Canadian Geotechnical Journal
 Revue canadienne de géotechnique

A modified method for estimating the permeability of clayey soils based on piezocone sounding results

Journal:	<i>Canadian Geotechnical Journal</i>
Manuscript ID	cgj-2017-0058.R2
Manuscript Type:	Article
Date Submitted by the Author:	01-Nov-2017
Complete List of Authors:	Chai, Jinchun; Saga University, Chanmee, Nachanok; Saga Daigaku, Civil Engineering and Architecture
Is the invited manuscript for consideration in a Special Issue? :	N/A
Keyword:	piezocone sounding, permeability, OCR, clay.

SCHOLARONE™
 Manuscripts

A modified method for estimating the permeability of clayey soils based on piezocone sounding results

Jinchun Chai and Nachanok Chanmee

J.-C. Chai¹ Department of Civil Engineering and Architecture, Saga University, Saga 840-8502,
Japan. (chai@cc.saga-u.ac.jp)

N. Chanmee Department of Civil Engineering and Architecture, Saga University, Saga 840-8502,
Japan.
(nachanok.ch@gmail.com)

¹ Corresponding author (e-mail: chai@cc.saga-u.ac.jp).

Abstract: A modified method for estimating the permeability of clayey deposits in the horizontal direction (k_h) based on piezocone sounding (uCPT) results was proposed. Most of the currently available methods relate the value of k_h with the $\Delta u/\sigma'_{v0}$ ratio (Δu is the measured excess pore pressure from uCPT at the shoulder of the cone, and σ'_{v0} is the initial vertical effective stress at the measurement location). However, it has been understood that the effect of the overconsolidation ratio (OCR) on the $\Delta u/\sigma'_{v0}$ ratio is more significant than that on the value of k_h . As a result, for an overconsolidated deposit, there is no suitable method for estimating the value of k_h based on uCPT results. Based on the results of laboratory model uCPT, an empirical equation for considering the effect of the OCR on $\Delta u/\sigma'_{v0}$ and then k_h corresponding to the yield vertical effective stress (σ'_{vmax}) and initial vertical effective stress (σ'_{v0}) for $OCR > 1.0$ is established. The modified method for k_h corresponding to σ'_{vmax} requires two parameters: OCR and a model parameter (α). In addition, for k_h corresponding to σ'_{v0} , two more soil parameters are needed: the swelling index (C_s) and the initial void ratio (e_0). The methods for estimating these parameters are also proposed. Finally, the modified method was applied to three field case histories to show its usefulness.

Keywords: piezocone sounding, permeability, OCR, clay.

1. Introduction

Piezocone sounding (uCPT) is an economic and efficient in situ testing method for determining the engineering properties of soils (Campanella and Robertson 1988; Lunne *et al.* 1997). uCPT provides nearly continuous measurements of tip resistance (q_t), sleeve friction (f_s) and pore water pressure (u_2) at the shoulder of a cone (standard cone). These measurements can be used to estimate the undrained shear strength of a deposit (s_u) (Campanella and Robertson 1988; Teh and Houlsby 1991; Lu *et al.* 2004), overconsolidation ratio (OCR) (Wroth 1984; Mayne and Kemper 1988; Robertson 1990; Chanmee *et al.* 2017) and permeability of soil in the horizontal direction (k_h) (Robertson *et al.* 1992; Lunne *et al.* 1997; Sully *et al.* 1999; Elsworth and Lee 2005; 2007; Robertson 2010; Chai *et al.* 2011). The cone can be held at a predetermined depth to measure the pore pressure dissipation process, and these dissipation results can be used to estimate the coefficient of consolidation of the soil at that depth (Teh and Houlsby 1991; Robertson *et al.* 1992; Burns and Mayne 1998; Chai *et al.* 2012a; Chai *et al.* 2015).

Regarding the method for estimating the value of k_h of soil strata, Elsworth and Lee (2005, 2007) proposed a method that is applicable only to sands and is not applicable to clayey soil deposits. Chai *et al.* (2011) modified Elsworth and Lee's method and made it applicable to almost all soil types in normally or lightly overconsolidated states ($\text{OCR} < 2.0$). While there are many clayey soil deposits with an $\text{OCR} \geq 2.0$ and even $1.0 < \text{OCR} \leq 2.0$, the effect of the OCR on the estimated value of k_h is still not clear. Robertson (2010) proposed a relationship between soil behavior type index (I_c) and permeability (k). However, Agung (2011) applied the methods of both Robertson (2010) and Chai *et al.* (2011) to two field cases in Japan, and it was shown that Robertson's method had poorer performance.

In this paper, the effect of the OCR on the measured pore water pressure (u_2) from uCPT, and therefore on the estimated value of k_h according to the method of Chai *et al.* (2011), is investigated using the results of laboratory model tests. Then, based on the model test results, a modified method for estimating the value of k_h of soil has been proposed. Finally, the modified method was applied to three field cases reported in the literature, and the effectiveness of the method is demonstrated.

2. Existing method

Wroth (1984) defined the dimensionless pore water pressure ratio (B_q) and dimensionless tip resistance (Q_t) of uCPT as:

$$[1] \quad B_q = \frac{\Delta u}{q_t - \sigma_{v0}}$$

$$[2] \quad Q_t = \frac{q_t - \sigma_{v0}}{\sigma'_{v0}}$$

where Δu is the measured excess pore water pressure, q_t is the total cone tip resistance, and σ'_{v0} and σ_{v0} are the initial effective and total vertical stresses, respectively.

Chai *et al.* (2011) proposed that the value of permeability (k_h) can be estimated as:

$$[3] \quad k_h = \frac{K_D U a \gamma_w}{2 \sigma'_{v0}}$$

where a is the radius of the cone, U is the rate of cone penetration, γ_w is the unit weight of water, and K_D is a dimensionless permeability index.

