

DETERMINISTIC AND PROBABILISTIC ANALYSES OF THE BEARING CAPACITY OF SCREW CAST IN SITU DISPLACEMENT PILES IN SILTY SOILS AS MEASURED BY CPT AND SDT

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Abstract. The bearing capacity of screw cast in situ displacement piles is mostly unexplored. There is also insufficient research on piles in silty soils. Therefore, five cone penetration tests (CPT) and one piezocone penetration test (CPTu) using direct methods were utilised to determine the load-bearing capacity of four displacement piles in Estonia. In addition to the CPT sounding data, static-dynamic test (SDT) results were used to analyse the load-bearing capacity of the piles. Both deterministic and probabilistic methods were used in the analysis. Characteristic values as a 95% reliable mean and 5% fractile values for sounding parameters, according to the Eurocode 7, were included. Additionally, Monte Carlo simulation was included in the reliability-based design (RBD). The bearing

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capacities of screw cast in situ displacement piles in silty soils, here based on the CPT and SDT sounding data, were similar. The adaptation of SDT results for the CPT direct methods for pile load-bearing capacity analysis certainly deserves attention and further investigation. For both sounding types, the Eurocode 7 method provided the best results for all piles. The results of pile-bearing capacities in the absolute difference varied within $\pm 11\%$ between the average, RBD and characteristic values.

Keywords: bearing capacity of pile, cone penetration test (CPT), Monte Carlo simulation, pile, static-dynamic probing test (SDT), static load test.

Introduction

In the Tallinn area, geological conditions vary substantially. There are areas where, on average, limestone hard stratum is at a depth of 1 m. On the other hand, there are areas where the hard stratum is cut by a complex system of several ancient valleys that are buried by dense fluvioglacial sands and soft limnoglacial and marine clayey sediments, which are often tens of meters thick (Arbeiter, 1962; Map applications of the Estonian Land Board, 2020). The use of piles under buildings in these conditions is often required. Bored piles and displacement piles are the most frequently used pile types in Estonia (Mets & Leppik, 2016). As a result of the diversity of soil types and pile installation methods, it is generally complicated to anticipate the vertical bearing capacity of the pile. The static axial compression resistance of a single pile (R_c) is calculated as the sum of the pile base resistance (R_b) and shaft resistance (R_s); this is done by implementing the following formula (EN 1997-2:2007, 2007):

$$R_c = R_b + R_s = q_b \cdot A_b + \sum_{i=1}^n q_{s,i} \cdot A_{s,i}. \quad (1)$$

The pile base capacity (R_b) is found by multiplying the unit end bearing or base resistance (q_b) by the pile toe area (A_b). The shaft friction capacity (R_s) is calculated as the sum of the product of the unit shaft friction ($q_{s,i}$) and the outer pile shaft area ($A_{s,i}$) for each soil layer. The static pile load test is the most accurate method for defining pile capacity after installation, which results in a load–settlement relation. Because of its high cost, the static loading test is often not used in the early phases of construction planning or in small piling sites.

Soil properties are often defined from the in situ sounding resistance. Simultaneously, pile-bearing capacity calculation methods based on the results of in situ tests, which are informative and useful method, are applied more regularly nowadays (Eslami & Fellenius, 1997; Cai et al., 2012; Moshfeghi & Eslami, 2016). The cone penetrometer test (CPT) is

one of the most generally implemented methods for pile-bearing capacity analysis (Niazi & Mayne, 2013). In addition, there are methods that utilise all readings of piezocone test (CPTu) (Eslami & Fellenius, 1997).

In Estonia, the most popular sounding type is the dynamic probing–super heavy test (DPSH-A). Some investigation companies have also actively adopted the static-dynamic probing test (SDT). Various variants of this method are widely used in the Nordic countries. In fine-grained soils or below groundwater level, DPSH-A may lead to erroneous results (Gadeikis et al., 2010; Žaržojus, 2010). CPT has been used in Estonia in a small number of investigations. CPT provides continuous, repeatable and reliable data. However, in some cases, anchoring or a larger reaction mass is needed to reach deeper layers when using CPT. Comparatively, the SDT method pushes the probe until upper anchoring resistance is reached. After that, the denser layers penetrate with dynamic blows. If there is a weaker layer under the denser layer and the anchoring is sufficient, probing can be continued by pushing the probe. This could be a good alternative in fine-grained soils, in which the CPT method cannot penetrate to the required depth. Passing through deep soils is essential for calculating the base bearing capacity of piles.

Because the cone penetrometer can be considered a mini-pile foundation (Bandini & Salgado, 1998; Mayne, 2007; Jardine et al., 2013), this has led to the evolution of a significant number of CPT-based pile-bearing capacity calculation methods (e.g., Nottingham, 1975; Schmertmann, 1978; de Kuitert & Beringen, 1979; Bustamante & Gianeselli, 1982; Eslami & Fellenius, 1997; Kempfert & Becker, 2010). In the present research, direct methods were used to find the load-bearing capacity of piles. As the SDT method does not have direct methods for defining the load-bearing capacity of a pile, CPT-based direct methods have been adjusted for SDT by converting sounding resistance to cone tip resistance (q_c).

The current study focuses on finding the load-bearing capacity of four piles from the CPT and SDT results in silty soils for screw cast in situ displacement piles from the Soodi site in Tallinn. One CPTu-based and five CPT-based calculation methods were used and analysed. The results were compared with a static pile load test with French criterion $s/B = 10\%$. In the criterion, s denotes the settlement of the pile head, while B denotes the diameter of the pile tip.

As the reliability and economic requirements of the design become increasingly important and related to the new generation of design codes around the world, the reliability-based design (RBD) method was also used in the analysis of the load-bearing capacity of the piles based on the LCPC method. Monte Carlo simulation (MCS) based on 10 000 simulations was implemented for this purpose. Based on the LCPC

method, 95% and 5% fractiles of the distribution of soil characteristic values were included.

Niazi (2014) has presented four alternatives for interpreting pile axial capacity based on in-situ geotechnical investigations (Figure 1). In the paper, three of the four possibilities were used in the analysis: correlation (empirical methods), statistics (analytical methods) and full-scale load test (experimental tests). Numerical methods should be included in the subsequent research.

1. Use of sounding methods and data for the prediction of pile-bearing capacity

1.1. CPT, CPTu and SDT soundings

The CPT is one of the most common probing methods to be widely used, studied and developed around the world for the past hundred years (Massarsch, 2014). The method is fast and economical. The probe has a 1000 mm² base area and 15 000 mm² sleeve surface area. The drive rod has the same diameter as the probe (35.7 mm). During the test, the probe is pushed at a constant speed of 20 mm/s to the required depth or until the compressive force runs out. The readings of the cone tip resistance (q_c) and sleeve friction (f_s) with short depth intervals (from 10 to 20 mm) create a nearly continuous representation of the soil layers. Additionally, pore pressure (u_2) data are collected if the CPTu is employed. These three independent parameters allow us to determine the properties of the soil, including its strength and compressibility (Massarsch, 2014).

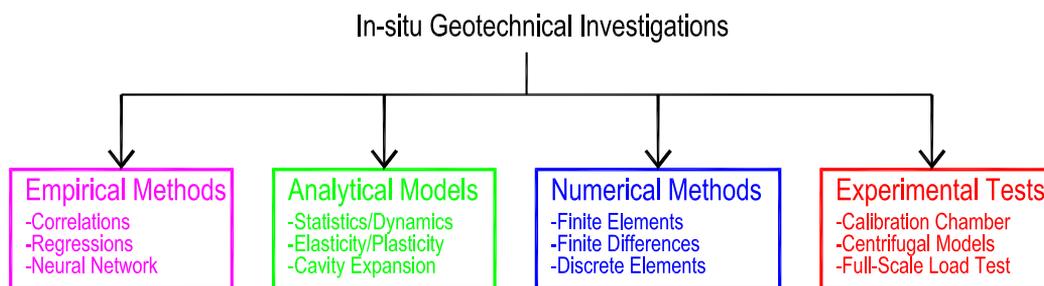


Figure 1. Alternatives to interpret axial pile response from in-situ geotechnical investigations (Niazi, 2014)

The SDT method was developed in Finland in the early 1980s. Especially in the Department of Geotechnics of the City of Helsinki, the method has been studied more intensely (Melander, 1989; Rantala & Halkola, 1997) and used for years. This method combines static and dynamic penetration tests. The test started as a static penetration test in which the drill rods with the cone were pressed and rotated simultaneously. The equipment usually has a maximum compressive force of 30 kN. When the maximum compressive force is reached, the device switches to the dynamic penetration phase (hammering). The dynamic phase switches to static penetration again if the amount of the blows (N_{20}) value is less than or equal to five within 0.4 m. During the test, compressive force, torque, number of strokes, sounding depth and speed of rotation are measured in the intervals of 20 mm to 40 mm (Finnish Geotechnical Society, 2001).

