Investigation of Dense Sand Properties in Shallow Depth using CPT and DMT

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ABSTRACT: The present paper is an investigation of the soil parameters of the given Aalborg University Sand No. 1 using the Flat Dilatometer Test (DMT) and the Cone Penetration Test (CPT). This clean sand type is considered to be similar to the sands found in the North Sea area. The research is mainly based on experimental laboratory testing, followed by computer assisted data interpretation. The mentioned tools are used in testing the sand properties in shallow depth and examining any occurrence of an effect induced by the limited size of the laboratory set-up.

1 INTRODUCTION

Until today, soil behavior analyzed by CPT with pressure on the soil surface application has not been used in any laboratory conditions at Aalborg University. This research has the goal to interpret the soil behavior of Aalborg University Sand No. 1 in shallow depths by conducting an analysis of the strength and stiffness parameters. The investigation is made using CPT and DMT. In addition, by the virtue of DMT, the horizontal stresses are analyzed at different positions in order to observe any influence coming from the limited dimensions of the laboratory sandbox.

This research is provides additional laboratory information for future experimental analyses at Aalborg University involving similar soil conditions in the same test set-up.

Output values from the laboratory are going to be processed analytically and results are being interpreted. Outcomes from the different types of testing procedures and data interpretation, as well as comparison between the values of common parameters coming from different sources are also going to be displayed. The values that are considered to be the most reliable according to each test apart are used.

2 TESTS DESCRIPTION

2.1 *Test set-up*

The determination of the soil properties and laboratory soil conditions is made at Aalborg University Laboratory. The herein study uses tri-axial test data of the Aalborg University Sand no. 1 from Borup et. al. (1995).

Laboratory tests are performed using a cylindrically shaped steel container with 2.5 m in diameter. The elements inside the box illustrated in Fig. 1 and described as: i) a network of uniformly placed pipes which serve as even distributors of the input water level through the lower valve system; ii) on top of the pipe system, 0.3 m of gravel is placed to ensure an undisturbed flow for the sand layer above; iii) a thin permeable geotextile membrane with the role of preventing the sand above being washed out and access the gravel or water inlet/outlet pipes; iv) a layer of Aalborg University Sand No. 1 with depth of 1.2 m. A detailed explanation of the laboratory set-up together with representative illustrations can be found in Vaitkunaite et. al. (2014).

2.2 *Soil preparation and specifications*

The soil conditions have to be re-created prior to each test. A mechanical rod vibrator is used for reaching the desired soil compaction level according to Vaitkunaite et. al. (2014).

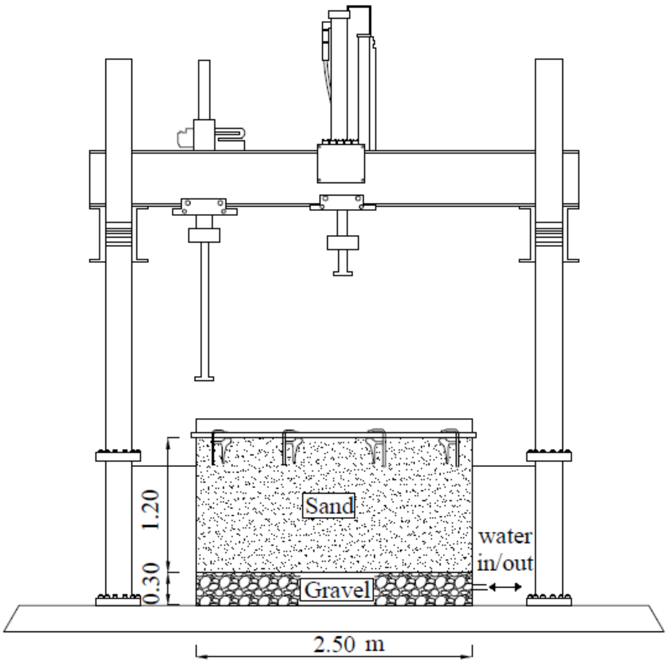

Fig. 1. Laboratory set-up at Aalborg University after Manzotti et. al. (2014).

The soil classification parameters are obtained from Borup et. al. (1995) for the Aalborg University Sand No. 1. The quartz sand with fines content of less than 1% is described in Table 1.

Table 1. Characteristic properties of Aalborg University Sand no.1 from sieve analysis.