For $B_q Q_t < 0.45$ (sandy soils),

$$[4] \quad K_D = \frac{1}{B_q Q_t} = \frac{\sigma'_{v0}}{\Delta u}$$

and for $B_q Q_t \geq 0.45$ (clayey soils),

$$[5] \quad K_D = \frac{0.044}{(B_q Q_t)^{4.91}} = 0.044 \cdot \left(\frac{\sigma'_{v0}}{\Delta u} \right)^{4.91}$$

From Eqs (4) and (5), it can be seen that for estimating the value of k_h , $\Delta u/\sigma'_{v0}$ ratio is a key parameter.

3. Laboratory model tests

To investigate the effect of the OCR on the measured $\Delta u/\sigma'_{v0}$ ratio of uCPT, laboratory model tests were conducted using different soil types and OCR values.

3.1 Equipment

The device used for the laboratory model tests is shown in Fig. 1. The cylindrical chamber was made from PVC, with an inner diameter of 0.485 m and a height of 1.0 m. Air pressure was applied to the piston to preconsolidate the model ground. The settlement of the model ground was measured using a dial gauge.

The piezocones used in these laboratory experiments had a tip angle of 60° with diameters of 20 and 30 mm (first, a cone with a diameter of 30 mm was used; then, a new cone with a diameter of 20 mm was made and used for most of the tests reported in this study). The pore water pressure gauge was located at the shoulder of the cone (standard type). The penetration system consisted of a reaction frame and a motor to control the penetration rate.

3.2 Set-up and test procedures

(1) Model setup. First, silicon grease was smeared over the inner surface of the cylindrical chamber to reduce possible friction between the chamber and soils. Then, three layers of non-woven geotextile (approximately 138 g/m^2) were placed at the bottom of the cylindrical chamber to act as a drainage layer, and then six 0.1 m-wide geotextile strips were lined vertically along the inner periphery of the chamber to facilitate drainage via outward radial flow of the pore water. The soils were thoroughly mixed at a water content of approximately 1.2 times their liquid limits and left in the laboratory for one day before being used in the model. Then, the soil was carefully poured into the chamber layer by layer until the thickness of the model ground was 0.8 m. Each layer was thoroughly stirred by a stainless steel rod to avoid possible air bubbles trapped in the model. Therefore, it is considered that the initial model ground was almost in a saturated condition. Piezometers were placed at predetermined locations in the model to measure the pore water pressure during the preconsolidation process. Finally, three layers of the non-woven geotextile were placed on top of the model ground to act as a drainage layer, and the piston system was set-up.

(2) Preconsolidation. Air pressure was applied to the piston to preconsolidate the model ground. Once the degree of preconsolidation was more than 95% (based on the measured settlement curves as an example shown in Fig. 2), the air pressure was adjusted to achieve the desired OCR value. For example, first, the model ground was preconsolidated with a pressure of 96 kPa (applied air pressure was 100 kPa, but considering the effect of the shaft installed at the top of the piston, the effective consolidation pressure was approximately 96 kPa), and then the consolidation pressure was adjusted to 12 kPa (air pressure of 12.5 kPa) to yield an OCR value of 8.0. After the preconsolidation, the thickness of the model ground was approximately 0.6 m.

In the case of $OCR > 1$, unloading induced negative pore pressure in the model ground (Fig. 3). The initial measured excess pore water pressure was almost equal to the applied load increment, which implies that the model ground was almost saturated. In this case, the piezocone penetration was conducted after the negative excess pore pressure dissipated, which generally took a few days.

(3) Penetration test. The rate of the cone penetration adopted was 25 mm/min (0.4 mm/s), which is 50% of the maximum rate of the system. During the penetration process, the pore water pressures at the shoulder of the cone were recorded. When the shoulder of the cone reached 0.3 m from the surface of the model ground, the penetration was stopped, and then the dissipation tests were conducted (not analyzed in this study).

(4) Oedometer consolidation test. After uCPT, soil samples were taken to determine the value of permeability via multistage-load (MSL) oedometer tests and, for some cases, constant rate of strain (CRS) consolidation tests.

3.3 Limitations of the laboratory model test

(1) Lower penetration rate. The adopted cone penetration rate of 0.4 mm/s is much lower than that used in the field, i.e., 20 mm/s. Although under a rate of 0.4 mm/s, the model ground still deformed very close an undrained condition (Kim *et al.* 2008). The behavior of clays is strain-rate dependent, and a lower rate will have an effect on the cone penetration-induced excess pore water pressure.

(2) Boundary effect. For any model test, there is a scale and boundary effect. For the model test setup of this study, considering a disc soil slice with a diameter of 485 mm, when a cone with a diameter of 30 mm is used to penetrate the center of the disc, the volume of the portion of

the cone that has penetrated into the soil slice will be approximately 0.38% of the volume of the disc soil slice. For a cone with a diameter of 20 mm, the volume will be approximately 0.17% of the slice. Due to the constraining effect of the wall of the model, there was almost no horizontal radial displacement at the boundary, while heaving of the model ground was monitored. Chai *et al.* (2014) reported that the radial boundary constraining effect increased the measured pore pressure of the piezometer located near the boundary. The larger the diameter of the cone, the larger the effect. Regarding the pore pressure measured at the shoulder of the cone, for the conditions adopted, the effect was less significant when compared with that near the boundary.

The drainage boundary is also different from the field condition. In the field, in the horizontal radial direction, at an infinite distance drainage occurs. While, in the laboratory, though the six geotextile strips provided certain drainage, other locations remained undrained.

(3) Difference between natural deposit and reconstituted model ground. Most natural clays are micro-structured, i.e. there are certain cementation bonds between the clay particles. While the mechanically reconstituted clayey soils normally do not have this kind of micro-structure. The mechanical behavior of a natural soil and its reconstituted one is difference (Watabe et al. 2002).

Because of above limitations, the main purpose of the laboratory tests is to investigate the relative effect of OCR on measured $\Delta u/\sigma'_{v0}$ ratio, and not for validating the absolute value of k_h estimated from the laboratory uCPT results.

