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Physical Modeling of the Cone Loading Tests (CLT and CLT-u) in the Calibration Chamber with a Focus on the Load-transfer Curves

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ABSTRACT

The cone loading tests (CLT/CLT-u) represent an innovative alternative to the conventional CPT test, allowing the measurement of soil stiffness. This paper aims to present the results of an experimental study of the (CLT/CLT-u) within the framework of physical modeling at natural gravity by using the calibration chamber. This technique makes it possible to carry out a full-scale penetration test within a soil slice. After describing the experimental devices and the loading program, the paper will focus on the presentation and discussion of the results of cone penetration (CPT) and cone loading (CLT) tests in Fontainebleau sand and Speswhite Kaolinite clay. Experimental load-transfer curves were then derived, and their parameters were determined. It was found that the cone modulus and the friction stiffness linearly increase with depth, according to the model of Gibson's soils. Comparison with the conventional soil properties of the soils tested led to the conclusion that these parameters may be used for a practical characterization of the soil stiffness.

Keywords: Cone loading test, Sand, Clay, Calibration chamber, Penetration, Stiffness.

INTRODUCTION

The usual cone penetration test (CPT) consists of penetrating a cone connected to a set of rods with a standardized rate of 20 mm/s, which provides a continuous profile of cone penetration resistance $q_c(z)$ as a function of depth z. During this test, the soil is subjected to large deformations following spherical cavity expansion scheme, without measurement of the soil stiffness. As an alternative to this shortcoming of the CPT test, the Ménard pressuremeter test (PMT) provides a cylindrical expansion curve, considered as an experimental stress-strain curve of the soil at a given depth. Interpreting such a curve provides, among other things, the soil stiffness quantified by the "Pressuremeter deformation modulus".

However, the PMT test presents some inherent disadvantages, including the discontinuity of the stiffness profile, because the PMT test is indeed carried out at 1 m-depth intervals. In this regard, to combine the advantages of the two previous tests; namely, the continuity of the soil-resistance profile provided by the CPT and the possibility to determine the soil stiffness as given in the PMT test, a new *in-situ* geotechnical test, called the CLT (Cone Loading Test) test has recently been developed in France. According to Figure (1), this test consists of carrying out the CPT test at a depth z to measure $q_c(z)$, then at this depth, a cone loading test is performed by applying small increments of vertical pressure at a relatively low rate of penetration, about 2 mm/s, to obtain two load-transfer curves. The first one, as depicted in Figure (2), describes the mobilization of the vertical pressure q_p as a function of the normalized cone settlement v_p/B, v_p and B being respectively the cone settlement and the cone diameter (equal to 35.7 mm). The interpretation of this curve allows determining the soil stiffness under the cone, quantified by the "cone modulus E_c", as well as the "CLT cone resistance" q_l^{clt}.

As illustrated by Figure (2), the second curve describes the evolution of the sleeve friction stress f_s as a function of v_p, from which the soil stiffness around the cone, quantified by the "friction stiffness E_f", and the limit sleeve friction f_s^{clt} are derived.

Moreover, since the rate of penetration during the CPT test is high enough to mobilize a short-term behavior within the fine saturated soils, like clayey and silty soils, characterized by an undrained cone resistance and a total pressure, the necessity to analyze the drained cone resistance led to the development of the piezocone penetration test (CPT-u), by instrumenting the cone by pore water pressure transducer, which allows measuring the pore water pressure u(z) and deriving the profile of the effective cone resistance q_c'(z). For the same arguments presented above, the CLT-u test was developed to determine the profiles $q_l^{clt}(z)$, u(z), and q_l^{clt}'(z), as well as those of the stiffnesses $E_c(z)$ and $E_f(z)$. It is noteworthy that the load-transfer curves derived from CLT/CLT-u tests may serve as practical tools for load-displacement analysis of foundations, piles, and retaining wall design.

As reported by Hassan (2010), the first generation of the CLT device was developed in 1979 by Gourvès and Faugeras, who used a standard Gouda CPT device with a capacity of penetration of 25 kN. After penetrating up to a given depth, the cone is subjected to a series of increments of vertical pressures of 50 kPa, until the occurrence of the soil failure, each increment for 30 seconds to 1 minute. Some improvements of the basic apparatus were subsequently made (Zhou 1997; Arbaoui et al. 2006; Hassan 2010), which enhanced the possibilities of measuring the soil properties and opened interesting perspectives of soil characterization.

Although the CLT/CLT-u tests were not yet standardized, they are conducted with the same devices of the CPT/CPT-u tests, which are prescribed by two international standards; namely, ISO-22476-12 (AFNOR 2010) and ISO-22476-1 (AFNOR 2013).

It should be noted that the CLT/CLT-u tests, like any newly developed geotechnical test, are not currently widely used within the scope of a usual geotechnical investigation, compared to the CPT test. However, extensive research work is being undertaken to explore the capabilities of soil characterization by these tests.

Hassan (2010) carried out a full-scale CLT test in three experimental sites; namely, the Flanders clay site in Merville (France), a sandy site in Utrecht (The Netherlands), and a sandy site in Limelette (Belgium). The results demonstrated that this test applies to a variety of soil types and is more representative of the initial state of the soil than other *in-situ* tests.

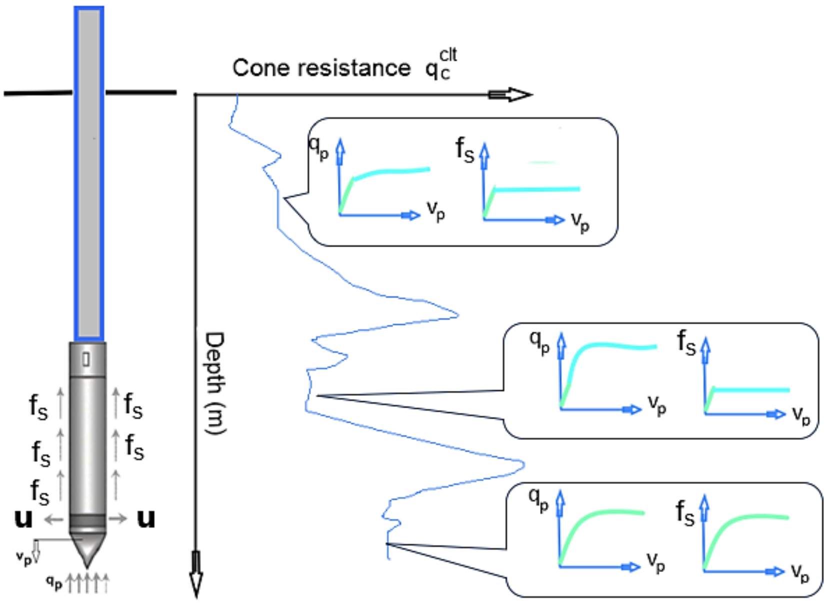

Figure (1): Scheme explaining the principle of the CLT/CLT-u tests

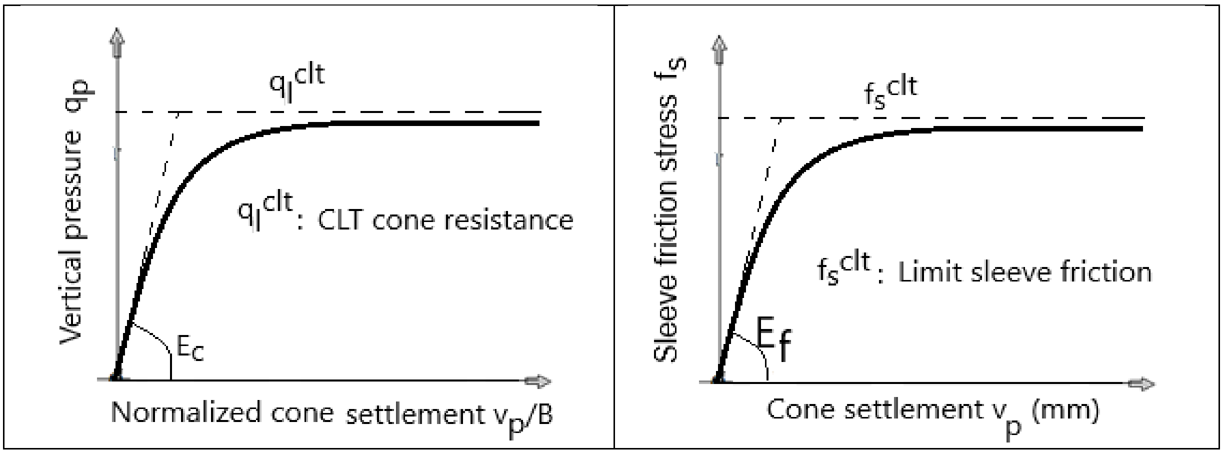

Figure (2): Typical load-transfer curves (q_p *versus* v_p/B and f_s *versus* v_p)

Moreover, the boundary conditions are well controlled. Physical modeling in macro-gravity has also been conducted by using small-scale models in the geotechnical centrifuge of the University of Gustave Eiffel, France, and the influences of several key parameters on the experimental results given by the CLT test in the Fontainebleau sand, such as the cone loading rate, the geometry of the cone, and the density of the sandy sand has been studied.