Characteristic	Symbol	Value
50% quantile [mm]	d_{50}	0.14
Coefficient of uniformity [-]	d_{60}/d_{10}	1.78
Specific grain density [g/cm^3]	d_s	2.64
Maximum void ratio [-]	e_{max}	0.858
Minimum void ratio [-]	e_{min}	0.549

2.3 *Tests for analysis of soil parameters*

The considered relevant tests for this research comprising soil parameters analysis and boundary effect analysis are presented in Table 2.

Cone Penetration Tests are done down to 0.9 m depth, Flat Dilatometer readings (testing illustrated in Fig. 3) start at 0.3 m depth and are sampled every 0.15 m until 0.9 m.

At least two CPTs are run before each test to inspect uniformity of the soil parameters, such as friction angle and relative density at different relevant points (e.g. see CPT1 and CPT2 in Fig. 2).

Table 2. Tests overview.

Test name	Overburden pressure [kPa]	Amount of tests performed		
		CPT	DMT	Soil Sample
Aim: Soil parameter analysis				
140901	0	-	2	-
141201	0	2	-	3
	~40	3	-	-
150101	0	2	-	3
	~40	3	-	
	~60-63	2	-	
150102	0	3	-	3
	~70	3	-	
Aim: Boundary effect analysis				
150103	0	3	4	-
150105	0	3	2	-
150201	0	3	2	-
150202	0	3	2	-
150203	0	3	2	-
150204	0	3	2	-
150205	0	3	2	-
150205	0	3	2	-

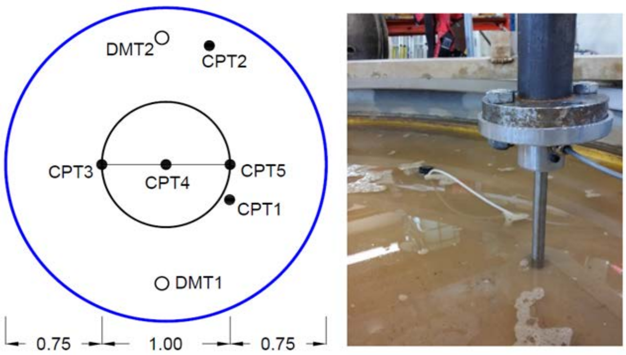

Fig. 2 CPT and DMT positions – (Test No. 141201, CPT1-2 before test; CPT3-5 during pressure appliance).

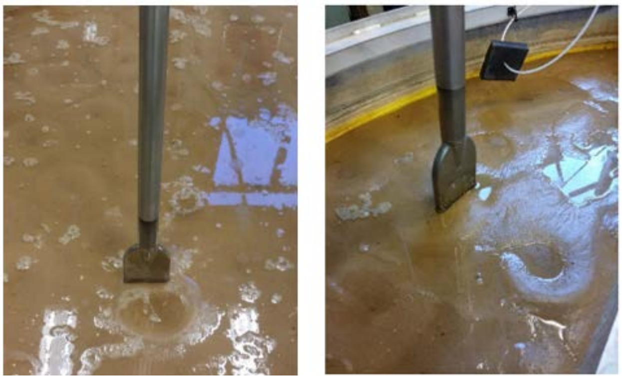

Fig. 3 Flat Dilatometer testing.

A sheet of geotextile is applied on the sand surface acting as a filter layer for the top membrane to prevent soil particles being pulled through the suction pipes. The overburden pressure is applied using a latex membrane to which suction pipes are connected, as illustrated in Fig. 4.

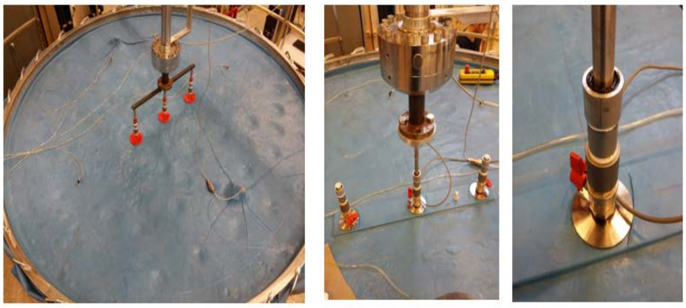

Fig. 4. Applied suction membrane and access valves for CPT during overburden pressure appliance.

Suction under the membrane can reproduce up to 75 kPa of applied overburden pressure on the soil surface simulating greater soil depth (for more details, see Vaitkunaite et. al. (2014)). Once the desired pressure level is reached, the suction should be kept constant for 10-12 hour time interval in order to assure uniform pressure application over the entire soil volume.