3.4 Soil samples, cases tested and test results

Five types of soil, namely, remolded Ariake clay and Ariake clay mixed with sand with sand/clay ratios (by dry weight) of 20:80 (Mixed soil 1), 50:50 (Mixed soil 2), 60:40 (Mixed soil

3), and 70:30 (Mixed soil 4) were used in the laboratory tests. The grain size distributions of the soils are shown in Fig. 4. The reconstituted model grounds had OCR values from 1 to 8. Some physical properties of these soils are listed in Table 1. The liquid limit (LL) and plastic limit (PL) were measured following JIS A 1205 (JIS, 1995). The cases tested and the results are summarized in Table 2, in which the values of Δu were the values measured when the shoulder of the cone reached 0.3 m depth from the surface of the model ground. The results of using the reconstituted Ariake clay and Mixed soil 3 are from Hossain (2014). In Table 2, the cases of OCR = 1.0, the measured Δu is larger than the corresponding yield vertical effective stress. In clay deposits, there are quite a number of cases reported in the literature that the value of Δu is larger than the yield vertical effective stress of the soil. For example, Kim *et al.* (2007) reported some laboratory model test results that for OCR = 1.0, Δu was about 1.6 times of the yield vertical effective stress.

3.5 Comparison of measured and estimated permeabilities

Using the method of Chai *et al.* (2011), the values of k_h were estimated, as listed in Table 2, and compared with the values of permeability in the vertical direction (k_v) as shown in Fig. 5. The values of k_v was interpreted from the results of MSL oedometer test under the corresponding effective vertical pressure. Since in engineering practice the values of k_v are generally measured in a laboratory and the estimated value of k_h is compared with the measured value of k_v in most studies, in this study, the estimated value of k_h is also directly compared with the measured value of k_v , but the measured ratios of k_h/k_v are provided and discussed.

For a given yield vertical effective stress (σ'_{vmx}), in the case of a higher OCR, a smaller value of σ'_{v0} acts on the sample. The measured values of k_v slightly increased with the OCR due to

swelling increased the void ratio, while the estimated values of k_h decreased with the OCR significantly. Eqs. [4] and [5] indicate that K_D is a function of $\Delta u/\sigma'_{v0}$ ratio, which increased with OCR from the measured data. As a result, the value of K_D and therefore, the estimated value of k_h (Eq. (3)) reduced with the OCR, which is in the opposite direction when compared with the measured value of k_v .

Tunne et al. (1989) reported the results of field uCPT from four sites that the ratio of $\Delta u_1/\sigma'_{v0}$ significantly increased with OCR as shown in **Fig. 6** (data from Tunne et al. 1989). Here Δu_1 is the excess pore pressure measured on the face of the cone, which is normally higher than that measured at the shoulder of the cone. While the tendency is the same as the results of the model tests reported in this study. Therefore, it can be said that the existing method for estimating the value of k_h from $\Delta u/\sigma'_{v0}$ ratio is not applicable for overconsolidated soils.

4. Modified method

4.1 Basic considerations

Since k_h is a function of $\Delta u/\sigma'_{v0}$ in most existing methods, the basic idea is to include the effect of the OCR on $\Delta u/\sigma'_{v0}$ into the method for estimating k_h . The following steps are proposed to do so.

(1) Based on the results of the laboratory test, estimate the value of $(\Delta u/\sigma'_{v0})_{NC}$ corresponding to $OCR = 1.0$ based on the measured value of $(\Delta u/\sigma'_{v0})_{OC}$ corresponding to $OCR > 1.0$, and then obtain the value of k_h corresponding to the yield vertical effective stress (σ'_{vmax}) via the method of Chai *et al.* (2011) and designate it as k_{h0} .

(2) When the value of k_h corresponding to the initial vertical effective stress (σ'_{v0}) is required, it can be approximately evaluated using Taylor's (1948) equation:

$$[6] \quad k_h = k_{h0} \cdot 10^{\left(\frac{\Delta e}{C_k}\right)}$$

where C_k is a constant and Δe is the change in the void ratio. Taylor's equation was originally proposed for clay soils. However, according to the test results, it can also be successfully applied to clayey sand for OCR values up to 8. The value of Δe can be calculated as:

$$[7] \quad \Delta e = C_s \log(OCR)$$

where C_s is the swelling index. In Eq. [6], if the laboratory measured value of permeability is available, it should be used to calibrate the value of C_k . If it is not available, $C_k = 0.5e_0$ can be used as suggested by Tavenas *et al.* (1983). Substituting Eq. [7] and $C_k = 0.5e_0$ into Eq. [6], k_h can be represented as:

$$[8] \quad k_h = k_{h0} \cdot OCR^{\frac{2C_s}{e_0}}$$

4.2 Obtaining k_{h0}

For the tests with the same value of σ'_{vmax} , $(\Delta u/\sigma'_{v0})_{OC}$ is normalized using $(\Delta u/\sigma'_{v0})_{NC}$. Then, the test results are plotted in accordance with the $OCR - (\Delta u/\sigma'_{v0})_{OC}/(\Delta u/\sigma'_{v0})_{NC}$ relationships, as shown in Fig. 7. It can be seen that $(\Delta u/\sigma'_{v0})_{OC}/(\Delta u/\sigma'_{v0})_{NC}$ increased with the OCR in a non-linear form. By observing the shape of the curves, it is proposed that:

$$[9] \quad \left(\frac{\Delta u}{\sigma'_{v0}}\right)_{OC} \bigg/ \left(\frac{\Delta u}{\sigma'_{v0}}\right)_{NC} = OCR^\alpha$$

where α is a model parameter. From the results of the laboratory tests, for the Ariake clay, $\alpha = 0.52$; Mixed soil 1, $\alpha = 0.56$; for Mixed soil 2, $\alpha = 0.60$; for Mixed soil 3, $\alpha = 0.66$; and for Mixed soil 4, $\alpha = 0.75$ (Fig. 7).

Kim *et al.* (2007) conducted a series of large scale (diameter of the model 1.2 m) laboratory model test with OCR of 1 to 20. The soil tested was a mixture of pure kaolinite and a sand with a mixing ratio of 1:1. Their results in term of OCR versus $(\Delta u/\sigma'_{v0})_{OC}/(\Delta u/\sigma'_{v0})_{NC}$ ratio relationship is shown in Fig. 8. The estimated relationship using $\alpha = 0.52$ is included into the figure also. Although there are some discrepancies for OCR = 10 and 20, generally it can be said that there is a fair agreement between the measured and estimated values. Therefore, Eq. (9) is not only applicable to the test results of this study but also the test results in the literature.