Due to the experimental aspect of geotechnical engineering, physical modeling is one of the most important tools used within the scope of a geotechnical project, as well as in research work. To extrapolate the experimental results from a physical model to its prototype, some similitude laws should be respected. A fundamental law of similitude prescribes that to keep the model and its prototype to the same level of stress and strain, it is necessary to increase the unit weight N times that of the prototype, 1/N being the reduction scale of the model. Consequently, physical modeling is usually carried out by testing the model in a macro-gravity environment, like in a geotechnical centrifuge, where the small-scale model is subjected to a centrifuge force during the rotation, leading to an increase in its apparent weight (Bouafia & Garnier, 1991). However, this type of modeling allows testing the whole prototype, and it may sometimes be of interest to focus the modeling on a part of the prototype, which is usually undertaken by using a calibration chamber, allowing for the analysis of the local behaviour of the prototype at natural gravity. This local modeling corresponds to a full-scale test focusing on a local aspect within the prototype.

This paper aims to present the results of an original research work, based on local physical modeling of the CLT test within sandy soil and the CLT-u test within clayey soil, undertaken in the calibration chamber of Navier's laboratory at the Ecole Nationale des Ponts & Chaussées, France. After a brief description of the calibration chamber, the CLT/CLT-u devices, the test procedure, and the methodology of interpretation are presented with a focus on the analysis of the parameters of the load-transfer curve involved in cone/soil interaction; namely, the CLT cone resistance q_l^{clt}, the limit sleeve friction f_s^{clt}, the cone modulus E_c and the friction stiffness E_f. These parameters will ultimately be compared to the conventional soil properties.

DESCRIPTION OF THE EXPERIMENTAL DEVICES

The Calibration Chamber

Within the framework of local physical modeling, the CLT/CLT-u tests are carried out in the calibration chamber within a slice of soil surrounding the penetration cone, under fixed boundary conditions simulating the real stress conditions at a given depth in the soil. As shown in Figure (3), the calibration chamber encompasses a 524-mm diameter cylindrical container with a height of 700 mm, a piston, two bases (lower and upper), and a cover.

The loading frame, equipped with a hydraulic installation jack and a servo-controlled loading jack, enables the installation and loading of the model. The piston, which also serves as the base of the device, has a diameter of 524 mm with a stroke of 300 mm and allows the application of vertical stress σ_{v0} to the base of the soil mass using a pressurization device (air-water cell) (Khouaouci, 2023).

The soil model is contained within a waterproof neoprene membrane, with a thickness of 2 mm, and held by 2 O-rings at each base. The lateral confinement chamber is a tube section with a diameter of 650 mm and a thickness of 6.35 mm, allowing the application of the fixed horizontal stress σ_{h0} through the pressurized water, which is located between the soil mass (protected by the membrane) and the metal chamber. Among many boundary conditions, in terms of stress or strain that may be applied within the calibration chamber, it was chosen to simulate a soil layer at rest, at a given depth z within a semi-infinite mass, and subjected to cone penetration and cone loading, which leads to define the K_0-stress state at its boundaries, such as:

$$\sigma_{h0}= K_0.\sigma_{v0}= K_0.\gamma.z \tag{1}$$

K_0 is the at-rest coefficient of lateral pressure, and γ is the soil unit weight.

Experimental Devices Used on Clay

The Soil Material

The CLT-u test was carried out on a mass of clayey soil composed of speswhite kaolinite.

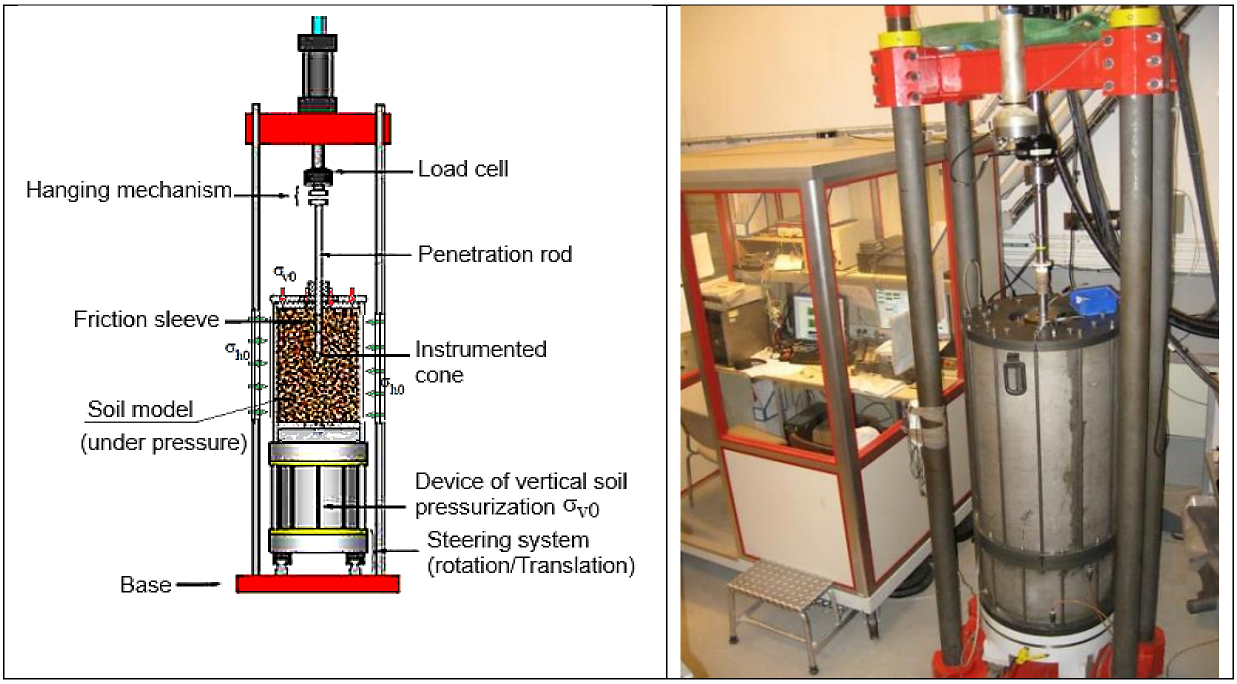

Figure (3): Scheme and photo of the calibration chamber components (Bekki, 2013)

This clay is widely used as a reference material in French geotechnical research laboratories, because it is high-swelling and has a relatively high permeability, which helps accelerate the consolidation phase. Extensive mechanical and physical laboratory tests were carried out to characterize this clay in detail, and their main results are summarized in Table 1.

The Piezocone

As illustrated in Figure (4), the electric piezocone used in the CLT-u test has a cone of a diameter of 36mm (an area of 10 cm²), an apex angle of 60°, and a total length of 28.8 cm, with the possibility of using extensions of various lengths. The lateral friction sleeve is 13.5 cm long (a lateral surface of 150 cm²). The piezocone is instrumented with several sensors; namely, a 25-kN load cell for measuring the tip resistance Q_c, a 25-kN load cell for measuring the lateral friction Q_s, and a pore water pressure sensor with a capacity of 2 MPa. The pore pressure measurement chamber is protected by a porous plastic ring (filter).

Table 1. Summary of the main geotechnical properties of the speswhite kaolinite

Property	Value	Comment
Specific density	2.64	
Plastic limit (%)	28	
Liquid limit (%)	58	
Plasticity index I_p (%)	30	
Passing at 80 µm (%)	98	
USCS class	CH	Highly plastic clay
Compression index C_c	0.41	
Swelling index C_s	0.07	High swelling clay
Initial void index e_0	1.32	
Water content used in tests (%)	45-50	
Bulk unit weight γ (kN/m^3)	17.10	
Compressibility $C_c/(1+e_0)$	0.160	Medium compressibility
Internal friction angle φ' (°)	21.6	CU+u triaxial test σ_c: Confining stress in kPa
Effective cohesion C' (kPa)	0.0	
Undrained shear strength C_u (kPa)	$9.68 + 0.21\sigma_c$	

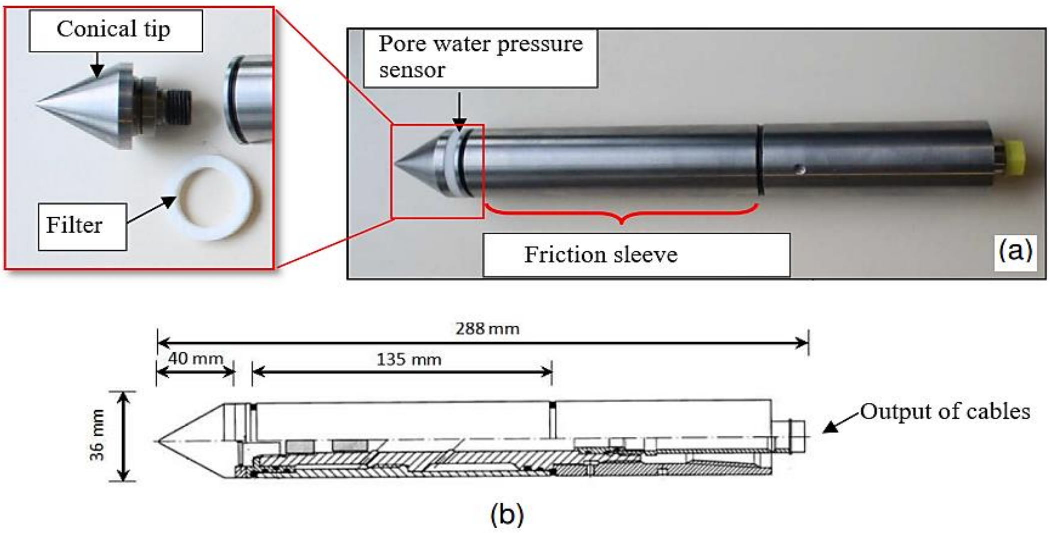

Figure (4): Photo (a) and scheme (b) of the piezocone used in the CLT-u test (Khouaouci, 2023)

The Consolidometer

As illustrated in Figure (5), the soil mass is prepared and consolidated within a consolidometer, consisting of a cylindrical plexiglass container, with internal and external diameters of 524 mm and 560 mm, respectively, and 900 mm high. The loading system and the consolidometer are assembled and connected to a data acquisition system.