Three centrally positioned Cone Penetration Tests are done through the three custom-made valves as shown in Fig. 4, allowing penetration of the instrument, while maintaining the inner pressure constant.

2.4 *Boundary effects tests*

Laboratory models come with some quantity of uncertainty. The goal of a boundary effect analysis is related to whether or not the limited size of the sand tank affects the results by not behaving as an infinite soil volume. In this respect, two types of tests are done: i) Type 1 - by using readings from DMT testing in two positions both facing the boundary as shown in Fig. 5; ii) Type 2 - by using the DMT readings at two positions facing in opposite directions towards the boundary as shown in Fig. 6, simulating the boundary of a laboratory foundation model inserted in the soil.

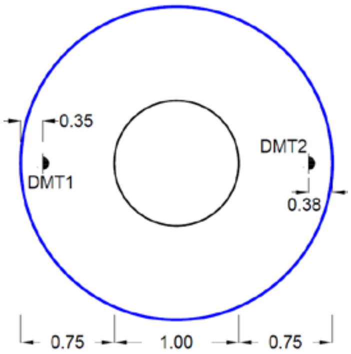

Fig. 5. Boundary effects tests positon of Type 1.

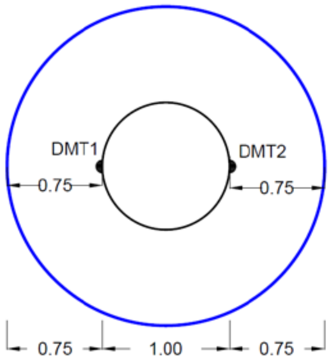

Fig. 6. Boundary effects tests position of Type 2.

3 EXPERIMENTAL DATA

3.1 *Interpretation of CPT data at shallow depth*

In this study, dense to very dense sand conditions are tested. By means of relative density D_r, this refers to dense (65-85%) and very dense (85-100%). Due to the local failure which is composed both of general and punching shear elements of failure, caused by the cone of the CPT, the first 100 mm of the sand are not taken into consideration as level of accuracy is considered low.

Soil compaction and condition of the sand for the related tests during the study case are presented in Table 3.

From the provided tests it can clearly be understood that in connection to the friction angle φ, the difference between DMT and CPT is less than 7%. The friction angle derived from DMT data is more conservative and it is known as $\varphi_{safe,DMT}$, details of the derivation can be found in Marchetti (1997). Furthermore, based on these results, it can be estimated that higher values of friction angles are observed due to the low acting mean stresses in such shallow depths during the test performance. This phenomenon is observed in the beginning stage of the Mohr-Coulomb failure envelope.

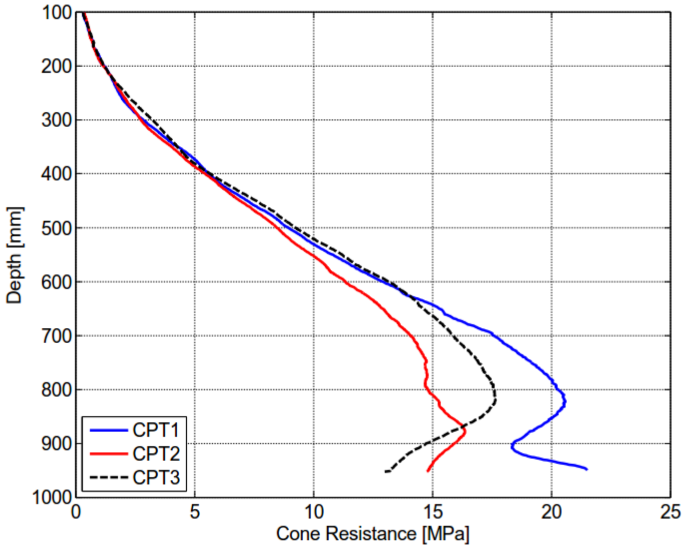

Fig. 7. CPT Cone resistance q_c (Test No. 150201, before pressure application).

