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Estimating coefficient of consolidation and hydraulic conductivity from piezocone test results - Case studies

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Abstract. The methods for estimating in-situ hydraulic conductivity (k_{hp}) and coefficient of consolidation (c_{hp}) in the horizontal direction from piezocone penetration and dissipation test results have been investigated using test results at two sites in Saga, Japan. At the two sites the laboratory values of hydraulic conductivity (k_v) and coefficient of consolidation (c_v) in the vertical direction are also available. Comparing k_{hp} with k_v and c_{hp} with c_v values, suitable methods for estimating k_{hp} and c_{hp} values are recommended. For the two sites, where $k_{hp} \approx k_v$ and $c_{hp} \approx 2c_v$. It is suggested that the estimated values of k_{hp} and c_{hp} can be used in engineering design.

Keywords: cone penetration test; coefficient of consolidation; hydraulic conductivity; field dissipation test; case study

1. Introduction

Hydraulic conductivity and coefficient of consolidation are important engineering parameters for clayey deposit. These parameters can be evaluated by laboratory odometer test and/or constant rate of strain (CRS) consolidation test (i.e., Chai *et al*. 2012b, Jia *et al*. 2013). However to obtain undisturbed soil sample is cost and the test results may not represent the field value because of small size of sample used (e.g., Chai and Miura 1999). To obtain more reliable field values in an economic way, some in-situ test methods have been developed. Direct test methods include the self-boring permeameter test (SBPT) (e.g., Chandler *et al*. 1990, Arulrajah *et al*. 2005), the BAT permeameter test (BAT) and the flat dilatometer test (DMT) (e.g., Arulrajah *et al*. 2005). Indirectly, hydraulic conductivity and coefficient of consolidation can be estimated from piezocone tests (uCPT) (e.g., Baligh and Levadoux 1980, Teh and Houlsby 1991). The piezocone test (uCPT) is widely used as an economic and efficient site investigation technique (e.g., Campanella and Robertson 1988, Lunne *et al*. 1997, Liu, *et al*. 2008). uCPT provides near continuous measurements of tip resistance (q_t), sleeve friction (f_s), and pore water pressure (u) at the shoulder (standard) of the cone. Furthermore, the cone can be halted at pre-determined locations, and the dissipation process of the u value can be observed. From the results of the piezocone penetration and dissipation tests, the soil profile and other engineering properties of the sub-soil, such as undrained shear strength (s_u) of clayey deposits (e.g., Campanela and Robertson 1988, Arulrajah *et*

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al. 2005), in situ hydraulic conductivity (k_{hp}) (e.g., Baligh and Levadoux 1980, Song and Pulijala 2010, Robertson 2010, Chai *et al*. 2011, Wang *et al*. 2013) and coefficient of consolidation (c_{hp}) of the sub-soils in the horizontal direction (e.g., Teh and Houlsby 1991, Arulrajah *et al*. 2007, Chai *et al*. 2012a) can be estimated. Subscript "p" means the value from piezocone test results. Further cone penetration tests have been used to evaluate liquefaction of sandy and silty soils (Samui and Sitharam 2010). However, the methods for estimating c_{hp} and k_{hp} values are under development, and for many cases, the estimated values are only orderly correct when compared with the laboratory measured values (e.g., Chai *et al*. 2011).

There are two types of dissipation curve observed from the field piezocone dissipation tests. One type shows monotonic decreasing of measured pore water pressure with elapsed time, and this is designated a "standard" curve (Baligh and Levadoux 1986, Teh and Houlsby 1991). Most dissipation tests conducted in normally or lightly over consolidated clayey deposits using a cone with a filter element at the shoulder, exhibit the standard dissipation curve. Another type of curve occurs when the dissipation test starts, the measured pore water pressure is first increasing from an initial value to a maximum, and then decreasing to a hydrostatic value (Burns and Mayne 1998, Sully *et al*. 1999), as illustrated in Fig. 1. This kind of dissipation curve has been referred to as "non-standard", and it often occurs in heavily over-consolidated clay deposits or dense sand deposits. Several methods have been proposed to estimate c_{hp} values from the standard dissipation curve, and perhaps Teh and Houlsby's (1991) method is the widely used one. As for the non-standard curve, only a few methods are available, such as Sully *et al*.'s (1999) shifting time origin and extrapolation root-time verses pore water pressure curve methods, and Chai *et al*.'s (2012a) method which corrects the time corresponding to 50% dissipation of the measured maximum u value.

There are efforts to estimate k_{hp} values from piezocone sounding records. Most of the proposed methods (Robertson *et al*. 1992, Lunne *et al*. 1997, Robertson 2010) are empirical, and some of them only provide a likely range of k_{hp} value (Robertson *et al*. 1992). Baligh and Levadoux (1980) proposed a method to evaluate k_{hp} values from c_{hp} values. Elsworth and Lee (2007) proposed a semi-theoretical equation for estimating k_{hp} value, but the method is only applicable to sandy soils. Chai *et al*. (2011) modified Elsworth and Lee's (2007) method and the proposed equations are applicable to most soil types, from fine sand to soft clay deposits. It is desirable to check the applicability and accuracy of the existing methods for estimating k_{hp} and c_{hp} values, by comparing the estimated values with corresponding directly measured data.