The SDT method uses a loose cone, which remains almost always on the ground when the rods are pulled out. The cone must be 45 ± 0.2 mm in diameter and 90 ± 2 mm in length. The apex angle is 90° . The cross-sectional area of the cone end is 1600 mm^2 , and the area of the side surface is 12700 mm^2 . The diameter of the drive rod is 32 mm, which is smaller than the diameter of the cone (45 mm). During the compression stage, the rods are compressed at a constant speed of 20 ± 5 mm/s. A hammer weighing 63.5 ± 0.5 kg and a lowering height of 0.5 m are used for dynamic penetration (Finnish Geotechnical Society, 2001).

Determining the geotechnical parameters from the results of the SDT method is based on calculation formulas developed for CPTu sounding. Unlike the CPTu, the diameter of the SDT cone is larger than the driving rod (Figure 2). Accordingly, the relationship between the SDT and CPT test results must be known. Based on laboratory experiments, Rantala & Halkola (1997) have determined that the cone tip resistance ($q_{c,CPT}$) of the CPTu can be found from SDT results of static pressure penetration using Equation (2). According to Sounding guidelines 6-2001 (Finnish Geotechnical Society, 2001), the net resistance of the static pressure penetration of the SDT test can be calculated based on the total torque (M_{tot}) and total compressive force (Q_{tot}) values using Equation (3). Based on the results of the dynamic penetration of the SDT test, Equation (4) can be utilised to convert the blow numbers to cone tip resistance ($q_{c,CPT}$) of the CPTu. The net stroke rate N_n is defined from Equation (5) with the help of the total stroke rate (N_{20}) and total torque (M_{tot}) (Finnish Geotechnical Society, 2001).

$$q_{c,CPT} = 1.07 \cdot q_{n,SDT} \quad (2)$$

$$q_{n,SDT} = \frac{Q_{tot}}{1000 \cdot A_c} - k_p \cdot (M_{tot} - \mu_1 \cdot Q_{tot}), \quad (3)$$

$$q_{c,CPT} = 0.83 \left[\frac{MPa}{l} \right] \cdot N_n, \quad (4)$$

$$N_n = N_{20} - 0.04M_{tot}, \quad (5)$$

where

$q_{c,CPT}$ is cone tip resistance of CPT;

$q_{n,SDT}$ is the net resistance to static pressure penetration, MPa, of SDT;

Q_{tot} is the total compressive force, kN, of SDT;

k_p is a standard ($k_p = 1/(A_c \cdot r \cdot 10^6) = 0.039$ (1/m³));

M_{tot} is the total torque value, Nm, of SDT;

μ_1 is a device-specific constant (e.g., for GM4000 $\mu_1 = 1$ Nm/kN) to estimate the effect of axial loading of the compression phase on the friction of the transmission thrust bearing;

N_n is the net stroke rate [l/0.2 m] of SDT;

N_{20} is the total stroke rate [l/0.2 m] of the SDT.

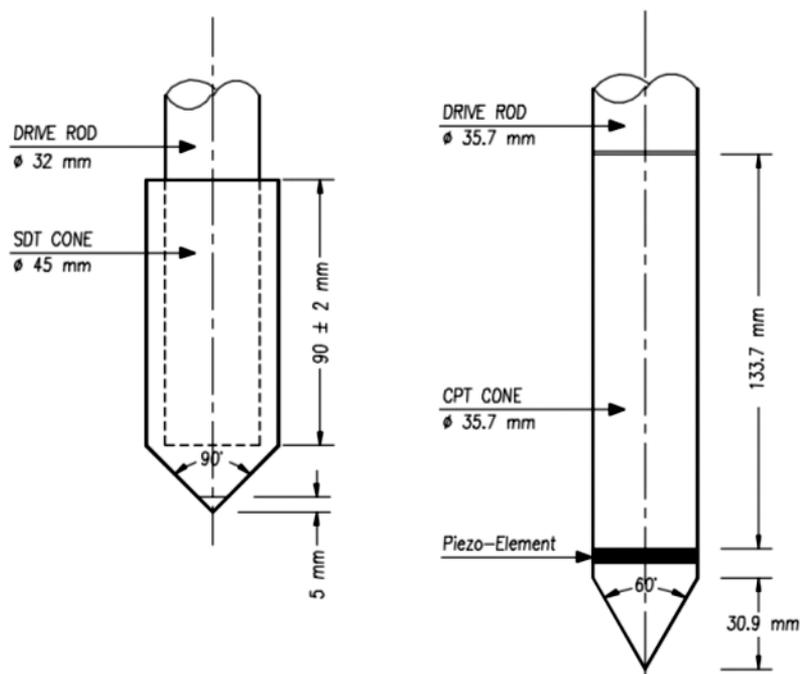


Figure 2. SDT penetrometer cone on the left and CPTu cone on the right.

1.2. Direct approaches for CPT and CPTu soundings to define pile capacity

The mean effective stress, compressibility and rigidity of the surrounding soil medium have an effect on the CPT cone and pile in a comparable manner (Eslami & Fellenius, 1997; Ardalan et al., 2009). Direct cone penetration methods for the CPT apply cone sleeve friction for unit shaft resistance and cone bearing for the unit end-bearing resistance of the pile, here by the analogy of the cone penetrometer as a model pile (Mayne, 2007). This concept has led to the development of many direct CPT methods around the world, whereby CPT readings are simply scaled up and used to evaluate the load-bearing capacity of full-scale piles (Niazi & Mayne, 2013). More than around 30 different CPT- and CPTu-based direct methods have been developed (Niazi & Mayne, 2013). Six direct methods were applied in the present study: five CPT methods and the Unicone method, which is based on CPTu results. The methods are the Nottingham (1975) and Schmertmann (1978) method, de Kuiter & Beringen (1979) method (Dutch method), LCPC method (Bustamante & Gianeselli, 1982; Bustamante & Frank, 1997), Eurocode 7 (EN 1997-2:2007, 2007) method and German method (EA-Pfähle, 2014). The Unicone (Eslami & Fellenius 1995, 1996, 1997; Fellenius & Eslami, 2000; Eslami, 1996; Fellenius, 2020) method is certainly a remarkable method because it is the first method to use all three readings of the CPTu sounding (q_v , f_s and u_2) in the pile load-bearing capacity analysis. In addition, the Unicone method developed a new soil profiling chart. The methods were chosen based on the fact that most of them were suitable for all soil types and for a wide range of piles. The only exception is the German method (EA-Pfähle, 2014), which is suitable for sandy soils. Concurrent, the German (EA-Pfähle, 2014) method offered good results in similar soils for screw cast in situ displacement piles (Leetsaar et al., 2022). The current study applies these methods to piles installed in silty soils. A summary of the methods used is presented in Table 1. Based on the SDT data, three of the six methods were used to analyse the load-bearing capacity of the piles. The Nottingham (1975) and Schmertmann (1978) method, together with the de Kuiter & Beringen method (1979), utilises the value of f_s to determine the load-bearing capacity of the pile. In addition to reading f_s , the Unicone (Eslami & Fellenius, 1997) method also exploits u_2 readings. SDT sounding does not record either of these readings.