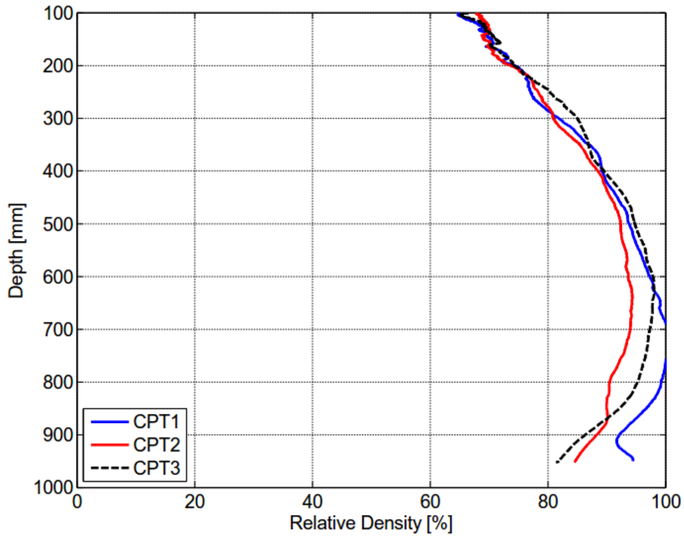

Fig. 8. CPT Relative Density D_r (Test No. 150201, before pressure application, σ_1 = 9.95; σ_2 = 7.80; σ_3 = 8.93).

Table 3. Compaction level and friction angle of sand prior to each test.

Test No.	CPT Relative Density D_r [%]	CPT Friction Angle φ [°]	DMT Friction Angle φ [°]
140901	76.2	52.2	52.7
141201	81.7	53.1	n/a
150101	83.6	53.4	n/a
150102	89.7	54.3	n/a
150103	82.0	53.6	52.2
150105	89.1	54.2	52.0
150201	88.4	54.1	51.1
150202	88.1	54.0	51.3
150203	93.8	54.7	51.1
150204	88.5	54.1	51.4
150205	89.5	54.5	51.7
150206	84.1	53.4	51.9

The second phase of the usage of CPT in this research corresponds to the application of an external overburden pressure. Determination of the vertical overburden pressure at a particular depth d then is turning into Eq. (1).

$$\sigma_v{}' = \sigma_{ex} + (\gamma{}' \cdot d) \quad (1)$$

where, σ_{ex} is the external applied pressure on the soil surface and γ' is the effective soil unit weight. During each test implying overburden pressure, as seen in Fig. 9, three CPTs were performed in the centre line of the sand box. It is visible that by applying overburden pressure the resistance of the sand reaches greater values compared to the ones from Fig. 7.

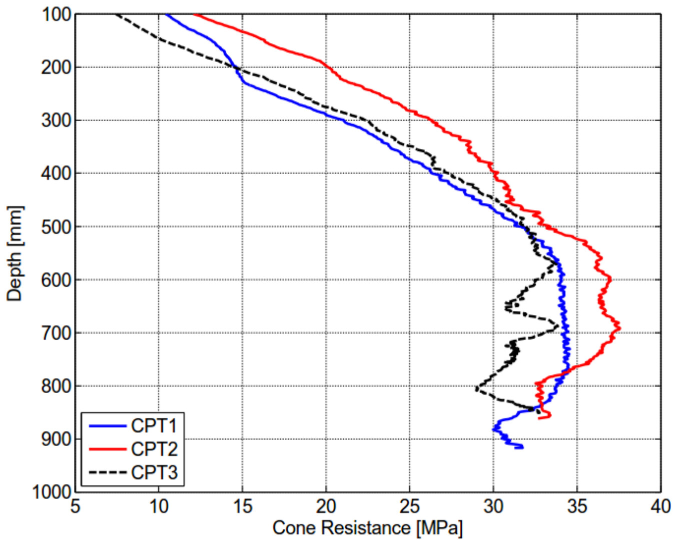

Fig. 9. CPT Cone resistance q_c (Test No. 150102, 70 kPa).

Based on the measured data from the tests with zero external pressure appliance the cone resistance becomes similar to the model described in Puech & Foray (2002). Two visible phases are also observed in all the test data. In the first phase the cone resistance is increasing in a parabolic shape, whereas in the second phase it is slowly increasing and becoming stationary.

Puech & Foray (2002) have described the cone response by dividing it into two phases and expressed the initial phase by Eq. (2), based on the general expression of the bearing capacity of the shallow foundation.

$$q_c(D) = \gamma' \cdot D \cdot N_q (1 + K \sin\varphi' D / L) \quad (2)$$

where, q_c is the cone resistance in the initial phase, D is the depth, γ' effective soil unit weight, N_q is approximated dimensionless bearing factor for shallow foundations, K is dimensionless factor governing the friction between the cylinder and the soil at rest, φ' is the effective friction angle of sand and L is the lateral extension of the slip lines in dimension of the cone cylinder.

