

Prediction of embankment settlements over marine clay using piezocone penetration tests

Songyu Liu · Guojun Cai · Anand J. Puppala · Qizhu Tu

Received: 1 February 2010 / Accepted: 13 August 2010 / Published online: 26 October 2010
© Springer-Verlag 2010

Abstract The paper discusses the use of piezocone penetration testing (CPTU) and evaluates four methods of interpretation for predicting the deformation constrained modulus (M) used to calculate the consolidation settlement of cohesive soil. CPTU tests were performed at eight embankment sites in the Lianyungang marine clay and high-quality samples for laboratory testing were collected close to the CPTU test locations. The constrained modulus (M) values predicted using the different interpretation methods were compared with the reference values determined from the laboratory consolidation tests. A new correlation was also developed to predict M from the net cone tip resistance ($q_t - \sigma_{v0}$). The paper discusses a simple approach to the use of the CPTU method to estimate the settlements of expressway embankments

Keywords CPTU · Soft soil · Settlement · Constrained modulus

Résumé L'article discute de l'utilisation du test de pénétration au piézocône (CPTU) et évalue quatre méthodes

d'interprétation pour la prévision du module de déformation oedométrique (M) utilisé pour calculer le tassement de consolidation de sols cohérents. Les tests CPTU ont été réalisés sur huit sites de remblais reposant sur l'argile marine de Lianyungang et des échantillons de haute qualité ont été prélevés près des sites d'essais au CPTU pour des essais de laboratoire. Les valeurs de module de déformation oedométrique (M) prévus en utilisant différentes méthodes ont été comparées avec les valeurs de référence déterminées à partir des essais de consolidation au laboratoire. Une nouvelle corrélation a également été développée pour prévoir M à partir de la résistance au piézocône ($q_t - \sigma_{v0}$). L'article discute d'une approche simplifiée concernant l'utilisation de la méthode CPTU pour estimer les tassements de remblais d'autoroutes.

Mots clés CPTU · Sol mou · Tassement · Module oedométrique

Introduction

In geotechnical practice, settlement and stability are two primary considerations related to the design of an embankment on soft ground (Ladd and Foott 1977; Ladd 1991; Yin 1999). With the development of expressways and high-speed railways in China, the prediction of embankment settlement becomes increasingly important. Generally, elastic theory is used to predict settlement, based on the deformation modulus and coefficient of consolidation of the soil. Both these parameters can be estimated from laboratory tests, such as the one-dimensional (1-D) consolidation test, conducted on the samples extracted from the field at different depths. However, most recovered samples have been subjected to a certain degree

S. Liu · G. Cai (✉) · Q. Tu
Institute of Geotechnical Engineering, Southeast University,
Nanjing 210096, China
e-mail: focuscai@163.com

S. Liu
e-mail: liusy@seu.edu.cn

Q. Tu
e-mail: 277142342@qq.com

A. J. Puppala
Department of Civil and Environmental Engineering,
The University of Texas at Arlington,
Arlington, TX 76019, USA
e-mail: anand@uta.edu

of disturbance during sampling, handling and transportation. In addition, the laboratory testing on small soil samples with some fissures or layering can be misleading hence laboratory-derived geotechnical parameters are unlikely to be entirely representative of the actual in situ soil conditions (Pitts 1984; Hight et al. 1992; Nash et al. 1992; Zhang 1999).

The piezocone penetration test (CPTU) has become more acceptable and popular for in situ testing, subsurface investigation and evaluation of soil properties and there is now much in the literature on this robust, fast and economical method which provides continuous soundings of the subsurface soil with depth (Lunne et al. 1997; Liu and Wu 2004; Cai et al. 2010). The piezocone penetrometer is capable of measuring the cone tip resistance (q_c), sleeve friction (f_s), and pore pressures at different locations, depending on the location of the pore water pressure transducer [at the cone face (u_1), behind the base (u_2), or behind the sleeve (u_3)]. These measurements can be used for soil stratification and identification and to evaluate different soil properties such as the strength and deformation characteristics, hence the CPTU is valuable for a wide range of geotechnical engineering applications (Konrad and Law 1987; Abu-Farsakh 2004; Abu-Farsakh and Nazzal 2005; Abu-Farsakh et al. 2007; Cai et al. 2009).

Several attempts have been made to calculate the consolidation settlement of soils using parameters derived from in situ cone and piezocone penetration tests (e.g., Crawford and Campanella 1991; Oakley and Richard 1990; Kuo-Hsia et al. 1994). Crawford and Campanella (1991) compared the measured settlements of earth embankments with settlements calculated from laboratory consolidation tests, in situ piezocone tests and dilatometer tests and reported that whilst there was good agreement between the three methods, the actual settlement was approximately 60% greater than the average calculated value. Oakley and Richard (1990) found that there was reasonable comparison between the calculated settlement using cone penetration test (CPT) data and the actual settlement, but the time rates of settlement were only within 150% of the actual field measurements. Kuo-Hsia et al. (1994) compared the CPT predicted settlement with the measured settlement of an instrumented test embankment. They reported that the CPT appeared to be the most valuable basis for assessing the constrained moduli and hence predicting the total settlement of soft soils.

