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Shallow foundation design based on load-transfer curves obtained from DCLT test

Conception des fondations superficielles basée sur les courbes charge-tassement obtenue à partir de l'essai DCLT

Philippe Reiffsteck, Tuan Anh Luong & Fabien Szymkiewicz
Université Gustave Eiffel, IFSTTAR, 77420 Marne la Vallée, France

Miguel Angel Benz Navarrete, Quoc Anh Tran & Caroline Forestti Oliveira
Research and Development, Sol Solution, 63204 Riom Cedex, France

Alexandre Teyssier & Catherine Jacquard
Research and Development, FONDASOL, 84000 Avignon, France

ABSTRACT: This paper proposes a methodology for the calibration of the curves from the dynamic cone loading test (DCLT) and the settlement curves of the shallow foundations according to the load-settlement curve method. A procedure for the analysis of obtained DCLT curves is proposed to separate the dynamic and static components. The fitting of a hyperbolic model and normalization with respect to cone resistance is carried out. The obtained DCLT curves are compared and adjusted based on different tests carried out either in the laboratory or in situ. The results show the practical and technical feasibility of the method.

RÉSUMÉ : Cet article propose une méthodologie pour le calage des courbes issues de l'essai de chargement dynamique de cône (DCLT) et les courbes de tassement des fondations superficielles selon l'approche proposée par la méthode de la courbe charge-tassement. Une procédure pour l'exploitation des courbes de l'essai DCLT est proposée afin de découpler les composantes dynamique et statique. L'ajustement d'un modèle hyperbolique et une normalisation par rapport à la résistance à la pénétration sont effectués. Les courbes DCLT obtenues sont comparées et ajustées sur la base de différents essais réalisés soit au laboratoire soit in-situ. Les résultats montrent la faisabilité pratique et technique de la méthode.

KEYWORDS: Shallow Foundation, Dynamic Cone Penetrometer, Load-Settlement Curve, Dynamic Cone Loading Test, DCLT.

1 INTRODUCTION.

Shallow foundations are considered as the most geotechnical structures encountered in civil engineering, because it represents the simplest form of load transfer from a structure to the soil beneath. In most cases, shallow foundations are the most cost-effective choice for support of a structure. However, its design is an essential requirement to ensure the general stability of the structure as well as its long-term durability.

Conventional design of shallow foundations consists of evaluating the bearing capacity and estimating its settlements based on the applied loads. Conventional methods such as the one-dimensional approach often underestimate settlement, which leads to overly optimistic design or differential settlement, both being detrimental to the structures (Poulos et al. 2001).

Independent of the design method, the input parameters are mainly obtained through in-situ tests or by means of full-scale loading tests, such as plate load test. In-situ tests are preferred because they are simple, fast, and cost-efficient techniques with a high vertical resolution, which facilitates the identification of soil layers as well as their spatial variability.

Among the various field tests, static and dynamic cone penetration tests (respectively, CPT and DPT) are widely used in many countries around the world for site exploration and soil characterization. CPT & DPT provide one major measure, the cone resistance (*qc* or *qd*), which is related to the failure strength of soils. However, in conventional procedures, these tests only allow a fairly rough assessment of soil deformation, since no relationship between load and settlement (stress strain relation) is obtained during the test. Some modifications in the test procedure would make it possible to respond to this shortcoming.

Due to its similarity with a circular embedded foundation, the measurements of settlement of an incrementally loaded CPT tip have been earlier developed to measure compressibility of soils (Haefeli and Fehlmann 1957; Ladanyi 1976).

Most advanced experiences have been achieved in France (Gourvès 1979; Faugèras et al. 1983; Arbaoui et al. 2006), where (Reiffsteck et al. 2008, 2009) adapted and applied this procedure in order to obtain cone load-penetration curve. This test is known as CLT (Cone Loading Test) test and allows to assess soil deformability as well as to obtain input data for design of foundations. Nevertheless, beyond the factors affecting the measurements (Ali et al. 2009), limitations related to the time required and thus the quantity of tests that can be carried out are the main limitations for the application of the CLT test for shallow soil characterization.

Thus, the development of a fast and simple method, with high vertical resolution, accurate and which can be a complement and an alternative for these techniques, is necessary.