3.2 *Interpretation of DMT at shallow depth*

The main soil strength parameter (φ - friction angle) and stiffness parameters (M_{DMT} - constrained modulus and E - Young's Modulus) for the sand type that is used in the laboratory are analyzed in depth from 0 to 0.9 m, as the results are considered for future usage in correspondence to other ongoing research projects at Aalborg University. The procedure is followed from Marchetti (2001).

3.2.1 *Initial DMT parameters*

The initial pressure readings A and B, must be corrected in order to determine the pressures p_0 and p_1 by using Eqs. (3) and (4), respectively.

$$p_0 = 1.05(A - Z_M + \Delta A) - 0.05(B - Z_M - \Delta B) \tag{3}$$

$$p_1 = B - Z_M - \Delta B \tag{4}$$

3.2.2 *Intermediate DMT parameters*

Having the corrected initial values, the determination of the intermediate DMT parameters such as I_D - Material Index, K_D – Horizontal Stress Index and E_D – Dilatometer Modulus can be based on the two readings.

The material index I_D can identify the soil type i.e. clay, silt or sand. The defining formulation used and the identifying ranges according to Marchetti (1980) are shown in Eq. (5).

$$I_D = \frac{p_1 - p_0}{p_0 - u_0} \tag{5}$$

Clay: $0.1 < I_D < 0.6$
Silt: $0.6 < I_D < 1.8$
Sand: $1.8 < I_D < (10)$

The tested sand is characterized by a material index with a greater values than 2.8, corresponding to clean sand properties.

The horizontal stress index K_D provides information about several soil parameters and it is one of the main parameters, as it is directly correlated for determining the friction angle of the sand. The horizontal stress index is determined by Eq. (6).

$$K_D = \frac{p_0 - u_0}{\sigma'_v} \tag{6}$$

where, σ'_v is the pre-insertion in situ overburden stress.

The dilatometer modulus E_D is obtained from p_0 and p_1 by Eq. (7).

$$E_D = 34.7 \cdot (p_1 - p_0) \tag{7}$$

3.2.3 *Derivation of geotechnical parameters*

The procedure of determining the strength parameter φ is taken from Marchetti (1997), who proposed the formulation related to K_D as shown in Eq. (8).

$$\varphi = 28^\circ + 14.6^\circ \log K_D - 2.1^\circ \log^2 K_D \tag{8}$$

According to Terzaghi (1996), the coefficient of earth pressure at rest K_0 is the ratio of the effective horizontal pressure σ'_h to the vertical soil pressure σ'_v in a soil that currently exists in zero horizontal deformation, as described in Eq. (9).

$$K_0 = \frac{\sigma'_h}{\sigma'_v} \tag{9}$$

Baldi et. al. (1986) performed series of DMT tests on Ticino and Hokkasund sands where K_0 was determined. It was found that the coefficient of earth pressure depends on K_D and the ratio of $q_c/\ \sigma'_v$. The last coefficient D_3 is recommended to be 0.0017 for freshly deposited sands. However, it has been inspected that for the sand used in this study, it results in negative K_0 values which can be explained by several reasons: the sand particles are finer than Ticino and Hokkasund sands and, perhaps more importantly, the sand was compacted in a different manner. Baldi et. al. (1986) reported that D_3 of 0.00093 gave the best fit to the data points from 42 tests performed on the mentioned artificial sands which was applied in this study.

$$K_0 = 0.376 + 0.095 \cdot K_D - D_3 \cdot \left(\frac{q_c}{\sigma'_v} \right) \tag{10}$$

Here, it is important to notice that the implemented cone resistance is used from the same depths as the desired DMT depths. By combining the obtained results from DMT and CPT, the horizontal pressure is finally obtained and used further in this paper. Example of the typical properties determined for sand from DMT data are presented in Fig. 10.