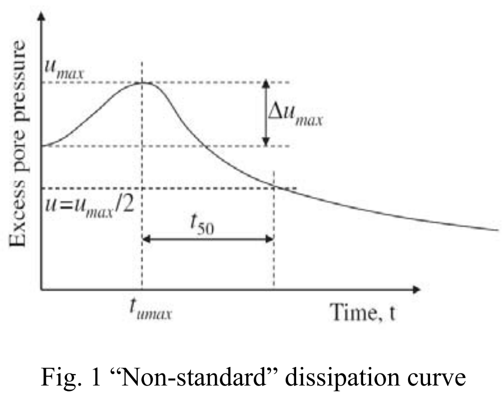


Fig. 1 "Non-standard" dissipation curve

Field uCPTs and dissipation tests were conducted at two sites in Saga, Japan. At both the sites, laboratory vlues of the hydraulic conductivity (k_v) and the coefficient of consolidation (c_v) in the vertical direction at certain depths are also available. In thais study using these field and laboratory test results, the effectiveness of currently available methods for estimating k_{hp} and c_{hp} values from piezocone test results has been investigated. The suitable methods are recommended.

2. A brief review of some existing methods

The newest and the most widely used methods of estimating c_{hp} and k_{hp} values from field piezocone test results are briefly described, and they will be used to interpret field test results at two sites in Saga, Japan.

2.1 Methods for estimating c_{hp} value

For a standard dissipation curve, among the methods available for evaluating c_{hp} value, the one proposed by Teh and Houlsby (1991) is probably most widely used, in which

$$c_{hp} = \frac{c_p \cdot r_0^2 \cdot \sqrt{I_r}}{t_{50}} \tag{1}$$

where c_p = a time factor corresponding to 50% degree of consolidation, which is related to the location of the filter element. For a cone with a shoulder filter element, c_p = 0.245, r_0 = the radius of the piezocone, t_{50} = time for 50% of maximum excess pore pressure dissipation, and I_r = rigidity index of sub-soil which is calculated as follows

$$I_r = \frac{G}{s_u} \tag{2}$$

where G = shear modulus and s_u = undrained shear strength of sub-soil.

Sully *et al.* (1999) proposed two methods for estimating c_{hp} value from a “non-standard” dissipation curve. One of the methods involves shifting the origin of time to the point where the measured pore water pressure is a maximum. Another one extrapolates the measured u values versus the root-time curve of the portion after the maximum u value to the time origin, to estimate a “true” maximum u value. These two methods are simple, but there are fundamental shortcomings (Chai *et al.* 2012a). Chai *et al.* (2012a) developed an empirical equation to modify t_{50}, the time period for u to dissipate from the maximum value to 50% of the maximum value. The modified t_{50} is designated as t_{50c}, and then using t_{50c} instead of t_{50} in Eq. (1) to estimate c_{hp} value. The expression for t_{50c} is as follows

$$t_{50c} = \frac{t_{50}}{1 + 18.5\left(\frac{t_{u\max}}{t_{50}}\right)^{0.67}\left(\frac{I_r}{200}\right)^{0.3}} \tag{3}$$

where $t_{u\max}$ = time for measured excess pore pressure to reach the maximum value (see Fig. 1).

2.2 Methods for estimating k_{hp} value

Robertson (2010) proposed an empirical method, which relates hydraulic conductivity in the horizontal direction, k_{hp}, to a parameter called the Soil Behavior Type (SBT) Index, I_c. I_c is a function of uCPT tip resistance (q_t) and sleeve friction (f_s).

$$I_c = \left[(3.47 - \log Q_{tn})^2 + (\log F_r + 1.22)^2\right]^{0.5} \tag{4}$$

$$Q_{tn} = \left[(q_t - \sigma_{vo})/p_a\right]\left(p_a/\sigma'_{vo}\right)^n \tag{5}$$

$$F_r = \left[f_s/(q_t - \sigma_{vo})\right]100\% \tag{6}$$

where q_t = uCPT corrected tip resistance, f_s = sleeve friction, σ_{vo} = initial in-situ total vertical stress, σ'_{vo} = initial in-situ vertical effective stress, and p_a = atmospheric pressure. $n \leq 1.0$, and it is calculated as follows (Robertson 2010)

$$n = 0.381(I_c) + 0.05(\sigma'_{vo} - p_a) - 0.15 \tag{7}$$

Clearly, to calculate I_c values using Eq. (4) certain iterations are needed.

Then the relationships between k_{hp} and I_c are as follows

$$\text{When} \quad 1.0 < I_c \leq 3.27 \quad k_{hp} = 10^{(0.952 - 3.04 I_c)} \quad \text{(m/s)} \tag{8}$$

$$\text{When} \quad 3.27 < I_c \leq 4.0 \quad k_{hp} = 10^{(-4.52 - 1.37 I_c)} \quad \text{(m/s)} \tag{9}$$

Chai *et al*. (2011) modified Elsworth and Lee's (2007) method and proposed semi-theoretical equations for calculating k_{hp} values from uCPT sounding records. Firstly, two dimensionless parameters, B_q and Q_t are defined as follows (Senneset and Janbu 1985)

$$B_q = \frac{u - u_0}{q_t - \sigma_{vo}} \tag{10}$$

$$Q_t = \frac{q_t - \sigma_{vo}}{\sigma'_{vo}} \tag{11}$$

where u = measured pore water pressure and u_0 = hydrostatic pressure.

Then a dimensionless hydraulic conductivity index, K_D, has been defined as a function of $B_q.Q_t$. The bi-linear $K_D - (B_q.Q_t)$ relationship proposed is as follows (Chai *et al*. 2011)

$$K_D = \frac{1}{B_q Q_t} \quad (B_q Q_t \leq 0.45) \tag{12}$$

$$K_D = \frac{0.044}{(B_q Q_t)^{4.91}} \quad (B_q Q_t > 0.45) \tag{13}$$

Finally the relationship between K_D and k_{hp} is as follows (Chai *et al*. 2011)

$$k_{hp} = \frac{K_D . r \gamma_w . V}{2\sigma'_{vo}} \tag{14}$$

where γ_w = the unit weight of water, and V = the rate of piezocone penetration. The method adopted by Wang *et al*. (2013) is basically the same as the method by Chai *et al*. (2011).