A reliable estimate of the settlement of structures on soft soil deposits requires a more accurate evaluation of the consolidation parameters of foundation soils. The total consolidation settlement can be estimated from deformation moduli, such as the constrained modulus (M). In this paper, different interpretation methods were reviewed to estimate the constrained modulus (M) from the CPTU data.

The consolidation settlements of soils were calculated using parameters derived from in situ piezocone penetration tests in Lianyungang marine clay sites and compared with field measured settlements of a bridge embankment.

Objective and scope

The main objective of this study was to evaluate the current methods and develop a new correlation to reasonably predict the deformation constrained modulus (M) needed to estimate the magnitude of consolidation settlement of cohesive soils using the CPTU data. The methods investigated included those proposed by Sanglerat (1972), Senneset et al. (1989), Kulhawy and Mayne (1990) and Jones and Rust (1995). In situ CPTU tests were performed on saturated marine clays at eight sites in Jiangsu Province and a laboratory testing program was undertaken to establish reference soil parameters. The capability of the different methods to reasonably estimate M was evaluated and a new correlation based on net tip resistance was developed. The results of this study were verified by comparing the magnitude of consolidation settlements predicted from the proposed CPTU method with the actual field measured settlements at two selected sites.

Interpretation of constrained modulus based on CPTU data

Many correlations have been developed to relate the laboratory measured constrained modulus, M , and cone tip resistance. The cone tip resistance can be measured (q_c), corrected (q_t) or net (q_n). Generally, the one-dimensional constrained modulus, M , as measured in an oedometer test, has been expressed in terms of a material coefficient, α_m , and cone resistance:

$$M = \alpha_m q_c \quad (1)$$

where α_m = correlation factor and q_c is the measured cone resistance.

In practice, it has been usual to correlate the modulus M with penetration resistance (e.g., Mitchell and Gardner 1975; Schmertmann 1978; Jamiolkowski et al. 1985; Senneset et al. 1989; Kulhawy and Mayne 1990). Sanglerat (1972) developed a comprehensive array of α_m values for different soil types with different cone resistance values (Table 1). Jones and Rust (1995) found that for South African alluvial clay, a value of $\alpha_m = 2.75 \pm 0.55$ gives a good correlation with M . Senneset et al. (1988) developed a correlation between M and corrected cone resistance (q_t). For silty soils, they obtained the following linear correlation:

Table 1 Estimation of constrained modulus, M , for clays (after Sanglerat 1972)

$M = a_m \times q_c$			
$q_c < 0.7$ MPa		$3 < a_m < 8$	Clay of low plasticity (CL)
$0.7 < q_c < 2.0$ MPa		$2 < a_m < 5$	
$q_c > 2.0$ MPa		$1 < a_m < 2.5$	
$q_c > 2.0$ MPa		$3 < a_m < 6$	Silts of low plasticity (ML)
$q_c < 2.0$ MPa		$1 < a_m < 3$	
$q_c < 2.0$ MPa		$2 < a_m < 6$	Highly plastic silts and clays (MH, CH)
$q_c < 1.2$ MPa		$2 < a_m < 8$	Organic silts (OL)
$q_c < 0.7$ MPa	$50 < w < 100$	$1.5 < a_m < 4$	Peat and organic clay (P_t , OH)
	$100 < w < 200$	$1 < a_m < 1.5$	
	$w > 200$	$0.4 < a_m < 1$	

w = water content

$$M = 2 \times q_t \quad q_t < 2.5 \text{ MPa} \tag{2}$$

$$M = 2 \times q_t \quad q_t < 2.5 \text{ MPa} \tag{3}$$

$$M = 4 \times q_t - 5 \quad 2.5 < q_t < 5 \text{ MPa} \tag{4}$$

The corrected cone tip resistance (q_t) is given by the following equation:

$$q_t = q_c + u_2(1 - a) \tag{5}$$

where a is the cone area ratio, equal to 0.80 for the 10 cm² piezocone used in this study.

For interpretation of the CPTU data, the constrained modulus, M , may be expressed as a function of the net cone resistance (q_n). For Glava clayey soils, Senneset et al. (1989) related the constrained modulus (M) to a linear interpretation of the net cone tip resistance (q_n) and proposed the followings:

For over-consolidated clays:

$$M_{oc} = \alpha_o \times q_n = \alpha_o \times (q_t - \sigma_{vo}) \tag{6}$$

For normally consolidation clays:

$$M_{nc} = \alpha_n \times q_n = \alpha_n \times (q_t - \sigma_{vo}) \tag{7}$$

where M_{oc} is the constrained modulus for the over-consolidated range; M_{nc} is the constrained modulus for the

normally consolidated range. For most clays, $\alpha_o = 10 \pm 5$, $\alpha_n = 6 \pm 2$; σ_{vo} is the total overburden stress.

