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Analysis of load-settlement behaviour of shallow foundations in saturated clays based on CPT and DPT tests

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Abstract. Static Penetration Test (CPT) and Dynamic Penetration Test (DPT) are commonly used in-situ tests in a routine geotechnical investigation. Besides their use for qualitative investigation (lithology, homogeneity and spatial variability), they are used as practical tools of geotechnical characterization (resistance to the penetration, soil rigidity) and modern foundation design as well. The paper aims at presenting the results of an extensive research work on the evaluation of the 1D primary consolidation settlement of saturated clayey soils on the basis of the CPT or DPT tests. The work is based on an analysis of the correlations between the tip resistance to penetration measured in these tests and the parameters of compressibility measured by the compressibility oedometer test, through a local geotechnical database in the northern Algeria. Such an analysis led to the proposal of two methods of calculation of the settlement, one based on the CPT test and the other one on the DPT. The comparison between the predicted settlements and those computed on the basis of the oedometer test showed a good agreement which demonstrate the possbility to use the CPT and DPT tests as reliable tools of computation of foundation settlements in clayey soils.

Keywords: settlement; shallow foundation; Cone Penetration Test (CPT); Dynamic Penetration Test (DPT); clay; oedometer test; consolidation

1. Introduction

The settlement analysis of shallow foundations is a key parameter of serviceability limit state design of foundations and an important field of research on soil structure interaction. Settlement calculation based on in-situ tests has been a subject of several studies (Abu-Farsakh 2004, Abu-Farsakh *et al*. 2007, Yu and Abu-Farsakh 2011, Damasceno and Badu-Tweneboah 2011, Abu-Farsakh and Yu 2013).

It is often admitted the settlement of shallow foundations is the sum of three components, which are the immediate settlement, the primary consolidation settlement (s_c) and the creep settlement (Skempton and Bjerrum 1957).

Six decades ago, the calculation of 1D primary consolidation settlement of shallow foundations in saturated clays was often made based on the compressibility parameters measured from the conventional compressibility oedometer test in the laboratory on samples supposed undisturbed.

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This settlement calculated using the Terzaghi's 1D consolidation theory has been widely used (Chunlin 2014).

However, the emergence of the in-situ tests as powerful tools of site investigation which are faster, easier and do not require any sampling procedures generated a revolution within the field of foundation design methods. When modeling the soil response, these methods often avoid the theoretical complications due to complex behavior of soils and follow a pragmatic approach based on direct correlations between the penetration resistance (static or dynamic) and the soil compressibility parameters.

Such an approach led to practical methods of calculation of the settlement on the basis of the CPT or DPT tests (DIN 2003).

The 1D primary consolidation settlement s_c of a soil slice, thick of H_0 and subjected at its mid-height to an increment $\Delta\sigma'_v$ of effective stress, is generally calculated based on the secant constrained modulus M determined from the oedometer loading curve as follows

$$s_c = \varepsilon_z H_0 = \frac{\Delta\sigma'_v}{M} H_0 \tag{1}$$

As shown in Fig. 1, due to the non linear soil behaviour and the prevented lateral deformations, the modulus M increases with the effective stress increment $\Delta\sigma'_v$ defined as (Briaud 2001)

$$\Delta\sigma'_v = \sigma'_v - \sigma'_{v0} \tag{2}$$

σ'_v and σ'_{v0} are respectively the final and initial effective vertical stress.

In order to benefit from the major advantage offered by the penetration tests, which is the solicitation of the soil in its natural environment, several proposals have emerged for the estimation of M by correlation to the tip resistance measured during the CPT or DPT tests (Janbu 1963, Kantey 1965, Thomas 1968, Meigh and Corbett 1969, Sanglerat 1972, Senneset *et al*. 1989, Briaud 2001, Mayne 2001, AbdelRahman *et al*. 2005, RohitRay 2007, Jovan 2009, Rito and Sugawara 2009, Robertson and Cabal 2015, Vendel 2013, Mayne 2007, Becker 2010, Cai *et al*. 2010, Hong *et al*. 2011, Duncan and Bursey 2013, Lin *et al*. 2014, McNulty and Harney 2014). A general formula of the constrained modulus M is given as follows

$$M = m\sigma_{ref}\left[\frac{\sigma'_{v0} + \beta.\Delta\sigma'_v}{\sigma_{ref}}\right]^n \tag{3}$$

m is the "soil modulus number", n is the "stress exponent", σ_{ref} is a "reference stress" taken equal to 100 kPa, and β is a factor depending on the overconsolidation ratio (OCR) of the clayey soil.

The initial constrained modulus M_0 (equal to the slope of the initial tangent to the curve, as illustrated by Fig. 1) is then defined by

$$M_0 = m\sigma_{ref}\left[\frac{\sigma'_{v0}}{\sigma_{ref}}\right]^n \tag{4}$$

Combining Eqs. (3)-(4) leads to the definition of the "stiffness ratio" M/M_0

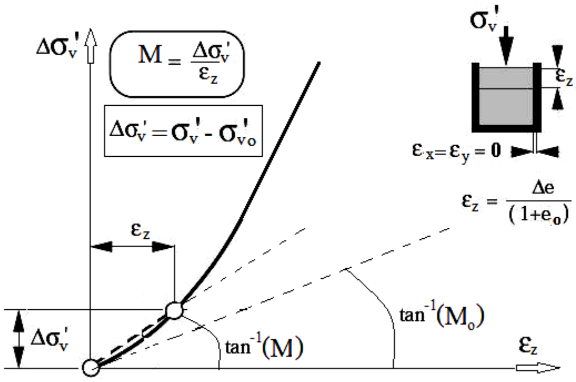

Fig. 1 Explanatory scheme of 1D loading curve

Table 1 Typical values of m according to CFEM - 1992

Soil type	Values of *m*
Silty clay, hard, stiff	20-60
Silty clay hard, stiff, firm	10-20
Clayey silt, soft	5-10
Marine clays, soft	5-20
Organic clays	5-20
Peat	1-5

$$\frac{M}{M_0} = \left[1 + \beta \frac{\Delta'\sigma_v}{\sigma'_{v0}}\right]^n \tag{5}$$

According to Janbu (1963) and Senneset *et al.* (1989), $\beta = n = 0$ for an overconsolidated clay which means within the preconsolidated stress range ($\sigma'_{v0} < \sigma'_v < \sigma'_c$), where σ'_c is the effective preconsolidation stress, the constrained modulus is constant ($M = M_0$).