Table 1. Summary of direct CPT-based pile design methods

Method/reference	Design equations	
	Pile unit shaft friction (q_s)	Pile end bearing resistance (q_b)
Nottingham (1975) and Schmertmann (1978) (for driven concrete, steel and timber piles, and drilled shafts in all soil types)	<p>In clay: $q_s = K_f \cdot f_s \leq 120$ kPa, $K_f = 0.2-1.25$ K_f is a function of the sleeve resistance</p> <p>In sand: $q_s = c_s \cdot q_c$ or $f_p = k \cdot f_s$ $c_s = 0.8-1.8\%$, $k = 0.8-2.5$</p>	<p>$q_b = C \cdot q_{ca} \leq 15$ MPa (in sands) and 10 MPa (in very silty sands) $C = 0.5-1.0$ depending on overconsolidation rate (OCR) $q_{ca} = (q_{c1} + q_{c2})/2$</p>
Dutch method (de Kuitert & Beringen 1979) (for offshore piles in all soil types)	<p>In clay: $q_s = \alpha \cdot s_u \leq 120$ kPa; $\alpha = 1$ for NC clay and 0.5 for OC clay; $s_u = q_{ca}/N_{kt}$; $N_{kt} = 15-20$</p> <p>In sand: $q_s = \min[f_s, q_c/300]$ for compression, $q_c/400$ for tension, 120 kPa]</p>	<p>In clay: $q_b = N_c \cdot s_u \leq 15$ MPa, $s_u = q_{ca}/N_{kt}$, $N_c = 9$; $N_{kt} = 15-20$; $q_{ca} = (q_{c1} + q_{c2})/2$</p> <p>In sand: similar to Nottingham (1975) and Schmertmann (1978) method</p>
LCPC or French method (Bustamante & Ganeselli, 1982; Bustamante & Frank, 1997) (for all pile types in all soil types)	<p>$q_s = q_{side}/k_s \leq f_{p(max)}$ $k_s = 30-150$ depending on soil type, pile type and installation procedure</p>	<p>$q_b = k_b \cdot q_{eq}$ depending on soil types: $k_b = 0.15-0.375$ for non-displacement piles $k_b = 0.375-0.60$ for displacement piles</p>
EUROCODE 7 (EN 1997-2:2007, 2007) (for all pile types in all soil types)	<p>$q_s = \alpha_s \cdot q_{c,z}$ $\alpha_s = 0.005 - 0.030$ depending on soil type or pile type and installation procedure</p>	<p>$q_b = 0.5 \cdot \alpha_p \cdot \beta \cdot s$</p> $\left\{ \frac{q_{c,I,mean} + q_{c,II,mean}}{2} + q_{c,III,mean} \right\}$ <p>$q_{b,max} \leq 15$ MPa; $\alpha_p = 0.6-1.0$ depending on soil type, pile type and installation procedure; β factor that takes into account the shape of the pile tip; s factor that takes into account the shape of the bottom of the pile</p>
German method (EA-Pfähle, 2014) (for piles in sandy soils)	<p>Provides upper and lower bound estimates of q_s, kPa, based on q_c (measured in MPa)</p>	<p>Provides upper and lower bound estimates of q_b, MPa, based on q_c (measured in MPa)</p>
Unicone method (Eslami & Fellenius, 1995, 1996, 1997; Fellenius & Eslami, 2000; Eslami, 1996; Fellenius, 2020) (all piles in all soils)	<p>$q_s = C_{se} \cdot q_E$ $q_E = q_t - u_2$ $C_{se} = 0.8-8\%$</p>	<p>$q_b = C_{te} \cdot q_{EG}$; q_{EG} is the geometric average of q_c C_{te} is generally taken as 1; for pile diameter $d > 0.4$ m $C_{te} = 1/(3d)$ $q_b = C_{te} \cdot q_{EG}$; q_{EG} is the geometric average of q_c</p>

1.3. Statistical determination of sounding data

In the current study, the characteristic value (here as a 95% reliable mean) and 5% fractile values for the sounding parameters were included in analysis of the direct methods in the pile capacity calculations. The aim is to determine how the characteristic values of soil properties based on EC7 change the results of these direct methods. For homogeneous soil without a significant trend in the ground, the characteristic value X_k as a 95% reliable mean value of the parameter can be determined from a set of individual values according to Frank et al. (2005):

$$X_k = X_{\text{mean}} (1 - k_n V_x), \quad (6)$$

where

X_{mean} is the arithmetical mean value of the individual sample parameter value;

V_x is the coefficient of variation of the parameter X ;

k_n is a statistical coefficient.

$$V_x = \frac{s_x}{X_{\text{mean}}}, \quad (7)$$

where s_x is the standard deviation of the n sample test results.

The value of the coefficient $k_{n,\text{mean}}$ for the assessment of a characteristic value as a 95% reliable mean value equation is as follows:

$$k_n = 1.645 \sqrt{\frac{1}{n}}. \quad (8)$$

The value of the coefficient $k_{n,\text{low}}$ for the assessment of a characteristic value as a 5% fractile value equation is as follows:

$$k_n = 1.645 \sqrt{\frac{1}{n} + 1}, \quad (9)$$

where n is the number of test results of the soundings.

1.4. Reliability-based design (RBD)

Direct methods usually assume that the pile is located in soil layers with homogeneous properties. Odd 'peaks and troughs' in the sounding data are reduced when using mean values (Eslami et al., 1997). In cases where in situ soil variability is considerable, deterministic analysis based on the mean values could be inefficient. One possible solution is to use statistical distributions of soil properties and implement them in a deterministic analysis with simulations.

One of the most commonly used techniques of reliability analysis is Monte Carlo Simulation (MCS), which is a repetitive simulation process that generates a set of values based on random variables with the known probability distribution. The increase in the number of simulations increases the accuracy of the MCS outcome. However, a very large number of simulations make the analysis slow and has little effect on the results. Typically $N=10^4$ or 10^5 number of simulation is chosen (Orr & Denys, 2008). The probability distribution (i.e., beta, normal, lognormal, etc.) for each independent variable is provided. The outcome can be presented in a histogram or an average value can be highlighted. In addition, the RBD determines the probability of failure or reliability index.

In the present study, the soil was divided into four layers. Three soil layers were analysed around the piles. The fourth layer was formed based on the influence zone for the pile according to the LCPC method. For the LCPC method the influence zone is 1.5 D below and above the pile base. The variable of the soil layers was the q_c value. The layers were assigned a mean value and standard deviation. Three of the four layers used a normal distribution for the variable. For the layer dominated by clayey soils, a lognormal distribution was used. Using software RiskAMP and MCS, 10 000 pile capacity values for each pile were generated. The average value of the results for each pile was used in the analysis.

A choice was made between those methods that allow the load-bearing capacity of the pile to be determined from both CPTu and SDT sounding data. The German method is based on tabulated values; therefore, it cannot be used in MC simulations. The Eurocode method was also excluded because it is too complicated, needing three different q_c values for calculating the bearing capacity of the pile base. Only the LCPC method was used for the RBD simulations.

2. Test site and tested pile types

The test site was Soodi, which is located in Tallinn (see Figure 3), northern Estonia. It lies above an old valley buried in Quaternary sediments. Marine, lacustrine and alluvial deposits consist of varying layers of clay, silty clay, sand and silty sand. To a depth of 4.1 m, there are mainly alternating silty and sandy silt layers. At a depth of 11.7–11.9 m, the sand and silty sand deposits appear again. Between the soft clayey and silty soil, the layers alternate. The hard stratum of gravel/moraine is found at a depth of almost 30 m. The ground water table varies between 0.05 and 0.65 m below the ground surface. A map of the sounding points and tested piles is shown in Figure 4. The SDT soundings SLP9 and SLP10 were performed before the erection of the test piles. The

CPTu soundings were assembled after the construction of the building and, therefore, are located more than 46 m from the test piles. A non-normalised CPT soil behaviour type (SBT) chart (Robertson, 2010) was used for soil classification. Internationally, the SBT chart is a global and favourable basis for comparing soils and test results.