To estimate the one-dimensional constrained modulus M , the correlation with net cone resistance ($q_t - \sigma_{vo}$) is used in the form (Kulhawy and Mayne 1990):

$$M = 8.25 \times q_n = 8.25 \times (q_t - \sigma_{vo}) \tag{8}$$

Some correlations have been proposed between China's single or double bridge CPT data and the constrained modulus M . Table 2 presents a summary of the main correlations available to calculate the constrained modulus from CPTU data.

Evaluation of methods to predict constrained modulus

Several sites were selected in three expressways in Jiangsu province and boreholes were drilled in each site in order to recover high-quality 76 mm Shelby tube samples from different depths. The laboratory testing program included basic soil characterization tests (water content, unit weight, Atterberg limits, grain size distribution including hydrometer tests and specific gravity) and one-dimensional consolidation tests (Table 3). CPTU tests were conducted

Table 2 Summary of methods to estimate M based on CPT/CPTU data

Methods	Correlations	CPTU parameters	Correlation factor	Comments
Sanglerat (1972)	$M = \alpha_m \times q_c$	q_c	α_m depends on soil types and cone tip resistance	
Jones and Rust (1995)	$M = \alpha_m \times q_c$	q_c	$\alpha_m = 2.75 \pm 0.55$	South African alluvial clay
Senneset et al. (1988)	$M = 2 \times q_t$	$q_t = q_c + u_2(1 - a)$	Linear correlation	$q_t < 2.5$ MPa
	$M = 4 \times q_t - 5$	$q_t = q_c + u_2(1 - a)$	Linear correlation	$2.5 < q_t < 5$ MPa
Senneset et al. (1989)	$M_{oc} = \alpha_o \times q_n$	$q_n = q_t - \sigma_{vo}$	$\alpha_o = 10 \pm 5$	OC
	$M_{nc} = \alpha_n \times q_n$	$q_n = q_t - \sigma_{vo}$	$\alpha_n = 6 \pm 2$	NC
Kulhawy and Mayne (1990)	$M = 8.25 \times q_n$	$q_n = q_t - \sigma_{vo}$	8.25	
Tongji University	$M = 3.11 \times p_s + 1.44$	p_s	Linear correlation	Coastal clay of Southeast China
Wuhan	$M = 3.72 \times p_s + 1.26$	p_s	Linear correlation	$0.3 < p_s < 5$ MPa

p_s = specific penetration resistance

Table 3 Summary of soil properties for investigated sites

Site	Soil layer	Depth (m)	Unit weight/ (kN m^{-3})	Water content (%)	Specific gravity	Liquid limit (%)	Plasticity index (%)	M (MPa)
Lianyan Expressway (Lianyungang)	Clay	1.55–2.10	17.7	42.1	2.76	54.3	24.5	4.54
	Marine clay	10.50–12.85	15.1–16.4	57.0–81.4	2.75–2.76	55.0–63.5	21.6–24.6	0.79–1.57
Huaiyan Expressway (Yancheng)	Silty clay	21.6–24.10	18.0–19.8	23.4–38.9	2.72–2.76	26.1–52.4	12.5–23.5	4.98–13.5
	Clay	1.75–2.75	18.4–18.5	36.8–39.6	2.71–2.74	30.5–38.3	11.8–18.5	3.1–3.7
	Soft clay	12.25–13.1	14.2–17.3	42.8–95.6	2.74–2.76	42.3–76.1	20.1–39.6	1.1–1.7
Linlian Expressway (Lianyungang)	Silty clay	15.40–26.1	19.6–20.0	23.4–28.8	2.71–2.72	31.0–33.4	11.8–14.9	3.5–7.9
	Clay	1.8–2.4	1.83–1.86	32.0–38.0	2.74	50.4–51.3	25.1–26.5	2.6–6.8
	Marine clay	11.4–12.7	1.56–1.57	69.2–71.9	2.74	57.0–62.6	30.4–34.1	0.82–1.3
	Clay	15.8–17.1	1.86–1.91	24.6–34.8	2.74	42.5–52.0	19.8–27.6	6.8–7.62

close to the boreholes and the profiles used to calculate the average cone tip resistance (q_t) values at the same depths as the Shelby samples. Average total overburden pressures (σ_{vo}) were estimated.

The predicted constrained modulus (M_p) obtained following the methods of Sanglerat (1972), Senneset et al. (1989), Kulhawy and Mayne (1990) and Jones and Rust (1995) were compared with the measured constrained modulus (M_m) obtained from the one-dimensional consolidation tests (Fig. 1a–d). The figures also present the best-fit line (M_{fit}) of the predicted (M_p) to measured constrained modulus (M_m) and the corresponding coefficients of determination (R^2).