Beyond this range ($\sigma'_v > \sigma'_c$), $\beta = n = 1$ which corresponds to a linear increase of M with effective stresses. Ranges of the typical values of the modulus number m are summarized in Table 1 as function of the soil porosity (Fellenius 2006, Robertson 2009a, Robertson and Cabal 2015).

The geotechnical European code Eurocode 7 recommends to take $\beta = 0.5$ and $n = 0.6$ for low plasticity clays ($I_p \leq 10\%$ and $\omega_l \leq 35\%$). The modulus number m is correlated to the tip resistance q_c (CPT test) or the number of blows N_d (DPT test) for low plasticity clays above the water table and having a consistency index I_c lying between 0.75 and 1.30, as indicated respectively in Tables 2 and 3.

Table 2 Typical values of m according to Eurocode 7 (CPT test)

m	Domain of validity	
	Resistance q_c (MPa)	Type of soil
$15.2q_c + 50$	$0.6 \leq q_c \leq 3.5$	Low plasticity clay with $0.75 \leq I_c \leq 1.3$ and above the water table

Table 3 Typical values of m according to Eurocode 7 (DPT test)

DPT equipment	m	Domain of validity	
		Number of blows N_{10}	Type of soil
DPL (Light DPT)	$4(N_{10})+30$	$6 \leq N_{10} \leq 19$	Low plasticity clay with $0.75 \leq I_c \leq 1.3$ and above the water table
DPH (Heavy DPT)	$6(N_{10})+50$	$3 \leq N_{10} \leq 13$	

A direct correlation of the modulus M_0 to the cone resistance q_c has also been recommended by the Eurocode 7, on the basis of the following equation:

$$M = K_{CPT} q_c \tag{6}$$

The coefficient K_{CPT} is given for different types of soils within rather wide margins which limits the use of the Eq. (6) to a rough estimation of the settlements. Such an approach is only valid in preliminary phase of geotechnical design or when the settlement analysis is not a key factor in the study of the structure.

On the basis of the CPT-u test (Piezocone test), Senneset *et al*. (1989) suggested to correlate the secant modulus M to the net cone tip resistance $(q_t - \sigma_{v0})$, q_t being the total cone tip resistance measured during the CPT-u test, using the following empirical formula

$$M = \alpha_M \left(q_t - \sigma_{v0}\right) \tag{7}$$

The dimensionless coefficient α_M ranges from 5 to 15 for overconsolidated clays and from 4 to 8 for normally consolidated clays (Lunne *et al*. 1997). Mayne (2007) proposes a value of α_M = 5 for Vanilla Clays whereas Hamza and Shahien (2013) propose α_M = 3.5. Finally, Abu-Farsakh (2004) found α_M = 3.15 and Abu-Farsakh *et al*. (2007) found α_M around 3.58.

Robertson (2009a, b), Robertson (2012) and Robertson and Cabal (2015) suggested the use of the previous equation where α_M is function of the normalized cone tip resistance Q_t and the soil behaviour index I_c

$$I_c = \left(\left(3.47 - \log Q_t\right)^2 + \left(\log F_r + 1.22\right)^2\right)^{0.5} \tag{8}$$

Q_t is the normalized cone tip resistance and F_r is the friction ratio given respectively by

$$Q_t = \frac{q_t - \sigma_{v0}}{\sigma_{v0}} \tag{9}$$

$$F_r(\%) = \frac{f_s}{q_t - \sigma_{v0}} x100 \tag{10}$$

α_M is calculated by the following formulae, depending on the values of I_c

If $I_c > 2.2$: $\alpha_M = Q_t$ if $Q_t < 14$, and $\alpha_M = 14$ if $Q_t > 14$. Otherwise ($I_c < 2.2$)

$$\alpha_M = 0.0188\left(10^{(0.55 I_c + 1.68)}\right) \tag{11}$$

The results presented in this paper are part of a research program on the design of shallow foundation undertaken in the University of Blida, on the basis of the analysis of correlations between the cone tip resistance quantified by q_c (CPT test) or the number of blows N_d (DPT test) and the 1D compressibility parameters measured by the oedometer test A local geotechnical database covering many sites located in the northern Algeria was used. For each penetration test (CPT or DPT), three approaches to interpret the oedometer compressibility curve were developed. Further comparison of predicted settlements to those computed based on the oedometer test led to select the approach exhibiting the highest predictive capability, which led to suggest two methods of calculation of the settlement, one based on the CPT test and the other e on the DPT.

2. Methodology of analysis of the settlement

The constrained modulus M may be obtained either by interpretation of the experimental compressibility curve e-Logσ_v illustrated in Fig. 2, or by fitting the 1D loading curve (see Fig. 1) to derive M and M_0, then fitting the curve of the stiffness ratio M/M_0 versus the stress increment ratio $\Delta\sigma'_v/\sigma'_{v0}$ for each sample tested in the oedometer.