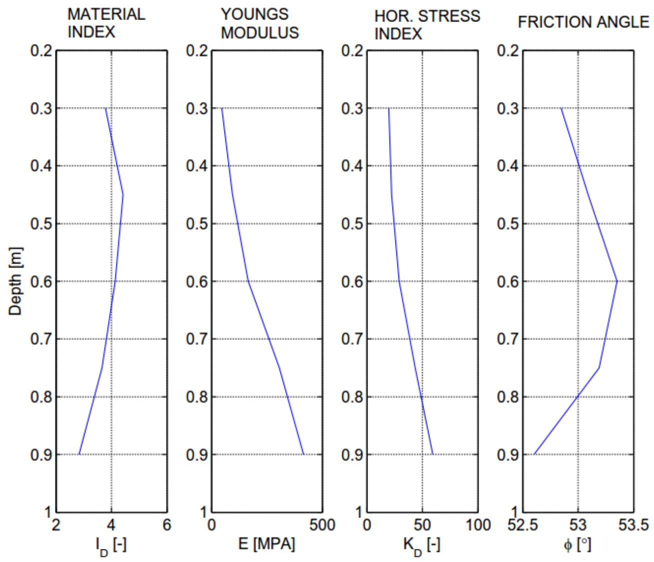

Fig. 10. Relevant DMT data output parameters for sand (Test No. 150103).

4 RESULTS

4.1 *CPT tendency with low and high pressure on soil surface*

From all the tests that were performed in the laboratory with soil pressure on the surface and tests with similar relative densities with no external pressure are presented in Fig. 11. The relative density of the sand is varying in a range of 80% to 90% which corresponds to dense and very dense conditions. Results showing low overburden pressure of ~9 kPa are done with external pressure σ_{ex} equal to zero.

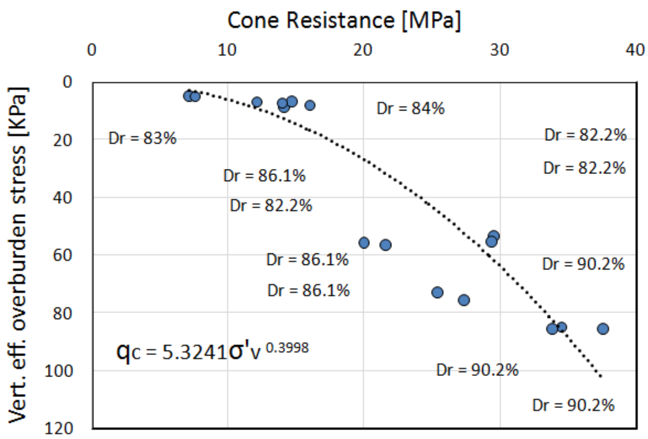

Fig. 11. Maximum cone resistance as function of vertical stresses in respect to the relative density of sand.

For data showing more than ~53 kPa, the tests are done with the suction application. Estimation of the vertical stress on the soil is presented in Eq. (1) and the relationship in terms of the quasi-stationary cone resistance is presented in Fig. 11.

The tendency of this output can be directly related to further tests with the use of the same external pressure on the sand surface application. Once setting the desired overburden pressure on the soil surface and determine the relative density, the cone resistance after the applied pressure can be estimated based on these tests by matching the ranges of compaction level as seen in Fig. 11. A conclusion in this regard to CPT can be drawn, that cone resistance increases when approaching higher vertical stresses.

An analysis of the initial phase of different tests with range of relative density found between 77% to 86% by Eq. (2) into a curve fitting procedure proposed by Puech & Foray (2002) is done.

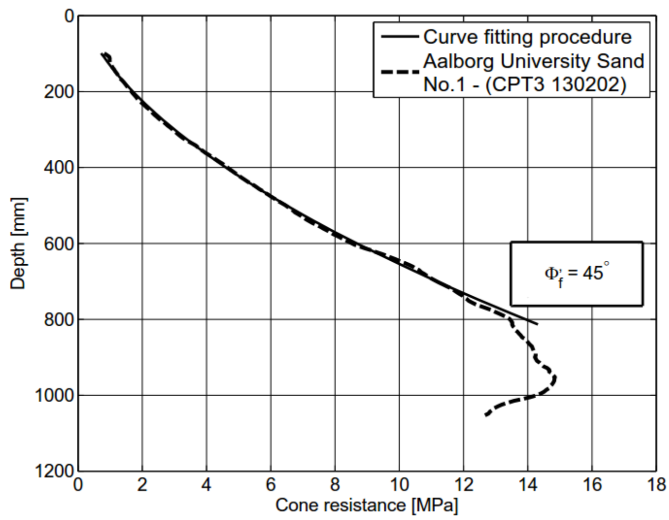

Fig. 12. Determination of effective friction angle by curve fitting procedure from the CPT initial phase.

