Evaluation of the degree of compaction of levees by a CPT-based method

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Abstract –

Permeability and strength parameters of compacted soils may be correlated to their degree of compaction. Unfortunately, the use of conventional and recent testing methods for the assessment of density and water content of earthworks under construction cannot be applied to existing levees. Therefore, the development of an expeditious and accurate method for the assessment of the degree of compaction of existing and new levees, after their completion, appears extremely useful. The purpose of this research is to develop a simple tool for the assessment of the degree of compaction of "compacted", partially saturated, fine grained soils. The proposed method combines in situ testing like electric CPT or CPTu with laboratory penetration testing performed with a mini–cone in a calibration chamber.

Keywords –

Compacted soils; Partially saturated soils; In situ testing; Calibration chamber

1 Introduction

Nowadays adverse weather conditions are more and more frequent because of global climatic changes. Particularly adverse climatic conditions such as repeated floods, very prolonged rain periods and very intense rainfalls can lead to an almost complete saturation of the levees and cause their failures [1]; [2]; [3]; [4]. Since budgets for levees refurbishments are limited, priority lists become mandatory.

The assessment of the safety factor against possible Ultimate Limit States of existing levees requires, at least, the knowledge of strength and permeability parameters and it is well recognized that these parameters mainly depend on the degree of compaction and the degree of saturation [5]. Therefore, the assessment of soil density and water content can contribute to a correct estimate of the required parameters. The use of both conventional and recent testing methods for the assessment of density and water content of earthworks, under construction, cannot be applied to existing levees. The Rubber Balloon Method [6], the Sand Cone Method [7], the Time Domain Reflectometry [8] and the Nuclear Methods [9] are only applicable at shallow depths. On the other hand, the use of specially equipped piezocones for electrical resistivity measurements [10]; [11], is not very popular and its application is restricted to fully saturated soils. Also nuclear density probes ([12]; [13]) are not very popular.

Therefore, an expeditious and accurate method for the assessment of the degree of compaction of existing and new levees appears extremely useful.

The proposed method combines in situ testing like electric CPT or CPTu with penetration testing with a mini–cone in a calibration chamber (CC).

2 Hypotheses

In the literature, many calibration chamber were designed for different purposes and very advanced mini-cones were realized. On the contrary, the purpose of this research is that of developing the simplest tool for the assessment of the degree of compaction of "compacted", partially saturated, fine grained soils.

A reference tip resistance, q_{cLAB}, is inferred from laboratory tests in a mini CC using a mini CPT. q_{cLAB} is expressed as a function of the expected dry density and of the relevant influential factors. Such an empirical relationship is obtained by carrying out a number of repeated tests in the CC at given densities and different values of the influential factors. A comparison between q_{cLAB} and the tip resistance inferred from in situ CPT gives the possibility of assessing the dry density of existing embankments, while, for new embankments, the method defines the expected in situ tip resistance for a given target dry density.

A similar procedure is described in the technical standards by AFNOR [14] and [15]. This procedure is applied to coarse grained soils and requires the construction of a trial embankment (physical soil model) and the performance of dynamic penetration tests. As a results a reference "penetrogramme" (i.e. displacement per blow vs. depth) is obtained from the experiments.

The standards also state the criteria for the acceptance of the in situ controls in comparison to the design “penetrogramme”. According to Setra–Lcpc [16], [17], this methodology should be applied to the control of the compaction degree of trenches.

The proposed method is based on the following considerations and assumptions:

1. The tip resistances of a standard cone (d = 35.7 mm) and a mini–cone (dC = 8 mm) are the same irrespective of the cone diameter when carried out in the same soil under the same conditions. This hypothesis involves two different aspects. The first one is related to the ratio between the cone diameter and the grain size of the soil and is discussed with the fourth hypothesis. The second aspect is related to the normalized penetration rate ([18]; [19]):

(1)

where V = normalized penetration rate; d = cone diameter, v = penetration rate, c_v = coefficient of consolidation.