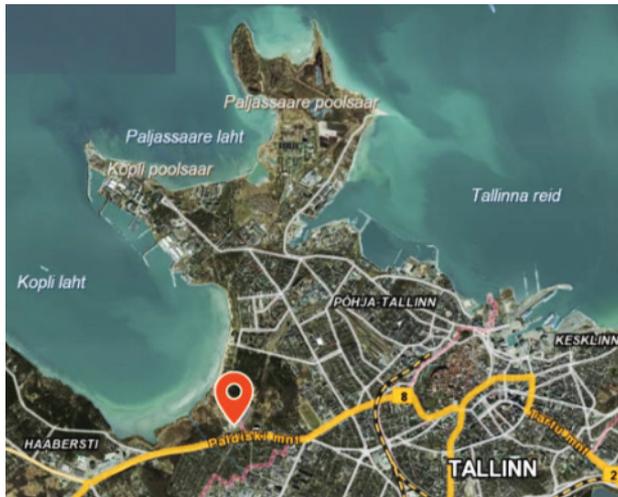


Figure 3. On the left, the location of the research point in Tallinn (Map applications of the Estonian Land Board, 2020). The red mark indicates the location of Soodi Street 4

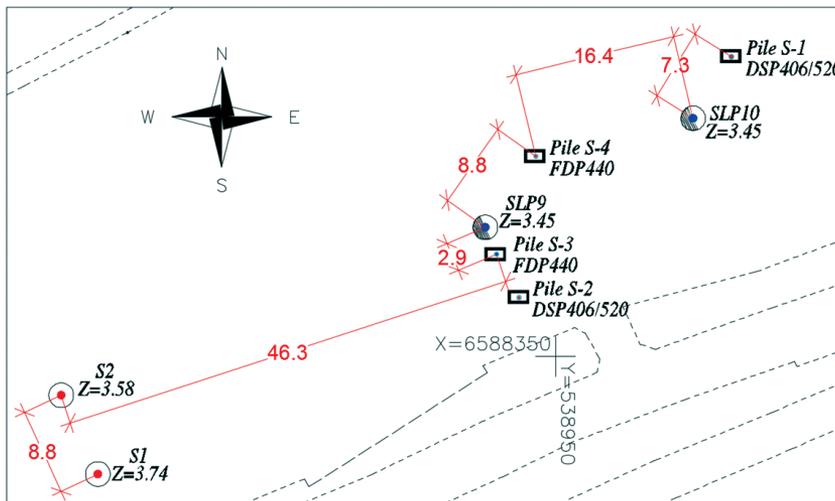


Figure 4. Site map with the tested piles and sounding points. S1 and S2 indicate CPTu soundings. SLP9 and SLP10 indicate SDT soundings. The pile symbols are S-1 to S-4, and the type of pile is shown next to the name of the tested piles. The dimensions given in the map are in metres

The two tested pile types were the Bauer full displacement pile (FDP) and displacement pile (DSP). During FDP pile installation (Figure 5 on the left), a displacement tool with a widening shape is drilled into the ground by pushing and rotating it. The displacement tool includes a starter auger, which first loosens the soil, and then, the widening displacement tool pushes it laterally into the surrounding soil. After reaching the designed depth, the displacement tool is removed and the cavity is simultaneously filled with concrete through an opening at the end of the drill stem. After this, reinforcement casing is pushed into the wet concrete. ‘Lost bit’ technology was used to install DSP piles (Figure 5 on the right). By rotating and pushing, the jacket pipe with a closed end is drilled into the desired depth. The drill head is unscrewed

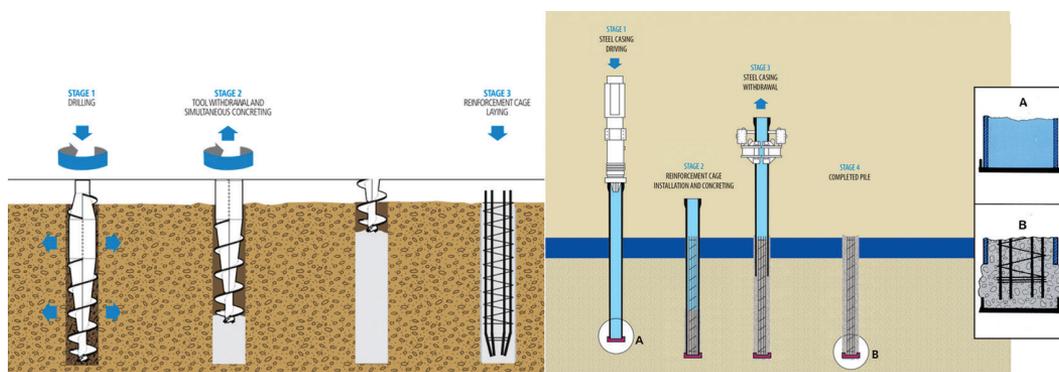


Figure 5. The principle of installing an FDP pile is shown on the left; the principle of installing a DSP pile is shown on the right (<https://www.trevispa.com/en/Technologies>)

Table 2. Summary of pile and sounding data

Pile name	S-1	S-2	S-3	S-4
Pile type	DSP 406/520	DSP 406/520	FDP 440	FDP 440
Pile length, m	12.69	11.34	12.39	12.5
Max load from pile load test, kN	1870	1700	1870	1870
Max settlement from pile load test, mm	35.3	22.8	17.0	22.7
Max depth of CPTu sounding, m	25.18	25.18	25.18	25.18
Max depth of SDT sounding, m	21.49	21.13	21.3	21.3
Max q_c reading from CPTu, MPa	21.7	21.7	21.7	21.7
Max q_c reading from SDT, MPa	11.1	14.5	14.5	14.5
Max N_{20} reading from SDT	15	26	26	26

and left in the ground as a pile toe. While lifting the jacket pipe up, the pile cavity is filled with concrete. Both pile types can be classified as cast in situ concrete displacement piles. At the test site, the diameter of the FDP pile head and shaft was 0.44 m. The shaft diameter of the DSP pile was 0.406 m, and the pile tip was 0.52 m. The length of the piles varied between 11.34 and 12.69 m. Piles of this type are classified in Europe as 'screw piles' (Basu et al., 2010). Table 2 shows the lengths of the statically tested piles with the maximum testing loads and respective settlements. In addition, the maximum resistance values of the CPTu and SDT soundings are given.

The German method contains tabular values for driven precast piles, simplex piles, Atlas piles, Fundex piles and bored piles in sandy and clayey soils (EA-Pfähle, 2014). The DSP pile most closely resembles the Fundex pile. Both pile types have a pile tip with overlap, i.e., the diameter of the pile tip is bigger than the diameter of the pile shaft. The pile head remains in the ground after the pile installation on both types of piles. Driven precast piles and simplex piles are both driven piles. When installing the Atlas pile, the screw-shape shaft remains. Bored piles are not displacement piles. Thus, the FDP pile is also calculated on the basis of the Fundex pile tables using the German method.

3. Pile tests and soundings

3.1. Static axial pile load tests

The piles were tested in accordance with EVS-EN 1997-1:2006 (based on EN 1997-1:2004) before the other parts of the pile field were constructed. The largest load on both types of test piles was 1870 kN. This is 83.7–98.8% from the ultimate capacity. The pile head settlement was between 17.0 mm and 35.3 mm. The piles were tested two to three weeks after installation.

According to Hirany & Kulhawy (1989), there are different methods to determine the ultimate capacity of a pile from the results of a static load test. One of the oldest definitions of pile-bearing capacity is the load at which the pile movement exceeds 10% of the diameter of the pile. This principle is also known as the French criterion (Vesić, 1977) and is used in Eurocode.