To accelerate the consolidation of the clayey mass, the container is equipped with a lateral drainage system composed of 36 drainage holes distributed across 3 levels, allowing for radial drainage of the water outside the container.

Experimental Devices Used on Sand

The Soil Material

The sandy mass is prepared by using the Fontainebleau silica sand of class NE-34, also considered as a reference material used in many geotechnical research projects in France. The main geotechnical properties of this sand are summarised in Table 2 (Bekki, 2013).

The Penetrometer Cone

The CLT test was carried out on the sandy mass by using a standard cone penetrometer the cone of which is characterized by a diameter of 35.7 mm (10 cm² cross-section) and a length of 950 mm.

The load cells incorporated into the cone and the friction sleeve have, respectively, a capacity of 20 kN and ±5 kN. The ratio of the calibration chamber diameter to the CPT diameter is about 14.6.

The Hopper

To obtain a homogeneous sandy mass with controlled density, an automatic hopper is usually used, as illustrated in Figure (7). The sand pluviation method, commonly used for calibration chambers and centrifuge tests, relies on the principle that the sand density achieved through pluviation primarily depends on the sand flow rate and the free fall height by pluviation.

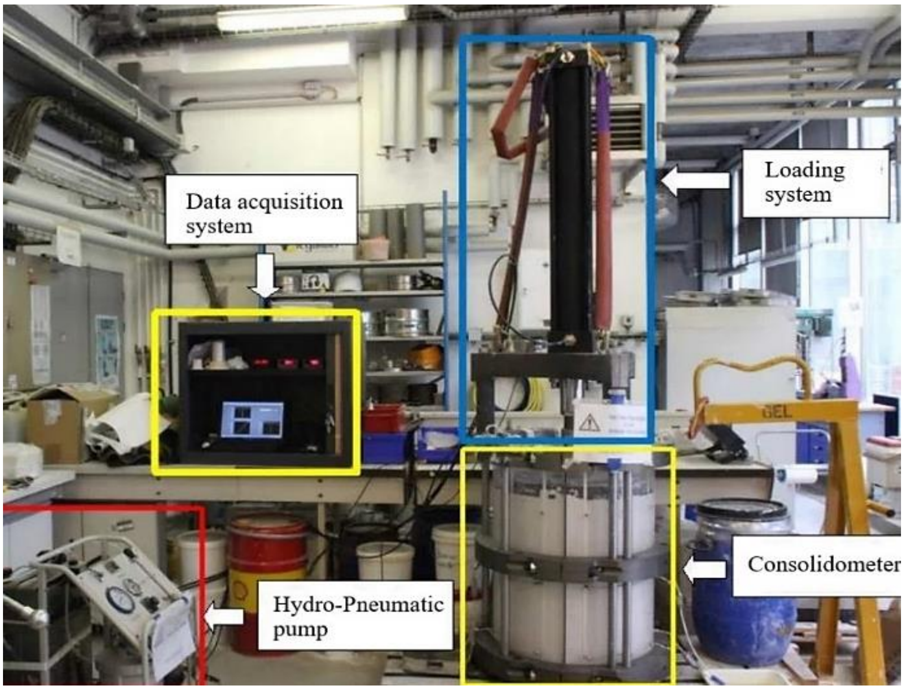

Figure (5): Photo illustrating the consolidometer (Khouaouci, 2023)

EXPERIMENTAL PROCEDURE IN THE CALIBRATION CHAMBER

Tests on Clay

The laboratory procedure for conducting the CLT-u test on clay includes the following steps:

1) Mixing the speswhite kaolin powder with water within the container.
2) Consolidation of the clayey mass under a vertical pressure σ_{v0} equal to that applied further in the calibration chamber tests, which may take up to 24 days to 59 days.
3) Installing the clayey mass within the calibration chamber and applying the radial stress σ_{h0} and the vertical stress σ_{v0}, according to Table 3.
4) Jacking the piezocone into the soil mass at a rate of 1 mm/s up to the given depth.
5) Configuring the test parameters in the control software and initiating the test.

Table 2. Summary of the main geotechnical properties of Fontainebleau sand

Property	Value	Comment
Specific density	2.65	
Median grain size D_{50} (mm)	0.21	
Coefficient of uniformity C_U	1.49	
Coefficient of curvature C_c	1.00	
Passing at 80 µm (%)	0.0	
USCS class	Sp	Poorly graded sand
Maximum void index e_{max}	0.882	
Minimum void index e_{min}	0.551	
Maximum dry unit weight γ_d^{max} (kN/m^3)	17.1	
Minimum dry unit weight γ_d^{min} (kN/m^3)	14.1	
Maximum internal friction angle φ (°)	43.9	Corresponding to I_D=100%
Minimum internal friction angle φ (°)	31.1	Corresponding to I_D=0%
Effective cohesion C' (kPa)	0.0	

Tests on Sand

The loading procedure for the CLT test on sand involves the following successive operations:

1) Creating the sand mass with the desired density by using the technique of air pluviation from the sand hopper.
2) Placing the sandy mass in the calibration chamber and applying the radial stress σ_{h0} and vertical stress $_{\sigma v0}$, as indicated in Table 3.
3) Following steps 4 to 5 as in the clay tests.

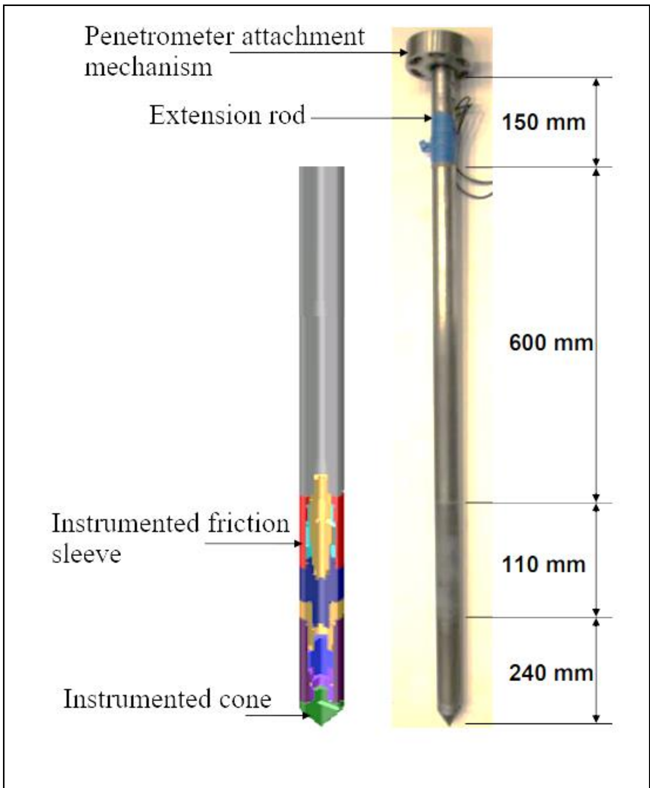

Figure (6): Photo illustrating the penetrometer

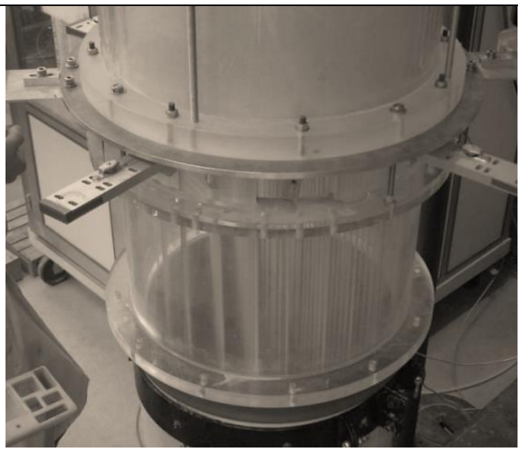

Figure (7): Photo of the sand hopper (Bekki, 2013)

PROGRAM OF TESTS

The CLT and CLT-u tests were conducted in the calibration chamber using a static penetration of the penetrometer cone into the soil mass, followed by three phases of loading. Two monotonic loading sequences at the specified depth are applied in the first phase. The second phase consisted of an extensive series of two-way cyclic loading up to 10^5 cycles, considered to date a world record for tests conducted in a calibration chamber. The last phase focuses on the post-cyclic behaviour of the penetrometer cone subjected to two monotonic vertical loads. In this paper, only the results of the first phase are presented.

In all the tests, the penetration test is conducted at a rate of 1 mm/s, which is significantly slower than the typical rate used in a standard cone penetration test (CPT) of 20 mm/s. This slower rate is necessary to ensure that the clayey material exhibits drained behaviour.