It is evident that for the mini-cone penetration occurs at a lower normalized penetration rate since the mini cone has a normalized velocity four times smaller than that of a standard cone. According to many researchers, higher tip resistances should be measured at lower normalized penetration rates, especially in the case of saturated silty clay ([20]; [21]; [22]). For the soils under consideration, unsaturated silt mixture, the correctness of the hypothesis, has been experimentally verified by performing, in situ, at close distances 4 standard and 4 mini cone tests in the Calendasco site (Piacenza, Italy) [1]. Since the obtained tip resistance profiles are very similar and do not show systematic differences it is possible to conclude that in the case of unsaturated silt mixtures standard and mini–cone give very similar tip resistances. It is worth noticing that the silt mixtures that were tested in this research are similar to the Calendasco soil in terms of texture.

2. Tip resistance in pluviated dry sand, can be expressed by the following equations ([23]; [24]; [25]; [26], [27]):

(2)

(2bis)

where: Q_c = tip resistance; C_0, C_1, C_2, C_3 = experimental constants; σ'_{v0}, σ'_{h0} = vertical and horizontal effective stress respectively; D_R = relative density as a fraction of 1 and σ'_m = mean effective stress. It is widely accepted that for dry or saturated clean sands the tip resistance is mainly controlled by relative density, soil type and stress state. As for the stress state, other equations are also available in literature.

In the case of silt mixtures, compacted at a given water content, the boundary stresses are no more representative of the effective stress state which depends on suction (i.e. saturation degree or water content during formation). Moreover, the compaction energy is also a relevant parameter because of the pre-stressing (or pre-straining) of the compacted soil. Tatsuoka [28] suggests that the degree of compaction, defined for certain compaction energy, is more appropriate than the relative density for the evaluation of the compacted state of soil including a large amount of fines content. Therefore, the influence of the effective stress state in the case of compacted silt mixtures should be defined in a different way.

3. A ratio between the calibration chamber diameter (D_{CC}) and that of the cone (d_C) equal to 40 is considered acceptable in order to consider the CC as an infinite medium. There is evidence in literature that this type of size effect in sands depends on the boundary conditions and soil dry density ([29]; [30]; [31]; [25]; [26], [27]). Even if, for very dense sands and zero lateral strain, higher value of the D_{CC}/d_C ratio are necessary in case of silt mixtures this assumption seems acceptable. In fact, a number of Cone Penetration Tests (CPTs) were carried out in a recently constructed levee at increasing horizontal distances from a Marchetti Flat Dilatometer Test (DMT) blade [32]. The blade was used as a cell pressure: it was maintained at a given fixed depth and continuously monitored. The tests show that when the horizontal distance between the DMT and the CPT is 20 times that of the cone diameter the DMT is no longer sensitive to the passage of the cone.

4. It is considered acceptable that the ratio of the cone diameter to the mean grain size be equal to or greater than 300 ([33]; [34]; [35]; [36]).

This assumption is necessary to perform tests using a cone having a diameter of only 8 mm in the case of silt mixtures. This hypothesis is not verified for the Ticino sand. It is worth noticing that it is not verified even in the case of standard CPT in Ticino sand. The ratio is about 70 for standard cone and only 16 for the mini–cone.

3 Laboratory procedure

The equipment consists of a cylindrical aluminum mold with an inner diameter of 320 mm and a height of 210 mm . Lattice membranes are located at the bottom of the mold and all around the internal lateral surface. Air pressure can be inflated inside the membranes in order to apply horizontal and vertical stresses to the sample: there are manual air pressure regulators for the vertical and horizontal stresses. The mold is housed in a stainless steel frame with a lower and upper plate that are connected to each other by means of four stainless steel rods. A locking system is located in the lower plate in order to push up the mold and put it in contact with the upper plate. Therefore, the bottom and lateral surfaces of the CC are flexible boundaries, while the top is rigid. A nozzle is located in the upper plate for the passage of the mini–cone. The mini–cone (8 mm in diameter) has an external sleeve along its full length and the tip resistance is measured by means of a load cell located above the cone. Since the external sleeve is not in contact with the load cell the sleeve friction is not measured. An electric step motor is used to drive the mini-cone at a constant rate of 20 mm/s. The system uses proximity transducers to automatically stop the penetration when the cone is close to the bottom (30 mm above the base).

