

Schmertmann (1974) pointed out that excess pore pressures affect the soil resistance values:

- Negative pore pressures⇒increase on the effective stress⇒larger shear strength⇒larger resistance
- Positive pore pressures⇒decrease on the effective stress⇒smaller shear strength⇒lower resistance

Influence of the density

The measured excess pore pressures will be positive or negative depending on the result of the combination of the two determining processes, namely, compression and shear deformation. Compression induced excess pore pressures are always positive, but shear-induces excess pore pressures can be either positive or negative. Volumetrical response to shear deformation is principally governed by soil density; density of the soil in relation to critical density can indicate whether the soil is contractant or not.

1 Compression-induced excess pore pressures: Always positive

2 Shear-induced excess pore pressures:
   - Dense soil⇒Dilatant behavior: there is an increase of volume, and so the void ratio, giving negative values of the excess pore pressure.
   - Loose soil⇒Contracting behavior: there is a decrease in volume, and so the void ratio, giving positive values of the excess pore pressure.

So, in general, a penetration process in sand, that can often display a dilatant behavior, generates excess pore pressures as follows:

1 Soil is compressed by the cone⇒Positive excess pore pressure generation in the displaced material
2 Soil around the shaft is subjected to local shear deformation ⇒ Negative excess pore pressure until critical state is reached

However, it is also possible that in the case of the dynamic test, with rapid loading (high penetration rate), the shear stresses may be applied very quickly, not giving time for the volume to expand (Te Kamp [9]). This postulate reinforce the influence of loading rate into pore pressure generation.

Influence of the loading rate

However, there is another role factor in the generation of excess pore pressures, the loading rate. Thus, the two governing parameters on the generation of excess pore pressures in sand are:

1 Relative density
2 Loading rate

And these two factors are linked. Canou et al. especially constructed a minipiezoecone of 1cm$^2$ cross-section in a 180mm diameter triaxial cell at penetration rates between 0.1 and 100mm/s and found out that excess pore pressure in loose sand depends on relative density as well as on the penetration rate.

Firstly note that increasing penetration speeds (or loading rates) will shorten the dissipation time between the generation of the excess pore pressures and the observation by the piezometer, consequently the possibility of observing the excess pore pressures in contractant soil before they dissipate will be larger.

Broere [16] performed high-speed CPT tests in the same calibration chamber at Geotechniek as this research. His results show that although larger excess pore pressure occur and are measured at higher than regular CPT penetration speed, there is no correlation with relative density other than that at higher densities greater negative excess pore pressures occur. Therefore, large negative excess pore pressures can indicate high relative density. On the contrary, large positive excess pore pressures do not provide any information about the sand density as positive excess pore pressures manifest themselves similarly for all density range.

1.4 Outline of the report

The report is structured in accordance with the research fields. After the introduction and the problem definition, the core of the research is structured in 3 parts.

Part 1 deals with the experimental testing. First, the test set-up and regime are described in chapter 3; the results are presented and evaluated in chapter 4.

Part 2 focuses in the analytical modeling. Based on the conclusions of the literature research, the adapted model is presented in chapter 5 and the calculations can be found in chapter 6. Once more, the last chapter of the part, number 7, is reserved to the discussion of the results.

The last part is part 3 and concerns the numerical modeling with PLAXIS. A theoretical review of the Hardening Soil model as well as the reasons for its preference is the subject of chapter 8. Chapter 9 details the mode definition, input, geometry, calculations. Chapter 10 shows some significant results and these are contrasted with those of analytical modeling and experimental testing.
After part 3, chapter 11 consists in a wide perspective and general evaluation of the results of the full research (the three parts) as well as some proposals and recommendations for the applicability of the method and prospective research.