As indicated by Eq. [9], under a given σ'_{vmax} condition, for OCR = 1.0, $(\sigma'_{v0})_{NC} = \sigma'_{vmax}$, and for OCR > 1.0, $(\sigma'_{v0})_{OC} = \sigma'_{vmax}/OCR$. Thus, if $\alpha = 1.0$, $(\Delta u)_{OC} = (\Delta u)_{NC}$. For $\alpha < 1.0$, $(\Delta u)_{OC} < (\Delta u)_{NC}$. Therefore, the more clay content as soil has, the smaller the value of α , and the smaller the $(\Delta u)_{OC}/(\Delta u)_{NC}$ ratio.

Then, the calculated value of $(\Delta u/\sigma'_{v0})_{NC}$ by Eq. (9) is used to calculate K_D in Eqs. [4] or [5] and designated the results as $(K_D)_{NC}$. Therefore, the value of k_{h0} corresponding to σ'_{vmax} can be calculated as:

$$[10] \quad k_{h0} = \frac{(K_D)_{NC} U a \gamma_w}{2 \sigma'_{v0} \cdot OCR}$$

by substituting Eq. [10] into Eq. [8], while the value of k_h (OCR > 1.0) corresponding to σ'_{v0} can be calculated as:

$$[11] \quad k_h = \frac{(K_D)_{NC} U a \gamma_w}{2 \sigma'_{v0} \cdot OCR^{\left(1 - \frac{2C_s}{e_0}\right)}}$$

4.3 Comparison with measured data

Applying the modified method (Eq. [11]) to the laboratory test results, the estimated values of k_h are compared with the measured value of k_v in Fig. 5. It can be seen that the modified method yielded much better results than those resulting from the method of Chai *et al.* (2011). The k_h/k_{h0} (k_{h0} corresponds to σ'_{vmax} , and k_h corresponds to σ'_{v0}) ratios estimated via the modified method for the Ariake clay and the Mixed soil 2 are shown in Fig. 9. The corresponding laboratory measured k_v/k_{v0} ratios are also included for comparison. It can be seen that both the measured and estimated ratios increased with the OCR and they are comparable.

For the soils tested, the k_h/k_v ratios were investigated via laboratory oedometer tests. From the results of CRS test for the reconstituted Ariake clay samples with both vertical and horizontal radial drainage under a maximum consolidation pressure of 100 kPa, an average value of $k_h/k_v \approx 1.27$ was obtained (Table 3), which is smaller than that reported by Chai *et al.* (2012b), who considered a consolidation pressure of up to more than 1000 kPa. The anisotropic behavior of remolded clay is induced by consolidation stress under the one-dimensional condition. The higher the consolidation stress, the larger the k_h/k_v ratio. For the mixed soils, an attempt to measure both k_h and k_v via CRS tests was also made. However, the horizontal drainage CRS tests for the mixed soils were not successful. This occurred because in the horizontal drainage CRS test, which involves a porous drainage tube inserted into the center of a specimen (Chai *et al.* 2012b), the friction between the mixed soil (50% or more than 50% sand) and the drainage tube was significant. Then, MSL consolidation tests were conducted using specimens of reconstituted

Mixed soil 2 cut both in the vertical and horizontal directions. For example, under a consolidation pressure of 100 kPa, $k_h/k_v = 1.02$ ($k_h = 2.09 \times 10^{-9}$ m/s and $k_v = 2.05 \times 10^{-9}$ m/s) was obtained. Chai *et al.* (2014) also reported that for a sand/clay mixture with sand content of more than 50%, the stress-induced anisotropy of permeability can be ignored. Therefore, for mixed soils with a sand content equal or more than 50%, the stress-induced permeability anisotropy is small and can be ignored.

From Fig. 5, it can be seen that the estimated values of k_h are generally larger than $1.5k_v$, i.e., the estimated values of k_h are larger than the values resulting from the laboratory oedometer test, i.e. the laboratory model tests did not provide a validation of the proposed method in term of absolute value of k_h . The possible reasons are as follows.

- (1) The original method was developed assuming that during the cone penetration process there is a ‘dynamic steady’ pore water flow around the cone. Then equating the total radial flow rate to a portion of the volume penetration rate of the cone, and a link between $\Delta u/\sigma'_{v0}$ ratio and permeability (k_h) in the horizontal direction has been established. While the quantitative equation was established empirically based on the field data of $\Delta u/\sigma'_{v0}$ and the laboratory measured corresponding values of k_v . Therefore, basically Eq. (5) for clayey soils is an empirical one.
- (2) For an empirical equation, strictly speaking, it is only valid under the conditions the equation has been developed. There are certain differences between the field uCPT and laboratory model uCPT as discussed in Section 3.3, which can affect the value of $\Delta u/\sigma'_{v0}$. Watabe et al. (2002) reported some K_0 consolidated undrained triaxial compression test results of Osaka clay. The values of $\Delta u_T/\sigma'_{v0}$ (Δu_T is the change of excess pore pressure and σ'_{v0} is the initial vertical effective stress in the soil sample) of recompressed samples (using undisturbed soil

sample and reconsolidated under the in situ effective stresses) was obviously higher than that of the reconstituted samples. Therefore, there is a possibility that the laboratory model uCPT test with reconstituted clayey soils might result in relative lower values of $\Delta u/\sigma'_{v0}$, and therefore, higher values of k_h , which is qualitatively agreed with the results in Fig. 5.

Therefore, if modifying the original equation to fit the measured k_h (or k_v) values of the model ground, it may not be applicable to the field condition. While, considering that the OCR simulated in the laboratory model tests is induced by consolidation pressures, it is suggested that the method proposed should not be applied to highly structured overconsolidated deposits, in which part of the yield vertical effective stress may be due to the effect of cementation, aging and possible chemical reactions.