After reaching the designated depth within the clayey soil, a relaxation period of 12 hours is kept to allow for the complete dissipation of excess pore water pressure within the clay.

As depicted in Figure (8), the first loading sequence is performed using displacement-controlled loading with a rate of 0.5 µm/s, followed by a force-controlled unloading at a rate of 30 N/s. This is then followed by a relaxation period of 2 hours before starting the second loading sequence, which is conducted at a rate of 5 µm/s for loading and 10 N/s for unloading.

The protocol for the loading test on sand is identical to that on clay, except for the absence of a relaxation time, with the rates of loading/unloading indicated in brackets in Figure (8).

As summarized in Table 3, the CLT-u test was conducted on two clayey masses, M1C and M2C, each representing a slice surrounding the piezocone at a specified depth. Additionally, 5 CLT tests were performed on sandy masses (M1S through M5S).

The full-scale depth z simulated by each test corresponds to the vertical stress (σ_{v0}) applied to the model, divided by the bulk unit weight γ, as outlined in Equation (1). The coefficient K_0 was set to 0.6 for clay and 0.4 for sand in all tests.

To ensure that the results were representative, the repeatability of the tests in the calibration chamber was controlled by conducting additional tests on both M1C and M1S.

Table 3. Summary of the main parameters of CLT/CLT-u tests

	Speswhite Kaolinite (Clay)				Fontainebleau (Sand)					
Model	**σ_{v0} (kPa)**	**σ_{h0} (kPa)**	**K_0**	**Depth z (m)**	**Model**	**σ_{v0} (kPa)**	**σ_{h0} (kPa)**	**K_0**	**Depth z (m)**	**I_D (%)**
M1C	250	150	0.6	14.64	M1S	125	50	0.4	8.20	40
M2C	125	75	0.6	7.32	M2S	250	100	0.4	16.40	40
					M3S	500	200	0.4	32.9	40
					M4S	125	50	0.4	7.85	65
					M5S	250	100	0.4	15.70	65

PRESENTATION AND DISCUSSION OF RESULTS

Cone Penetration Test (CPT) on Sand

1) Presentation of Results

Figure (9) illustrates typical profiles of cone penetration in Fontainebleau sand. The cone resistance q_c exhibits a rapid mobilization over the first 100 mm of penetration, followed by a stabilization around 4 MPa, with a slight increase due to the proximity of the base of the container.

As can be seen in Figure (9), the sleeve friction f_s remains zero until a cone penetration of 150 mm, because f_s is calculated from the lateral surface of the friction sleeve of 135-mm length, and up to this depth, it remains outside the soil mass. Once the friction sleeve enters the soil mass, the friction gradually increases and reaches a value of 30 kPa.

For each sandy mass, the stabilized values of q_c and f_s were retained as representative values and, therefore, correspond to the depth z of the slice simulated by the sandy mass in the calibration chamber, as indicated in

Table 3. As shown in Figure (10), the cone resistance profile at a given density was constructed point by point, each point corresponding to the value of q_c representative of a sandy mass.

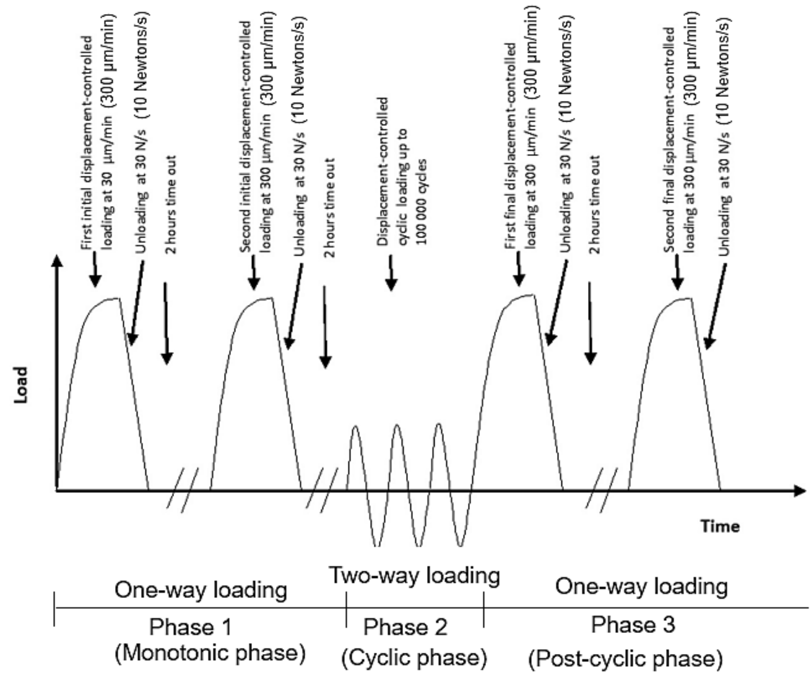

Figure (8): Program of the CLT/CLT-u tests in the calibration chamber (Khouaouci, 2023)

At a given density, the cone resistance exhibits a linear increase in depth, similar to the profile of the initial vertical overburden pressure σ_{v0}. In this regard, the normalized cone resistance q_c^1 is usually defined as the ratio of these two profiles, such that:

$$q_c^1(z) = \frac{q_c(z)}{\sigma_{v0}(z)}\,. \qquad (2)$$

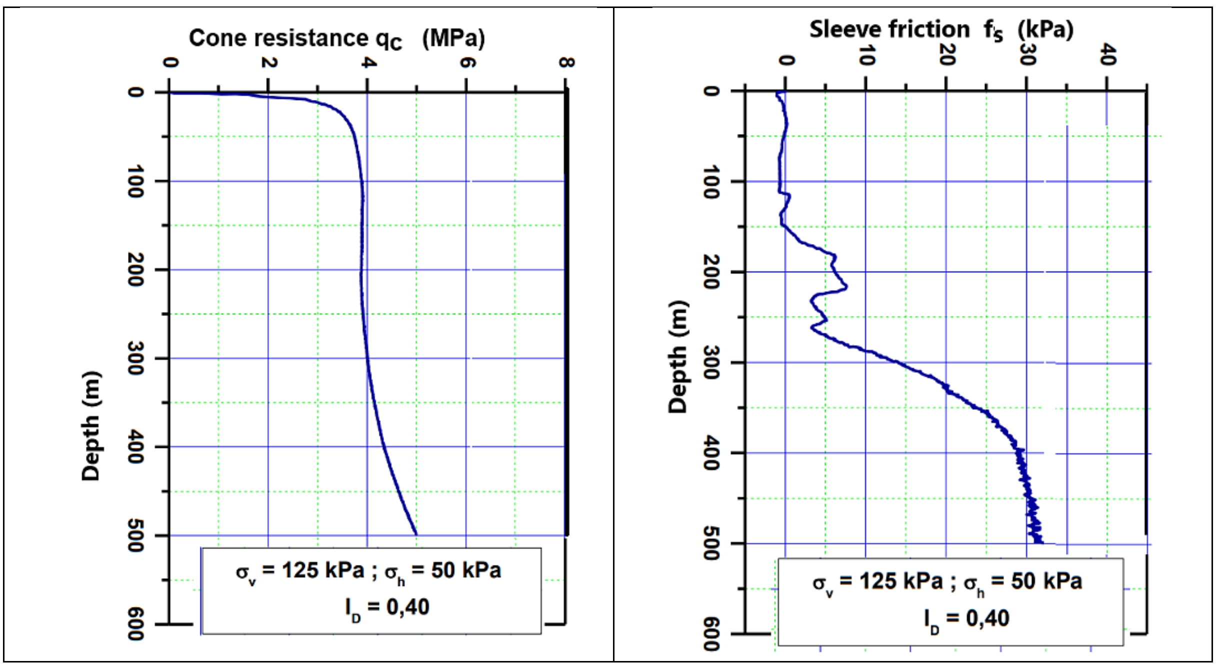

Figure (9): Typical profiles of penetration in Fontainebleau sand (Mass M1S) (Bekki, 2013)