The initial constrained modulus M_0 and the effective preconsolidation stress σ'_c may be correlated to the cone tip resistance q_c (CPT test) or the number of blows N_d (DPT test) as follows

$$M_0 = K_{CPT} q_c \tag{12}$$

$$M_0 = K_{DPT} N_d \tag{13}$$

$$\sigma'_c = \lambda_{CPT} q_c \tag{14}$$

$$\sigma'_c = \lambda_{CPT} q_c \tag{15}$$

The three following approaches used to compute M and M_0 are presented as follows

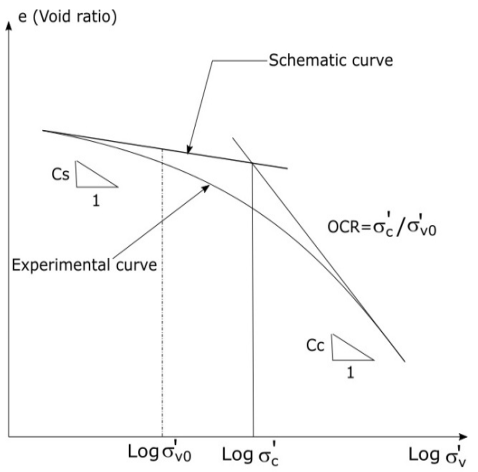

Fig. 2 Schematic compressibility curve of an oedometer test

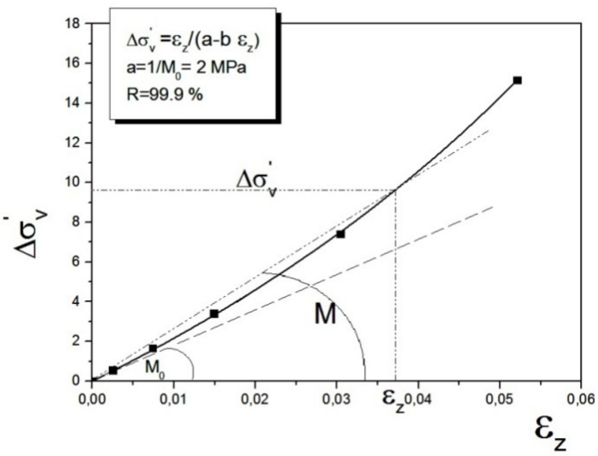

Fig. 3 Typical experimental 1D loading curve

- *Hyperbolic Empirical Approach* (*HEA*)

The experimental compressibility curve e-Logσ_v, illustrated in Fig. 2, was used to compute for each applied stress σ'_v the settlement s_c according to Eq. (16), the initial void ratio e_0 and the initial height H_0 of the sample being known

$$s_c = H_0 \frac{\Delta e}{1+e_0} = H_0 \frac{e-e_0}{1+e_0} = \varepsilon_z H_0 \tag{16}$$

The vertical 1D strain ε_z is then computed, and the loading curve $\Delta\sigma'_v = f(\varepsilon_z)$ is plotted. The analysis of such curves showed in almost all the cases a hyperbolic shape followed by the experimental points (see typical curve in Fig. 3) which may be fitted by the hyperbolic function

$$\Delta\sigma'_V = \frac{\varepsilon_Z}{a\text{-}b.\varepsilon_Z} \tag{17}$$

The initial constrained modulus M_0 is equal to $1/a$ which may be obtained by a least squares fitting procedure

$$M_0 = \frac{1}{a} \tag{18}$$

The stiffness ratio M/M_0 and the stress ratio $\Delta\sigma'_v/\sigma'_{v0}$ are then computed for each loading increment and the curve M/M_0 versus the stress ratio $\Delta\sigma'_v/\sigma'_{v0}$ for all the samples studied in the database is analysed and fitted according to Eq. (5) in order to evaluate the coefficients β and n.

- *Bilinear Analytical Approach* (*BAA*)

Based on the bilinear simplified compressibility curve, illustrated in Fig. 2, it can be easily demonstrated the initial modulus M_0 may be written as follows

$$M_0 = \sigma'_{v0} \frac{(1+e_0)}{C_s} Ln(10) \tag{19}$$

Moreover, the stiffness ratio M/M_0 depends on the stress ratio $\Delta\sigma'_v/\sigma'_{v0}$ and the overconsolidation ratio OCR as follows

- Overconsolidated clay with $\sigma'_{v0} < \sigma'_v < \sigma'_c$

$$\frac{M}{M_0} = \frac{\dfrac{\Delta\sigma'_v}{\sigma'_{v0}}}{Ln(1+\dfrac{\Delta\sigma'_v}{\sigma_{v0}'})} \tag{20}$$

- Overconsolidated clay with $\sigma'_{v0} < \sigma'_c < \sigma'_v$

$$\frac{M}{M_0} = \frac{\dfrac{\Delta\sigma'_v}{\sigma'_{v0}}}{\dfrac{C_c}{C_s} Ln(1+\dfrac{\Delta\sigma'_v}{\sigma'_{v0}}) + \left(1-\dfrac{C_c}{C_s}\right) Ln(OCR)} \tag{21}$$

- Normally consolidated clay with $\sigma'_{v0} = \sigma'_c < \sigma'_v$

$$\frac{M}{M_0} = \frac{\dfrac{\Delta\sigma'_v}{\sigma'_{v0}}}{\dfrac{C_c}{C_s} Ln(1+\dfrac{\Delta\sigma'_v}{\sigma'_{v0}})} \tag{22}$$

These expressions may be compiled into the following compact equation

$$\frac{M}{M_0} = \frac{\dfrac{\Delta\sigma'_v}{\sigma'_{v0}}}{c + dLn(1+\dfrac{\Delta\sigma'_v}{\sigma'_{v0}})} \tag{23}$$

The factors c and d, depending on the ratios C_c/C_s and OCR, may be estimated by correlation to the penetration tests parameters, namely the cone tip resistance q_c (CPT test) or N_d (DPT test). The secant constrained modulus M may then be evaluated from Eq. (23) provided the initial modulus M_0, computed according to Eq. (19), is estimated by correlation to q_c or N_d.