4.4 Methods for determining the parameters

To use the modified method in the field to estimate the value of k_h corresponding to σ'_{vmax} , two additional parameters need to be determined, i.e., OCR and α .

(1) OCR

There are several methods in the literature for estimating the value of the OCR based on the uCPT results (Mayne and Kemper 1988; Mayne 1991; Ladd and DeGroot 2003). Chanmee *et al.* (2017) investigated the applicability of existing methods for estimating the OCR based on the uCPT results. The results show that the methods proposed by Chanmee *et al.* (2017) and Mayne and Kemper (1988) perform better and the method proposed by Mayne and Kemper (1988) is simpler. Therefore, it is suggested that if a locally validated method is available, it should be used. If not, the method proposed by Mayne and Kemper (1988) is recommended:

$$[12] \quad OCR = k \cdot Q_t$$

where k is an empirical parameter. If there is no locally validated value, $k = 0.3$ is suggested for clayey soils.

(2) α

The value of α is related to soil types. If the clay content of a deposit is known from the local geotechnical data base, it is suggested to use clay content to determine the value of α using Figs. 4 and 7. If determining the permeability solely using piezocone penetration test results, it is suggested to use the $B_q - Q_t$ chart (Fig. 10) of Robertson (1990) to classify soil types using the results of uCPT. According to the results of this study, only α values of 0.55, 0.65 and 0.75 for soil types 3, 4, and 5, respectively, are recommended.

If the value of k_h corresponds to σ'_{v0} is required, two more parameters are needed: swelling index, C_s , and initial void ratio, e_0 . If the measured values are not available, it is suggested to estimate them using a local geotechnical database.

4.5 Procedure for using the modified method

The steps for evaluating the value of k_h based on the results of uCPT are as follows:

- (1) Calculate B_q and Q_t based on the results of uCPT.
- (2) Calculate the OCR using Eq. [12]. If OCR = 1.0, go to step (5).
- (3) Use the $B_q - Q_t$ chart to classify the soil and determine the value of α (Fig. 10).
- (4) Calculate $(\Delta u/\sigma'_{v0})_{NC}$ based on the measured value of $(\Delta u/\sigma'_{v0})_{OC} = B_q/Q_t$ and the value of OCR using Eq. [9].
- (5) Calculate $(K_D)_{NC}$ using Eq. [4] or [5].

- (6) Calculate the value of k_{h0} corresponding to σ'_{vmax} using Eq. [10]. If the value of k_h corresponding to σ'_{v0} is not specifically required, stop here. Otherwise, continue to steps (7) and (8).
- (7) Estimate the values of e_0 and C_s based on the test results or on the local geotechnical database.
- (8) Calculate k_h corresponding to σ'_{v0} using Eq. [11].

5. Application to field cases

The modified method was applied to two field cases in Japan (at the Saga and Yokohama sites) and one case in Shanghai, China, to demonstrate its usefulness.

5.1 Case 1: Saga, Japan

In the Saga plain along the coast of the Ariake Sea (a semi-enclosed inland bay), Kyushu, Japan, clayey soils, called Ariake clay, with a thickness between 10 and 30 m, are deposited. For a highway construction in the Saga plain, several boreholes, together with uCPT adjacent to the boreholes, were created. The results of laboratory oedometer tests using undisturbed soil samples and the results of the corresponding uCPT at one of the sites are analyzed here. The results of uCPT are shown in Fig. 11 (ASCRDO 2008). The piezocone used at this site had a diameter of 35.7 mm (cross-sectional area of 1000 mm²), an apex angle of 60°, and a filter element for pore water pressure measurement at the shoulder of the cone (standard type). The rate of penetration adopted was 20 mm/s. The soil profile, groundwater level, laboratory-measured values of permeability (k_v) and values of the OCR based on the results of the oedometer tests are shown in Fig. 12.

The values of k_h estimated based on the piezocone sounding results using both the method of Chai *et al.* (2011) and the modified methods are compared with the measured values of k_v in Fig. 13. For the modified method, the values of e_0 , C_s and α adopted were 2.5, 0.08 (ASCRDO 2008) and 0.55, respectively.

Generally, the results of the modified method were closer to the $k_h/k_v = 1.5$ line. However, it is noted that there were three points at which the values of k_h were more than 10 times the corresponding value of k_v . These three points were located in a sandy clay layer. The value of k_h based on uCPT may include the effect of thin sand layers or sand seams in the deposit, but the laboratory oedometer test might fail to reflect the effect because a relatively uniform clay soil sample is usually selected for laboratory oedometer tests.

5.2 Case 2: Yokohama, Japan

The Japanese Geotechnical Society (JGS) organized an activity involving the simultaneous conducting of uCPT by several companies at a site in Yokohama, Japan, in 2007 (Suemasa *et al.* 2009). The data used in this study are from one of the organizations (organization C). From the ground surface, the soil deposit consists of a 4 m-thick sandy clay, underlain by a fine sand layer to a depth of approximately 10 m, which is then followed by a sandy clay layer that is approximately 10 m thick. The piezocone used at this site had the same specifications as the one used at the Saga site. The results of uCPT are shown in Fig. 14 (JGS 2009). The soil profile, groundwater level, and laboratory-measured values of OCR and k_v are shown in Fig. 15.

After applying both the method of Chai *et al.* (2011) and the modified method to the case, the results were compared, as shown in Fig. 16. In the modified method, the values of e_0 , C_s and α adopted were 1.55, 0.057 (JGS 2009), and 0.55, respectively. As shown, the results of the

modified method were closer to the $k_h/k_v = 1.0$ line. For the Yokohama site, the k_h/k_v ratio is not known. Additionally, it can be seen that there were two points at which the values of k_h were more than 100 times the corresponding value of k_v . Again, these two points were located in a sandy clay and fine sand layer. It is considered that the interpreted k_h value may reflect the effect of the thin sandy layers, but the laboratory oedometer tests failed to do so.