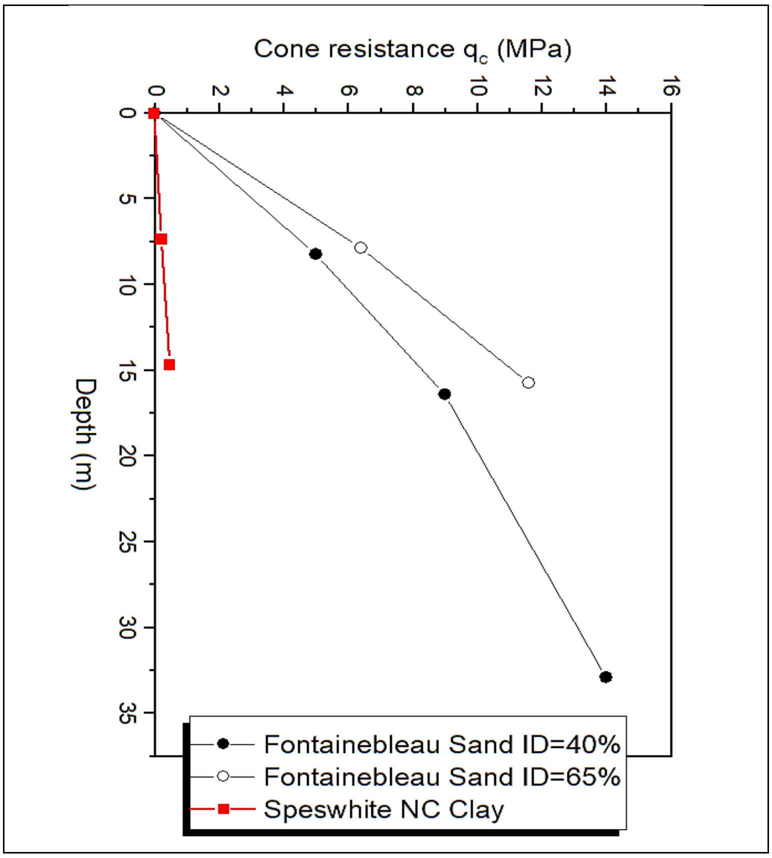

Figure (10): Profiles of the cone resistance in sand and clay

2) Effect of Sand Density

The values of q_c^1 increase with density from an average of 34.6 for I_D=40% to 48.8 for I_D=65%. To comprehensively evaluate the experimental variation of the cone resistance with the density index, to these values obtained in the calibration chamber, other experimental data obtained elsewhere were added. Previous research works by physical modeling of small-scale piles were carried out on the Fontainebleau sand within the IFSTTAR (Institut Français des Sciences et Technologies du Transport, de l'Aménagement et des Réseaux) centrifuge, Nantes, France (Mezazigh, 1995; Remaud, 1999; Hajialilue, 2003; Rosquoet, 2004; Rakotonindriana 2008). In these research works, penetration tests at various densities were carried out by a small-scale cone penetrometer. As shown in Figure (11), in a bi-logarithmic scale, the normalized cone resistance q_c^1 increases linearly with the density index I_D (expressed to unity), suggesting the following power function as the best fit of the experimental data (regression coefficient R=91%):

$$q_c^1 = 214 I_D^2. \quad (3)$$

This empirical correlation is specific to the Fontainebleau sand, which is an unaged, uncemented, normally consolidated quartz sand, and there are similar correlations cited in the literature for other sands exhibiting different properties (Lunne et al., 1997).

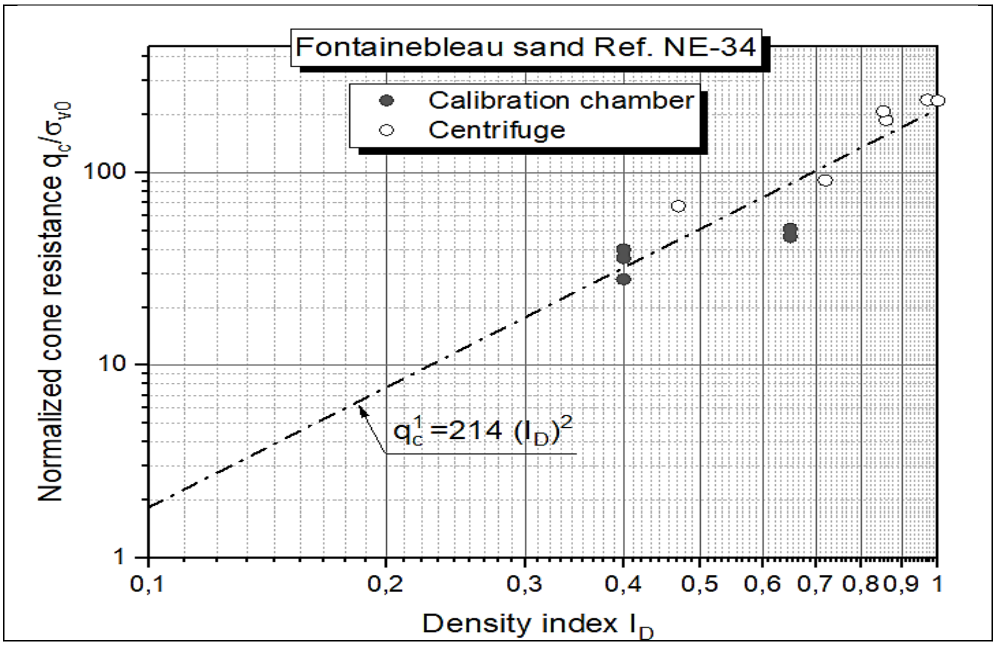

Figure (11): Chart of the correlation q_c^1-I_D for the Fontainebleau sand

Based on the standard ISO 14688-2, which gives an international classification of sand according to its density (AFNOR, 2005), one can use the previous equation to define the margins of density as a function of q_c^1, as summarized in Table 4. One practical implication of this classification is the adoption of 26 as the threshold value of q_{c1}, above which the Fontainebleau sand is no longer loose and, therefore, not susceptible to liquefaction.

3) Shear Strength of Sand

Since the angle of internal friction φ characterizes the shear resistance of clean dry sand, it was correlated to the normalized cone resistance q_c^1, as shown in Figure (12). An exponential increase of q_c^1 as a function of φ can be seen.

Additionally, this variation aligns very well with the theoretical values provided by Willson and Smith (1988) and Durgunoglu and Mitchell (1975), with a coefficient of correlation of 75%.

Willson and Smith carried out finite element modeling of the problem of cone penetration within an elastic-plastic medium, obeying Mohr-Coulomb's failure criterion, and fitted their results by the following exponential function:

Table 4. Classification of sand density based on q_c^1

Density	Margin of I_D (%)	Margin of q_c^1
Very loose	0-15	0.0-4.8
Loose	15-35	4.8-26.0
Medium	35-65	26.0-90.4
Dense	65-85	90.4-154.6
Very dense	85-100	154.6-214.0

$$q_c^1 = \frac{1+\sin(\varphi)}{1-\sin(\varphi)} \exp\left[\left(4\varphi + \frac{\pi}{3}\right) tg(\varphi)\right] \qquad (4)$$

Durgunoglu and Mitchell (1975) analyzed this problem using the bearing capacity theory, modeling the soil as a rigid-plastic material. For practical purposes, this chart is very useful for the design of foundations and retaining walls on this sand based on the CPT test.

4) Sleeve Friction in Sand

Concerning sleeve friction f_s, it was observed that the normalized sleeve friction f_s/σ_{v0} remains nearly constant at a value of 0.3. Therefore, it is independent of sand density.

Piezo-cone Penetration Test (CPT-u) on Clay

1) Presentation of Results

Figure (13) illustrates typical profiles of cone penetration in speswhite clay, where the cone resistance and the excess pore water pressure exhibit progressive mobilization up to 65 mm of penetration, followed by a stabilization of around 0.22 MPa (Figure (13a)) for q_c and 105 kPa for the excess pore water pressure (Figure 13c)).

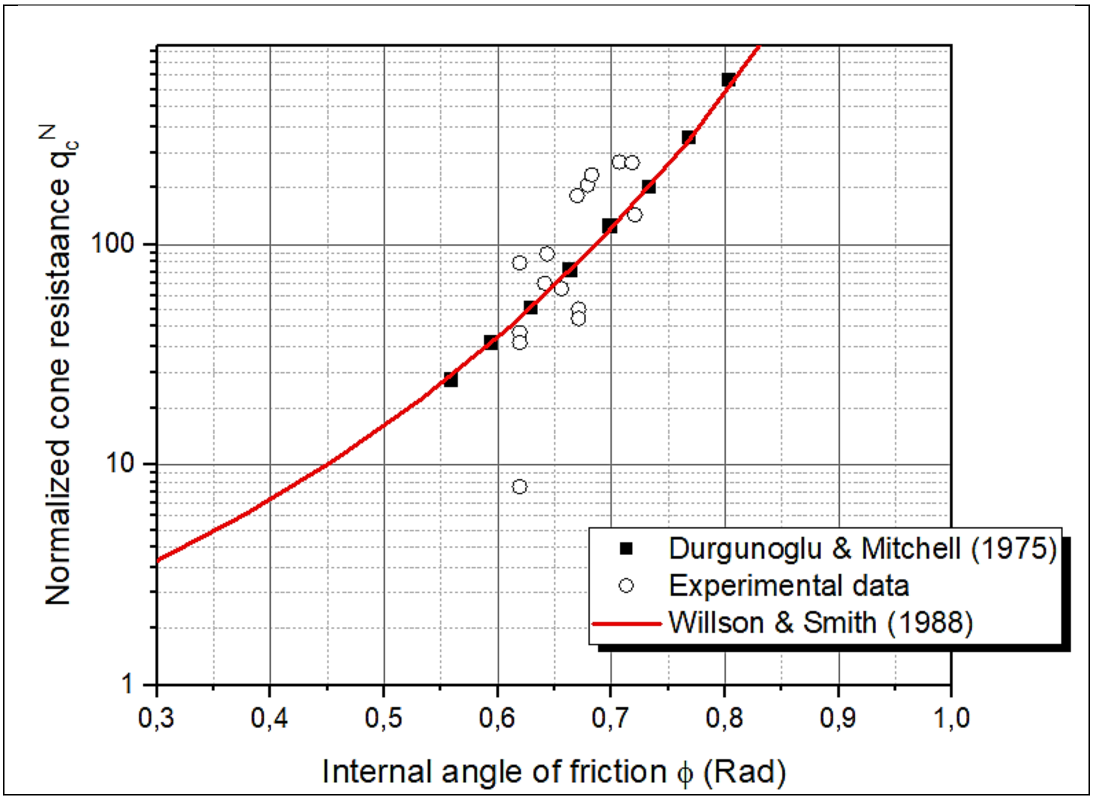

Figure (12): Chart of q_c^1- φ for the Fontainebleau sand

In Figure (13b), an increase can be observed in the local friction sleeve f_s up to a peak value of 9 kPa at a cone penetration of 150 mm corresponding to the end of the friction sleeve, followed by a decrease and a stabilization at a value of 6.2 kPa.