The results indicate that the Sanglerat (1972) and Kulhawy and Mayne (1990) methods over-estimate the constrained modulus by a factor of 1.24–2.44, whereas the Jones and Rust (1995) method tends to under-estimate the constrained modulus. The Senneset et al. (1989) method, however, gives a good prediction of the measured constrained modulus with the best-fit line of $(M_p/M_m) = 1.07$ and $R^2 = 0.97$. This is consistent with the comments by Lunne et al. (1997), who recommend using correlations specific to a given site and updating existing correlations with local experience.

It should be noted here that the different CPTU- M prediction methods evaluated in this study were developed using CPTU data collected from different location sites with possibly different soil deposits. The methods may perform well in some cases but not in others, depending on the local soil type. This emphasizes the importance of using local experience to calibrate existing methods or develop better correlations between the CPTU data and the different soil parameters such as the constrained modulus with greater reliability.

Case study

Piezocone penetration tests

The field investigation consisted of piezocone testing at existing embankments where settlement records were available. The piezocone penetration device used in this study is produced by Vertek-Hogentogler & Co. of USA. The CPTU equipment comprised a versatile piezocone system equipped with advanced digital cone penetrometers fitted with 60° tapered and 10 cm^2 tip area cones which can provide measurements of cone tip resistance q_t , sleeve friction f_s , and penetration pore-water pressures (u_2) with a porous filter 5 mm thick located at shoulder position u_2 . The data logging system was connected to both a chart recorder and a laptop computer so that instantaneous read-out of cone resistance and pore pressure was available. Water pressure can act on part of the base area of the cone in a direction opposite to the resisting forces developed during pushing, thus the measured total stress q_c will be reduced. To obtain the actual total stress q_t , a correction must be made to account for the design of the cone (Cai et al. 2005). This is made directly by the field computer data acquisition system (E4FCS) during data processing for each set of readings through the following relationship:

$$q_t = q_c + 0.2 \times u_2 \quad (9)$$

Due to the presence of fill material at the sites, prior to cone penetration testing shallow pre-drilling was required until saturated soil was detected. The penetration rate for all the tests in this study was 20 mm/s, and the readings were recorded every 50 mm of penetration. The only difference between the standard CPT and CPTU is that the piezocone of the CPTU must be saturated first.