Out of this process it is derived that the values of the effective friction angle φ' of Aalborg University Sand No. 1 vary around 45^{o} (-/+ 2^{o}) and is noted as the fitted friction angle φ'_f for each of the CPT initial parabolic phase, as illustrated in Fig. 12 for one measurement. The most suitable values for K in Eq. (2) vary from 0.4 to 1.7 and an optimal value is set to 0.9. Furthermore, the depth at which the quasi-stationary cone resistance starts is between 0.5 m – 0.6 m with average values of 0.56 m for medium sand and 0.5 m – 0.93 m with average value of 0.74 m for dense sand. The difference of these depths compared to Puech & Foray (2002), where the quasi-stationary phase begins at 1.5 m, is caused by the different cone diameter of 15 mm instead of the standard size of 37 mm.

Based on the tests performed in this study and tests performed by Larsen (2008) until 0.4 m depth in lower relative densities, the relationship between the quasi-stationary cone resistance and the relative density of the Aalborg University Sand No. 1 is expressed by Eq. (14) in Fig. 13.

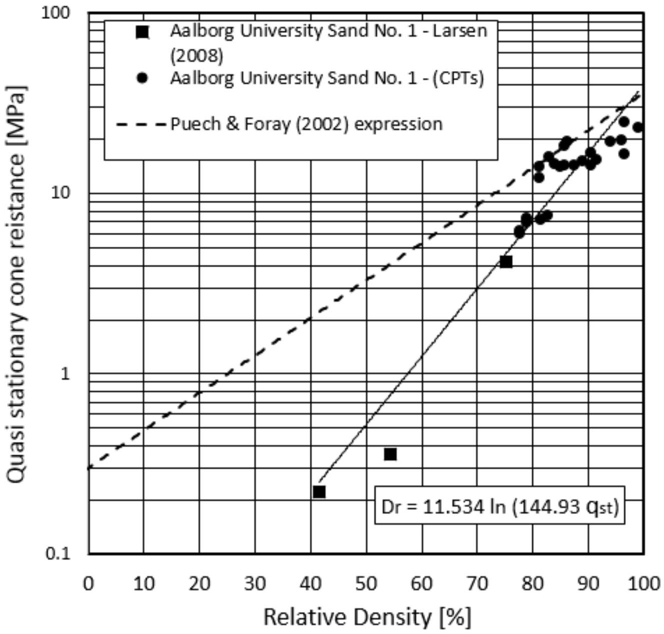

Fig. 13. Correlation of quasi-stationary cone resistance and relative density for Aalborg University Sand No. 1.

Eq. (14) can classify the relative density of the sand from obtaining only the cone resistance in the quasi-stationary phase.

$$D_r = 11.534 \cdot \ln(144.93 \cdot q_{st}) \tag{14}$$

Fig. 13 also illustrates a fitted function described by Puech & Foray (2002) in Eq. (15) for three different sands, namely Fontainebleau, Hostun and Loire types.

$$D_r = 0.209 \cdot \ln q_{st} + 0.25 \tag{15}$$

Furthermore, it can be concluded that the Aalborg University Sand no. 1 presents similar resistance behaviour at dense and very dense compaction levels, whereas in low relative density the difference is significant.

4.2 *DMT results*

This section is intended to the investigation of parameters affected by the limited size of the sand box. Fig. 14 presents p_0 data together with the average output curves gathered from both Type 1 and Type 2 tests. The measurements near the boundary wall from Type 1 DMT2 show lower results for p_0, which suggests that the horizontal stresses are lower when the boundary wall is close. It can be compared to Type 1 DMT1 measurements show greater p_0. Values from testing Type 2 show tendency to be rather close to the Type 1 DMT2 values which are the values measured near the boundary wall. This proves that the horizontal pressure is affected by the limited sizes of the cylindrical sand container.

It can be concluded that the central area of the test set-up at which DMT Type 2 tests are proceeded (as shown in Fig. 14), is affected by the boundary effects in terms of p_0 values. The obtained data shows 30.6% lower values (average over the total depth for the pressure applied from the sand in the center area of the box) with respect to the measurements obtained in Type 1 DMT1 (black line in Fig. 14).