- *Bilinear Empirical Approach* (*BEA*)

At each borehole, it is assumed a rectangular foundation embedded in a soil mass composed of N slices is subjected to a series of vertical pressures. Each slice is assumed characterized by a bilinear simplified curve e-Logσ_v, as illustrated in Fig. 2. The settlement $s_c(j)$ of the j^{th} slice thick of H_j is computed as follows (Duncan and Buchignan 1976, CGS 1992)

- For a normally consolidated slice

$$s_c(j) = \frac{C_c(j)}{1+e_0(j)} H_j Log\left(\frac{\sigma'_{v0}(j) + \Delta\sigma'_v(j)}{\sigma'_{v0}(j)}\right) \tag{24}$$

- For an overconsolidated slice, if $\sigma'_v < \sigma'_c$

$$s_c(j) = \frac{C_s(j)}{1+e_0(j)} H_j Log\left(\frac{\sigma'_{v0}(j) + \Delta\sigma'_v(j)}{\sigma'_{v0}(j)}\right) \tag{25}$$

If $\sigma'_v > \sigma'_c$ then the settlement is computed as

$$s_c(j) = \frac{H_j}{1+e_0(j)}\left[C_s(j) Log\left(\frac{\sigma'_c(j)}{\sigma'_{v0}(j)}\right) + C_c(j) Log\left(\frac{\sigma'_v(j)}{\sigma'_c(j)}\right)\right] \tag{26}$$

The vertical strain ε_z is then computed from Eq. (16) and the curve $\Delta\sigma'_v = f(\varepsilon_z)$ is plotted and analyzed following the same remaining steps of the HEA approach through the Eqs. (17)-(18) until obtaining the curve M/M_0 versus the stress ratio $\Delta\sigma'_v/\sigma'_{v0}$ and fitting it according to Eq. (5).

The advantage of such an approach is the possibility to investigate the shape effect (quantified by the ratio L/B) and the embedment effect (quantified by D/B) of the foundation on the 1D settlement behaviour.

3. Description of the geotechnical database

The northern Algeria contains on a wide area deposits of clayey layers exhibiting a variety of mineralogical and physical properties. Within the scope of the correlations study, a geotechnical database was built on the basis of geotechnical reports on site investigations carried out on many sites located in the northern Algeria, namely in the following counties: Algiers, Blida, Boumerdès,

Table 4 Summary of the main geotechnical properties of the clayey samples

Penetration Test	CPT	DPT
Number of sites	35	36
Number of boreholes	50	38
Number of samples	267	52
Soil class (USCS classification)	CL-CH	CL-CH
Saturation degree S_r (%)	82-100	82-100
OCR (Over-Consolidation Ratio)	1.1-5.6	1.3-10.0
$C_c/(1+e_0)$ (Compressibility)	0.015-0.300	0.015-0.300
Tip resistance (MPa)	$0.9 < q_c < 50$	$1 < q_d < 55$
Compression index C_c	0.02-0.73	0.04-2.30
Overconsolidation index C_s	0.001-0.16	0.001-0.016

Tipaza, Skikda and Annaba. 36 sites were selected for the correlations DPT/Oedometer and 35 sites for the correlation CPT/Oedometer.

All the samples studied were extracted from sampling boreholes and classified according to the USCS system as low to high plasticity clays (CL or CH class). They are almost saturated, since the saturation degree is more than 82%, and are moderately to highly overconsolidated. Table 4 summarizes the main geotechnical properties of the clayey samples studied.

The CPT test results usually correspond to a cone cross sectional area of 10 cm^2 and a rate of penetration of 20 mm/s.

4. Analysis of correlations

4.1 CPT/Oedometer correlation

The preconsolidation stress σ_c, the overconsolidation ratio OCR and the ratio C_c/C_s were correlated to q_c at the same depth of the sample subject of the oedometer test. According to each approach (HEA, BAA and BEA), M_0 was computed and correlated to q_c.

The results of statistical analysis of the coefficients K_{CPT} (Eq. (12)), λ_{CPT} (Eq. (14)), OCR and

Table 5 Comparison of values of *k*

Source	Soil	Value of k
Pant (2007)	Louisiana Soils	0.14
Abu-Farsakh *et al*. (2003)	Cohesive soil	0.15
Salem and El-Sherbiny (2014)	Silty clay	0.30
Becker (2010)	Beaufort Sea Clays	0.24-2.30
Cai *et al*. (2010)	Quaternary clays in China	0.37-0.47
Chung *et al*. (2002, 2012)	Pusan clays	0.18
Hamza and Shahien (2013)	Soft clay	0.23
This study	Clay (CL-CH)	0.18

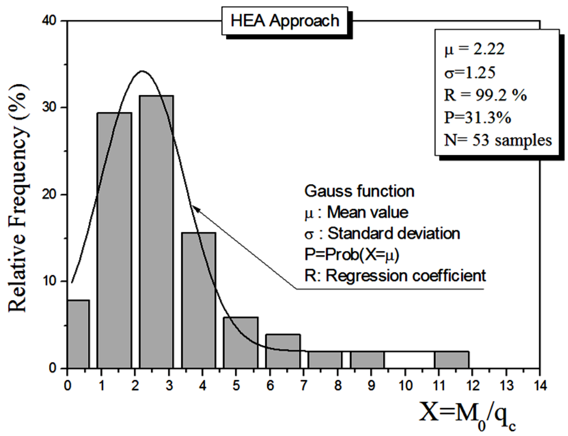

Fig. 4 Typical Histogram of KCPT (approach HEA)

Table 6 Results of statistical analysis of the correlation CPT/Oedometer

Approach	$K_{CPT} = M_0/q_c$				$\lambda_{CPT} = \sigma_c/q_c\,(= k)$				C_c/C_s			
	μ	σ	R (%)	P (%)	μ	σ	R (%)	P (%)	μ	σ	R (%)	P (%)
HEA	2.22	1.25	99.2	31.3					σ			
BAA	1.24	1.05	91.0	30.1	0.18	0.05	99	61.4	3.0	1.23	99	58.8
BEA												

* μ: Mean value; σ: Standard deviation;
R: Regression coefficient of the Gauss curve; P: Probability ($X = \mu$)

C_c/C_s are summarized in Table 5. Fig. 4 shows a typical histogram of analysis fitted by a Gauss normal function in order to characterize the ratio M_0/q_c by the mean value μ.