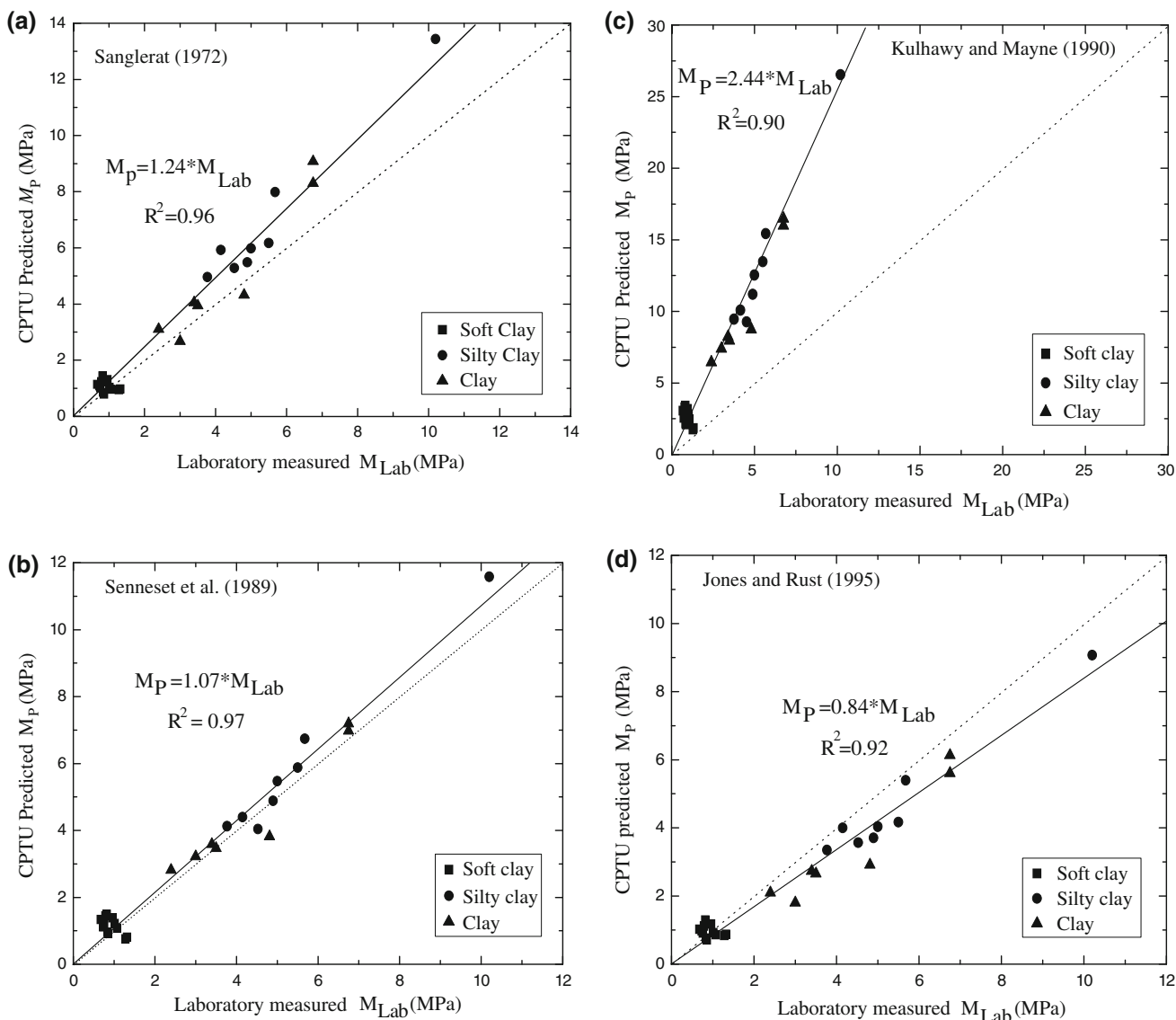


Fig. 1 Measured versus predicted M using four CPTU methods

Site conditions

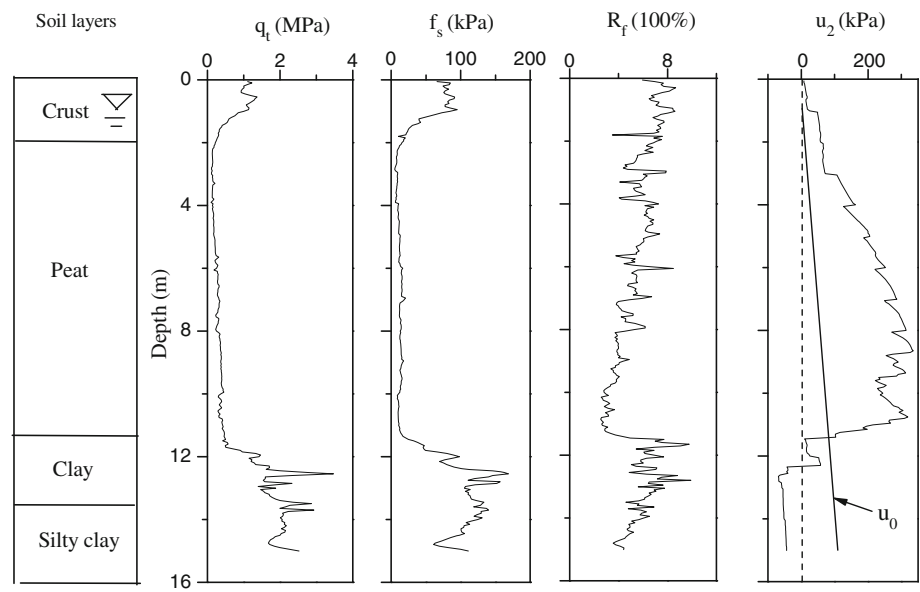
The Lianyan Expressway is located in the coastal area of Eastern Jiangsu province, where Quaternary marine clays are widely distributed. The 10–12 m thick marine sediment overlies silty clay, the top 1.5–2.0 m of which is a surface crust. The depth to groundwater varies from 0.5 m to 1.8 m. Sand interlayers can be found below 9.0 m and marine shells are scattered in the whole clay layer. These deposits are characterized by a very high water content, high plasticity, high sensitivity, high compressibility, low shear strength and low permeability (Liu et al. 2008).

The piezocones were located at K15 + 000, K15 + 200, K17 + 000, K20 + 800, K21 + 200, K27 + 550, K28 + 900 and K30 + 000 sections of Lianyan Expressway. As the geological profile was established at pre-construction stage,

the CPTU tests were carried out in virgin soil adjacent to the selected embankments. Typical profiles of q_t , f_s and u_2 , the depth of the piezocone tests and the groundwater level are shown in Fig. 2 for the chainage K15 + 000. The profiles of the CPTU sounding were used to classify the soil using the soil type classification chart.

Figure 2 indicates a relatively homogeneous marine clay deposit with a 2 m thick surface crust. The soft soil layer is present below the crust. The depth extends to a depth of 11.5 m and has a water content of about 75.4%. Below this layer, the average water content decreases to about 52.9%. Below 2 m q_t increases slightly, below which it increases markedly. This increase in pressure coincides with a notable drop in pore pressure. The change in tip resistance and pore pressure coincides with a change in the materials, as identified from the drilling reports.

Fig. 2 Typical CPTU profiles for chainage K15 + 000



The eight sections were all pre-loaded hence comparisons of measured settlements with laboratory test-based predictions and with CPTU-based predictions require a definition of settlement and its components. The measured total settlements were divided into immediate settlement, primary consolidation and secondary compression. In some cases this sub-division was possible on the basis of available field and laboratory results while in other cases some assumptions were necessary regarding stress conditions, relative values of drained and undrained moduli and Poisson’s ratio. The results show that the overall conclusions are not sensitive to these assumptions; nevertheless the distinction of the individual settlement components is important (Jones and Rust 1995).