Baligh and Levadoux (1980) proposed a method to calculate approximate k_{hp} values from c_{hp} values, in which

$$k_{hp} = \frac{\gamma_w}{2.3\sigma'_{vo}} \cdot RR \cdot c_{hp} \tag{15}$$

where RR represents the value of $C_s/(1+e_0)$ or $C_c/(1+e_0)$ (e_0 is initial void ratio, C_c and C_s are compression and swelling indexes respectively). One of the challenges of using Eq. (15) is how to determine the RR value. Ariake clay deposit is in a slightly overconsolidated state, and for most soft soil layers, e_0 is about 3.0 and C_c is more than 1.0 (Miura et al. 1998). Assuming $C_s = C_c/10 = 0.1$, $C_s/(1+e_0)$ of 2.5×10^{-2} can be estimated, and this value is used for calculating k_{hp} from c_{hp}.

3. Piezocone tests results at two sites in Saga, Japan

In Saga plain, around the Ariake Sea in Japan, there is a clayey soil (Ariake clay) deposit with a thickness of about 10 to 30 m. Piezocone penetration tests as well as dissipation tests at several depths were conducted at two sites (TA and TB) in Saga plain, as shown in Fig. 2. For the both sites laboratory consolidation test results using undisturbed soil samples retrieved from boreholes adjacent to piezocone test locations are also available. At the both sites, the piezocone used had a diameter of 35.7 mm (cross-section of 1000 mm^2), an apex angle of 60°, and the filter element for pore water pressure measurement at the shoulder of the cone. The filter element was boiled in water to remove the air and kept in water during the day prior to the field tests, and assembled underwater at the field just before the start of the test. The rate of penetration was 20 mm/sec.

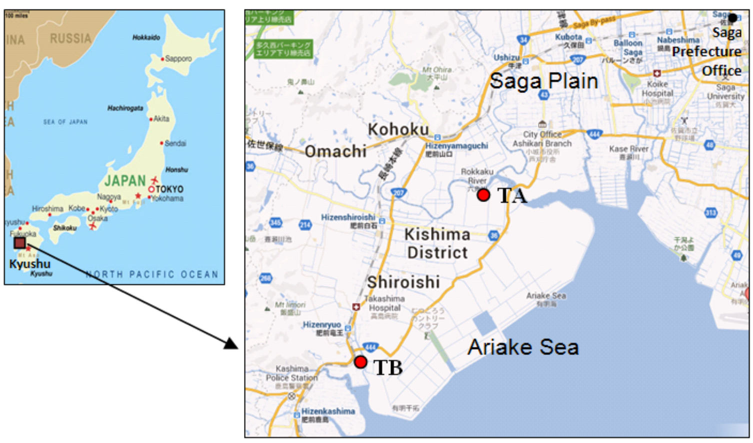


Fig. 2 Locations of the piezocone test site in Saga, Japan

3.1 Test results at TA site

The TA Site is at the toe of a river embankment (Chai *et al*. 2004). At this site, the thickness of soft clay soil is about 12-14 m. The top crust is about 2.0 m thick and is in an apparent overconsolidated state. Below it, the soil is slightly overconsolidated. There is a borehole (BH)

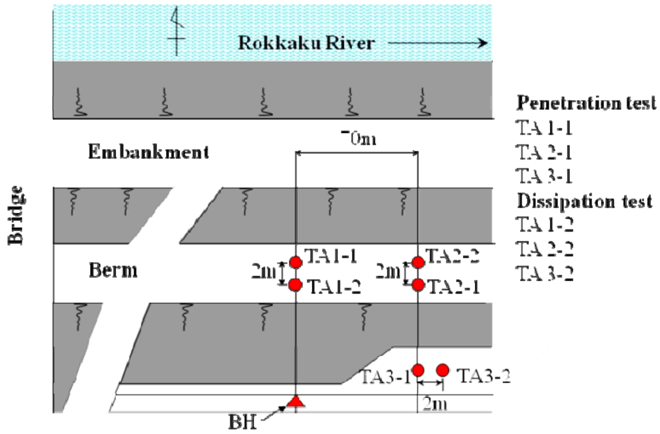
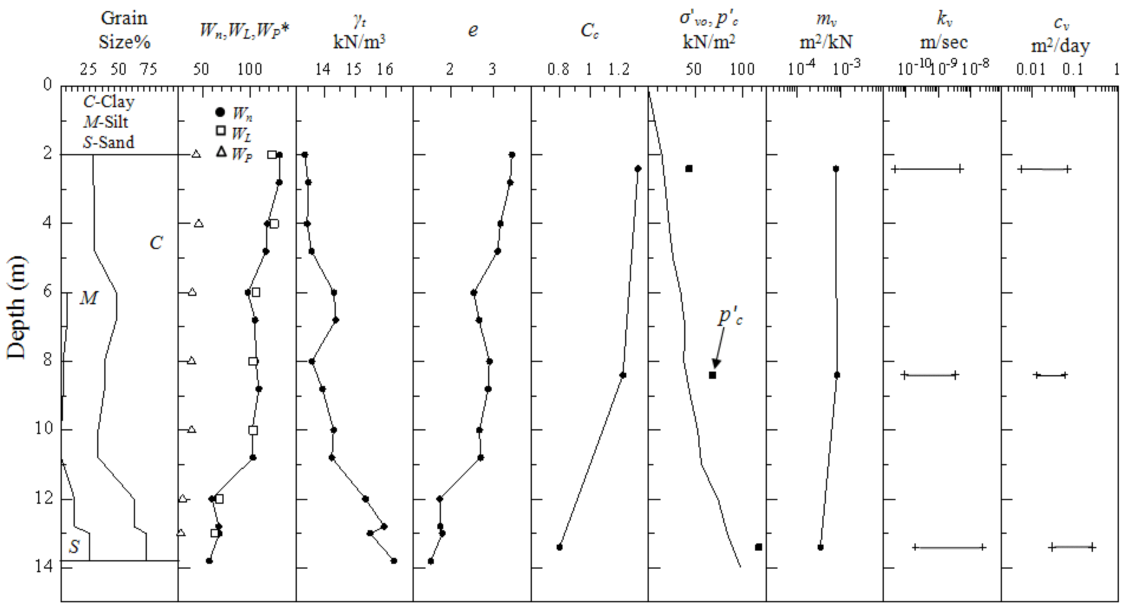