5.3 Case 3: Shanghai, China

Shanghai is located on the alluvial plain of the Yangtze River Delta, China. A case of uCPT, together with values of k_v obtained via oedometer tests, in Shanghai was reported by Xu *et al.* (2009) and Shen *et al.* (2015). At the site, from the ground surface, the soil deposit consists of a 5 m-thick gray mucky clay, underlain by a gray mucky silty clay layer to a depth of approximately 15 m, which is then followed by a gray silty clay layer that is approximately 15 m thick. The piezocone used at this site had a cross-sectional area of 1500 mm², an apex angle of 60°, and a filter element for pore water pressure measurement located at the shoulder of the cone (2 mm behind the conical tip). The rate of penetration adopted was 20 mm/s, and the results of uCPT are shown in Fig. 17 (Shen *et al.* 2010). The soil profile, groundwater level, and laboratory-measured values of k_v are shown in Fig. 18.

For the Shanghai site, the OCR values were not reported. The OCR values were estimated based on the results of uCPT using Eq. [12] with $k = 0.3$. The range of the estimated OCR values for this site was approximately 1.0 to 1.9, which agrees with the range of 1.0 to 2.0 reported by Li *et al.* (2012) and Ye and Ye (2016) for the clayey deposit in Shanghai, China. After applying both the method of Chai *et al.* (2011) and the modified method to the case, the results of the modified method were closer to the $k_h/k_v = 1.0$ line (for the Shanghai site, the k_h/k_v ratio is not

known), as shown in Fig. 19. The values of e_0 and C_s adopted, according to Xu *et al.* (2009), were 1.2 and 0.035 ($C_s \approx 0.1C_c$), respectively, and the value of α adopted was 0.55.

6. Conclusions

Most existing methods for estimating the field permeability in the horizontal direction (k_h) based on the results of piezocone sounding (uCPT) relate k_h to the $\Delta u/\sigma'_{v0}$ ratio (Δu is the measured excess pore pressure at the shoulder of the cone, and σ'_{v0} is the initial vertical effective stress at the depth considered) but do not consider the effect of the overconsolidation ratio (OCR) on $\Delta u/\sigma'_{v0}$. Therefore, they are not applicable to an overconsolidated deposit. Based on the results of the laboratory model tests of piezocone penetration (uCPT) conducted in this study, a modified method that considers the effect of the OCR was proposed.

When estimating the value of k_h corresponding to the yield vertical effective stress using the modified method, two parameters are required: the OCR and a model parameter, α . If the value of k_h corresponding to σ'_{v0} is required, two more parameters, i.e., the swelling index (C_s) and the initial void ratio (e_0), are needed. The methods for estimating these parameters were also established. The α value is related to the soil type. In case only the results from uCPT are available, it is recommended to classify the soil type using Robertson's (1990) method. Based on the limited test results of this study, it was suggested that $\alpha = 0.55$, 0.65 and 0.75 for soil types 3, 4, and 5, respectively.

The modified method was applied to three field cases reported in the literature. The measured values of permeability in the vertical direction (k_v) obtained via the oedometer test were available at all the sites. By comparing the values of k_h estimated based on the results of uCPT with the

measured values of k_v , it was shown that the modified method results in much better estimation of the field values of k_h .

7. Acknowledgments

Mr. A. Saito, technician at the Graduate School of Science and Engineering, Saga University, Japan, helped us conduct the laboratory model tests.

References

- Agung, P.M.A. 2011. Estimation of permeability and coefficient of consolidation from piezocone test results (Unpublished Doctoral dissertation). Saga University, Saga, Japan.
- ASCRDO. 2008. Site investigation reports, along the route of the Saga section, around Ariake sea Highway, Ariake Sea Coastal road Development Office; Saga, Japan (in Japanese).
- Burns, S.E., and Mayne, P.W. 1998. Monotonic and dilatory pore pressure decay during piezocone tests in clay, *Canadian Geotechnical Journal*, **35**(6), 1063-1073.
- Campanella, R.G., and Robertson, P.K. 1988. Current status of the piezocone test, In: Ruiter J, editor, *penetration testing*, Balkema, Rotterdam, 93-116.
- Chai, J.-C., Agung, P.M.A., Hino, T., Igaya, Y., and Cater, J.P. 2011. Estimating hydraulic conductivity from piezocone soundings, *Géotechnique*, **61**(8), 699-708.
- Chai, J.-C., Sheng, D.C., Carter, J.P., and Zhu, H.H. 2012a. Coefficient of consolidation from non-standard piezocone dissipation curves, *Computers and Geotechnics*, **41**, 13-22.
- Chai, J.-C., Jia, R., and Hino, T. 2012b. Anisotropic consolidation behavior of Ariake clay from three different CRS tests, *Geotechnical Testing Journal*, ASTM, **35**(6), 1-9.
- Chai, J.-C., Hossain, Md. Julfikar, Carter, J., and Shen, S.L. 2014. Cone penetration-induced pore pressure distribution and dissipation. *Computers and Geotechnics*, **57**, 105-113.

- Chai, J.-C., Hossain, Md. J., Yuan, D.J., Shen, S.L., and Carter, J. 2015. Pore pressures induced by piezocone penetration, *Canadian Geotechnical Journal*, **53**(3), 540-550.
- Chanmee, N., Chai, J.-C., Hino, T., and Wang, J. 2017. Methods for evaluating overconsolidation ratio from piezocone sounding results. *Underground Space*, **2**(2), doi: <http://dx.doi.org/10.1016/j.undsp.2017.05.004>.
- Elsworth, D., and Lee, D.S. 2005. Permeability determination from on-the-fly piezocone sounding, *Journal of Geotechnical Geoenvironmental Engineering*, **131**(5), 643-653.
- Elsworth, D., and Lee, D.S. 2007. Limits in determining permeability from on-the-fly uCPT sounding, *Géotechnique*, **57**(8), 769-685.
- Hossain, Md. J. 2014. Coefficient of Consolidation of Clayey Deposits from Piezocone Test Results (Doctoral dissertation, Saga University, Saga, Japan). Retrieved from http://portal.dl.saga-u.ac.jp/bitstream/123456789/121939/4/zenbun_fulltext_hossain.pdf
- JGS. 2009. Report of simultaneous field uCPT tests. Research Committee of Advanced CPT Technology and its Applications to Design, Environment Assessment and Disaster Prevention. Tokyo: Japanese Geotechnical Society.
- Japanese Industrial Standards (JIS) 1995. JIS A 1205, Test method for liquid limit and plastic limit of soils. Tokyo, Japan.
- Kim, K., Prezzi, M., Salgado, R., and Lee, W. 2008, Effect of penetration rate on cone penetration resistance in saturated clayey soils, *Journal of Geotechnical and Geoenvironmental Engineering*, **134**(8), 1142–1153.
- Kim, T, Kim, N, Tumay, M.T., and Lee, W. 2007, Spatial distribution of excess pore-water pressure due to piezocone penetration in overconsolidated clay. *Journal of Geotechnical and Geoenvironmental Engineering*, **133**(6): p. 674–68.