Table 4 presents a brief summary of each embankment giving the height and the laboratory predicted and field measured settlements from the settlement plate. An estimate is then made of the yield and secondary components of the measured settlement, and these are subtracted from

the total settlement so that the α_m calculated represents only the immediate and primary consolidation settlements. At a number of embankments more than one data set is given; this reflects the extent of the available settlement records which could be correlated with piezocone results.

Back analysis of correlation factor

The results indicate that, as expected, for the Lianyungang marine clay, the relations based on net cone tip resistance (q_n) predicted the measured constrained modulus (M) of the different sites better than the other two CPTU measured parameters (q_c and q_t). In order to calibrate existing methods or develop correlations with greater reliability, the correlation factor (α_m) was back-analyzed based on CPTU data and measured settlements of the K15 + 000 and K15 + 200 sections of the Lianyan Expressway. According to the layer-wise summation method widely used in China, the total settlement, S , can be predicted using the piezocone data through the evaluation of the constrained modulus, M , using the following equation:

$$S = \sum_{i=1}^n \frac{\Delta P_i \Delta h_i}{M_i} = \sum_{i=1}^n \frac{\Delta P_i \Delta h_i}{\alpha \cdot q_i} = \frac{1}{\alpha} \sum_{i=1}^n \frac{\Delta P_i \Delta h_i}{q_i} \tag{10}$$

$$\alpha = \frac{1}{S} \sum_{i=1}^n \frac{\Delta P_i \Delta h_i}{q_i} \tag{11}$$

where S is the total settlement; ΔP_i is the applied stress in the i soil layer (kPa); Δh_i is the thickness of the i soil layer; M_i is the constrained modulus of the i soil layer (kPa); q_i is the cone tip resistance of the i soil layer (kPa), such as measured cone tip resistance q_c , corrected cone tip resistance q_r , net cone tip resistance q_n . α is the correction factor, as shown in Table 2. α can be α_c corresponding to

Table 4 Measured and predicted settlement data for Lianyan CPTU sites

Number	Location	Embankment height	Lab test-based settlement (mm)	Measured settlement (mm)
1	K15 + 000	2.38	450	713
2	K15 + 200	1.96	476	651
3	K17 + 000	2.48	433	616
4	K20 + 800	2.28	374	282
5	K21 + 200	2.81	573	760
6	K27 + 550	2.20	375	343
7	K28 + 900	1.96	373	421
8	K30 + 000	1.46	227	392

Table 5 Constrained modulus correlation factor from back-analysis

Number	Embankment	Settlement (mm)	α_c	α_t	α_n
1	K15 + 000	713	2.81	3.27	3.67
2	K15 + 200	651	3.24	3.07	3.55
	Mean		3.03	3.17	3.61

measured cone tip resistance. α can be α_t corresponding to corrected cone tip resistance. α can be α_n corresponding to net cone tip resistance.

The back-analyzed correction factors are shown in Table 5 which indicates the mean of α_c is equal to 3. Evaluating the embankment settlement based on the CPTU data, the mean value of α_t is 3.20 and the mean value of α_n is 3.60.

Verification of settlement results

The results of this study were verified by comparing the predicted consolidation settlements using the three different back-calculated correlation factors with the measured field settlements in six other expressway sites. For settlement calculations in these sites, the constrained modulus (M) for each soil layer was predicted using both the measured cone tip resistance, corrected cone tip resistance and the net cone tip resistance value of the soil layer. The embankments were instrumented with settlement plates to monitor the consolidation settlement with time. The observed and predicted results for vertical settlement are shown in Table 6.

The comparison shows that the CPTU interpretation method predicted the total consolidation settlement better than the laboratory calculations. In addition, the variation in the laboratory-calculated results is much greater than with the CPTU interpretation method. This is an indication of sample disturbance. The laboratory method over-estimated the settlement values by 42% for the section of K17 + 000. However, the net tip resistance and corrected

cone tip resistance can predict the total consolidation settlements better than measured cone tip resistance; the corrected cone tip resistance gave a value of 98% of the actual field settlement. The net cone tip resistance correlation factor gave a value of 99.5% of settlement, indicating good agreement between CPTU-predicted and field measured settlements.

The study has shown that for the Lianyungang marine clay (mainly peat), the deformation constrained modulus can reasonably predict the magnitude of settlement based on corrected cone tip resistance and net cone tip resistance, with correlation factors of $\alpha_t = 3.20$ and $\alpha_n = 3.60$. The Senneset et al. (1989) method based on q_n shows a good prediction of the laboratory measured constrained modulus with the correlation factor α_n equal to 3.60 and $R^2 = 0.96$ (Fig. 3). This proved that the laboratory tests results on undisturbed samples are reliable. However, the laboratory tested constrained modulus over-estimated the settlement values by 21% for the six sections.