As shown in Figure (10), and similar to the Fontainebleau sand, the profile of cone resistance linearly increases with depth, which is typical of a normally consolidated homogeneous clay.

It was found that the normalized cone resistance q_c^1, as formulated by Equation (2), is nearly constant with a value of 1.85, whatever the depth.

2) Effective Shear Strength of Clay

Table 5 presents a summary of the results from the CPT-u test, where the effective cone resistance q_c' was calculated as the difference between the total cone resistance q_c and the pore-water pressure u. Additionally, the normalized effective cone resistance q_c^{1}' was determined according to Equation (2), using the effective initial vertical stress σ_{v0}'.

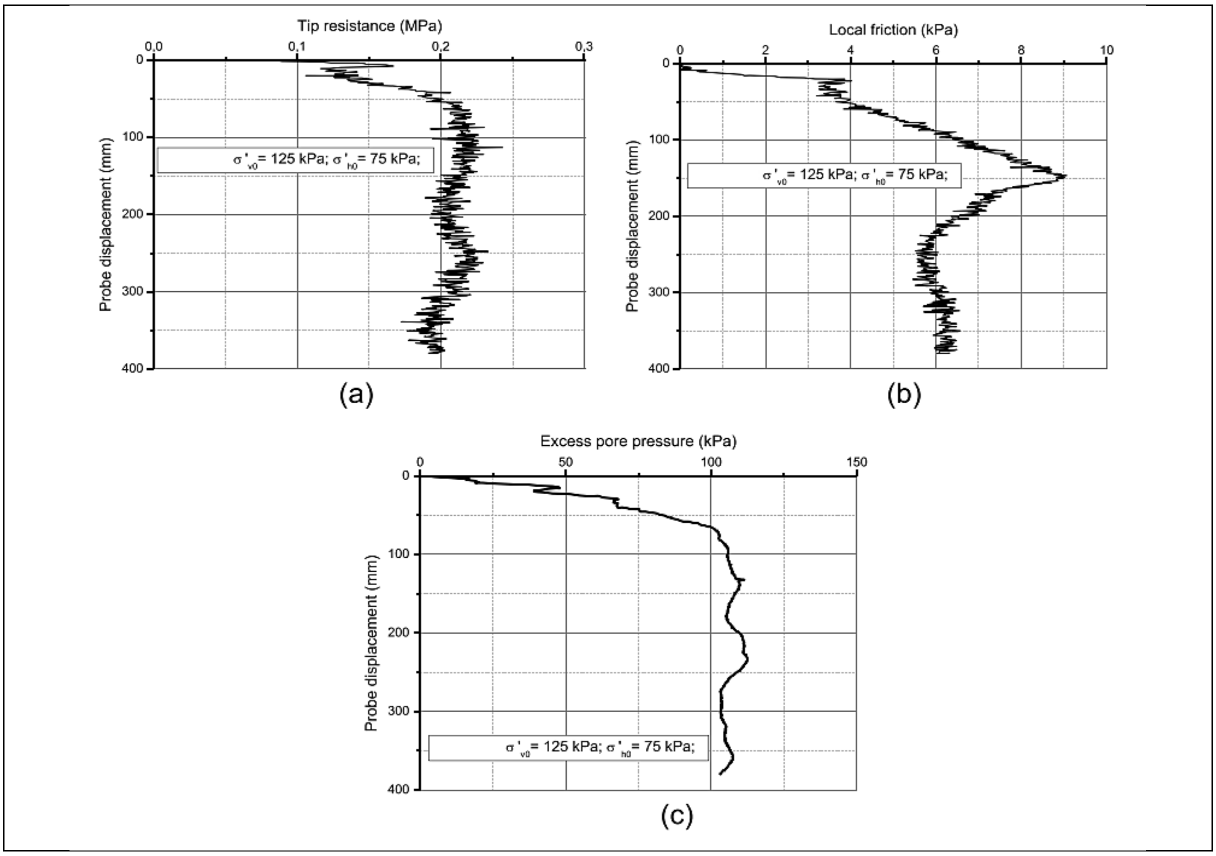

Figure (13): Typical profiles of penetration in speswhite clay (Mass M2C) (Khouaouci, 2023)

Table 5. Summary of the results of the CPT-u test

Mass	σ_{v0} (kPa)	σ_{h0} (kPa)	q_c (MPa)	u (kPa)	q_c^1	Depth (m)	σ_{v0}' (kPa)	q_{c1}'	f_s (kPa)	S_t
M1C	250	150	0.480	244	1.92	14.6	103.5	2.28	18.5	3.36
M2C	125	50	0.220	105	1.76	7.3	51.7	2.22	5.70	6.30

It was attempted to simply formulate the effective normalized cone resistance q_c^{1}' as follows:

$$q_c^{1\prime} = \frac{1+\sin(\varphi')}{1-\sin(\varphi')} = K_p. \quad (5)$$

By entering the value of 21.6° as the measured effective angle φ' of friction into this equation, the computed q_c^{1}' is equal to 2.16, which is in very good accordance with the average experimental value, which is 2.25. This equation involves the passive earth pressure coefficient K_p derived from Rankine's theory of earth pressures on retaining walls. This statement may lead to a simple interpretation of the cone penetration as a mobilization of the passive limit equilibrium state of the clay around the cone.

3) Sleeve Friction f_s and Sensitivity of Clay

According to Lunne et al. (1997), it is usually recognized that the sleeve friction f_s, measured in an electrical cone, is approximately equal to the remoulded undrained shear strength. In this regard, the clay sensitivity S_t may be evaluated as:

$$S_t = \frac{C_u}{f_s}\,. \quad (6)$$

C_u is the intact undrained shear strength depending on the confining pressure, as mentioned in Table 1. Equation (6) led to the values of S_t summarized in Table

5, indicating that this clay may be described as of medium-to-high sensitivity (Mitchell and Soga 2005).

Cone Loading Test (CLT) on Sand

1) Presentation of Results

Figure (14) illustrates typical load-settlement curves at the cone as well as at the friction sleeve, where a regular increase of the vertical pressure q_p and the sleeve friction f_s with the cone penetration v_p up to an asymptote was observed, corresponding to the full mobilization of resistance.

These load-transfer functions were fitted based on the least squares technique by a simple analytical function called PARECT (PArabola-RECTangle), which can capture the non-linear shape of the curve, as well as the horizontal asymptote from a threshold displacement v_p^R called "reference displacement". This function is defined as follows:

$$Y(z) = E_y(z).v_p(z).\left(1 - \frac{v_p(z)}{2v_p^R(z)}\right). \quad (7)$$

Y is q_p or f_s, and E_y is the initial slope of the curve to be fitted (E_c or E_f). The reference displacement may be formulated as follows, Y_u being q_l^{clt} or f_s^{clt}:

$$v_p^R(z) = 2\frac{Y_u(z)}{E_y(z)} . \quad (8)$$

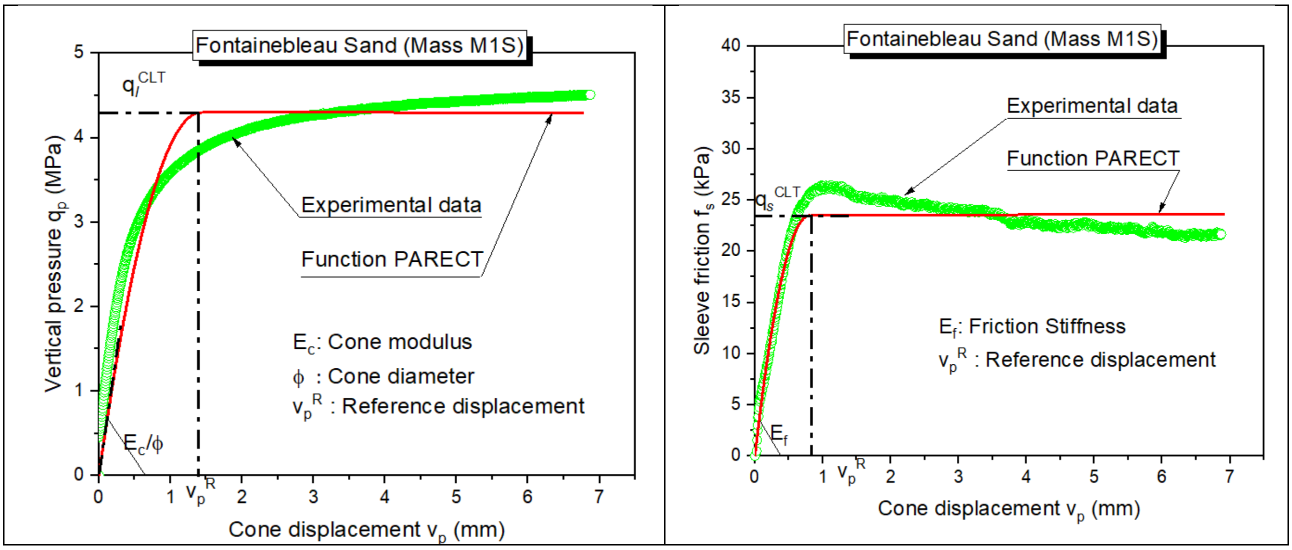

Figure (14): Typical load-transfer curves of the CLT test

2) The CLT Cone Resistance q_l^{clt} and Limit Sleeve Friction f_s^{clt} in Sand

The CLT cone resistance q_l^{clt}, defined as the asymptotic value of the load-settlement curve of the cone, was found to be almost equal to the cone resistance q_c measured during the CPT test. Similarly, the limit sleeve friction f_s^{clt} was also found to be nearly equal to the sleeve friction f_s measured during the CPT test.