Fig. 3 Plan layout of field tests at TA site (after Chai *et al*. 2004)



*W_n; Water content, W_l; Liquid limit, W_p; Plastic limit, γ_t; Unit weight, e; Initial void ratio, C_c; Compression index, m_v; volume compressibility; σ'_{vo}; vertical effective stress, p'_c; preconsolidation pressure, k_v; hydraulic conductivity and c_v; coefficient of consolidation in the vertical direction.

Fig. 4 Some physical and mechanical properties of Ariake clay at the TA site

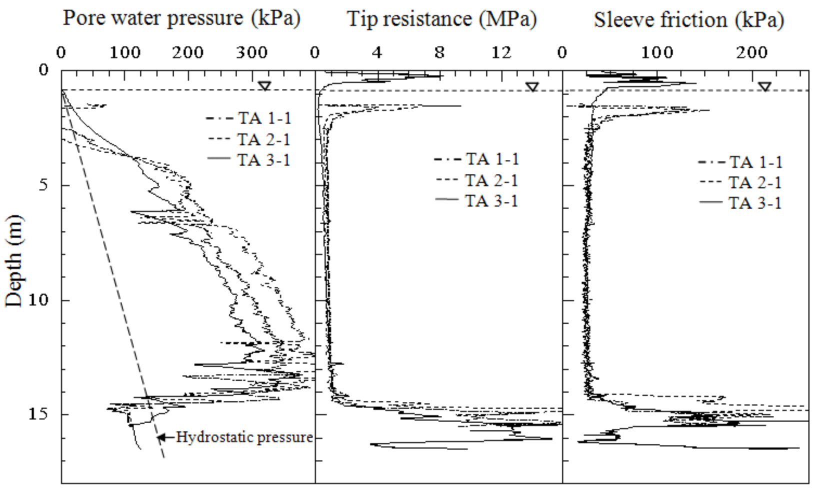
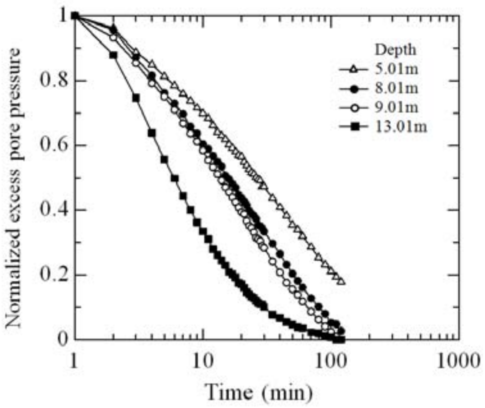


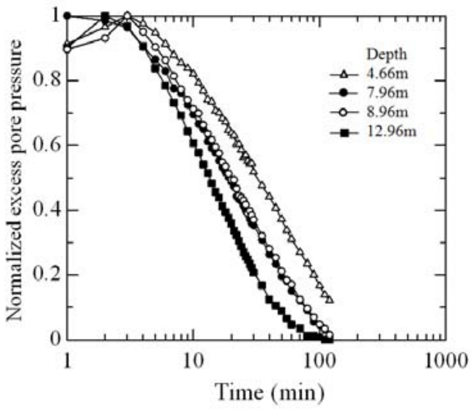
Fig. 5 Piezocone penetration test results at TA site

adjacent to the piezocone test points (Fig. 3). Fig. 4 shows some physical and mechanical properties of the soils retrieved from the borehole. The range of laboratory values of c_v and k_v indicated in the figure are for consolidation stresses (measured with an odometer) from 4.9 kPa to 1254 kPa. Ariake clay is sensitive and highly compressible clay. For most cases the plasticity index is about 50 and the natural water content is greater than 100%, slightly higher than the corresponding liquid limit.

As shown in Fig. 3, six piezocone penetration tests were arranged in three pairs, and each pair involved a continuous penetration test (Test point TA 1-1, 2-1 and 3-1), and a separate test that paused at about 1.0 m intervals to measure the dissipation of excess pore water pressure generated during the preceding penetration (Test point TA 1-2, 2-2 and 3-2). At the test site, the elevations of three pairs of the tests are different. In Figs. 4 and 5, the depth is defined by taking the ground surface at the BH location with an elevation of 0.80 m as 0. Although the elevations at TA 1-1, TA 2-1 and TA 3-1 are higher than that at the BH location, the piezocone readings were taken only when the cone entered the original soil deposit. In Fig. 5, the sections above elevation 0.80 m are not shown. Corrected tip resistances (q_t), total pore water pressures (u) and sleeve frictions (f_s) for tests TA 1-1, TA 2-1 and TA 3-1 are plotted in Fig. 5. For the dissipation tests conducted at TA 3-2, there are some abnormal phenomena, and we judged that they are less reliable (possibly due to a measurement problem) and are excluded here. At TA 1-2 and TA 2-2, a total of 25 dissipation tests were conducted. However, the dissipation tests at the depth of 2.01 m at TA 1-2, and at the depth of 2.96 m at TA 2-2 are excluded. TA 1-2 and TA 2-2 were on the berm of the river embankment, and due to the embankment loading induced settlements, the two depths may be near the interface between the original ground surface and the embankment fill material, and the data are judged as unreliable. The ground-water level was about 0.8 m below the ground surface at the BH location. Figs. 6(a) and (b) show some of normalized field excess pore pressure dissipation curves at TA 1-2 and TA 2-2, respectively. The normalization is made using the following equation.