Summary and conclusions

CPTU interpretation methods to predict the deformation constrained modulus (M) were evaluated for use in the calculation of the consolidation settlement of cohesive soil. Eight sites in the Jiangsu province of China were used in this study. In each site, CPTU soundings were performed and used to predict the M values using the different interpretation methods. The predicted M values obtained from the different CPTU interpretation methods were compared with the laboratory reference M values. The reliability of the different interpretation methods was evaluated. Based on the field measurement and back-analysis settlement data, a new CPTU-based method for predicting the settlement of embankment on soft clays is proposed, which uses the net cone resistance q_n and requires no additional experimental process for soil sampling and laboratory testing. The following conclusions are drawn:

Table 6 Comparison of CPTU-predicted, laboratory-calculated and field-measured settlements for the six embankments

Number	Location	Measured S (mm)	Laboratory-calculated S_{lab} (mm)	S/S_{lab}	$\alpha_c = 3.00$		$\alpha_t = 3.20$		$\alpha_n = 3.60$	
					CPTU-predicted S_{qc} (mm)	S/S_{qc}	CPTU-predicted S_{qt} (mm)	S/S_{qt}	CPTU-predicted S_{qn} (mm)	S/S_{qn}
1	K17 + 000	616	433	1.42	725	0.84	661	0.93	645	0.96
2	K20 + 800	282	374	0.75	299	0.94	297	0.95	270	1.04
3	K21 + 200	760	573	1.33	833	0.91	796	0.95	767	0.99
4	K27 + 550	343	375	0.91	398	0.86	363	0.94	361	0.95
5	K28 + 900	421	373	1.13	442	0.95	394	1.06	400	1.05
6	K30 + 000	392	227	1.72	439	0.89	368	1.06	402	0.98
Mean				1.21		0.90		0.98		0.995

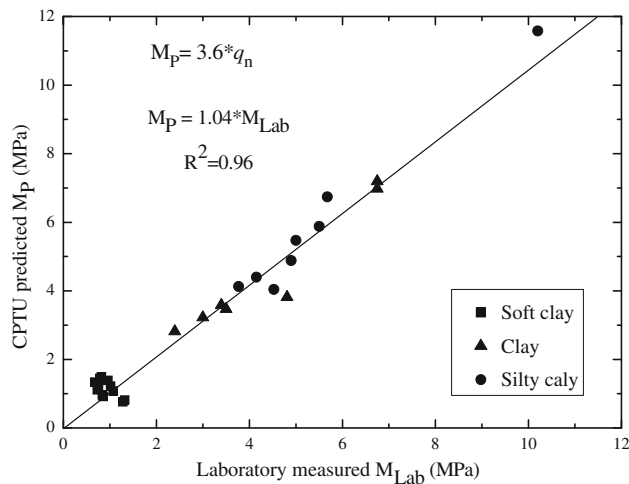


Fig. 3 Comparison of laboratory-measured and CPTU-predicted constrained modulus based on net tip cone resistance

1. The predicted constrained modulus (M_p) obtained from four CPTU interpretation methods were compared with the measured constrained modulus (M_m) from the one-dimensional consolidation laboratory tests, and the best fit line of (M_p/M_m) and the corresponding coefficient of determination (R^2) were determined. A comparison of the results and arithmetic analysis (mean and standard deviation) showed that the Senneset et al. (1989) method has a better prediction of M than the other three methods. The Sanglerat (1972) and Kulhawy and Mayne (1990) methods over-estimate the constrained modulus by a factor of 1.24–2.40 times, while the Jones and Rust (1995) method under-estimates the constrained modulus by a factor of 0.84 times. Linear correlations were developed between M and the net cone resistance q_n with a correlation factor ($\alpha_n = 4.0$).
2. The findings of this study were verified by comparing the predicted settlements using the proposed CPTU correlation and the back-analysis, laboratory-calculated settlement, and actual field settlements measured using settlement plates in the eight selected sections. The results demonstrated that the CPTU method can predict the magnitude of settlement better than the laboratory-calculated estimates using parameters obtained from the consolidation tests. For the Lian-yungang marine clay, the correlation factors between the constrained modulus and CPTU parameters (e.g., q_c , q_t , q_n) are $\alpha_c = 3.00$, $\alpha_t = 3.20$, $\alpha_n = 3.60$ from the settlement back-analysis.
3. The results also showed that the constrained modulus from the net cone resistance q_n can reasonably predict the consolidation settlements.

In general, the CPTU method is fast, continuous, reliable, easy to operate and independent of all the operator-subjective influence involved in the traditional combination of borings, sampling and testing. As a consequence, it is considered particularly suitable for use in predicting embankment settlements.

Acknowledgments This research was supported by the National Natural Science Foundation of China (NSFC) (Grant No. 40702047), the Excellent Doctoral Dissertation Foundation of Southeast University, China (Grant No. YBJJ0714) and the Jiangsu Transportation Research Foundation (Grant No. 8821006021). These financial supports are gratefully acknowledged.