- Ladd, C.C., and DeGroot, D.J. 2003. Recommended practice for soft ground site characterization: Arthur Casagrande lecture, *In Proceedings of the 12th Panamerican Conference on Soil Mechanics and Geotechnical Engineering*, Cambridge, MA.
- Li, Q., Ng, C.W.W., and Liu, G. 2012. Low secondary compressibility and shear strength of Shanghai Clay, *Journal of Central South University*, **19**(8), 2323-2332.
- Lunne T., Lacasse, S., and Rad, N.S. 1989. SPT, CPT, pressuremeter testing and recent developments on in situ testing of soils. General Report from the 12th International Conference on Soil Mechanics and Foundation Engineering, Rio de Janeiro, 4: 2339-2303, Balkema Pub., Rotterdam.
- Lunne, T., Robertson, P.K., and Powell, J.J.M. 1997. Cone penetration testing in geotechnical practice, London: E & FN Spon.
- Lu, Q., Randolph, M.F., Hu, Y., and Bugarski, C.A. 2004. Numerical study of cone penetration in clay, *Géotechnique*, **54**(4), 257-267.
- Mayne, P.W. 1991. Determination of OCR in clays by piezocone tests using cavity expansion and critical state concepts, *Soil and foundations*, **31**(2), 65-76.
- Mayne, P.W., and Kemper, J.B. 1988. Profiling OCR in stiff clays by CPT and SPT, *Geotechnical Testing Journal*, **11**(2), 139-147.
- Robertson, P.K. 1990. Soil Classification Using the Cone Penetration Test, *Canadian Geotechnical Journal*, **27**(1), 151-158.
- Robertson, P.K. 2010. Estimating in-situ soil permeability from CPT and CPTu, 2nd International Symposium on Cone Penetration Testing, Huntington Beach, CA, USA, 2-43.

- Robertson, P.K., Sully, J.P., Woeller, D.J., Lunne, T., Powell, J.J.M., and Gillespie, D.G. 1992. Estimating coefficient of consolidation from piezocone test, *Canadian Geotechnique Journal*, **29**(4), 539-550.
- Shen, S.L., Wang, J.P., and Ma, L. 2010. Identification of soil stratigraphy of soft deposit in Shanghai from CPTU test, *In Proceedings of Sessions of Geoshanghai 2010*. ASCE, 384-391.
- Shen, S.L., Wang, J.P., Wu, H.N., Xu, Y.S., Ye, G.L., and Yin, Z.Y. 2015. Evaluation of hydraulic conductivity for both marine and deltaic deposits based on piezocone testing, *Ocean Engineering*, 110, 174-182.
- Suemasa, N., Katagiri, M., and Iizawa, M. 2009. Comparative study on the performance of various electric cone penetrations in 2007, *Japan Geotechnical Engineering Journal*, **57**(8), 16-19.
- Sully, J.P., Robertson, P.K., Campanella, R.G., and Woeller, D.J. 1999. An approach to evaluation of field CPTU dissipation data in overconsolidated fine-grained soils, *Canadian Geotechnique Journal*, **36**(2), 369-381.
- Taylor, D.W. 1948. *Fundamentals of Soil Mechanics*, John Wiley and Sons, Inc. New York.
- Tavenas, F., Jean, P., Leblond, P., and Leroueil, S. 1983. The permeability of natural soft clays: part II, permeability characteristics. *Canadian Geotechnique Journal*, **20**(4), 645-660.
- Teh, C.I., and Houlsby, G.T. 1991. An analytical study of the cone penetration test in clay, *Géotechnique*, **41**(1), 17-34.
- Watabe, Y., Tsuchida, T., and Adachi K. 2002. Undrained Shear Strength of Pleistocene Clay in Osaka Bay. *J. Geotech. Geoenviron. Eng.*, 128(3), 216-226.
- Wroth, C.P. 1984. The interpretation of in situ soil tests, 24th Rankine Lecture, *Géotechnique*, **34**(4), 449-489.

- Xu, Y.S., Shen, S.L., and Du, Y.J. 2009. Geological and hydrogeological environment in Shanghai with geohazards to construction and maintenance of infrastructures, *Engineering Geology*, **109** (3-4), 241-254.
- Ye, G., and Ye, B. 2016. Investigation of the overconsolidation and structural behavior of Shanghai clays by element testing and constitutive modeling, *Underground space*, **1**(1), 62-77.

Draft

List of tables

Table 1. Physical properties of soils tested.

Table 2. Conditions and results of model tests conducted.

Table 3. Comparison of k_h and k_v values from CRS test on Ariake clay samples

Draft

List of figures

Fig. 1. Schematic diagram of laboratory model test.

Fig. 2. Typical settlement curve of preconsolidation of Mixed soil 2 with $\text{OCR} = 8.0$.

Fig. 3. Typical pore water pressure curves at P_4 , P_5 and P_6 during preconsolidation of the model ground for Mixed soil 2 with $\text{OCR} = 8.0$.

Fig. 4. Grain size distributions of soils.