Ticino sand, and four different silt mixtures (classified as A4 to A6 according to [37]) were used for the testing program. Table 1 summarizes the main characteristics of the silt mixtures (FR, PC, DD, TC).

Tests on the well known Ticino sand were carried out only for a preliminary check of the equipment.

Ticino sand samples were reconstituted by dry pluviation. In practice the sand was poured into the mold using a funnel that moved over the entire mold surface. This method gave a repeatable relative density of about 40%. The mold was also subject to slight vibrations. This method gave a repeatable relative density of about 60%. Moist tamping would be more appropriate to simulate the behavior of compacted sand fills but tests on Ticino sand samples were carried out only to validate the equipment, by comparison of the results obtained with the mini-cone in the mini-CC with those available in literature [27].

The silt mixtures were used for the construction of new levees and for the refurbishment of existing structures. The soils were sieved in order to eliminate the fraction with a diameter greater than 2 mm.

Samples of fine grained soil were reconstituted in four layers (each 52.5 mm high) using a stainless steel mold with an internal diameter of 310 mm (smaller than that of the CC). The soil was prepared at a given water content and compacted to a given density by applying a vertical pressure to the upper surface of the sample via a loading piston and an upper plate of 300 mm in diameter (i.e. under K_0 conditions). Each layer was compressed to the desired density by applying a static pressure on the upper surface of the layer. Since the applied force (pressure) and the associated displacement were measured, it was possible to compute the compaction energy per unit volume of soil for each layer and for the whole sample:

$$ - \tag{3} $$

where: F_i = force applied for each layer; δ_i = displacement caused by each applied force; V_i = soil volume of each layer.

After the sample had been reconstituted, it was transferred into the CC. There was a gap between the sample and the lateral membrane. The CC was then put inside the frame and the locking system was used to push up the CC and put the upper surface of the soil in contact with the upper aluminum plate.

The consolidation stresses were applied to the sample in two steps: first the isotropic component of horizontal and vertical boundary stresses was simultaneously applied and after that, the deviatoric component of the consolidation stresses was imposed.

The penetration test was carried out few minutes after the application of the consolidation stresses.

All the tests were performed under constant boundary stresses (Boundary Condition 1: BC1).

Table 1. Main characteristics of the tested fine grained soils: FR, PC, DD, TC.

Soil	γ_{dmax} kg/m^3	w_{opt} %	e_{opt}	$(Sr)_{opt}$ %	LL	PL	PI	AASHTO M 145	Gs	d_{50} mm
FR	2047	9.43	0.33	78	26÷31	18÷24	7÷10	A4÷A6	2.72	0.002÷0.025
PC	1950	10.7	0.39	74	25	19	6	A4	2.71	0.085
DD	1820	13.1	0.49	73	31.5	23.5	8	A4	2.71	0.01
TC	1895	12	0.42	77	25	6	19	A6	2.69	0.02

4 Test results

Table 2 sums up test results for Ticino sand. It reports boundary stresses (σ'_v, σ'_h), estimated relative density (D_R), measured average tip resistance (Q_c) and that evaluated by means of equation 2bis. A single Ticino sand sample was reconstituted in the laboratory. Indeed, moving the CC in the horizontal plane of about 40 mm in various directions it is possible to perform at least six penetration tests on the same sample. Therefore, a single relative density of about 40% was considered and different boundary stresses were applied on the same sample. In a first phase, the vertical stress was kept constant while the horizontal stress took different values.