(a) Test point TA 1-2



(b) Test point TA 2-2

Fig. 6 Dissipation test results at TA site

$$U = \frac{u(t) - u_0}{u_{\max} - u_0} \tag{16}$$

where $u(t)$ = total pore water pressure at time t, $u_{\max}$ = maximum measured total pore water pressure, and u_0 = the equilibrium in situ pore water pressure at the depth of interest.

Most reported non-standard excess pore water pressure dissipation curves have been for heavily over-consolidated clayey deposits (Burns and Mayne 1998, Sully *et al.* 1999). The test results at the TA site indicate that the phenomenon can also occur in some lightly over-consolidated soils.

With the methods described in Section 2, for both standard and non-standard dissipation curves, the value of I_r is needed to calculate c_{hp} values. It has been reported that the Ariake clay deposits in Saga area has a ratio of E_{50}/S_u (E_{50} is the secant modulus at 50% of peak deviator stress from unconfined compression tests, s_u is undrained shear strength) between 100 and 200 (Chai *et al.* 2005). Assuming a Poisson's ratio of 0.5 (undrained) and E_{50}/S_u ratio of 150, an I_r value of 50 can be obtained and it has been used in calculations. c_{hp} values have been calculated by Teh and Houlsby's (1991) method for the standard dissipation curves, and Chai *et al.*'s (2012a) method for the non-standard curves. The calculated values are listed in Table 1 together with the available laboratory measured c_v values.

k_{hp} values have been estimated using the methods of Baligh and Levadoux (1980) (Eq. (15)), Robertson (2010) (Eqs. (8) and (9)) and Chai *et al.* (2011) (Eq. (14)), and are listed in Table 2. In Tables 1 and 2, the c_v and k_v values were interpolated from oedometer test results using in-situ vertical effective stress, σ'_{vo}, estimated at the depth where the sample was retrieved.

3.2 Test results at TB site

Two piezocone test locations were arranged adjacent to a borehole (approximately 2.0 m apart) at this site. At one location continuous penetration was carried out (Test TB-1) and at another location the piezocone was halted at about 1.0 m intervals to measure the dissipation of excess pore water pressures generated during the preceding penetration, and in total 9 dissipation tests were conducted (Test TB-2) (Ariake Sea Coastal Road Development Office (ASCRDO), 2008). At the site, the thickness of the soft clay deposit is about 17 m. The soil profile and some of the measured physical and mechanical properties of the soils are given in Fig. 7. There is a sandy clay or clayey sand layer with a thickness of about 4.0 m at the ground surface. Below this is an approximately 12 m thick clay layer. The ground-water level was about 0.6 m below the ground surface.