As the piles were loaded into the settlement equal to 10% of its nominal diameter, Chin's (1970) extrapolation method and load-settlement curve were used to determine the ultimate pile capacity. The extrapolation results for all four piles are summarised in Figure 6. Based on Kondner's (1963) work, Chin's extrapolation method is familiar and

extensively used in practice (Al-Homoud et al., 2003; Basu et al., 2010; Elsamee, 2012; Niazi, 2014; Camacho et al., 2018). In agreement with the 10% criteria, this method allows the ultimate resistance to be defined, even if the pile head settlement does not reach 10% of the pile diameter (Holeyman et al., 1997; Borel et al., 2004; De Cock, 2009; Basu et al., 2010).

3.2. CPTu tests

Two CPTu soundings were made in 2019. CPTu soundings were performed with a Nova cone manufactured by Geotech AB and mounted on a lightweight truck. The Nova cone has a 1000 mm² project area and 15 000 mm² sleeve surface area, according to ASTM D-5778 (2000) and EN ISO22476-1 standards. The covering fill layer was penetrated by predrilling. The lightweight truck was anchored with two ground anchors to achieve a higher compression force. The tests were performed according to Lunne et al. (1997) guidelines.

The distance between CPTu sounding points and test piles varied from 44.9 to 75.3 m. The sounding profiles of the corrected cone resistance (q_c), unit sleeve friction resistance (f_s), friction ratio (R_f) and pore pressure measured behind the cone (u_2) are shown in Figure 7. Sounding S1, which reached a depth of 25.18 m, is marked in blue. Sounding S2 reached a depth of 20.42 m and is marked in red.

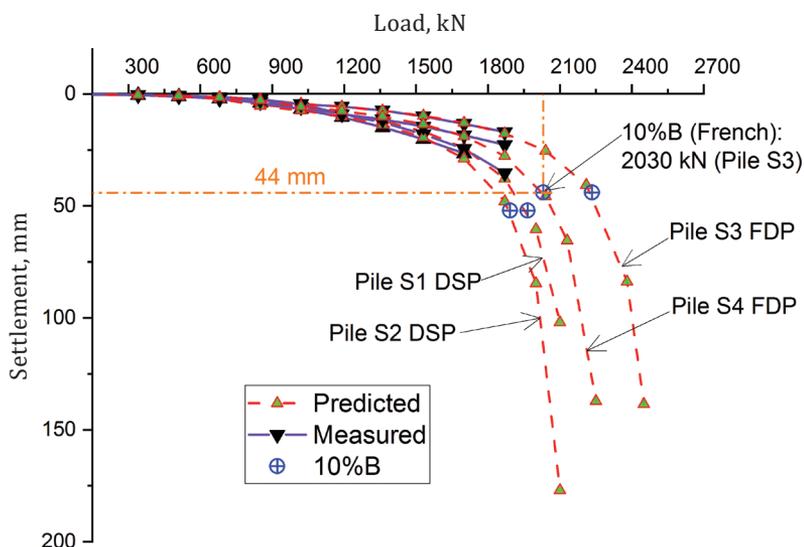


Figure 6. Load-displacement curve of the pile load test and the extrapolation results for the four piles

R_f is defined as $f_s/q_t \times 100\%$. The types of soil layers defined by SBT (Robertson, 2010) are shown on the left. Different mixtures of silt predominate in soils. The pore water pressure image also includes the water table and in situ pore pressure (u_0) profile.

3.3. Static-dynamic probing tests

SDT soundings were performed in spring 2015 with the GM 65 GTT unit, here according to Melander's (1989) instructions. A total of 10 soundings were conducted. The results of the two soundings closest to the piles tested were utilised. The distance of the piles from the nearest sounding point varied between 2.9 and 8.8 m (see Figure 4). The depths of the SDT soundings SLP9 and SLP10 were 21.13 and 21.49 m, respectively. The results of the soundings are shown in Figure 8. The figure on the left shows the results of SLP9. The figure clearly demonstrates the alternation of static penetration (q_c -SDT) with dynamic penetration (N_{20} -SDT) of the SDT test. The middle figure shows the SLP9 and SLP10 test results (q_c -SDT + N_{20} -SDT) with the q_c values derived from the SLP9 and SLP10 results (q_c -SDT-CPT) after applying Equations (2) to (5). In the figure on the right, the q_c values derived from the results of SLP9 and SLP10 (q_c -SDT-CPT) are compared with the CPTu tip resistance values of S1 and S2 (q_c -CPTU).

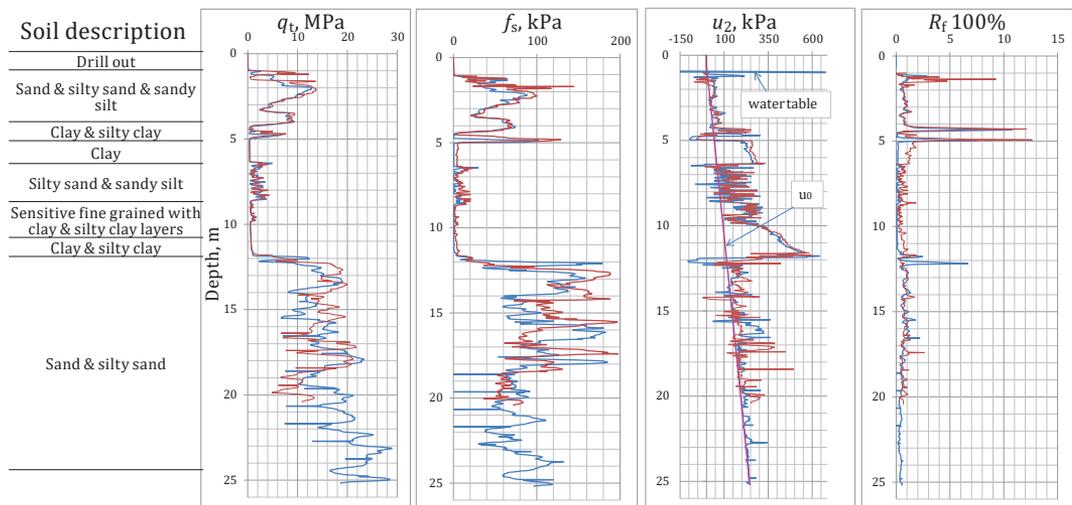


Figure 7. Soil description and the results of CPTu tests S1 (blue figures) and S2 (red figures) at the Soodi site. q_t , cone resistance corrected for the pore pressure effects; f_s , sleeve friction; R_f , friction ratio; u_2 , pore pressure

4. Estimated versus measured ultimate pile capacity

The current study has analysed piles ranging in length from 11.34 to 12.69 m. Settlement of the pile equalling 10% of the pile diameter should be 56 mm for the DSP pile and 44 mm for the FDP pile. The two tested DSP piles had a maximum settlement of 37% and 59% less than the required size of 10% of the pile base diameter. On the two FDP piles, the percentage was 61% and 48%, respectively. The results might be affected by extrapolation, which was done to define the ultimate pile capacity.

A high groundwater level could also affect sounding results and the time of the pile-bearing capacity, especially for fine-grained cohesive soils. The effect of time on the pile load-bearing capacity certainly depends on the type and length of the pile. It is also important to distinguish whether the clayey soil layers are around the pile base or only above it. In conditions where nearly 50% of the pile is surrounded by clayey soils, the load-bearing capacity of the pile may increase for up to 100 days after construction of the pile (Togliani & Reuter, 2014).