Given that loading rates during the CPT and CLT tests were, respectively, 1 mm/s and 0.005 mm/s, it appears that the cone resistance q_l^{clt} and the limit sleeve friction f_s^{clt} do not depend on the loading rate for this sand.

3) Analysis of the Cone Modulus E_c and Friction Stiffness E_f in Sand

As shown in Figure (15), the profiles of the cone modulus E_c and the friction stiffness E_f in Fontainebleau sand are almost linear, meaning a regular increase of the soil stiffness with depth. Additionally, there is a slight dependence on the density of sand. This characteristic of soil stiffness is often seen in what are termed Gibson's soils, which include homogeneous sandy layers or saturated, normally consolidated clays.

As a reference material, the Fontainebleau sand was subjected during many years to an extensive series of laboratory tests to measure its stiffness, and the maximum shear modulus G_{max}, corresponding to very small strains (up to 10^{-5}) was obtained by using some laboratory dynamic tests; namely, the resonant column, the bender elements and the hollow cylinder. A simple empirical formulation of G_{max} for the Fontainebleau sand was then proposed as follows:

$$G_{max} = \frac{57.75}{e}\left(\frac{p}{p_{ref}}\right)^{0.56} . \quad (9)$$

p and e are, respectively, the mean effective stress and the void index, and p_{ref} is a reference stress equal to 100 kPa. This equation is dimensionally non-homogeneous, where p is in kPa and G_{max} in MPa.

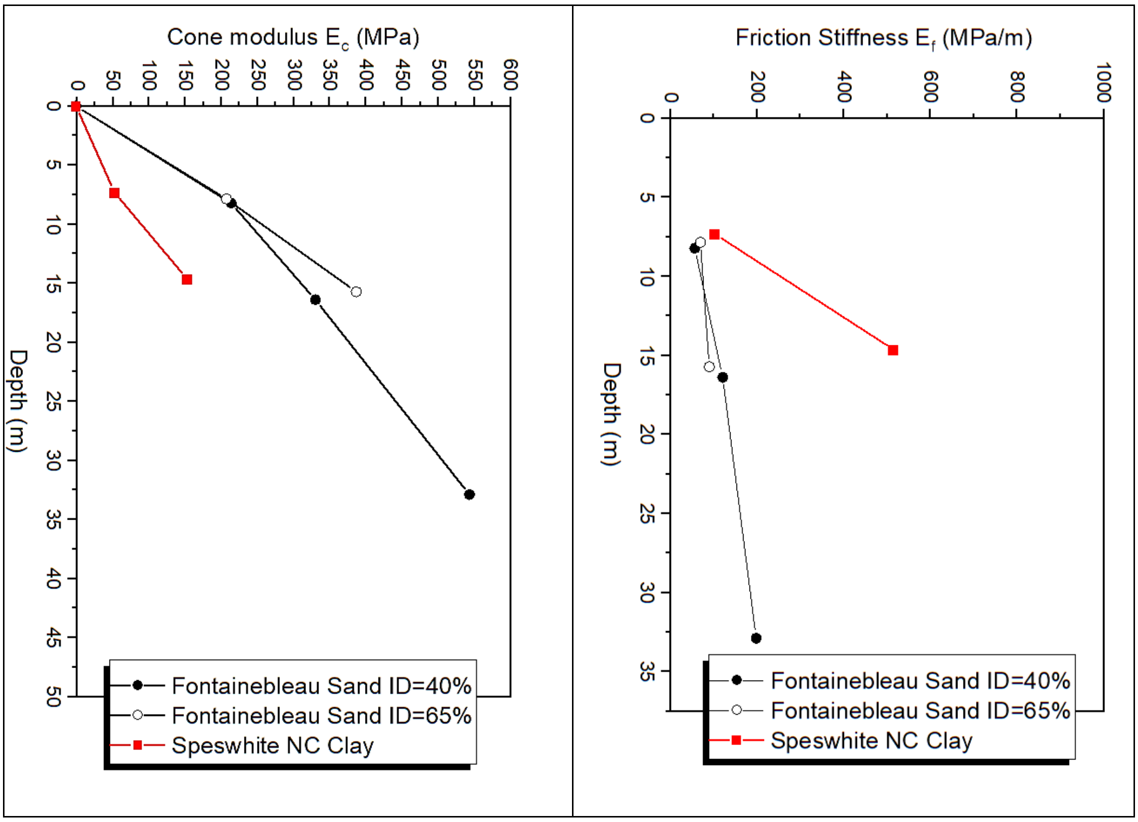

Figure (15): Profiles of the cone modulus in sand and clay

For each sandy mass tested in the calibration chamber, G_{max} was computed, and Figure (16) shows that E_c and ($E_f.\Phi$) are linearly proportional to G_{max} according to the following equations, where Φ is the cone diameter (Φ=35.7 mm):

$$E_c = 3.52G_{max}; \tag{10}$$

$$E_f B = 0.04G_{max}. \tag{11}$$

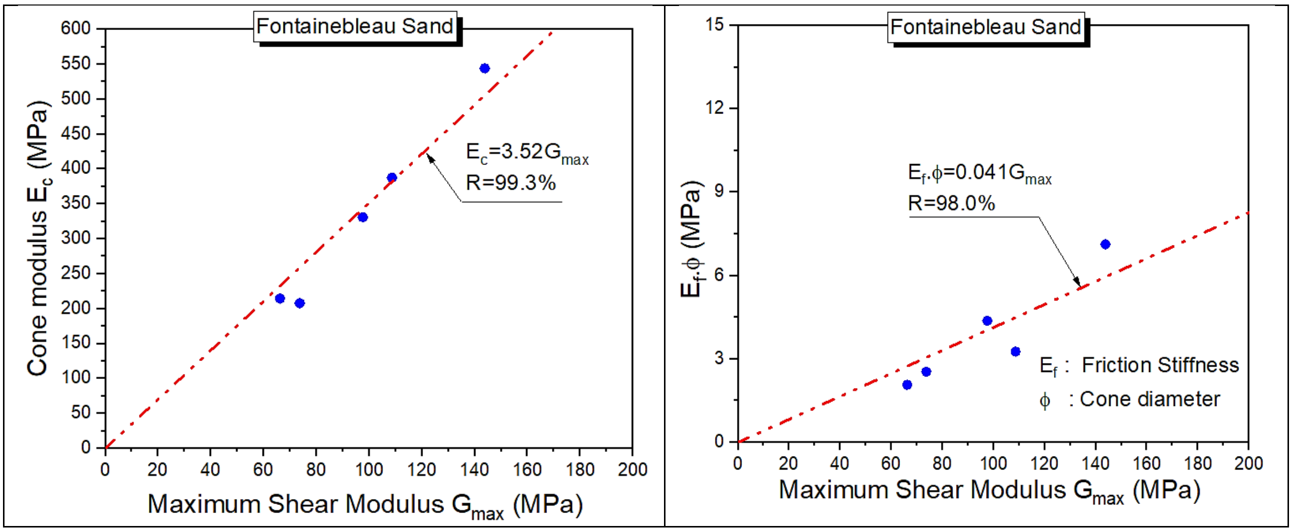

Figure (16): Experimental charts E_c-G_{max} and E_fB-G_{max} for Fontainebleau sand

These equations provide a useful method for estimating the sand deformation modulus, denoted as G_{max}, by utilizing the stiffness values E_c and E_f measured by the CLT test. Subsequent CLT tests on other sandy materials may provide a general formula to be used to estimate the deformation modulus of sand.

Cone Loading Test (CLT-u) in Clay

1) Presentation of Results

The load-transfer curves in clay are similar to those shown in Figure (14). For brevity, typical curves will not be shown.

2) The CLT Cone Resistance q_l^{clt} and Limit Sleeve Friction f_s^{clt} in Clay

As summarized in Table 6, the CLT-u test conducted within the clayey mass resulted in low values of the pore water pressure u, suggesting a drained behaviour. However, it was found that the value of the CLT cone resistance q_l^{clt} is about twice that of q_c obtained in the CPT, and similarly, the limit sleeve friction f_s^{clt} is 2-3 times f_s measured in the CPT test. One possible explanation of the increase in cone resistance and sleeve friction is that previous primary consolidation during the CPT test mobilized relatively higher values of the pore water pressures, as shown in Table 5, and increased the effective shear strength of clay. Using Equation (5) and entering (q_l^{clt}'/σ_{v0}') instead of (q_c'/σ_{v0}') resulted in effective friction angles φ' of 34.7° for M1C and 32.8° for M2C, indicating a significant increase in φ' compared to the value of 21.6°.