References

- Abu-Farsakh MY (2004) Evaluation of consolidation characteristics of cohesive soils from piezocone penetration tests. FHWA/LA.04/386, Louisiana Transportation Research Center, Baton Rouge
- Abu-Farsakh MY, Nazzal MD (2005) Reliability of piezocone penetration test methods for estimating the coefficient of consolidation of cohesive soils. *Transp Res Rec* 1913:62–76
- Abu-Farsakh MY, Zhang ZJ, Gautreau G (2007) Evaluating deformation modulus of cohesive soils from piezocone penetration test for consolidation settlement. *Transp Res Rec* 2004:49–59
- Cai GJ, Liu SY, Tong LY, Du GY (2005) General factors affecting interpretation and corrections of primary data for the piezocone penetration test (CPTU) data. *J Eng Geol* 14(5):632–636 in Chinese
- Cai GJ, Liu SY, Tong LY, Du GY (2009) Assessment of direct CPT and CPTU methods for predicting the ultimate bearing capacity of single piles. *Eng Geol* 104(1):211–222
- Cai GJ, Liu SY, Tong LY, Du GY (2010) Field evaluation of undrained shear strength from piezocone penetration tests in soft marine clay. *Mar Georesour Geotechnol* 28(2):143–153
- Crawford CB, Campanella RG (1991) Comparison of field consolidation with laboratory and in situ tests. *Can Geotech J* 28(1):103–112
- Hight DW, Boese R, Butcher AP, Clayton CRI, Smith PR (1992) Disturbance of the Bothkennar clay prior to laboratory testing. *Géotechnique* 42(2):199–217
- Jamiolkowski M, Ladd CC, Germaine JT, Lancellotta R (1985) New developments in the field and laboratory testing of soils: State of the art. In: *Proceedings of the 11th international conference on soil mechanics and foundation engineering*, vol 1, San Francisco, pp 57–153
- Jones GA, Rust E (1995) Piezocone settlement prediction parameters for embankment on alluvium. In: *Proceedings of the international symposium on penetration testing, CPT95*, vol 2, Linköping, pp 501–508
- Konrad JM, Law KT (1987) Preconsolidation pressure from piezocone tests in marine clay. *Géotechnique* 37(2):177–190
- Kulhawy F, Mayne PH (1990) *Manual on estimating soil properties for foundation design*. Electric Power Research Institute, Palo Alto
- Kuo-Hsia C, William DK, Ming-Jiun W (1994) Comparison of predicted and measured settlement of a test embankment over soft soil. In: *Proceedings of the settlement '94*. ASCE geotechnical special publication, vol 40, pp 16–18
- Ladd CC (1991) Stability evaluation during staged construction. *J Geotech Eng ASCE* 117(4):540–615

- Ladd CC, Foott R (1977) Foundation design of embankments constructed on varved clays. United States Department of Transportation, Report FHWA TS-77-214, p 234
- Liu SY, Wu YK (2004) On the strategy and development of CPT in China. *Chin J Geotech Eng* 26(4):553–556 in Chinese
- Liu SY, Cai GJ, Tong LY, Du GY (2008) Approach on the engineering properties of Lianyungang marine clay from piezocone penetration tests. *Mar Georesour Geotechnol* 26(7):189–210
- Lunne T, Robertson PK, Powell JJM (1997) Cone penetration testing in geotechnical practice. Blackie Academic and Professional, London
- Mitchell JK, Gardner WS (1975) In situ measurement of volume change characteristics, In: Proceedings of the ASCE specialty conference on in situ measurement of soil properties, vol 2, Raleigh, pp 279–345
- Nash DFT, Powell JJM, Lloyd IM (1992) Initial investigations of the soft clay test site at Bothkennar. *Géotechnique* 42(2):163–181
- Oakley III, Richard E (1990) Case history: use of the cone penetrometer to calculate the settlement of a chemically stabilized landfill. In: Proceedings of the geotechnics of waste fills-theory and practice. ASTM STP 1070, American society for testing and materials, Philadelphia, pp 345–357
- Pitts J (1984) A review of geology and engineering geology in Singapore. *Q J Eng Geol* 17:93–101
- Sanglerat G (1972) The penetrometer and soil exploration. Elsevier, Amsterdam
- Schmertmann JH (1978) Guidelines for cone penetration test, United States Department of Transportation, Report FHWA TS-78-209, 145 pp
- Senneset K, Sandven R, Lunne T, Amundsen T (1988) Piezocone tests in silty soils. In: Proceedings of the international symposium on penetration testing, ISOPT-1, A.A. Balkema, Rotterdam, pp 955–966
- Senneset K, Sandven R, Janbu N (1989) Evaluation of soil parameters from piezocone tests. *Transp Res Rec* 1235:24–37
- Yin ZZ (1999) Settlement and consolidation of soil. China Electric Power Press, Beijing in Chinese
- Zhang CH (1999) The application of piezocone penetration test. China Architecture and Building Press, Beijing in Chinese