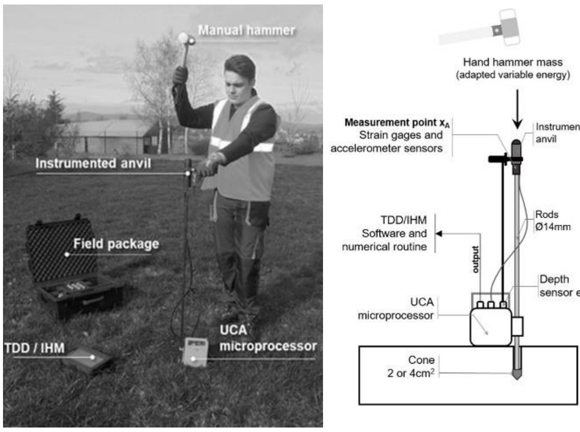
The dynamic cone loading test (DCLT), developed very recently in France (Benz Navarrete et al., 2013; Escobar Valencia et al., 2016), consists of analyzing the measurements made during the dynamic penetrometer driving in order to obtain a dynamic cone load-penetration curve at cone-soil interface. This loading curve is qualitatively comparable to the expansion curve obtained by the pressuremeter as well as those obtained from CLT test.

This paper proposes a methodology for the calibration of the curves from the DCLT test and the settlement curves of the shallow foundations according to the load-settlement curve method (Briaud 2007).

2 DYNAMIC CONE LOADING TEST (DCLT)

2.1 *Lightweight penetrometer and instrumentation*

The present instrumented DPT has been designed and based on the same functional principle as the P.A.N.D.A. (*from French Pénétromètre Autonome Numérique Assisté par ordinateur*). It is an instrumented lightweight dynamic cone penetrometer which is driven with variable energy (Gourvès and Barjot 1995). The driven energy is provided manually by means of a hand-hammer mass, which can be adapted according to the variations of soil stiffness. For each blow, the energy supplied as well as total penetration are directly measured (Benz Navarrete 2009).



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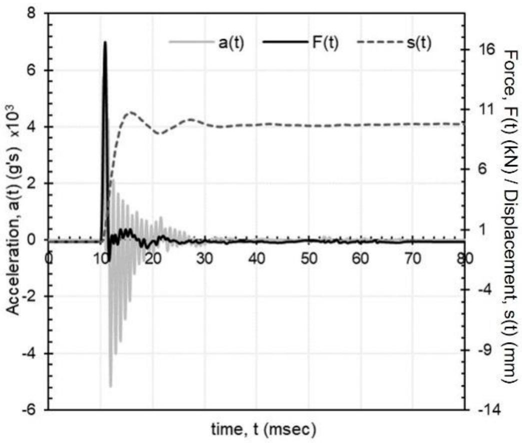


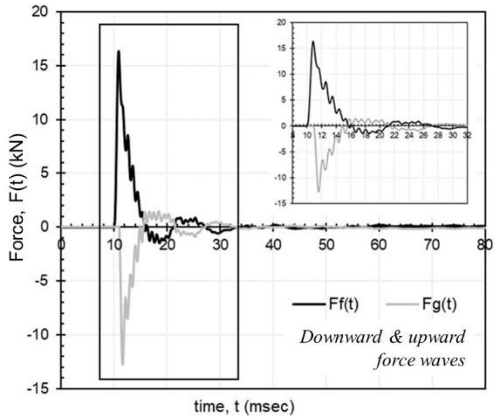
(b)

Figure 1. (a) Dynamic penetrometer P.A.N.D.A. 3 (Benz et al. 2013) and (b) raw measurements (acceleration, force and displacement)

The new version of P.A.N.D.A. employed in this work, incorporates new sensors and the general principle is based on wave equation solution (Benz Navarrete et al. 2013). Sensors are installed on the anvil and used to measure the strain *ε(x, t)* and acceleration *a(x, t)* caused by the compressional wave created immediately after each hammer blow. A third sensor is installed in the Central Acquisition Unit (UCA, from French Unité Centrale d'Acquisition) to measure simultaneously the cone penetration displacement $s_e(t)$ (Figure 1.a).

The instrumentation of the anvil is composed of strain gauges with a measurement range of ± 45 kN and installed in a Wheatstone bridge that compensates lateral deformations. The wave force, *F(x, t)*, is calculated from the measured strain *ε(x, t)* using Hooke's law. Miniature piezoresistive high-g shock accelerometers with a measurement range of ±20.000g, are equally installed close to the strain gauges. Velocity *v(x, t)* is obtained by means of frequency domain integration of signal *a(t)* (Brandt and Brincker 2014; Pan et al. 2016). Two displacement are measured; the first, *s(x, t)* by double integration and baseline correction of signal *a(x, t)* and the second $s_e(t)$ directly measured by the encoder sensor.