Table 2. Test conditions and results for Ticino dry sand samples.

σ'_v (kPa)	σ'_h (kPa)	Q_c (kPa)	Q_c by eq. 2 (kPa)	D_R
50	50	4277	5071	39.7
50	100	6560	6791	39.9
50	150	8269	8272	40.2
50	50	4377	5147	40.2
100	50	4501	6047	40.2
150	50	5772	6851	40.2

After that, a second set of stresses was applied by keeping the horizontal stress constant and applying different values of the vertical stress. When the initial boundary stresses of 50 kPa were restored for the second set of tests, the measured average tip resistance was very close to the first measurement. The agreement between measured and computed (eq. 2bis) tip resistances seems acceptable, even though a certain scatter is observed (Table 2). The low ratio between cone and grains diameters could be a reason for the observed scatter. The following parameters were used to compute the tip resistance by means of eq. 2bis [27]: $C_0 = 23.19$; $C_1 = 0.56$ and $C_2 = 2.97$. From a multiple – variable linear regression analysis of experimental data, the following values of the parameters of eq. 2 were obtained: $C_0 = 52.4$; $C_1 = 0.22$ and $C_2 = 0.61$. C_3 constant could not be assessed as the data referred to a single relative density. Therefore it was assumed $C_3 = 2.97$ [25]. The exponent C_2 is greater than C_1, i.e. the effect on Q_c of the horizontal stress is greater than that of the vertical one and this result is qualitatively in agreement with the results of a numerical simulations carried out by Arroyo *et al.* [38] and with experimental evidences ([26], [27]).

Table 3 sums up test results for the fine-grained soils. It reports soil type; boundary stresses (σ_v, σ_h); sample dry unit weight (γ_d); maximum dry unit weight (Modified Proctor), $\gamma dmax$; sample water content (w), optimum water content (Modified Proctor), w_{opt}, compaction energy per unit volume (E); maximum vertical stress applied during sample formation (σ'_{pmax}) and average tip resistance (Q_c).

Samples of fine grained soils were reconstituted at densities in between 80 and 92% of the maximum (Modified Proctor) with a water content approximately corresponding to the optimum value. For the FR samples a value of the water content higher than the optimum (9.43%) was used and a test series at constant density (equal to 80% of the optimum) and variable water content (4, 8 and 12%) was also performed. Therefore, these samples were produced by moist-compaction as in the field compaction. After testing, measurements of sample heights and diameters were performed by means of calipers. The maximum vertical strain (in the centre of the sample) was of less than 4 %. Since the after testing evaluation of current sample volume was not considered too much accurate, the dry densities reported in the tables refer to the values just after formation.

For the fine–grained soils, it has been observed that, for a given soil and a given water content a correlation exists between:

- the dry density (γ_d) and the compaction energy per unit volume (E) as shown by Figure 1. FR soil shows a certain scatter especially at higher densities. This scatter could be a consequence of the fact that various batches of FR soil were used and the various batches exhibit small differences.
- the average tip resistance (Q_c) and the compaction energy per unit volume (E) as shown by Figure 2.
- the dry density (γ_d) and the average tip resistance (Q_c) as shown by Figure 3.

Table 3. Test conditions and results for fine grained soils.

Soil	σ_v [kPa]	σ_h [kPa]	γ_d [kN/m^3]	γ_{dmax} [kN/m^3]	γ_d/γ_{dmax}	W [%]	w_{opt} [%]	E [MJ/m^3]	σ'_{pmax} [kPa]	Qc [MPa]
DD	30	30	14.56	17.85	0.82	13.2	13.1	0.395	8224	2.807
DD	50	50	14.56	17.85	0.82	13.2		0.238	6157	1.786
DD	80	80	14.56	17.85	0.82	13.2		0.299	6752	1.512
DD	30	30	16.38	17.85	0.92	13.2		1.324	24474	4.751
DD	50	50	16.38	17.85	0.92	13.2		1.413	24523	4.063
DD	80	80	16.38	17.85	0.92	13.2		1.501	24523	4.990
PC	30	30	15.60	19.13	0.82	10.8	10.7	0.62	13731	3.274
PC	50	50	15.60	19.13	0.82	10.8		0.697	14712	3.648
PC	80	80	15.60	19.13	0.82	10.8		0.545	13731	3.850
PC	30	30	17.55	19.13	0.92	10.8		2.407	39627	7.191