Table 6. Summary of the results of the CLT-u test

Mass	q_l^{clt} (MPa)	f_s (kPa)	u (kPa)	q_l^{clt}/q_c	f_s^{clt} (kPa)	f_s^{clt}/f_s
M1C	0.876	18.5	15	1.825	40.9	2.20
M2C	0.400	5.70	6	1.820	17.3	3.03

3) Analysis of the Cone Modulus E_c and Friction Stiffness E_f in Clay

Similar to Fontainebleau sand, the speswhite clay exhibits a regular increase of stiffnesses as a function of depth, as illustrated in Figure (15).

Based on the results of triaxial compression tests conducted on samples of speswhite clay under different values of mean stress p, the initial triaxial modulus E_0 was obtained. According to Figure (17), the measured stiffnesses E_c and E_f were found to be linearly increasing with the mean stress as follows:

$$\frac{E_c}{E_0} = 1.76\left(\frac{p}{p_{ref}}\right); \tag{12}$$

$$\frac{E_f\Phi}{E_0} = 0.19\left(\frac{p}{p_{ref}}\right). \tag{13}$$

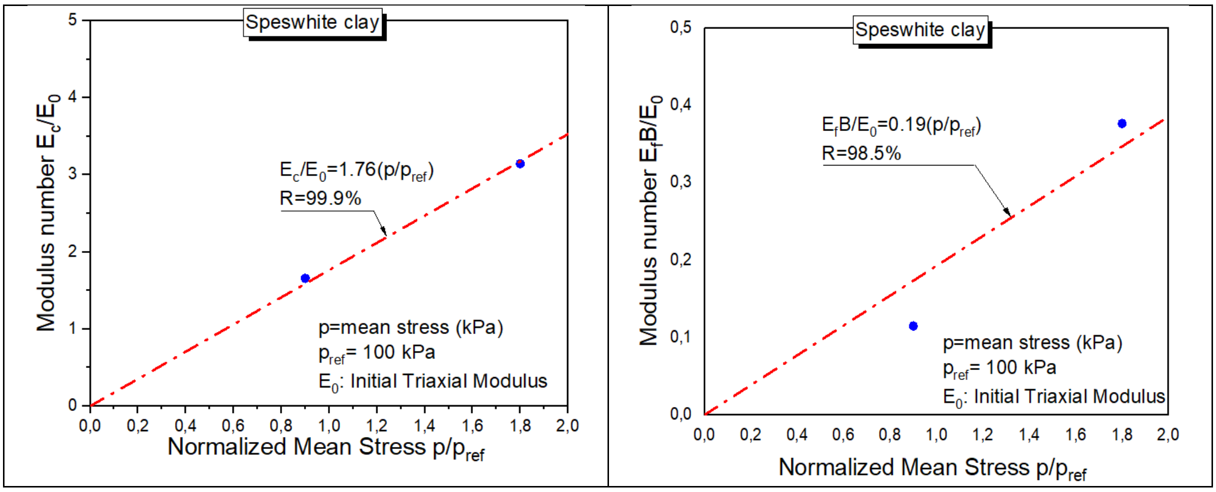

Figure (17): Experimental charts E_c/E_0-p/p_{ref} and E_fB-p/p_{ref} for speswhite clay

The mean stress p is computed as:

$$p = \frac{\sigma_{v0} + 2\sigma_{h0}}{3}. \tag{14}$$

The two previous equations provide a practical means to estimate the deformation modulus of this clay based on E_c or E_f measured by the CLT test.

ILLUSTRATIVE EXAMPLE OF THE APPLICABILITY OF THE CLT TEST

The practical aspects of the correlations presented above based on the CLT test are demonstrated through the following numerical example, describing the project of an embedded cylindrical diaphragm wall retaining a bi-layered soil. As shown in Figure (18), the top layer is composed of a soft, saturated, normally consolidated clay, overlying a deep, fine, saturated sand. The project site belongs to a seismically active zone, and the representative values of the CLT test results are summarized.

A finite element analysis of the long-term behavior of the wall in axisymmetric conditions is to be conducted, and the parameters of an elastic-perfectly plastic stress-strain law for the soil are required; namely, Mohr-Coulomb's parameters (cohesion and angle of friction), the elastic soil modulus, and Poisson's ratio.

- Clay: Based on the effective normalized cone resistance $q_c^{1'}=3$, Equation 5 gives an effective friction angle φ' equal to 30°. It is usually assumed that the effective cohesion of a normally consolidated clay c'=0 and the Poisson ratio ν'=0.33.

  Equation (14) gives $p=\sigma_{v0}'$, because $K_0=1$. E_c increases linearly with depth, which implies from Equation (12): E_0=56.8 MPa, whereas from equation (13), one finds E_0=58.2 MPa, and an average value of 57.5 MPa may be taken as an estimation of the elastic modulus.
- Sand: Equation (4) gives φ'=30.8°. The cohesion is taken 0, and ν' is estimated to be 0.33.

  Based on Equations (10) and (11), the small-strain shear modulus G_{max} equals 71.0 MPa and 72.3 MPa, with an average value of 71.6 MPa. The elastic modulus $E_0 = 2(1+\nu)G_{max}$=190.5 MPa. Additionally, Equation (3) gives a density index I_D equal to 31%, which corresponds, according to Table 4, to a loose sand and may be liquefiable during a seismic event.

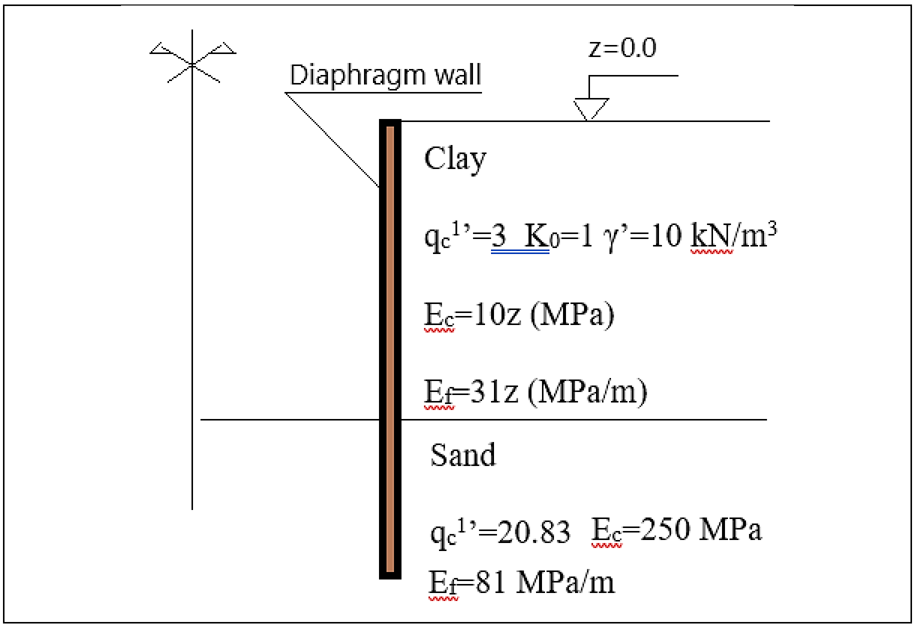

Figure (18): Illustrative example of the applicability of CLT test

CONCLUSIONS

In this paper, an innovative *in-situ* geotechnical test, the Cone Loading Test (CLT/CLT-u), was described and modelled in full scale in the calibration chamber. The main feature of this test is its ability to measure the soil resistance as well as its stiffness, combining the advantages of two commonly used *in-situ* tests; namely, the CPT and PMT tests. Based on the experimental results obtained during the penetration test (CPT) and cone loading test (CLT/CLT-u) conducted on models of Fontainebleau sand and speswhite kaolinite saturated clay in a calibration chamber, practical correlations were established and compared to the conventional models available in geotechnical literature. The major conclusions of this work can be summarized as:

- The normalized cone resistance q_c^1 of sand was found to vary as a power function of the density index I_D, as well as an exponential function of the internal angle of friction φ; providing a practical tool to estimate the sand density as well as its angle of friction based on the CLT test.
- In speswhite clay, the effective cone resistance $q_c^{1'}$ showed very good correlation with the effective internal angle of friction φ', according to a simple model of Rankine's passive limit equilibrium state. This straightforward relationship provides an efficient method for estimating the drained angle of friction of the saturated clay. However, being limited to only two standard materials, these correlations should be extended and confirmed by studying other materials using the CLT/CLT-u tests.
- The normalized sleeve friction f_s/σ_{v0} in sand remains nearly constant at a value of 0.3, and is therefore independent of sand density.
- The load-transfer curves describing the soil/cone and soil/sleeve interaction were derived and fitted by the PARECT function, which led to defining, respectively, the cone modulus E_c and the friction stiffness E_f. These parameters were found to be proportional to the small-strain shear modulus G_{max} in sand, the mean stress p, and the initial triaxial modulus E_0 in clay.

It was demonstrated that these two deformability parameters may simply be used to estimate the soil deformation modulus based on the CLT test.

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Conflict of Interests

The authors declare that there is no conflict of interests regarding this work.

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