Sensors are connected to the UCA, that integrates the digital acquisition cards. During penetration test, each blow is continuously recorded, and each signal are sampled up to a 250 kHz frequency at 24-bits resolution. Time record per blow is either 100 or 200 milliseconds.



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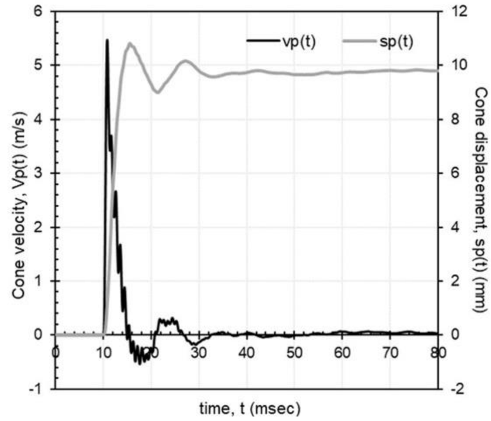

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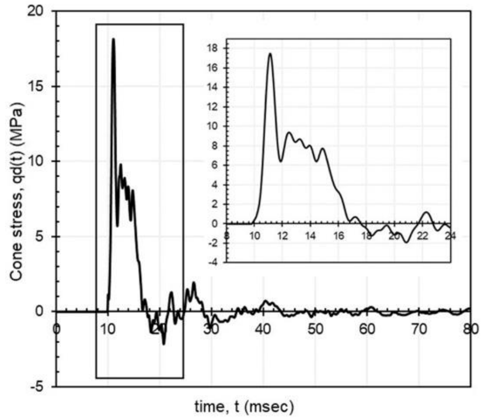

(c)

Figure 2. Example of (a) force decoupled waves in measurement point x_A, and reconstructed cone signals (b) velocity and displacement; and (c) cone stress.

After each hammer blow, UCA sends data to the Transfer Dialog Data box (TDD) which conditions, processes, and stores each measured signal by mean of a specially designed software containing the algorithms presented below. The signal processing of raw measurement includes a baseline correction and a signal filtering by mean of a low pass FIR (Finite Impulse Response) filter by using a rectangular window and a cut-off frequency of 25 kHz. An example of raw measurements performed during the test is shown in Figure 1.b.

Penetrometer is composed of rods with a diameter of 14 mm and 500 mm in length and of overflowing conical tips with a cross-section of 2 or 4 cm^2 (respectively, 15.9 or 22.5 mm in diameter). In accordance with the standard ISO-22476-2 (2005), the apex angle of cones is 90°.

To avoid the effects of skin friction along the rods, the use of overflowing conical tips (having a larger diameter than that of the rods) has proved to be sufficient. However, if it is not enough, jacking or mud injection can be implemented, as described in ISO 22476-2.

2.2 *Measurement principle : DCLT curve*

The main principle is based on pile dynamic test, rapid shocks tests, Split Hopkinson Pressure Bars (SPHB) as well as rock percussion borehole test (Rausche 1970; Karlsson et al. 1989; Bussac et al. 2002; Omidvar et al. 2014, 2015).

As shown in Figure 2 and as presented by (Benz Navarrete et al. 2013; Escobar Valencia et al. 2013; Tran et al. 2019), - supposing no skin friction along the rods - for each blow and from measurements carried out in the upper end of rod string (close to the anvil), elementary downward and upward waves are decoupled, from which, force $F_p(t)$, velocity $v_p(t)$ and settlement $s_p(t)$ are computed at cone/soil interface. Dynamic cone load-penetration (DCLT) curve is obtained plotting cone stress $qd(t)$ as a function of cone settlement $s_p(t)$ (Figure 3).