PC	50	50	17.55	19.13	0.92	10.8		2.76	40707	7.877
PC	80	80	17.55	19.13	0.92	10.8		2.211	36979	7.603
FR	30	30	18.50	2.05	0.92	12.0		4.123	46864	6.533
FR	30	30	18.50	2.05	0.92	12.0		3.315	43136	6.535
FR	30	30	18.50	2.05	0.92	12.0		2.938	37465	6.767
FR	30	30	18.00	2.05	0.90	12.0		1.735	22730	3.254
FR	30	30	18.00	2.05	0.90	12.0		1.735	24005	3.568
FR	30	30	18.00	2.05	0.90	12.0		1.828	24400	4.056
FR	30	30	16.00	2.05	0.80	12.0		0.511	8608	1.843
					0.80	12.0	9.43	0.463	8313	1.736
					0.80	12.0		0.475	7823	2.022
					0.80	4.0		0.26	10103	2.036
					0.80	4.0		0.307	9809	1.479
					0.80	4.0		0.346	10790	1.827
					0.80	8.0		0.579	15990	3.077
					0.80	8.0		0.622	15891	2.533
					0.80	8.0		0.564	15303	2.455

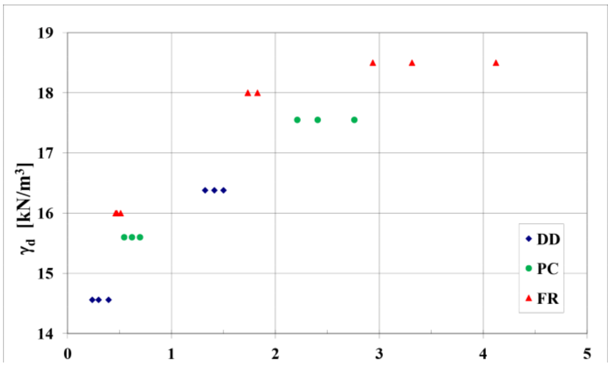

Figure 1. Partially saturated fine grained soils: correlation between dry density (γ_d) and compaction energy per unit volume (E).

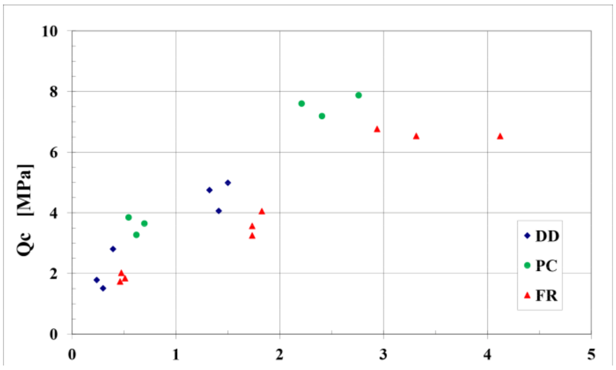

Figure 2. Partially saturated fine grained soils: correlation between average tip resistance (Q_c) and compaction energy per unit volume (E) for a given water content and a given soil.