As it can be seen in Table 5, the values of k given by Eq. (27) and evaluated from the CPT-u test, vary between 0.15 and 0.5. The value of 0.18 found in Table 6 lies within this margin.

$$OCR = k\frac{\left(q_t - \sigma'_{v0}\right)}{\sigma'_{v0}} \tag{27}$$

Regarding the ratio C_c/C_s, some researchers found it independent of q_c. Giao and Hien (2007) found a value of 4.4 for a soil database in the North of Vietnam, which is comparable to that mentioned in Table 6.

Table 6 shows two different values of K_{CPT}, namely 1.24 and 2.22, due to two different techniques used in these approaches to derive M_0: bi-linearization of the compressibility curve (approaches BAA, BEA) versus a hyperbolic fitting of the 1D loading curve.

According to the Eurocode 7, the factor K_{CPT} vary from 1 to 8 for low to high plasticity clays. The value found by correlation in Table 6 lies within this margin (Sanglerat 1972, CEN 2007).

According to Table 7 summarizing the values of λ_{CPT} found in the literature, the value of 0.18 has the same order of magnitude.

4.2 DPT/Oedometer correlation

The DPT is an in-situ impact driving test of rods connected to a cone. It consists of counting the number of blows N_d at the top of rods to make penetrating the cone to 10 or 20 cm. Due to the dynamic phenomenon associated to this procedure, this test is unadapted to the investigation of fine soils below the water table and may lead to unreliable measurements. In this regard, the correlation study was limited in this section to saturated clayey samples above the water table.

Belonging to the category of energy-based driving formulae, the Dutch formula allows

Table 7 Comparison of the values of λ_{CPT}

Reference	Soil	Value of λ_{CPT}
Tavenas and Leroueil (1979)	Eastern European database	0.33
Sunitsakul *et al.* (2010)	Taipei clay	0.21-0.24
Kulhawy and Mayne (1990)	Bangkok clay	0.29
This study	Clay (CL-CH)	0.18

estimating the so-called dynamic tip resistance q_d as function of N_d, as described by the following equation (Washkowski 1983)

$$q_d = \frac{MgH}{A\frac{h}{N_d}} \frac{M}{M+M'} \tag{28}$$

M', M and H are respectively the driven mass (rods, cone and anvil), the driving mass (hammer) and the free fall height, whereas A is the cross sectional area of the cone and h is the penetration (usually 10 or 20 cm).

In spite of its current use, this driving formula leads to some inconsistencies among which the unbalance in driving energy and the ignorance of the time effect. Moreover, according to this formula a refusal to penetration (h = 0) corresponds to an infinite value of q_d, whereas increasing the driving mass M leads to facilitate the penetration of the cone (Gonin 1978).

In order to avoid such inconsistencies related to the concept of dynamic resistance, the number of blows N_d used in this database was directly correlated to the oedometer test parameters. Almost all of the DPT tests were carried out with devices corresponding to the standard device DPSH-A (Super Heavy Dynamic Penetrometer-Category A) according to the international standard ISO-22476-2: M = 63.5 kg, H = 0.50 m, A = 16 cm^2, h = 20 cm (ISO 2005). The number of blows N_d is hereafter noted by $N_d = N_{20}$.

Statistical analysis of the coefficients K_{DPT} in Eq. (13), λ_{DPT} in Eq. (15), OCR and C_c/C_s are

Table 8 Results of the statistical analysis of the correlation DPT/Oedometer

Approach	$K_{CPT} = M_0/N_{20}$				$\lambda_{DPT} = \sigma_c/N_{20}$ (= k) (σ_c in kPa)				C_c/C_s			
	μ	σ	R (%)	P (%)	μ	σ	R (%)	P (%)	μ	σ	R (%)	P (%)
HEA	1.08	1.08	98	39.4	12	6.5	90.5	29.2				
BAA BEA	0.93	0.45	98	34.6					3.0	1.23	99	58.8

* μ: Mean value; σ: Standard deviation;
R: Regression coefficient of the Gauss curve; P: Probability ($X = \mu$)

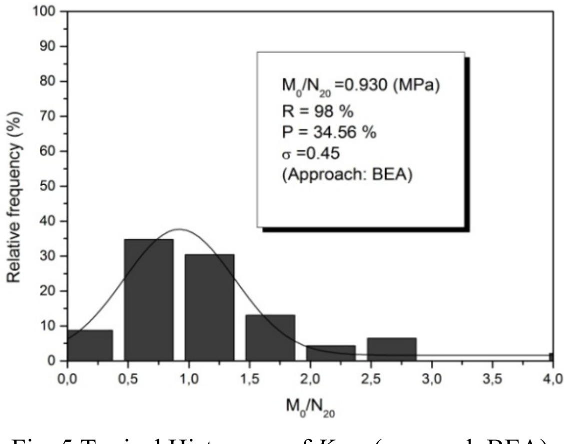

Fig. 5 Typical Histogram of K_{DPT} (approach BEA)

summarized in Table 8 and a typical histogram of analysis of λ_{DPT} is illustrated in Fig. 5

5. Study of the stiffness ratio M/M_0

Whereas the stiffness ratio is directly computed through the Eq. (23) according to the approach BAA, in the other approaches (BEA and HEA) the curve M/M_0 versus the stress ratio $\Delta\sigma'_v/\sigma'_{v0}$ for all the samples studied in the database is fitted by the least squares procedure according to the Eq. (5) in order to evaluate the coefficients β and n.