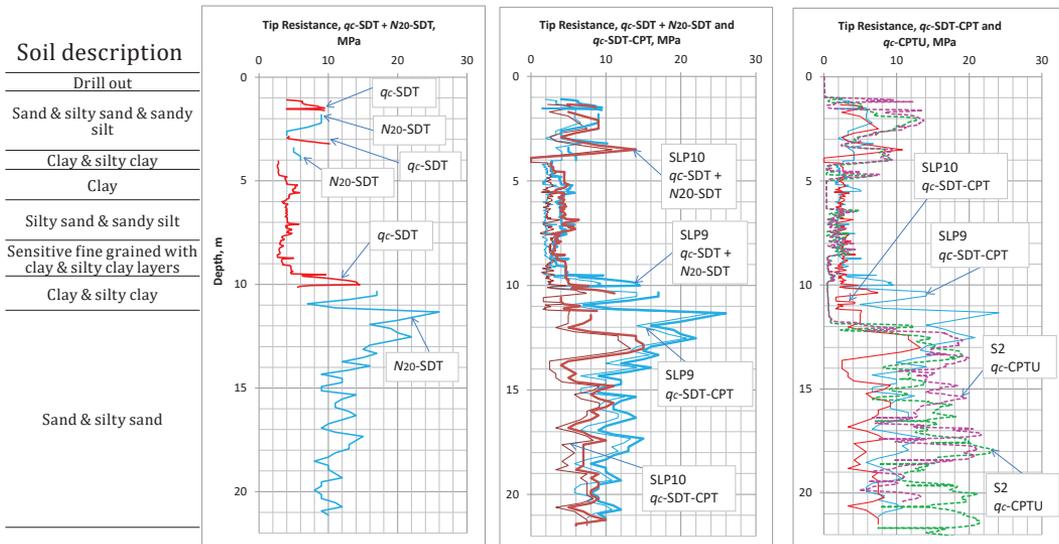


Figure 8. SLP9 sounding results in the figure on the left. The results of SLP9 and SLP10 (q_c -SDT + N_{20} -SDT) converted to the q_c value of CPT (q_c -SDT-CPT) in the middle. The results of SDT test compared with the CPTu test at the Soodi site on the right. q_c -SDT, cone resistance from static readings of SDT; N_{20} -SDT, cone resistance from dynamic readings of SDT; q_c -SDT-CPT, measured and derived q_c values from SDT q_c and N_{20} values; q_c -CPTu, cone resistance from CPT

The piles included in the present study had clayey soils layers above the pile base; thus, generating adhesion on the pile shaft affected the load-bearing capacity of the pile. Therefore, it is important to know how quickly the load tests were performed after installation and how much of the total pile load-bearing capacity was the pile shaft capacity. The pile shaft resistance was analysed based on the average results of *S1* and *S2*, as well as on the *SLP9* and *SLP10* soundings. As an average of the results for the five CPT methods and one CPTu method, pile shaft resistance accounted for 30% of the total bearing capacity for DSP piles and 37% for FDP piles. Using the average results of the two SDT soundings, the average values of the results of the three CPT methods were 41% and 49%, respectively. The higher proportion of the bearing capacity of the base of the DSP piles can be explained by the larger diameter of the pile base compared with the pile shaft. Because the load-bearing capacity of the pile shaft is a significant part of the total load-bearing capacity, it can be assumed that the actual load-bearing capacity of the tested piles was higher than that measured during the static load test.

The five CPT methods and one CPTu method referred to in Section 2.2 were used for analyses. The estimated pile capacities (R_{cp}) from the different sounding data were compared with the measured capacities (R_{cm}). The results for pile S-1 based on *S1* and *SLP10* soundings are shown in Table 3. In Table 3, the ratios of the measured total capacity, R_{cm} , to estimated pile capacity, R_{cp} , are given, along with the absolute percentage difference between the estimated and measured capacities. The minus sign indicates that the calculation method underestimates the load-bearing capacity of the pile. In addition to the total load-bearing capacity (R_c) of the pile, the base capacity (R_b) and shaft friction capacity (R_s) are also shown separately for different calculation methods. The sounding ID indicates which investigation point data were used to calculate pile capacity. Only the soundings made during the research after the installation of the piles carry the abbreviation CPTu in sounding ID.

As the distance between two CPTu soundings was only 8.8 m and the tested piles were more than 46 m away from the sounding points, the results of axial bearing capacity based on two CPTu soundings were first compared. The results of the four piles in bearing capacity absolute differences based on CPTu soundings *S1* and *S2* are summarised in Figure 9. The calculation methods with reference to the type of probing test used are shown on the horizontal axis in Figure 9. The vertical axis shows the absolute percentage difference between the predicted and calculated capacities. The values calculated from soundings CPTU-S1 (*S1*) and CPTU-S2 (*S2*) are presented side by side in pairs. In the figure, $\pm 10\%$ and $\pm 20\%$ areas are indicated by hatching. The more the results

Table 3. Comparison between the static load test results and bearing capacity prediction for the test pile S-1

Method	R_{br} kN	R_{sr} kN	R_{cr} kN	R_{cm}/R_{cp}	Absolute difference, %	Sounding ID
Static Loading Test ($s/B=10\%$)			1965	1.0000	0	
Nottingham (1975) and Schmertmann (1978)	1797	325	2121	0.9264	7	CPTU-S1
de Kuiter and Beringen (1979)	1797	680	2436	0.8067	19	CPTU-S1
LCPC (1982; 1997)	1128	640	1727	1.1378	-14	CPTU-S1
EUROCODE 7 (EN 1997-2:2007, 2007)	1263	763	2025	0.9704	3	CPTU-S1
German method (EA-Pfähle, 2014)	1448	659	2107	0.9326	7	CPTU-S1
Unicone method (1997)	961	663	1582	1.2421	-24	CPTU-S1
LCPC (1982; 1997)	1012	629	1600	1.2281	-23	SLP10
EUROCODE 7 (EN 1997-2:2007, 2007)	1005	916	1880	1.0452	-5	SLP10
German method (EA-Pfähle, 2014)	1058	616	1632	1.2040	-20	SLP10

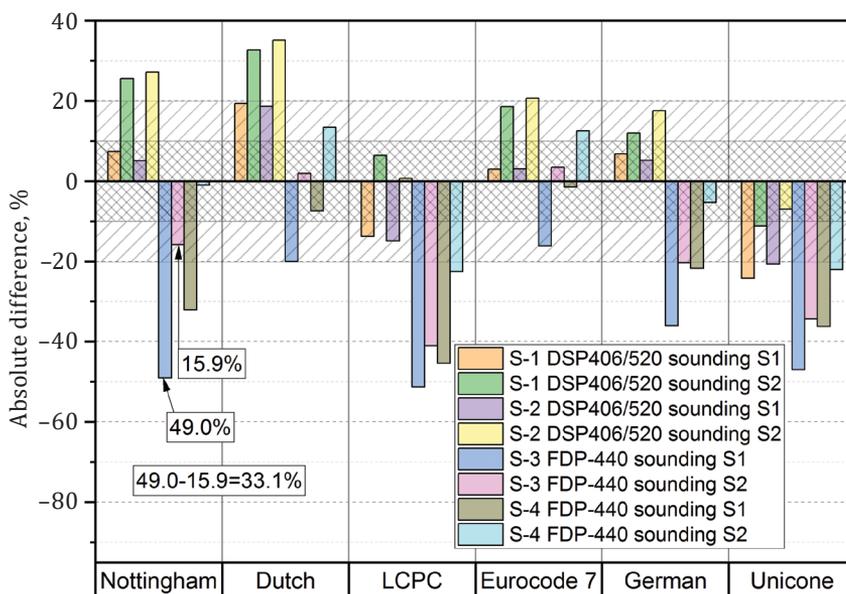


Figure 9. Comparison of the measured and predicted capacity for the piles at the Soodi site based on CPTU-S1 and CPTU-S2; + overestimates, - underestimates

were within $\pm 20\%$ or even $\pm 10\%$, the better the method. Figure 9 shows the difference between the results of soundings *S1* and *S2* as calculated for pile *S-3*, here based on the Nottingham (1975) and Schmertman (1978) method. Differences in the results of all four piles have been calculated using the same principle. The results are shown in Table 4. The average values in the bottom row clearly show that the results differ in the range of 14.2–19.9% based on the results of the two CPT soundings. The smallest difference is 5.2%, and the largest difference is 33.1%. Hereinafter, the average values of *S1* and *S2* were used in load-bearing capacity analysis of the piles. The average values of SLP9 and SLP10 were also used.