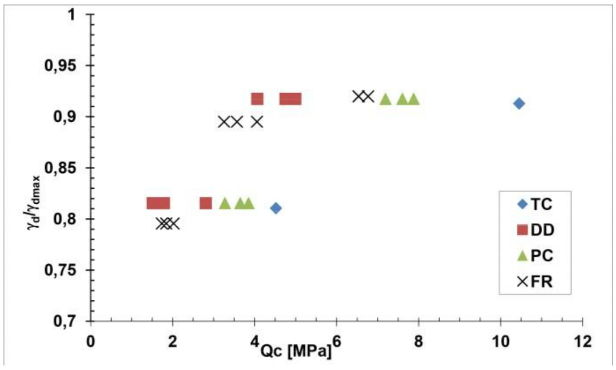

Figure 3. Partially saturated fine grained soils: correlation between dry density (γ_d) and average tip resistance (Q_c) for a given water content (w_{opt}) and a given soil.

The effect of boundary stresses seems negligible. This is supposed to be a consequence of two facts:

- effective stresses are mainly controlled by the suction;
- the compaction stresses, applied during sample formation, are several hundreds of times greater than the applied boundary stresses.

Therefore, it is possible to predict the dry density from the measured tip resistance irrespective of the boundary stresses. The water content during earthwork formation may be also an influent parameter. The use of compaction equipment measuring the compaction energy represents an alternative to infer the in situ density after an appropriate calibration. Moreover, the control of the compaction process in the laboratory offers a quantitative evaluation of the soil workability.

In fact, Table 3 and Figure 1 show that some soils are more workable than others. For example, for FR soil, the maximum compaction pressure or the compaction energy per unit volume that is necessary to obtain a given percentage of the optimum dry density is smaller in comparison with that required in order to compact the PC and DD soils.

5 Water content after sample formation and elapsed time effects

In order to study in the laboratory, the tip resistance variation with water content after the sample formation a sample of soil was prepared at the optimum water content and a dry density equal to 90% of the optimum value and several penetration tests were repeated on the same sample. In fact, it was observed that it is possible to horizontally move the CC of about 40 mm along all directions and to repeat the penetration tests along different verticals at least 6 times for the same sample. The possibility of performing repeated tests on the same sample was preliminary checked several times. The tests were carried out at different dates and water contents. The water content decreased with time because of evaporation and was increased by adding water to the sample. Water was sprayed on the top surface in several steps. For each step the water content was increased of about 2.5%. The penetration test was performed after seven days. Figure 4 shows the Qc (average value between 6 and 15 cm depth) vs. the water content for all the fine grained soils. While for the PC soil the experimental results (Table 5) show that the tip resistance linearly increases with a decrease of the water content and the phenomenon seems perfectly reversible, in the case of DD soil, the data (Table 6) show that the tip resistance also increases with time and not only with a water content decrease. In this case the phenomenon is not fully reversible.

Therefore, the effect of the elapsed time after sample formation was experimentally studied by performing repeated penetration tests, in the CC, on the same sample over a period of 2 months. The sample water contained remained constant over the time. The same testing program was repeated using two different material, TR and PE soil samples, in order to compare the results. The two soil samples were reconstituted at a water content equal to the optimum water content and at a dry density approximately corresponding to the 80% of the maximum value (Modified Proctor). Tables 8 and 9 sum up, for each soil, the average tip resistance values measured at different dates.

As TR soil is concerned, Table 8 reports, in the last column, the mass of the CC and of the sample. Measurements of such a mass were taken after each penetration tests. The reported values include 31 kg of CC. The only variations concern the water mass. For PE soil, the water mass variation over a period of time of two months was of 0.205kg.

Test results show an almost linear increase of the resistance with the time for both soils (Figure 5). From the regression analysis of the whole data it is possible to assume an increase of about 40% of the tip resistance per log cycle of time.