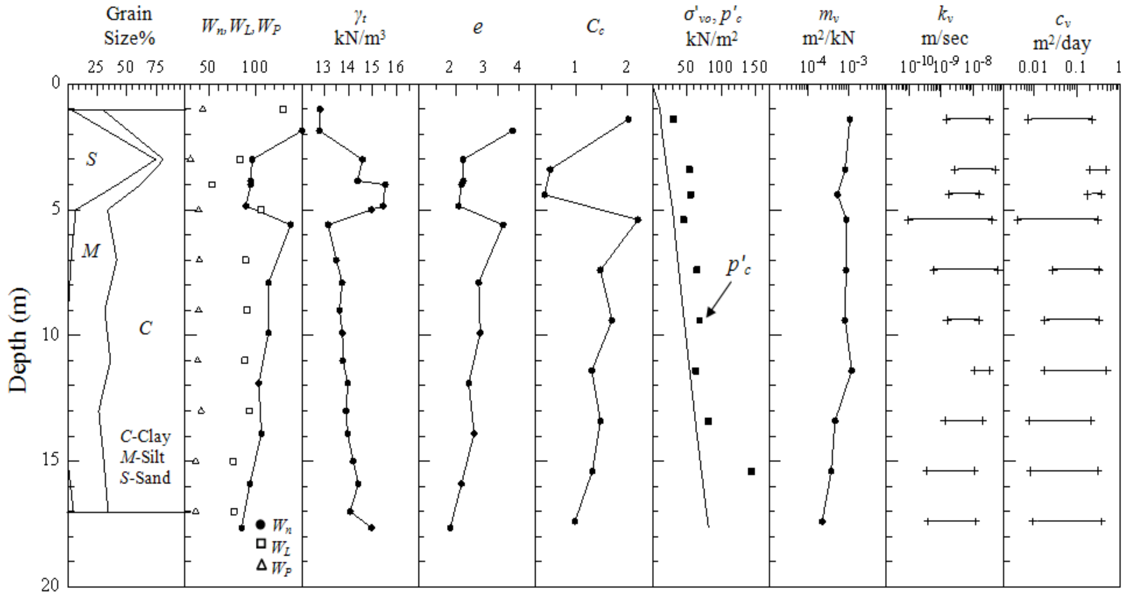


Fig. 7 Some physical and mechanical properties of soils at the TB site

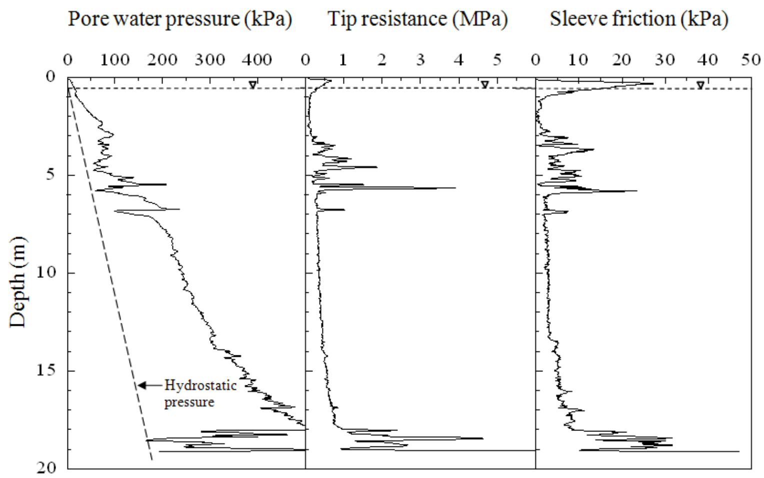


Fig. 8 Piezocone penetration test results at the TB site

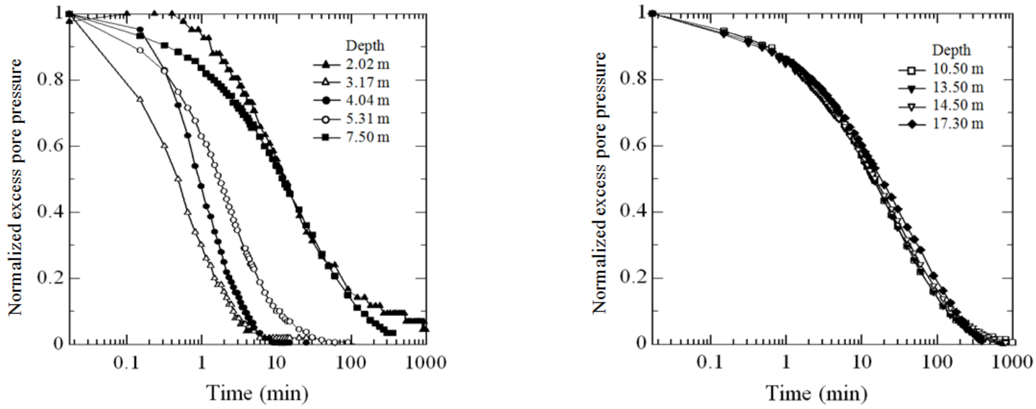


Fig. 9 Field dissipation test results at TB-2

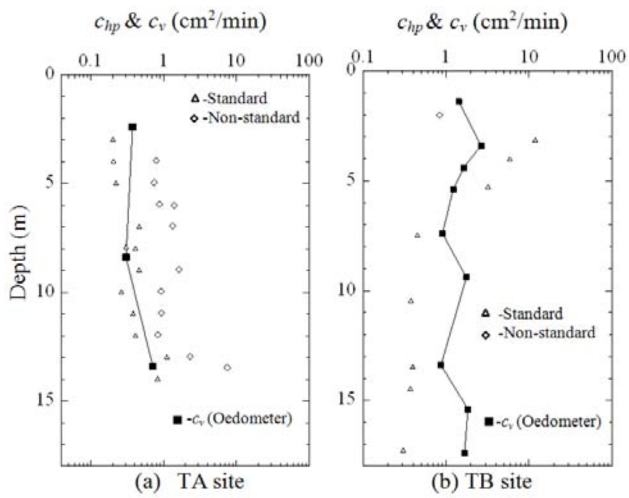


Fig. 10 Test results of c_{hp} (uCPT) and c_v (oedometer)

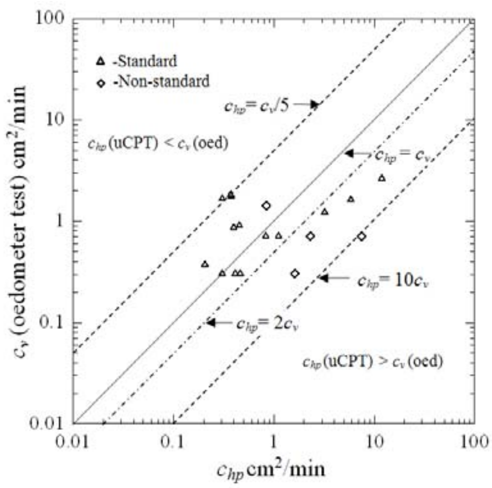


Fig. 11 Comparison of c_{hp} (uCPT) and c_v values

The measured q_t, u and f_s values from TB-1 are given in Fig. 8. Some of the normalized field excess pore water pressure dissipation curves are given in Fig. 9. A non-standard dissipation response is measured only at one depth (2.02 m). With the results given in Figs. 8 and 9, the calculated c_{hp} and k_{hp} values are listed in Tables 1 and 2, respectively. In the calculation, I_r of 50 has been used.

4. Comparison of c_{hp} and k_{hp} values with laboratory measured c_v and k_v values

4.1 Comparison of c_{hp} and c_v values

At the TA site, laboratory consolidation test results using undisturbed samples are available for

Table 1 Summary of the field dissipation and laboratory consolidation test results