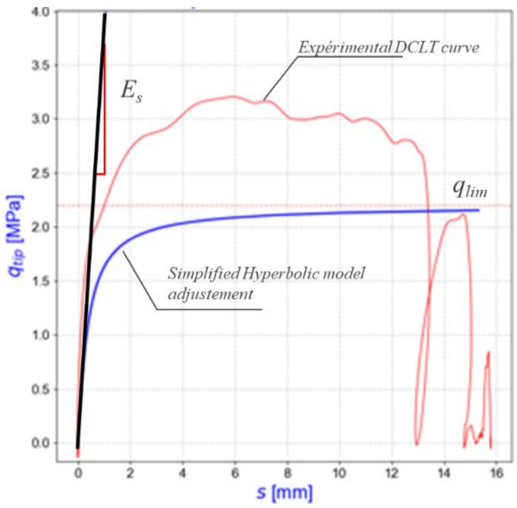

Figure 3. Field Dynamic Cone Loading (DCLT) curves (Jossigny, France).

In the Figure 3, cone resistance $qd(t)$ represents the total soil resistance under dynamic loading. In this way, DCLT curve can be modelled by means of a perfectly elasto-plastic and radiation dashpot model, where total soil resistance $qd(t)$ is the summation of pseudo-static q_{ps}, damping $q_{dyn}(t)$ and inertial q_{in} resistances (Eq. **Error! Reference source not found.**). In our cases, inertial resistance can be neglected due to the low mass of penetrometer. Pseudo-static resistance q_{ps} is displacement dependent and modelled by means of non-linear elastic perfectly plastic law while $q_{dyn}(t)$, is penetration rate dependent and modelled with a radiation dashpot (Eq. **Error! Reference source not found.**).

$$qd(t) = q_{ps} + q_{dyn}(t) + q_{in} \tag{1}$$

$$qd(t)Ap = Qd = k_b u + c_b \dot{u} + m_P \ddot{u} \tag{2}$$

In Eq. **Error! Reference source not found.**, $\ddot{u}$, $\ddot{u}$, and u are, respectively, acceleration, velocity, and displacement at penetrometer's cone/soil interface. Q_d is the total strength and A_p the cone section. c_b and k_b are the spring and dashpot constant (El Naggar and Novak 1996), defined by:

$$k_b = \frac{4G_s R_{(H)}}{(1-v_s)} \tag{3}$$

$$c_b = \frac{3.4 R_{(H)}{}^2 \sqrt{G_s \rho_s}}{(1-v_s)} \tag{4}$$

Where, $R_{(H)}$ is equivalent radius introduced by (Holeyman 1988) and computed according to (Novak and Beredugo 1972; El Naggar and Novak 1994). G_s, ρ_s and v_s are respectively shear modulus, density, and Poisson's ratio of soil.

Plastic resistance or soil limit resistance q_{lim} is obtained from velocity, displacement and strength signal according to the Unloading Point Method proposed by (Middendorp et al. 1992; Hölscher et al. 2012) and explained by (Brown 1994, 2016).

Moreover, spring constant or dynamic stiffness k_{dyn} is determined from DCLT experimental measurements by mean of FRF transfer functions, according with (Davis 1975; Paquet and Briard 1976) and as presented by (Tran et al. 2019). For the case of a rigid embedded foundation subject to transient dynamic loading, it is assumed that at low frequencies, k_{dyn} is close to k_b. Knowing k_b and assuming Poisson's ratio v_s, shear modulus G_s can be obtained from Eq. **Error! Reference source not found.**. Moreover, Elastic modulus (E_s) can be then obtained by applying the theory of elasticity. Other elastic parameters can be deduced.

According with the present work, in practice field DCLT curves are analyzed as follows:

- Measurement of dynamic stiffness k_b from FRF curves,
- Determination of shear modulus G_s (Eq. **Error! Reference source not found.**).
- Elastic modulus (E_s) assessment from G_s and assuming Poisson's ratio v_s.
- Determination of soil limit resistance q_{lim} from DCLTs curves and Eq. **Error! Reference source not found.** according with UPM method:
  - From cone velocity signal $v_p(t)$, find the moment (t_0) where $v_p(t)$ becomes zero after the blow.
  - Confirm that the t_0 moment coincides with maximum tip displacement $s_p(t)$ is at its maximum.
  - From strength signal $F_p(t)$, computed average value $F_p(t)^*$ at ($t_0 \pm$ dt) moment (with dt $\approx$0,1msec).
  - Limit resistance assessment is obtained after inertial correction, $q_{lim} = F_p(t)^* - m_p.a(t_0)$; where m_p is the mass of penetrometer and $a(t_0)$ is average acceleration at t_0.