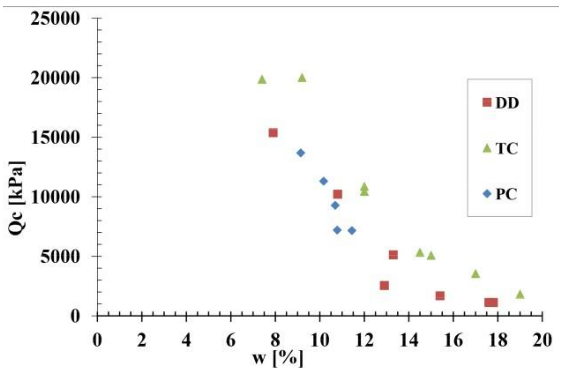

Figure 4. CC tests at variable water content: average tip resistance vs. water content for all the fine grained soils.

Table 5. CC tests on a PC soil sample: average tip resistance measured for the same sample (Qc), along different verticals, at different dates and water contents (w).

Test	Date	Time days	w %	Qc kPa
1	22/07/2014	0	10.78	7206
2	07/08/2014	15	10.69	9278
3	05/09/2014	45	10.17	11307
4	19/09/2014	59	9.14	13680
5	02/10/2014	72	11.44	7163

Note: Soil sample: PC; $\gamma_d = 0.9\gamma_{dmax}$

Table 6. CC tests on a DD soil sample: average tip resistance measured for the same sample (Q_c), along different verticals, at different dates and water contents (w).

Test	Date	Time days	w %	Qc kPa
1	16/10/2014	0	12.9	2548
2	27/10/2014	11	15.4	1685
3	03/11/2014	18	17.6	1124
4	10/11/2014	25	17.8	1120
5	21/11/2014	36	13.3	5125
6	05/12/2014	50	10.8	10216
7	22/12/2014	67	7.9	15377

Note: Soil sample: DD; $\gamma_d = 0.9\gamma_{dmax}$

Table 8. TR soil sample: average tip resistance values measured at different dates.

Test	Time [Days]	Qc [kPa]	Mass (kg)
1	7	4253	58.740
2	14	5738	58.730
3	21	5413	58.725
4	28	6461	58.685
5	39	6570	58.650
6	57	6597	58.605

Table 9. PE soil sample: average tip resistance values measured at different dates.

Test	Time [Days]	Qc [kPa]
1	4	4211
2	16	4451
3	28	5492
4	38	5784
5	50	5908
6	60	6044

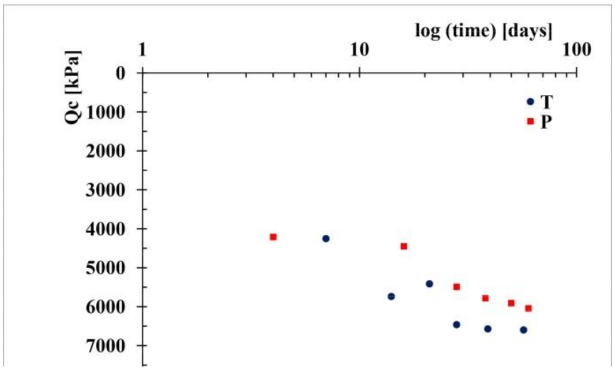

Figure 5. Average tip resistance versus time for TR and PE soils.

6 Conclusions and further research

The tests on the compacted partially saturated fine-grained soil samples suggest that the tip resistance mainly depends on the compaction degree and water content after sample formation. The total boundary stresses are not influent probably because the effective stress state, in this case, mainly depends on suction and pre-stressing during compaction.

The water content during sample formation has a lesser influence.

For practical application of this method, it is suggested to define, for a given soil, a design compaction degree. Therefore, it is possible to experimentally determine, for the given compaction degree, the design tip resistance vs. the water content after sample formation. For the experimental determination of this design curve it is sufficient to reconstitute a sample of a given soil at a given dry density and water content. On this sample it is possible to repeat the tests with variable water contents after sample formation.

The effect of the elapsed time since the sample formation has a great effect and also this aspect deserves further research. While for new earthworks, it is suggested to proceed with controls immediately after the work completion, for earthworks realized centuries ago it would be interesting to understand how this indication should be applied to.

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