According to the approaches BEA and HEA, the best fitting curve corresponds rather to n = 1 in Eq. (5) (linear relationship) with a regression coefficient R exceeding 94%, as follows

$$\frac{M}{M_0} = 1 + \beta \frac{\Delta\sigma'_v}{\sigma'_{v0}} \tag{29}$$

Such a linear trend may be justified by the fact that all the experimental loading curves (see Fig. 2) are hyperbolic shaped and thus the stiffness ratio may be derived from Eq. (17) as follows

$$\Delta\sigma'_V = \frac{\varepsilon_Z}{a\text{-}b.\varepsilon_Z} = \frac{\varepsilon_Z}{\frac{1}{M_0} - b.\varepsilon_Z} \tag{30}$$

$$\Delta\sigma'_V \frac{M}{M_0} = \frac{1}{1\text{-}bM_0\varepsilon_Z} = \frac{1}{1 - bM_0 \frac{\sigma'_{v0}}{\sigma'_{v0}} \frac{\Delta\sigma'_v}{M}} = \frac{1}{1 - b\sigma'_{v0} \frac{\frac{\Delta\sigma'_v}{\sigma'_{v0}}}{\left(\frac{M}{M_0}\right)}} \tag{31}$$

Let us take $\delta = b\,\sigma'_{v0}$. Eq. (31) may finally be written as follows

$$\frac{M}{M_0} = 1 + \delta \frac{\Delta\sigma'_v}{\sigma'_{v0}} \tag{32}$$

The factor β, identical to δ, is given by the Table 9 depending on the position of the final effective vertical stress σ'_v with respect to the preconsolidation stress σ_c and the approach used to derive M and M_0.

Table 9 Values of the factor β

Approach	Vertical stress	β
HEA	$\sigma'_v < \sigma'_c$	0.018
	$\sigma'_v > \sigma'_c$	0.035
BEA	$\sigma'_v < \sigma'_c$	0.458
	$\sigma'_v > \sigma'_c$	0.137

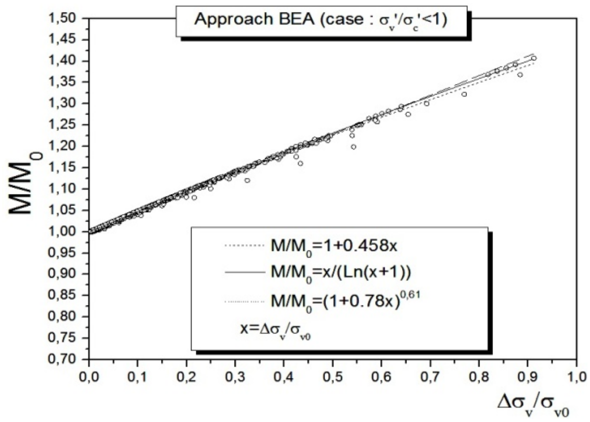

Fig. 6 Stiffness ratio versus the stress ratio (Approach BEA case : $\sigma'_v < \sigma'_c$)

The Fig. 6 is an illustrative example of the fitting procedure used in approach BEA in case of overconsolidated soil, where it can be seen the points may also be fitted with the same quality of regression by the logarithmic function given by Eq. (20) in the approach BAA. Moreover, the general formula given by Eq. (5) may fit well the points provided that β is equal to 0.78 and n = 0.61 instead of β = 0.5 and n = 0.6 as recommended by Eurocode 7 for low plasticity clays.

6. Settlement study

The stiffness ratio M/M_0 may be directly estimated from Eqs. (23)-(29) depending on the approach used to interpret the experimental results. It is then multiplied by the initial modulus M_0 estimated by correlation to the CPT or DPT tests in Eqs. (12)-(13).

The 1D consolidation settlement of a clayey layer thick of H_0 is then computed on the basis of Eq. (1)

$$s_c = \varepsilon_z H_0 = \frac{\Delta\sigma'_v}{M} H_0 = \frac{\dfrac{\Delta\sigma'_v}{\sigma'_{v0}}}{\left(\dfrac{M}{M_0}\right)\dfrac{M_0}{\sigma'_{v0}}} H_0 \tag{33}$$

When using the approach BEA or HEA and correlating M_0 for example to q_c, this equation becomes

$$s_c = C \frac{\dfrac{\Delta\sigma'_v}{\sigma'_{v0}}}{\left(1+\beta\dfrac{\Delta\sigma'_v}{\sigma'_{v0}}\right) K_{CPT}\dfrac{q_c}{\sigma'_{v0}}} H_0 = C\frac{F(\sigma'_v)}{K_{CPT}\dfrac{q_c}{\sigma'_{v0}}} H_0 \tag{34}$$

q_c/σ'_{v0} is usually called in the literature of the CPT test the normalized cone resistance. C is a coefficient of calibration of the settlements computed by the different approaches presented above

(BEA, BAA and HEA) with respect the one computed by the conventional oedometer test-based method.

When using the DPT test and the BEA approach, correlating M_0 to N_d leads to

$$s_c = C\frac{\frac{\Delta\sigma'_v}{\sigma'_{v0}}}{\left(1+\beta\frac{\Delta\sigma'_v}{\sigma'_{v0}}\right)K_{DPT}\frac{N_d}{\sigma'_{v0}}}H_0 = C\frac{F(\sigma'_v)}{K_{DPT}\frac{N_d}{\sigma'_{v0}}}H_0 \tag{35}$$

Finally, the function $F(\sigma'_v)$ given by the Eq. (36) is called in this paper "Function of effective stress distribution" and illustrated in Fig. 7 in case of the BEA approach

$$F(\sigma_v') = \frac{\frac{\Delta\sigma_v'}{\sigma_{v0}'}}{1+\beta\frac{\Delta\sigma_v'}{\sigma_{v0}'}} \tag{36}$$

Using the BAA approach and correlating M_0 for example to q_c leads to:

$$s_c = C\frac{\frac{\Delta\sigma_v'}{\sigma_{v0}'}}{\left(\frac{\frac{\Delta\sigma_v'}{\sigma_{v0}'}}{c+dLn(1+\frac{\Delta\sigma_v'}{\sigma_{v0}'})}\right)K_{CPT}\frac{q_c}{\sigma_{v0}'}}H_0 = C\frac{F(\sigma_v')}{K_{CPT}\frac{q_c}{\sigma_{v0}'}}H_0 \tag{37}$$

The function $F(\sigma'_v)$ is then given as follows:

$$F(\sigma_v') = c + dLn\left(1+\frac{\Delta\sigma_v'}{\sigma_{v0}'}\right) \tag{38}$$

Bv subdividing the active zone of settlement under a shallow foundation into N slices. the total 1D settlement s_c^{1D} of the soil foundation may be computed by summing the settlements $s_c(j)$ of the slices, j = 1 to N

$$s_c^{1D} = \sum_{j=1}^{j=N} s_c(j) \tag{39}$$

The conventional oedometer test-based method was considered as a reference method to which the predictions of the methods proposed herein will be compared and calibrated. At each borehole within the geotechnical database, it is assumed a square foundation embedded and subjected to a series of increments of vertical pressure at its base. The active zone of settlements was subdivided into thin slices and the distribution of vertical effective stresses along this zone was computed by rigorous elastic methods at mid-length of each slice (Poulos and Davis 1973, Holtz 1991).