Based on the average sounding results, the load-bearing capacity of the four piles found by all the utilised methods is shown in Figure 10. The results were compared with the measured results of four static pile load tests. In addition, the significant results are marked with a yellow circle. In Figure 11, the same results are shown as an absolute percentage difference between the predicted and calculated capacities.

As can be clearly seen in Figures 10 and 11, when comparing the measured and calculated results of all four piles, the best agreement is with the results obtained by the Eurocode 7 (EN 1997-2:2007, 2007) method. In addition, very comparable results were achieved with this method based on both the CPT and SDT sounding data. Most results of the Eurocode 7 (EN 1997-2:2007, 2007) method are within or close to the $\pm 10\%$ range. The largest absolute difference was 13%.

Table 4. Absolute percentage difference in load-bearing capacity of four piles as the distinction between the results based on the CPTU-S1 and CPTU-S2 soundings

Method	S-1 DSP406/520,	S-2 DSP406/520,	S-3 FDP-440,	S-4 FDP-440,
	%	%	%	%
Nottingham (1975) and Schmertmann (1978)	18.2	22.0	33.1	31.0
de Kuiter and Beringen (1979)	13.3	16.5	21.9	20.8
LCPC (1982; 1997)	20.2	15.6	10.2	22.8
EUROCODE 7 (EN 1997-2:2007, 2007)	15.5	17.6	19.6	13.9
German method (EA-Pfähle, 2014)	5.2	12.4	15.6	16.4
Unicone method (1997)	13.1	13.7	12.6	14.2
Average	14.2	16.3	18.9	19.9

Other methods gave significantly different results between the two types of piles. The results of the DSP piles tended to overestimate the measured capacity. In contrast, the calculated capacities of most FDP piles underestimated the measured values. The results of the DSP piles were most overestimated by the Dutch (de Kuiter & Beringen, 1979) method (28%). Nevertheless, all other methods offered results for the DSP pile within the $\pm 20\%$ range. The LCPC method and the Unicone (1997) method underestimated the load-bearing capacity of the FDP piles the most. The absolute differences were 59% and 66%, respectively. Further, the LCPC method and Unicone (1997) method underestimated the load-bearing capacity of all four piles compared to the measured capacity. Figure 11 shows that the LCPC method, together with the Eurocode method and German method, gave very comparable results based on both the CPT and SDT sounding data. The load-bearing capacity of the FDP piles varied within a $\pm 10\%$ range using the Dutch (de Kuiter & Beringen, 1979) method and the Eurocode 7 (EN 1997-2:2007, 2007) method. The variability ranged from -7% to 5% and -5% to 6%, respectively.

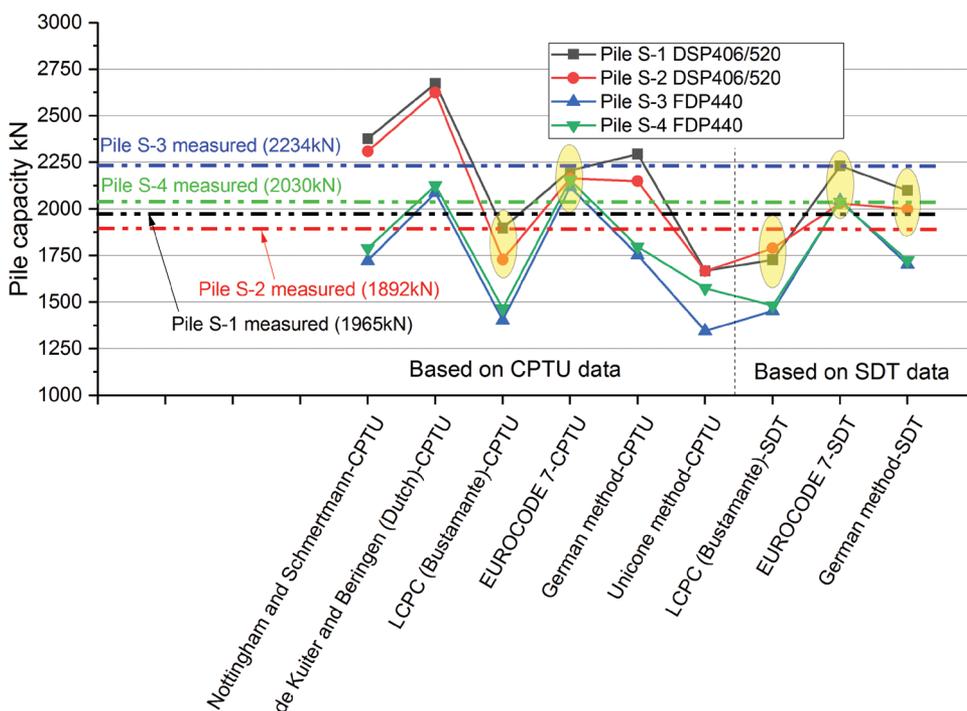


Figure 10. Comparison of the measured and predicted capacity for the piles at the Soodi site based on CPTU-S1 and CPTU-S2 average results

Figure 12 compares the average (average) and MCS results against the 95% reliable estimate of the mean value (characteristic) and 5% fractile (Low 5) results obtained by the LCPC method. The results shown in Figure 12 on the left are based on CPTu soundings S1 and S2. The results in Figure 12 on the right are based on SDT soundings SLP9 and SLP10. The results are presented and compared based on the absolute differences.

Compared with the absolute difference, the MCS and characteristic values based on the S1 and S2 soundings were similar for all four piles.

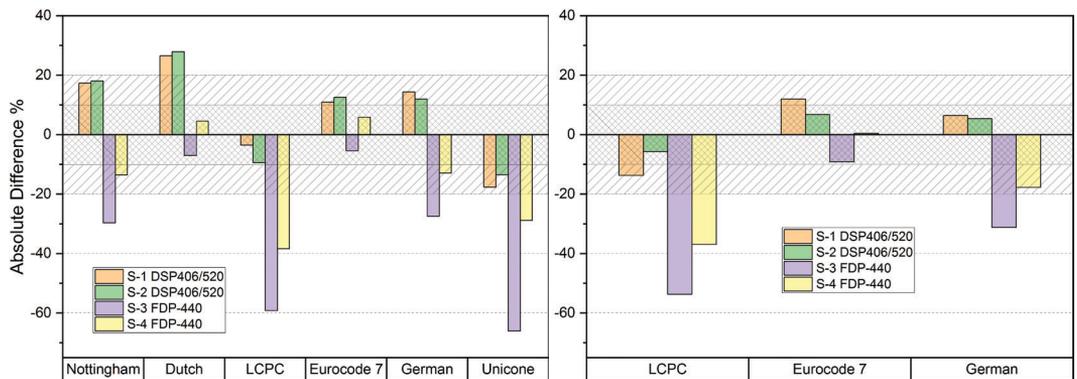


Figure 11. On the left based on a comparison of CPTU-S1 or CPTU-S2 average results between pile types; on the right based on a comparison of SLP9 and SLP10 average results between pile types; + overestimates, - underestimates

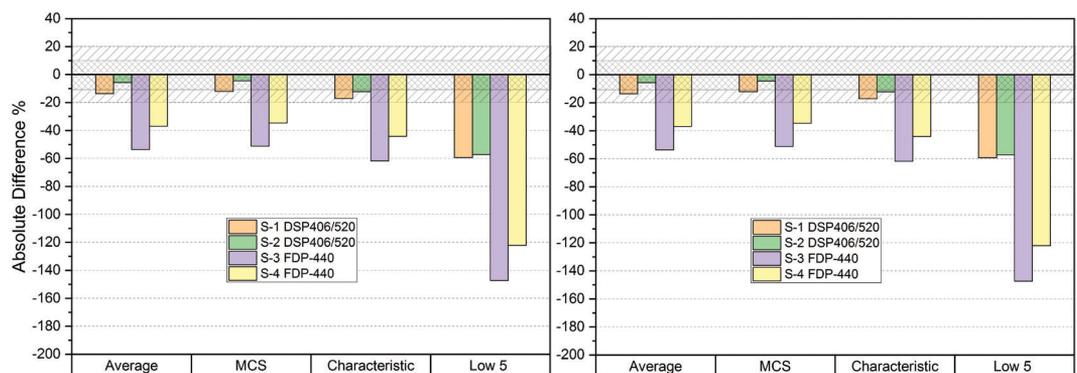


Figure 12. Comparison of average (Average), Monte Carlo simulation (MCS), 95% reliable estimate of the mean value (characteristic) and 5% fractile (Low 5) values based on LCPC method. On the left based on CPTU-S1 or CPTU-S2 result comparison between pile types; on the right based on SLP9 and SLP10 result comparison between pile types; + overestimates, - underestimates