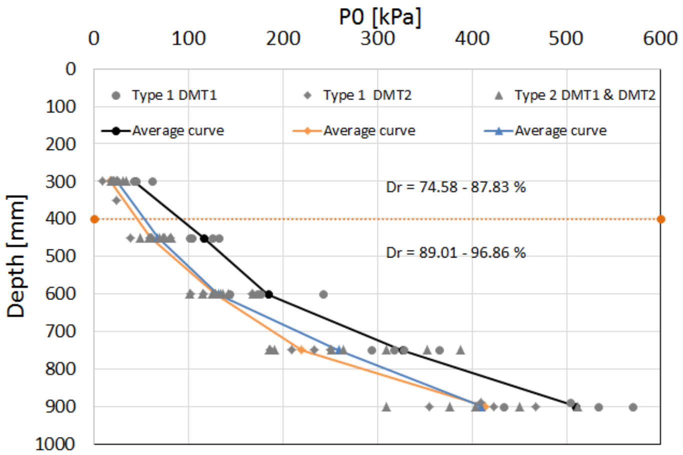

Fig. 14. p_0 vs. depth for Type 1 and 2 DMT tests.

By obtaining the K_0 coefficient with combination of CPT and DMT data as in Eq. (10), the horizontal effective stresses are obtained. Similar tendency to the corrected p_0 value can be seen in Fig. 15, where the horizontal stresses in the central area are affected by the limited size of the sand box. Detailed differences between the two DMT types for horizontal effective stresses σ'_h in the entire examined depth are presented in Table 4.

This imposes, that by virtue of DMT, a sufficient distance from the test set-up boundary should be kept for the experiments, where the horizontal stress variation has a significant influence.

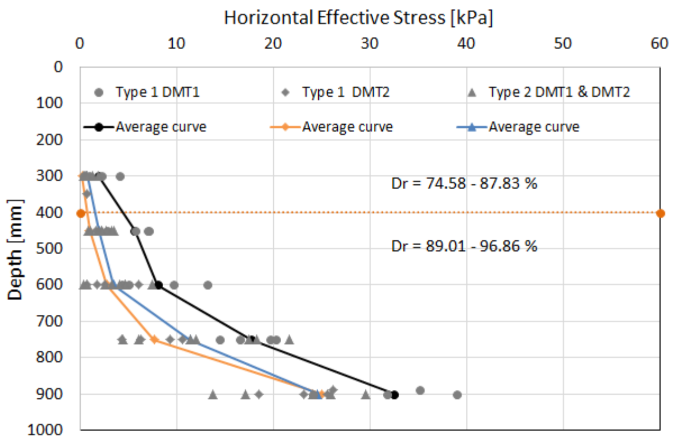

Fig. 15. Horizontal effective stress vs. depth for Type 1 and 2 DMT tests.

Table 4. Comparison between the averaged values of DMT1 Type 1 and DMTs Type 2 and their standard deviation.

Depth [m]	DMT1 Type1 σ'_{h0} [kPa]	STD σ	DMTs Type2 σ'_{h0} [kPa]	STD σ
0.30	1.90	1.65	0.73	0.57
0.45	5.62	1.98	1.93	1.23
0.60	7.99	4.15	3.45	2.29
0.75	17.7	2.71	11.38	7.52
0.90	32.45	6.32	24.54	4.63

5 CONCLUSION

This paper gave and approach to the laboratory testing of Aalborg University Sand No. 1 in shallow depth by using DMT and CPT. The sand container presented in section 3.1 was used for all the performed tests during which soil parameters were analyzed as well as the percentage of influence caused by its limited size. In this respect, three tests implying overburden pressures simulating larger depths of the soil and eight tests regarding the boundary effects of the test set-up were performed.

After analysis of the results presented in section 4, it can be stated that output values of the soil characteristics were accurate and reliable. The investigation, which was performed in depth of 0 m to 0.9 m via the DMT and CPT, showed similar outcome of the soil behavior in-between common parameters of the mentioned testing approaches. The applied levels of overburden stress were as follows: 0, 40, 60, 63 and 70 kPa. In these test results - CPT showed an increase of the resistance of the sand with increase of the applied vertical stress on the soil surface as function of the compaction level.

Tests performed with the Flat Dilatometer showed worthy strength and stiffness parameters as well as it described how the soil acted in horizontal direction. Clearly, it can be seen that the central area was affected by the limited size of the sand box. Consequently, this influence should be accounted in the upcoming tests. By combination of both CPT and DMT data, the relation between vertical and horizontal effective stresses was determined by the coefficient of earth pressure K_0, which can be crucial for experiments performed with this set-up.

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