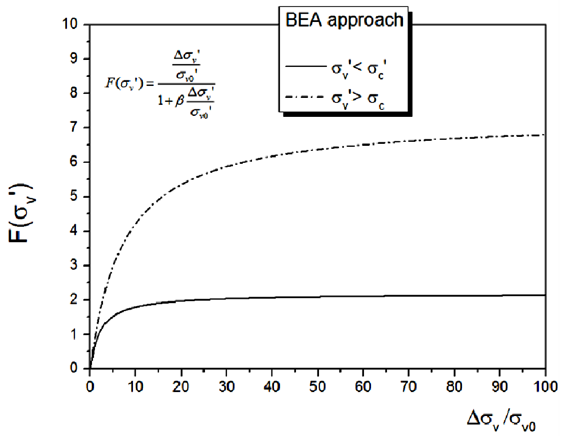

Fig. 7 Chart of the function $F(\sigma_v')$

Table 10 Comparison of the statistical features of the ratio s^{CPT} / s_c^{oed}

Approach	μ	P (%)	σ	R (%)
BEA	0.95	34.3	0.47	96.0
BAA	0.86	40.0	0.55	98.6
HEA	0.80	37.0	0.35	92.7

The total 1D settlement s_c^{1D} was computed by the conventional oedometer-based method as well as according to the same methodology of this method but with use of the approaches to evaluate the constrained modulus in correlation to the CPT or DPT test. In order to a undertake fast and automatic calculation procedure, a Fortran language based-computer program was written. The settlements $s_c^{oed}, s^{DPT}, s^{CPT}$ used hereafter refer respectively to those computed on the basis of the oedometer test, the DPT test and the CPT test.

In Table 10 are summarized the results of the statistical analysis of the ratio s^{CPT} / s_c^{oed} after calibration.

It can be seen that the computation based on the BEA approach offers a better predictive capability with a mean value of 0.95 for the ratio s^{CPT} / s_c^{oed} with an associated probability of 34.3%, which is encouraging seeing the multitude of approximations and hypotheses made along the process of development of such a method. Fig. 8 illustrates a comparative histogram of the ratio s^{CPT} to s_c^{oed} predicted by the BEA approach.

In order to investigate a possible foundation shape effect on the ratio s^{CPT} / s_c^{oed}, subsequent settlement computations on the basis of the BEA Approach were launched by using two other geometrical configurations of the foundation, namely with slenderness ratios L/B = 2 (isolated footing) and 10 (strip footing), L and B being respectively the length and the width of the foundation.

It was found that the ratio s^{CPT} / s_c^{oed} decreases from 0.95 for L/B = 1, to 0.91 for L/B = 2 and to 0.87 for L/B = 10 whatever the vertical pressure applied to the foundation and the soil stratigraphy in the database. The ratio s^{CPT} / s_c^{oed} decreases therefore slightly with the slenderness L/B.

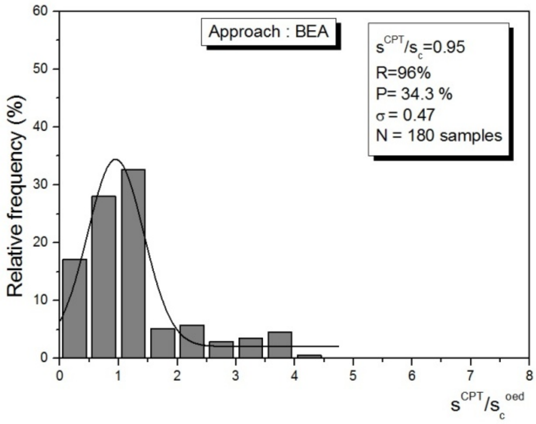

Fig. 8 Histogram of comparison of s^{CPT} and s_c^{oed}

Table 11 Comparison of the statistical features of the ratio s^{CPT} / s_c^{oed}

Approach	μ	P (%)	σ	R (%)
BEA	1.02	44.1	0.69	98.7
BAA	1.15	63.7	0.65	99.7
HEA	1.13	63.0	0.69	99.9

Regarding the computation based on DPT test, the better ratio s^{CPT} / s_c^{oed} was also found predicted by the BEA approach. According to the Table 11 summarizing the results of statistical analysis of this ratio after calibration, the mean value is 1.02 with an associated probability of 44.1%.

The histogram describing the statistical distribution of the ratio s^{CPT} / s_c^{oed} based on the BEA approach is illustrated in Fig. 9.

Identical analysis of the effect of the foundation shape on the ratio s^{CPT} and s_c^{oed} was carried out and showed this ratio decreasing from 1.02 for L/B = 1, to 1.00 for L/B = 2, and to 0.95 for

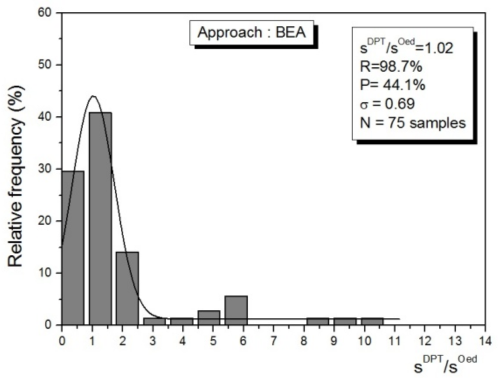

Fig. 9 Histogram of comparison of s^{CPT} and s_c^{oed}

L/B = 10 for all the vertical pressure increments applied to the foundation. The ratio s^{DPT} and s_c^{oed} depends slightly on the slenderness ratio L/B and there is likely no shape effect on this ratio.