Fig. 5. Comparison of measured k_v with estimated k_h using both method of Chai *et al.* (2011) and the modified method: (a) Ariake clay; (b) Mixed soil 1. (c) Mixed soil 2; (d) Mixed soil 3; and (e) Mixed soil 4.

Fig. 6. Relationships between $\Delta u_1/\sigma'_{v0}$ and OCR (data from Tunne et al. 1989)

Fig. 7. Values of model parameter (α) for different soils: (a) Ariake clay; (b) Mixed soil 1. (c) Mixed soil 2; (d) Mixed soil 3; and (e) Mixed soil 4.

Fig. 8 Normalized $\Delta u/\sigma'_{v0}$ ratio versus OCR relationship of Kim et al.'s (2007) test data

Fig. 9. Example of variation of measured k_v/k_{v0} and estimated k_h/k_{h0} ratios with OCR: (a) Ariake clay; (b) Mixed soil 2.

Fig. 10. CPT soil behavior type chart (modified version of that from Robertson 1990).

Fig. 11. Piezocone penetration test results for Saga site.

Fig. 12. Soil strata, measured values of OCR and permeability for Saga site.

Fig. 13. Comparison of measured k_v with estimated k_h using both method of Chai *et al.* (2011) and modified method for Saga site.

Fig. 14. Piezocone penetration test results for Yokohama site.

Fig. 15. Soil strata, measured values of OCR and permeability for Yokohama site.

Fig. 16. Comparison of measured k_v with estimated k_h using both method of Chai *et al.* (2011) and modified method for Yokohama site.

Fig. 17. Piezocone penetration test results for Shanghai site.

Fig. 18. Soil strata and permeability for Shanghai site.

Fig. 19. Comparison of measured k_v with estimated k_h using both method of Chai *et al.* (2011) and modified method for Shanghai site.

Table 1. Physical properties of soils tested.

Soil properties	Liquid limit	Plastic limit	Void ratio	Compression	Swelling
	(LL)	(PL)	(e_0)	index	index
	(%)	(%)		(C_c)	(C_s)
Ariake clay	114.00	60.60	2.36	0.80	0.16
Mixed soil 1 (20:80)*	70.28	30.46	1.48	0.50	0.10
Mixed soil 2 (50:50)*	56.71	24.13	1.02	0.35	0.07
Mixed soil 3 (60:40)*	44.24	21.28	0.90	0.28	0.06
Mixed soil 4 (70:30)*	39.85	21.29	0.81	0.23	0.05

* Ariake clay mixed with sand with the corresponding sand/clay ratio (by dry weight)

Table 2. Conditions and results of model tests conducted.

Soil properties	Case	σ'_{vmax} (kPa)	σ'_{v0} (kPa)	OCR	Piezocone diameter (mm)	Measured permeability k_v (m/s)	Δu (kPa)	Estimated permeability k_h (m/s) (Eq. [3])	Remark
Ariake clay	1	96	96	1	30	8.00×10^{-10}	123.16	3.18×10^{-9}	Hossain (2014)
	2	96	48	2	30	9.32×10^{-10}	89.24	1.34×10^{-9}	
	3	96	24	4	30	1.08×10^{-9}	69.13	3.17×10^{-10}	
	4	96	12	8	30	1.30×10^{-9}	47.39	1.32×10^{-10}	
Mixed soil 1	1	96	96	1	20	9.10×10^{-10}	101.50	5.48×10^{-9}	This study
	2	96	48	2	20	9.86×10^{-10}	72.084	2.54×10^{-9}	
	3	96	12	8	20	1.50×10^{-9}	42.917	1.43×10^{-10}	
Mixed soil 2	1	96	96	1	20	2.01×10^{-9}	97.33	6.73×10^{-9}	This study
	2	96	48	2	20	2.45×10^{-9}	81.08	1.43×10^{-9}	
	3	96	24	4	20	2.87×10^{-9}	63.83	3.07×10^{-10}	
	4	96	12	8	20	3.27×10^{-9}	45.17	1.12×10^{-10}	
Mixed soil 3	1	96	96	1	20	2.05×10^{-9}	99.55	6.03×10^{-9}	Hossain (2014)
	2	96	48	2	20	2.33×10^{-9}	85.29	1.11×10^{-9}	
	3	96	24	4	20	2.94×10^{-9}	60.82	3.89×10^{-10}	
	4	96	12	8	20	3.10×10^{-9}	54.89	4.29×10^{-11}	
Mixed soil 4	1	96	96	1	20	3.11×10^{-9}	85.25	1.29×10^{-8}	This study
	2	96	48	2	20	3.39×10^{-9}	68.58	3.25×10^{-9}	
	3	96	24	4	20	3.89×10^{-9}	59.33	4.40×10^{-10}	
	4	96	12	8	20	4.47×10^{-9}	53.83	4.72×10^{-11}	

* σ'_{vmax} is the maximum vertical consolidation stress, and σ'_{v0} is the initial vertical effective stress in the model ground before piezocone penetration.

Table 3. Comparison of k_h and k_v values from CRS test on Ariake clay samples

Test cases	Type of soil	σ'_{v0} (kPa)	OCR	k_v (m/s)	k_h (m/s)	k_h/k_v
1	Ariake clay	96	1	5.26×10^{-10}	5.89×10^{-10}	1.12
2		48	2	5.34×10^{-10}	6.71×10^{-10}	1.26
3		24	4	5.72×10^{-10}	8.93×10^{-10}	1.56
4		12	8	1.58×10^{-9}	1.77×10^{-9}	1.12
					Average	1.27

Draft

Fig. 2

Fig. 3

Fig. 4

Fig. 5

Fig. 6

Fig. 7

]

Fig. 8

Fig. 9

Fig. 10

Zone of soil behavior type

1. Sensitive, fine grained soils	5. Sand mixtures: silty sand to sand silty
2. Organic soils-peats	6. Sands: clean sands to silty sands
3. Clays: clay to silty clay	7. Gravelly sand to sand
4. Silt mixtures: clayey silt to silty clay	

Fig. 11

Fig. 12

Fig. 13

Fig. 14

Fig. 15

Fig. 16

Fig. 17

Fig. 18

Fig. 19