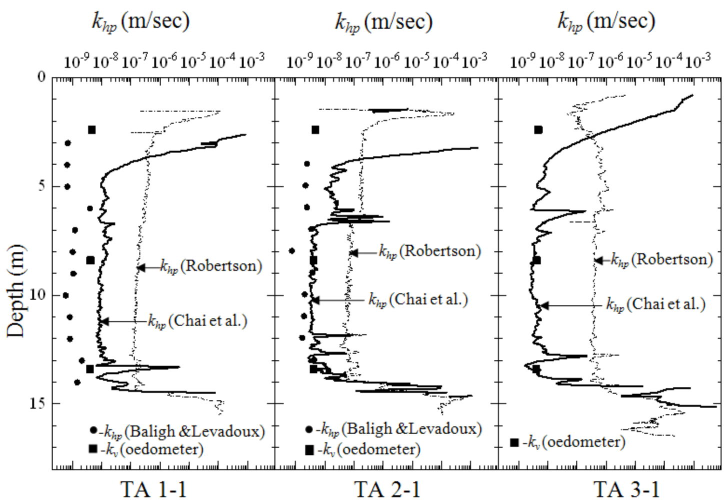
Test site	Test point No.	Depth m		$t_{u\max}$ min	$u_{\max}$ kPa	t_{50} min	t_{50m} min	c_{hp} cm^2/min		c_v cm^2/min (oedometer)
		CPTu	Oedometer					Standard	Non-standard	
TA	1-2	3.01	2.40*		186.62	27.54		0.202		0.375
		4.01			194.85	27.04		0.205		
		5.01			234.28	25.04		0.222		
		6.01		2	252.12	16.00	3.97		1.400	
		7.01			239.67	12.00		0.463		
	2-2	3.96		4	175.73	29.50	7.02		0.790	
		4.96		3	197.31	28.00	7.50		0.740	
		5.96		3	209.66	25.00	6.33		0.877	
		6.96		3	218.88	19.00	4.18		1.330	
	1-2	8.01	8.40		262.43	13.50		0.411		0.305
		9.01			275.76	12.40		0.461		
	2-2	7.96			251.25	18.00		0.305		
		8.96		3	277.04	16.70	3.43		1.620	
	1-2	10.01			306.85	21.00		0.264		
		11.01			328.81	14.50		0.383		
		12.01			350.39	13.50		0.411		
	2-2	9.96		2	301.35	21.00	5.95		0.932	
		10.96		2	324.01	21.00	5.95		0.932	
		11.96		2	349.90	22.50	6.59		0.842	
	1-2	13.01	13.40		349.51	5.00		1.110		0.711
		14.01			370.30	6.70		0.828		
	2-2	12.96		2	391.87	11.50	2.40		2.310	
		13.46		2	359.61	5.40	0.74		7.480	
TB	2	2.02	1.40	0.23	65.00	12.27	6.60		0.841	1.424
		3.17	3.40		88.00	0.47		11.895		2.639
		4.04	4.40		116.10	0.95		5.843		1.632
		5.31	5.40		209.10	1.73		3.202		1.222
		7.50	7.40		200.20	12.30		0.451		0.903
		10.50	9.40		244.90	14.73		0.377		1.757
		13.50	13.40		308.60	13.97		0.397		0.868
		14.50	15.40		355.80	14.97		0.371		1.840
		17.30	17.40		441.30	18.20		0.305		1.667

*The value is the average depth for an approximately 0.8m long sample obtained by a thin-wall tube only 3 depths, and at the TB site, laboratory test results are available for all depths of field dissipation tests. The estimated c_{hp} values and available laboratory measured c_v values are compared in Figs. 10 (a) and (b) for test sites TA and TB, respectively. It can be seen that at the TA site, there are more points where c_{hp} values are higher than the corresponding c_v values. However, at the TB site, at about 2.0 to 6.0 m depth, $c_{hp} > c_v$, while at other depths, $c_v > c_{hp}$.

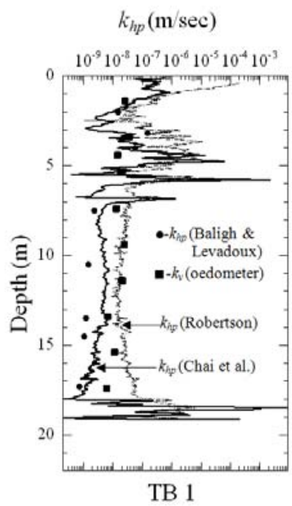
It is generally accepted that laboratory tests normally underestimate the field coefficient of consolidation (Chai and Miura 1999), and for most natural clayey soil deposits, the coefficient of consolidation in the horizontal direction (c_h) is larger than that in the vertical direction (c_v) (Chai *et al*. 2012b). Fig. 11 plots the relationship between c_{hp} and c_v values. It can be seen that almost all the data are within the range of $c_{hp} = c_v/5$ to $c_{hp} = 10c_v$, and a best fitted relationship is about $c_{hp} = 2c_v$. For undisturbed Ariake clay samples, the laboratory test gave a c_h/c_v ratio of about 1.6 (Chai *et al*. 2012b). $c_{hp}/c_v = 2$, is close to the laboratory measured ratio. Generally, it can be said that the methods adopted for estimating c_{hp} values have an acceptable accuracy. As for the points where $c_v > c_{hp}$, one of the possible explanations may be due to spatial variation of the soil, i.e., the soil at the piezocone test location may not be exactly the same as that at the borehole location. Another point is that the TB site is located in a serious land subsidence area, and the in-situ value of σ'_{vo} may be larger than that estimated assuming a hydrostatic water pressure condition. With a smaller σ'_{vo} value, a larger c_v (or k_v) value can be interpolated. However, the exact water pressure in the ground was not measured, and this kind of effect cannot be considered.

4.2 Comparison of k_{hp} and k_v values

A comparison of k_{hp} values estimated from uCPT results with k_v values from oedometer test results are plotted in Fig. 12. From Fig. 12(a) it is noted that at the TA site the values of k_{hp}, estimated using Robertson's (2010) method are higher than k_{hp} values estimated using Chai *et al*.'s



(a) TA site



(b) TB site

Fig. 12 Hydraulic conductivity from Piezocone test results and laboratory oedometer test results