7. Sensitivity analysis of the key parameters

Based on a correlation study of the initial constrained modulus M_0 and the preconsolidation stress σ_c to the CPT or DPT penetration tests data (see Eqs.(12)-(15)), the 1D settlement Eq. (1) was adapted to lead to Eq. (34) or Eq. (35) derived from the BEA approach. The correlation parameters, namely K_{CPT}, K_{DPT}, λ_{CPT} and λ_{DPT} were statistically derived, and therefore may exhibit some variability which affect the predicted settlements according to these equations.

It can be seen from Eq. (34) or Eq. (35) the relative uncertainty on the settlement may be derived as follows

$$\frac{\Delta s_c}{s_c} = \frac{\Delta F(\sigma'_v)}{F(\sigma'_v)} - \frac{\Delta K}{K} \tag{40}$$

where K refers to K_{CPT} or K_{DPT} depending on the penetration test carried out.

In case of $\sigma'_v < \sigma'_c$, one can write that for the CPT test

$$\frac{\Delta\sigma'_v}{\sigma'_{v0}} = \frac{\sigma'_v - \sigma'_{v0}}{\sigma'_{v0}} < \frac{\sigma'_c - \sigma'_{v0}}{\sigma'_{v0}} = \frac{\sigma'_c}{\sigma'_{v0}} - 1 = \frac{\sigma'_c}{q_c}\frac{q_c}{\sigma'_{v0}} - 1 = \lambda\frac{q_c}{\sigma'_{v0}} - 1 \tag{41}$$

The function F may therefore be written in this case as follows

$$F(\sigma'_v) = \frac{\dfrac{\Delta\sigma'_v}{\sigma'_{v0}}}{1+\beta\dfrac{\Delta\sigma'_v}{\sigma'_{v0}}} = \frac{x}{1+\beta x} < \frac{\lambda\dfrac{q_c}{\sigma'_{v0}} - 1}{1+\beta\left(\lambda\dfrac{q_c}{\sigma'_{v0}} - 1\right)} = \frac{y}{1+\beta y} < y \tag{42}$$

where x and y refer respectively to $\Delta\sigma'_v/\sigma'_{v0}$ and $\lambda q_c/\ \sigma'_{v0} - 1$. The relative uncertainty $\Delta F/F$ may then be bounded by $\Delta y/y$ which is itself equal to $\Delta\lambda/\lambda$. Rearranging Eq. (40) leads to bound the relative uncertainty on s_c such as

$$\frac{\Delta s_c}{s_c} = \frac{\Delta F(\sigma'_v)}{F(\sigma'_v)} - \frac{\Delta K}{K} \le \frac{\Delta\lambda}{\lambda} - \frac{\Delta K}{K} \tag{43}$$

In case of the CPT test, the analysis of the Gauss functions fitting the histograms of K_{CPT} and λ_{CPT} showed that $\Delta K_{CPT}/K_{CPT} = 32.6\%$ and $\Delta\lambda_{CPT}/\lambda_{CPT} = 24.1\%$, which leads to $\Delta s_c/s_c$ less than 8.5%. Similarly, in case of the DPT test, $\Delta K_{DPT}/K_{DPT} = 21.7\%$ and $\Delta\lambda_{DPT}/\lambda_{DPT} = 20.3\%$, finally the Eq. (43) leads to $\Delta s_c/s_c$ less than 2.0%.

These results show the slight effect of the variability of the parameters on the settlement prediction which encourages to recommend the two methods presented in this paper as practical tools of estimation of the 1D settlement based on the CPT or DPT tests data. Further work will focus on a direct validation of these methods by comparing their predicted settlements to the ones measured by the oedometer test.

It is finally recommended to follow the step-by-step procedure described hereafter to compute the 1D settlement of a shallow foundation embedded in a saturated clayey soil based on the CPT or the DPT tests.

8. Proposal of a methodology of computation of the settlement

(1) Divide the active zone of settlement beneath the foundation base into N thin slices and estimate q_c or N_{20} at mid-length of the layer. The thickness h of such a zone with respect to the foundation base corresponds to an effective stress increment $\Delta\sigma'_v$ negligible with respect to the net applied pressure at the base, say less than 10%,
(2) Estimate M_0 by correlation according to Eq. (12) or Eq. (13) by taking K_{CPT} = 1.24 and K_{DPT} = 0.93 respectively,
(3) Estimate σ_c by correlation according to Eq. (14) or Eq. (15) by taking λ_{CPT} = 0.18 or λ_{DPT} = 12 respectively,
(4) Compute the function F(σ'_v) according to Eq. (36) or graphically from the Fig. 7, the factor β taking the value 0.458 if $\sigma'_v < \sigma'_c$ and 0.137 if $\sigma'_v > \sigma'_c$,
(5) Compute the settlement of each slice according to Eq. (34) or Eq. (35), by assigning to the coefficient of calibration C a value of 1.32 for the CPT test and 0.81 for the DPT test,
(6) Compute the total settlement by the Eq. (39).

9. Conclusions

Static and dynamic penetration tests are the most commonly used in-situ tests carried out within a routine site investigation.

In this paper two practical methods of computation of the 1D consolidation settlement of shallow foundations in clayey soil on the basis of the CPT test or the DPT are proposed. The process of development of each method was based on the selection among three approaches of evaluation of the constrained modulus. It was found the approach of bi-linearization of the compressibility curve combined to some simple correlations with the cone resistance q_c or the number of blows N_{20} gives the best predictive capability with respect to the conventional oedometer-based method. The characteristic value of the ratio s^{CPT}/s_c^{oed} or s^{DPT}/s_c^{oed} is around 1 which is encouraging seeing the multitude of approximations and hypotheses made along the process of development of such methods.

Calibration procedure was undertaken which led to predict almost the same amount of settlement given by the oedometer test-based method.

Further validation process will be carried out by testing the quality of prediction of these two methods with respect to the case studies compiled in another database.

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