In this way, knowing E_s and q_{lim}, DCLT curves can be modeled by a nonlinear elastic plastic model. In this way, nonlinearity is introduced by mean of a simplified hyperbolic model as presented below.

2.3 *Simplified Hyperbolic Model adjustment to DCLT*

One possibility to reproduce the cone resistance–penetration curve of DCLT would be to apply a rheological model such as an elasto-plastic model taking into account the various phenomena occurring during the penetration of the cone. This choice was not made here, because it would require too many parameters to be fit and even tedious calculations. The option is to fit a predetermined curve type suitable for reproducing the test, with the choice of deducing the parameters set on the characteristic features of the curve which are the modulus E_s and the limit resistance q_{lim}. A similar development has been proposed by (Elhakim, 2005).

In order to reproduce the curves in a systematic way on the basis of the parameters that are the tip resistance and the dynamic modulus, we will use a hyperbola formulation inspired by the pressuremeter method. The final version homogenizes the terms of the formula and systemizes the determination of the A_i parameters, closer to the method proposed by (Baud & Gambin, 2008):

$$\varepsilon = \frac{s}{h_e} = A_1 + A_2 q + \frac{A_4}{A_6 - q} \tag{5}$$

With:

$$A_1 = -\frac{A_4}{A_6} \qquad A_2 = \frac{\beta}{E_{dyn}}$$

$$A_4 = \frac{(\alpha-\beta)(1-x)}{E_{dyn}} q_d^2 \qquad A_6 = q_d = q_{lim}$$

Where, $x = \frac{q_2}{q_d}$ a non dimensional term, where q_2 is the inflection point at the end of the linear part, corresponding to the conventional definition of $p_{LM} = 1.7 \cdot p_{fM}$. We can therefore estimate x with a value around 0,6 to 0,7.

$\alpha = \frac{s_2 E}{q_2 h_e} = (1-\nu^2)\frac{b}{h_e} C_f = \frac{b}{h_e} 0.7384$, a non dimensional term, with $C_f = 0.79$ corresponding to the form coefficient of the rigid circular cone of P.A.N.D.A, an influence depth $h_e = \frac{\pi R}{4}(1-\nu^2)$ (Butterfield & Banerjee, 1971), b and R being respectively diameters and radius of the cone.

β is equal to 0.95α, this ratio allowing to decrease the initial slope, in order to guide the hyperbola with an upward concavity.

Figure 4 shows an adjustment curve modelled by this version, based on data of DCLT test that are the tip resistance, q_d and the dynamic modulus, E_{dyn} at the depth from 1 m to 1.25 m, in Jossigny site.

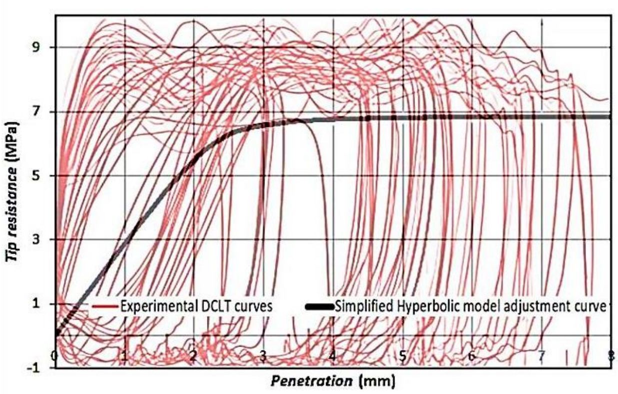

Figure 4. Curve adjusted in Jossigny (1 m to 1.25 m).

3 BEARING CAPACITY AND SETTLEMENT PREDICTION

3.1 *Bearing capacity*

Bearing capacity of the foundation is obtained using the French penetrometer design direct method with slight modification (AFNOR, 2013).

$$q_{net} = q_0 + k_d \cdot q_{de} \tag{6}$$

$$k_{d;\frac{B}{L}} = k_{d0} + \left(a + b.\frac{D_e}{B}\right)\cdot\left(1 - e^{-c\cdot\frac{D_e}{B}}\right) \tag{7}$$

$$k_{d;\frac{B}{L}} = k_{d;\frac{B}{L}=0}\cdot\left(1-\frac{B}{L}\right) + k_{d;\frac{B}{L}-1}\cdot\frac{B}{L} \tag{8}$$

With the q_{net} the net pressure under the foundation, q_0 the total vertical stress at the base of the foundation after construction, k_d the penetrometric bearing capacity coefficient, q_{de} the equivalent cone resistance and *a, b, c* and *d* the coefficients tabulated according to the type of soil and the shape of the foundation.