The biggest difference was 3%. The results based on the *S1* and *S2* soundings obtained with the average values are 3–8% lower than the MCS and characteristic values. Low 5 values of the same soundings are 43–132% lower than the RBD and characteristic values. The average values based on the results of SDT soundings SLP9 and SLP10 are similar to the RBD values. The difference is between 1% and 3%. The characteristic values are 5–11% lower than the average and RBD values. Low 5 values are 47–96% lower than the average and MCS values.

5. Discussion

As shown in Figure 11, in four out of the six methods, the direct methods tended to overestimate the bearing capacity of the DSP piles. The load-bearing capacity of the pile was underestimated by the LCPC and Unicone methods only. As the DSP piles have a pile tip with a larger diameter and the installation technology resembles a Fundex pile, they should behave in a similar way. According to Kemfert & Becker (2010), a pile tip with a larger diameter leads to loosening of the ground in the shaft area, resulting in a reduction of shaft resistance. In contrast, the screw-shaped shaft of the pile increases the load-bearing capacity of the pile when compared with a smooth pile shaft (Basu et al., 2010). This may be the reason why pile calculation methods do not always provide the actual load-bearing capacity of such screw-shaped type piles with the desired accuracy (Kemfert & Becker, 2010). The results of the two DSP piles in the present study are compared with the results of the three Fundex piles at the Paldiski mnt site (Leetsaar et al., 2022) in Table 5. Table 5 shows that the results have the same trend for most methods.

Table 5. Absolute percentage difference in load-bearing capacity of two DSP piles at the Soodi site and three fundex piles at the Paldiski mnt Site; + Overestimates, - Underestimates

Method	Soodi site	Paldiski mnt site
	Absolute Difference, %	Absolute Difference, %
Nottingham (1975) and Schmertmann (1978)	18	39
de Kuiter and Beringen (1979)	27	27
LCPC (1982; 1997)	-7	6
EUROCODE 7 (EN 1997-2:2007, 2007)	12	36
German method (EA-Pfähle, 2014)	13	14
Unicone method (1997)	-16	20

The Dutch (de Kuiter & Beringen, 1979) method and the German method offer identical results for both sites. The Unicone method demonstrates the most significant variability. This method underestimates the load-bearing capacity of the pile by an average 16% at the Soodi site and overestimates it at the Paldiski mnt site. The reason that direct methods may overestimate the load-bearing capacity of the pile could be related to the testing time of the pile after completion of the pile. The piles in the current study were loaded two to three weeks after completion. Togliani & Reuter (2014) have stated that the load-bearing capacity of the pile may increase even after 100 days from the construction of the pile in conditions where nearly 50% of the pile is surrounded by clayey soils. Further examination of the bearing capacity of the DSP pile in parallel with a Fundex pile is necessary.

The fact that some direct methods underestimate the load-bearing capacity of the FDP pile may indicate a lack of these methods considering the actual behaviour of this pile type in the soil. Bush et al. (2013) concluded that in silty and sandy soils, the cone resistance because of FDP pile installation was increased down to the depth of the displacement body. In addition, a slight decrease below the displacement body had no negative effect on the bearing capacity of the pile. Based on the calculations, in the soil around the pile, there were only minor changes in density and primarily changes in the horizontal stresses. It is very difficult to measure the horizontal stress state and void ratio in situ. Therefore, it is necessary to investigate this type of pile by static load tests in parallel with CPT and SDT soundings. Based on the load test and sounding results, direct methods can be better calibrated.

Based on average, MCS and characteristic values, the results differed by up to 11%. Characteristic values gave the pile capacity 3–8% lower than the average values. The results for the Low 5 fractile differed up to 132% compared with the average values. Eurocode 7 (Frank et al., 2005) recommends the use of the Low 5 fractile when the soil volume involved in a limited state is very small compared with the length of the fluctuation of the soil property. In the studied soils, Low 5 values gave the capacity of the pile with a large reserve.

Conclusion

In the current study, the load-bearing capacity of four screw cast in situ displacement piles in silty soils was analysed. The outcome of the static pile load tests were compared with the results of five CPT-based methods and one CPTu-based direct method. The results of static pile load tests were extrapolated, and the $s/B=10\%$ failure criterion was

applied to the piles. Data from both the CPT and SDT soundings were included in load-bearing capacity analysis of the piles. As there are no direct methods to apply the SDT results to pile-bearing capacity analysis, three CPT direct methods were used. The percentage of absolute difference was used to compare the methods. The percentage of absolute difference was found to be the difference between the calculated and measured load-bearing capacity from the pile load test. The comparison of the methods was based on the principle that the more the results were within $\pm 20\%$ or even $\pm 10\%$, the better the method. In addition to deterministic methods, the probabilistic method with MCS was used for the analysis.

The results indicated that the application of the SDT sounding outcome in the CPT direct methods provided comparable results to the utilisation of the CPTs sounding data. However, a comparison of the bearing capacities calculated from the results of two close CPT soundings showed significant variations. The percentage of absolute difference varied between 5.2% and 33.1%. This is a clear indication of the need for statistical processing of sounding data prior to pile-bearing capacity analysis. As a result, the soil around the piles was divided into three layers and treated statistically. The results of the CPT two soundings were considered together. The two SDT soundings were treated in the same way. The Eurocode 7 method showed the best performance based on analysis of the arithmetic average values of both the CPT and SDT soundings. Most of the results when using this method were within or close to the $\pm 10\%$ range, and the largest absolute difference was 13%. Other direct methods tended to overestimate the load-bearing capacity of DSP piles and underestimate the load-bearing capacity of FDP piles. Except for the Dutch (de Kuiter & Beringen, 1979) method, all other methods offered results for the DSP pile within a $\pm 20\%$ range. The LCPC method and Unicone (1997) method underestimated the load-bearing capacity of all four piles when compared with the measured capacity. The biggest absolute differences were 59% and 66%, respectively.

RBD analysis of the piles was performed using the LCPC method. Density functions were accomplished on the soil layers. Pile-bearing capacity was determined by the LCPC method with MCS, here based on the arithmetic average values of soils layers and standard deviation. Here, 10 000 simulations were used in the simulation. In addition, characteristic 95% reliable estimate of the mean value and 5% fractile results were obtained based on the LCPC method. Based on CPT and SDT soundings, the outcome of pile-bearing capacities in the absolute difference varied within 11% between the average, RBD and characteristic values. Based on CPT soundings, the 5% fractile was

43–132% lower than the RBD and characteristic values. Using SDT sounding data, 5% fractile results were 47–96% lower than the RBD and characteristic values.

The use of SDT sounding results in the CPT direct methods for analysing pile-bearing capacity in this case gives good results and deserves attention and further investigation. Studies should explore parallel soundings of the CPT and SDT in different soils along with different pile types tested statically. The piles must be loaded up to a $s/B=10\%$ failure criterion. Sounding data should be applied to load-bearing analysis of the pile by sounding separately and then being statistically combined. In addition to average values, characteristic values should be used in parallel. Compared with analytical methods, RBD takes account of the variability of the parameters, provides more information and gives a reliable assessment of the probability of failure or actual safety. RBD should increasingly be included in load-bearing capacity analysis of piles, including direct methods, in the future.

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