Table 2 Summary of the results of hydraulic conductivities

Test site	Test point No.	Depth m	k_{hp} m/sec Baligh and Levadoux (1980)	Chai *et al.* (2011)	Robertson (2010)	k_v m/sec (oedometer)
TA	1-1	8.40	1.009E-09	1.19E-08	1.53E-07	4.070E-09
	2-1	8.40	7.416E-10	3.27E-09	6.52E-08	4.070E-09
	3-1	8.40		3.40E-09	3.80E-07	4.070E-09
	1-1	13.40	2.076E-09	2.32E-07	1.67E-07	4.029E-09
	2-1	13.40	1.348E-08	4.39E-09	7.21E-08	4.029E-09
	3-1	13.40		2.75E-09	6.55E-07	4.029E-09
TB	1	1.40	1.496E-08	5.44E-08	2.20E-07	2.560E-08
		3.40	1.506E-07	5.41E-08	1.09E-06	3.624E-08
		4.40	5.676E-08	1.44E-08	6.41E-07	1.441E-08
		5.40	2.332E-08	6.08E-09	1.58E-08	1.799E-08
		7.40	2.310E-09	4.58E-09	2.27E-08	1.299E-08
		9.40		4.20E-09	2.40E-08	2.413E-08
		11.40		5.93E-09	1.46E-08	2.078E-08
		13.40	1.210E-09	6.57E-09	1.55E-08	6.671E-09
		15.40		3.18E-09	1.95E-08	1.143E-08
		17.40	7.146E-10	1.17E-09	5.99E-08	6.131E-09

(2011) method, and much higher than the laboratory k_v values for most locations (Fig. 12(a)). The values of k_{hp}, estimated using Baligh and Levadoux's (1980) method with $RR = 2.5 \times 10^{-2}$, are less than the k_{hp} values estimated using Chai *et al.*'s (2011) method and are less than the laboratory k_v values for most locations (Fig. 12). We believe that if an accurate value of RR can be estimated, Baligh and Levadoux's equation can result in reasonable k_{hp} values. For the point at a depth of about 2.2 m, the difference between k_{hp} and k_v values is very large. The borehole (BH) was located at the toe of the river embankment, where settlement due to the embankment load should be much less than that under the berm where TA 1-1 and TA 2-1 were located. At a relative depth of about 2.0 m, the soils in the BH location and in the piezocone test locations of TA 1-1 and TA 2-1 are most likely different due to different settlement. For this reason, this point is not included in Fig. 13 for comparison. Although for the results at the TB site, it is difficult to judge which method (i.e., Baligh and Levadoux's, Robertson's or Chai *et al.*'s (2011) method) is better, the results at the TA site clearly show that Robertson's method is not applicable. As explained, for comparing the c_{hp} and the c_v value at the TB site, the higher interpolated k_v values may be due to possible underestimation of σ'_{vo} values.

The relationship between k_{hp} values from Chai *et al.*'s (2011) method and the laboratory k_v values is shown in Fig. 13. Almost all data points are within the range of $k_{hp} = k_v/10$ to $k_{hp} = 10k_v$. It can be said that the k_{hp} values from Chai *et al.*'s (2011) method are orderly correct. For the clayey soil deposit in Saga plain, the laboratory measured k_h/k_v ratios using undisturbed soil samples are about 1.5 from incremental load odometer tests (Park 1994), and 1.7 from constant rate of strain

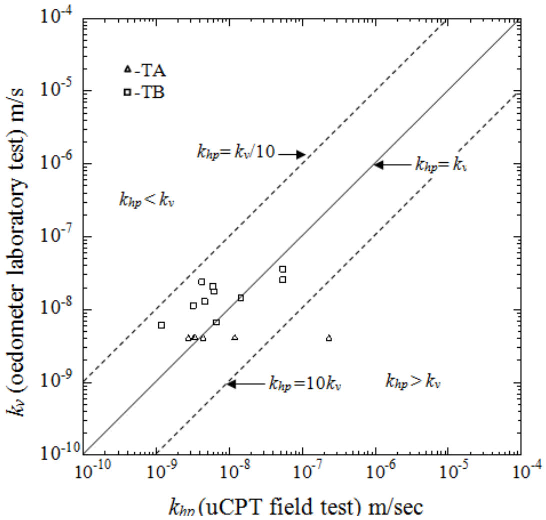


Fig. 13 Comparison of k_{hp} with k_v values

odometer tests with vertical and radial drainage respectively (Chai *et al*. 2012b). Where for some points at the TB site, $k_v > k_{hp,}$ this may be due to possible underestimation of in-situ σ'_{vo} values and then interpolated larger k_v values.

5. Conclusions

Some of the most widely used and newest methods for estimating the in-situ horizontal coefficient of consolidation (c_{hp}) and hydraulic conductivity (k_{hp}) using piezocone penetration and dissipation tests have been briefly reviewed.

There are two sites at Saga, Japan, where the results of field piezocone penetration, dissipation and laboratory consolidation tests using undisturbed soil samples are available. Comparing the estimated c_{hp} values by Teh and Houlsby's (1991) and Chai *et al*.'s (2012a) methods and comparing the k_{hp} values by Chai *et al*.'s (2011) method from the piezocone test results and the laboratory coefficient of consolidation (c_v) and hydraulic conductivity (k_v) in the vertical direction, the following conclusions/recommendations can be made:

- For most points, the values of both c_{hp} and k_{hp} are within a range of 1/10 to 10 times the corresponding laboratory values in the vertical direction. The best fitted relationships are $c_{hp} \approx 2c_v$ and $k_{hp} \approx k_v$. For Ariake clay deposits in Saga, Japan, the laboratory measured c_h/c_v and k_h/k_v (where c_h and k_h are the coefficient of consolidation and the hydraulic conductivity in the horizontal direction, respectively) ratios are 1.5 to1.7. Further considering that field values of c_h and k_h are higher than that of laboratory values, $c_{hp}/c_v \approx 2$ is quite reasonable. However, $k_{hp}/k_v \approx 1.0$ seems lower, and it may be due to possible underestimation of in-situ vertical effective stress (σ'_{vo}), which leads to a higher interpolated k_v value.
- Based on the results of this study, Teh and Houlsby's (1991) and Chai *et al*.'s (2012a) methods are recommended for estimating c_{hp} value from field standard and non-standard

dissipation curves respectively. Chai *et al*.'s (2011) method is suggested for calculating k_{hp} values from piezocone sounding records.

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