3.2 *The load-settlement curve method*

A method similar to the Load Settlement Curve Method approach proposed by Briaud has been developed to access the full load settlement curve (Briaud, 2007). Figure 5 shows the principle of the dynamic penetration test with the DCLT. The similarity of the cone resistance – penetration curve acquired by the DCLT at each hammer blow with the loading – settlement curve of the shallow foundation test supports proposition for adaptation of LSCM method.

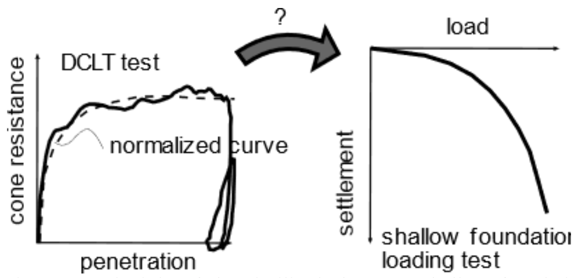

Figure 5. Principle of the similitude between shallow foundation loading test and DCLT with P.A.N.D.A.3.

This transformation based on full-scale tests and numerical simulations is achieved using the equation 5 and real load pressure anticipated up to q_{net} obtained with equation 6.

Results obtained with the proposed method are compared with settlement predicted by Schmertmann method as proposed by Eurocode 7 part 2.

4 EXPERIMENTAL CAMPAIGN

4.1 *Shallow foundation loading procedure*

Several loading tests of 1x1 m and 0.71 x 0.71 m square foundations have been performed on several sites. Ground investigation using P.A.N.D.A (DCLT), PMT, CPT, SPT and other techniques helped to define the ground model.

Foundations were loaded, using load steps of a 30 minutes duration, until failure occurred, with an average number of 10 steps. The shallow foundation loading tests were carried out using a device schematized in Figure 6.

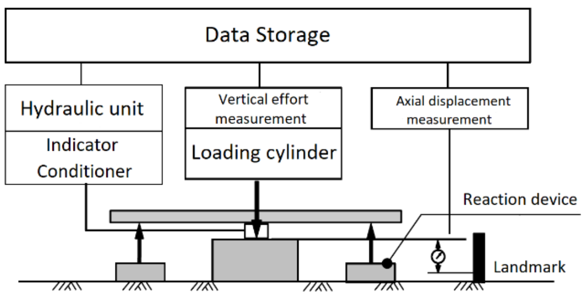

Figure 6. Schematic shallow foundation loading test.

4.2 *Sites presentation and tests performed*

Sites with different soil types were chosen to perform the DCLT tests and shallow foundation tests. The Table 1 below shows their soil types with the number of DCLT tests in each place.

Table 1. Sites and number of DCLT test

Site	Soil type	DCLT test	
		Number of test	DCLTs curves /ml
Jossigny	Silty	6	130 c/ml
Cran	Marine clay	6	128 c/ml

From the results acquired of several square foundation loading tests on any given site, a mean adjusted curve can be modelled, using the least-squares method. Figure 7 presents the mean curve of loading – settlement on Jossigny site for the case of 1x1m square foundations.

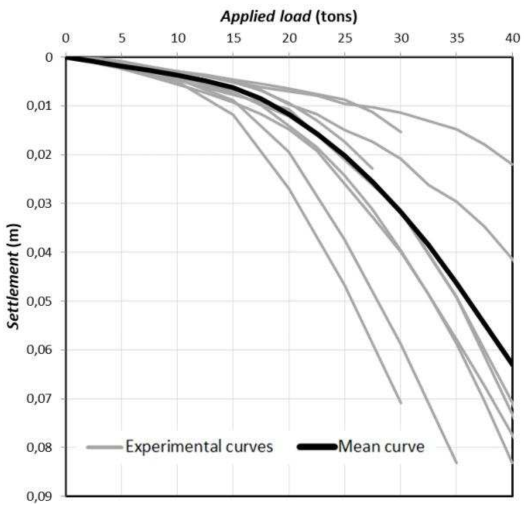

Figure 7. Mean curve of foundation loading tests performed on Jossigny (Square 1x1m).

4.3 *Tests results and comparison to calculation*

The bearing capacity prediction using the penetrometer methods in NF P94-261 the French national application standard of Eurocode 7 for shallow foundation design, in this case is a square 1x1m footing, is presented in figure 8. The distribution curves of the ratios between the calculated bearing capacity coefficients from these methods to the measured coefficients are compared with the curve of Normal law.

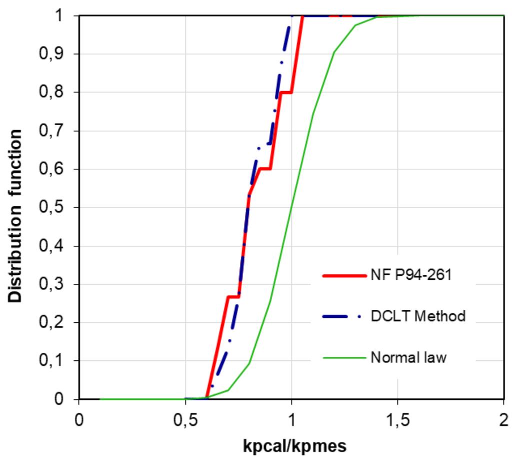

Figure 8. Distribution curves of the experimental and computed bearing capacity ratio on Jossigny.

We note that the results of the France application standard of EC7 for the design of shallow foundation (NF P94-261) and the DCLT method are superimposed. The safety level of the bearing capacity is represented by the deviation from an average value of 1. In Figure 8, we can however find that the prediction of these methods tends to a conservative result of bearing capacity, for the soil in Jossigny. The same approach has been realized for all the sites of the data base.

Figure 9 shows the prediction of settlement, according to the loading, by four design methods: the France application standard of EC7 (NF P94-261), the Load Settlement Curve Approach proposed by (Briaud, 2007), the Simplified Hyperbolic model for PMT and the Schmertmann method, for a shallow square 1x1m foundation in Jossigny. We also note that the value of the conventional bearing capacity of soil on this site is about 36 tons.

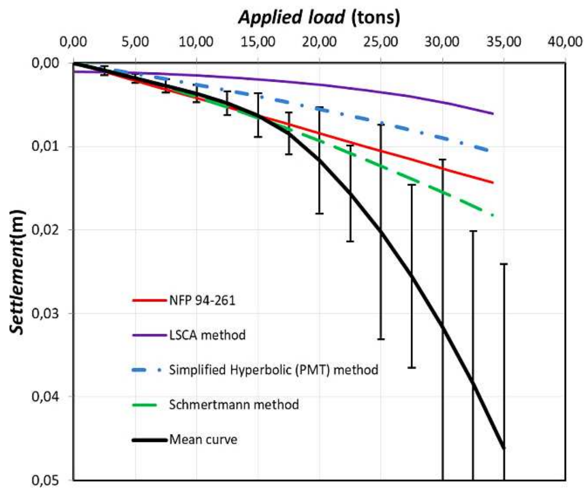

Figure 9. Comparison of experimental and predicted settlement curves. For the 1x1 m square foundation on silt

In the Figure 9, the mean curve is accomplished with the error bars representing the dispersion of experimental results (Figure 7). The calculated settlement curves of three methods NF P94-261, Schmertmann method and the Simplified Hyperbolic model for PMT are well matched with the experimental mean curve, in the first half the loading phase. However, the LSCA method proposed by Briaud underestimates the settlement. One drawback of this method is the non-zero settlement for a loading close to zero.

When the loading is over 50% of bearing capacity, the four methods all underestimate the settlement of foundation. The calculated curve of the LSCA method is always farthest from the experimental mean curve while that of Schmertmann method is the closest prediction.

4.4 *Results and discussions*

The log of experimental DCLT test (q_{lim} and E_s) obtained in sites of Jossigny and Cran are presented in the **Error! Reference source not found.Error! Reference source not found.**. From this log of q_{lim} and E_s curves, and this is for every 1 meter of depth, Hyperbolic model is fitting according to the method presented above. And bearing capacity and settlement curves derived.

5 CONCLUSIONS

This paper suggests that it is possible to estimate the bearing capacity of shallow foundations based on penertion resistance and modulus derived from the newly developed Dynamic Cone Loading Test. To improve the accuracy of the technique it is necessary of record more data points during the early stages of the penetration. A database of foundation loading test has been collected to validate the proposed design method. Good agreement between the predicted resistance and the measured one has been observed.

6 ACKNOWLEDGEMENTS

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