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Horizontal Coefficient of Consolidation of Soft Bangkok Clay

ABSTRACT: This paper presents the results of the coefficient of consolidation in the horizontal direction determined from the laboratory and the field through testing, along with back analysis from settlement measurement of embankment constructed with prefabricated vertical drains at the Suvarnabhumi Airport site in Bangkok. Constant rate of strain consolidation tests with radial drainage and standard oedometer tests were conducted to provide the consolidation characteristics of soft Bangkok clay. A 20-mm piezoprobe was used to measure the pore water pressure dissipation characteristics of the soft clay in the field, and the corresponding horizontal coefficients of consolidation were estimated based on established theory. The test results, such as the horizontal coefficients of consolidation and permeability, were compared with back analysis of the constructed runway embankment based on Asaoka method. The results show very good agreement in both horizontal coefficients of consolidation and permeability obtained from different testing methods and back analysis, implying the reliability of the testing methods adopted.

KEYWORDS: clay, consolidation, dissipation, permeability, settlement

Notations

A area of specimen
α constant as function of r_w and r_e
β constant as function of r_w and r_e
β_0 intercept in Asaoka method
β_1 slope in Asaoka method
c_h coefficient of consolidation in horizontal direction
c_u undrained shear strength
c_v coefficient of consolidation in vertical direction
d_e diameter of soil cylinder
d_w diameter of well
Δt time interval
E_u undrained Young's modulus at 50 % undrained shear strength
F factor in radial flow consolidation
G_u undrained shear modulus
γ_w unit weight of water
H height of specimen
I_r rigidity index, $G_u/c_u = E_u/3\ c_u$
k_h coefficient of permeability in horizontal direction
k_s coefficient of permeability of disturbed clay
k_v coefficient of permeability in vertical direction
m_v coefficient of volume compressibility
P vertical load
q_c cone resistance
q_w discharge capacity of drain
r_e radius of specimen
r_o radius for cylindrical model
r_w radius of well or central drain
RR recompression ratio
ρ settlement
ρ_f final settlement
$\rho\ (t)$ primary consolidation settlement at time t
σ'_v effective vertical stress
$\sigma'_{v.ave}$ average effective vertical stress
σ'_{vo} effective overburden stress
σ'_p preconsolidation pressure
σ'_{vf} final effective vertical stress
t measured time
t_{50} measured time at 50 % dissipation
T time factor
T_{50} time factor at 50 % dissipation
u_b excess pore water pressure at impervious outer boundary
u_i initial pore water pressure at $t = 0$
u_{max} maximum pore water pressure
u_o in situ pore water pressure
u_t pore water pressure at time t
$\overline{U}$ normalized excess pore pressure
v_p velocity of load piston
w_l liquid limit
w_n natural water content
w_p plastic limit

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Introduction

The coefficient of consolidation in the horizontal direction (c_h) is an important parameter for predicting the rate of settlement of soft soils undergoing ground improvement via prefabricated vertical drain (PVD) method. This quantity can be determined in the laboratory and in the field by various methods, such as Rowe cell, dissipation test, etc. In the laboratory, a constant rate of strain consolidometer with radial drainage developed at the Asian Institute of Technology provides a continuous compression curve, together with estimates of horizontal coefficient of consolidation. In the field, dissipation test by means of piezocone or piezoprobe is generally used for evaluating the flow characteristics of the in situ soil. The typical feature of this equipment consists of a 60° apex angle cone, a 35.7-mm-diameter shaft and a filter element located just above cone tip. Both piezocone and piezoprobe have the ability of measuring pore water pressure at a particular depth. From the decay in pore water pressure with time or dissipation curve, the horizontal coefficient of consolidation can be estimated from established theoretical solutions.

A large-scale ground improvement project by means of prefabricated vertical drains has been adopted for the construction of runways, taxiways, and aprons at the New Second Bangkok International Airport (SBIA) or Suvarnabhumi Airport in recent years. This method was used to improve the subsoil condition and to accelerate the consolidation of soft clay, hence reducing future settlement. Extensive field monitoring, such as settlement and piezometric pressure measurements, has also been conducted during construction of the embankments. This site was selected for the study due to the availability of field monitoring data throughout the construction period, and these data can be used for comparing with measurements made from laboratory and field tests.

Undisturbed soil samples were collected before embankment construction for determining the consolidation characteristics by means of constant rate of strain consolidometer with radial drainage and standard oedometer tests. These consolidation tests provided the compression curves of soil specimens, the stress history as well as the coefficients of consolidation and permeability.

For determining the coefficient of consolidation in the field, a small (20 mm) diameter piezoprobe was designed and fabricated with a 60° cone apex and a filter located just above cone tip. The reduction in diameter shortened the duration of pore water pressure dissipation by about 3.2 times compared with the standard 35.7 mm cone.

From the test results, a comparison was made between the test measurements and back analysis from settlement data of the embankment during construction. A method proposed by Asaoka (1978) was adopted to estimate the magnitude of final settlement as well as the horizontal coefficient of consolidation from the measured settlement data. This method adopts a curve fitting procedure based on the consolidation theory, and some essence of the method is explained below.

The solution of the consolidation equation under radial drainage condition takes the following form:

The settlement,

$$\rho(t) = \rho_f - \rho_f \exp\left[-\frac{8\,c_h}{d_e^2\,F}t\right] \tag{1}$$

or

$$\rho(t) = \rho_f\left[1 - \exp(\lambda t)\right] \tag{2}$$

where

$$\lambda = -\frac{8}{d_e^2\,F}c_h \tag{3}$$

Equation 2 can be expressed in the form of an ordinary differential equation:

$$\frac{d\rho}{dt} - \lambda\rho = \mathrm{f} \tag{4}$$

where f is a constant.

The variable ρ in Eq 1 will satisfy all conditions in Eq 4. If the time is evenly divided into Δt interval and $\rho_i = \rho(\Delta t_i)$, then Eq 4 becomes

$$\frac{\rho_i - \rho_{i-1}}{\Delta t} - \lambda\,\rho_i = f$$

$$\Rightarrow \quad \rho_i = \frac{1}{1-\lambda\,\Delta t}\,\rho_{i-1} + \frac{\Delta t}{1-\lambda\,\Delta t}\,f$$

or

$$\rho_i = \beta_1\rho_{i-1} + \beta_0 \tag{5}$$

where

$$\beta_1 = \frac{1}{1-\lambda\Delta t} \tag{6}$$

and

$$\beta_0 = \frac{\Delta t}{1-\lambda\Delta t} \tag{7}$$

The constants, β_0 and β_1, will be the intercept and the slope of the fitted straight line in $\rho_{i-1} - \rho_i$ axes, and they can be obtained graphically with examples given in the later section. It should be noted that the final settlement could also be obtained from the following expressions:

$$\rho_i = \rho_{i-1} = \rho_f \quad \text{as time approaches infinity, then}$$

$$\rho_f = \frac{\beta_0}{1-\beta_1} \tag{8}$$

The horizontal coefficient of consolidation can be obtained by substituting Eq 3 into Eq 6, giving

$$c_h = \frac{(1-\beta_1)\,d_e^2 F}{8\,\beta_1\,\Delta t} \tag{9}$$

Therefore, the horizontal coefficient of consolidation and the final settlement can be determined by this method.

Descriptions of CRS-R Consolidometer with Radial Drainage

Seah and Juirnarongrit (2003) developed a constant rate of strain consolidometer with radial drainage (CRS-R) as shown in Fig. 1 and established equations based on Barron's equal strain theory

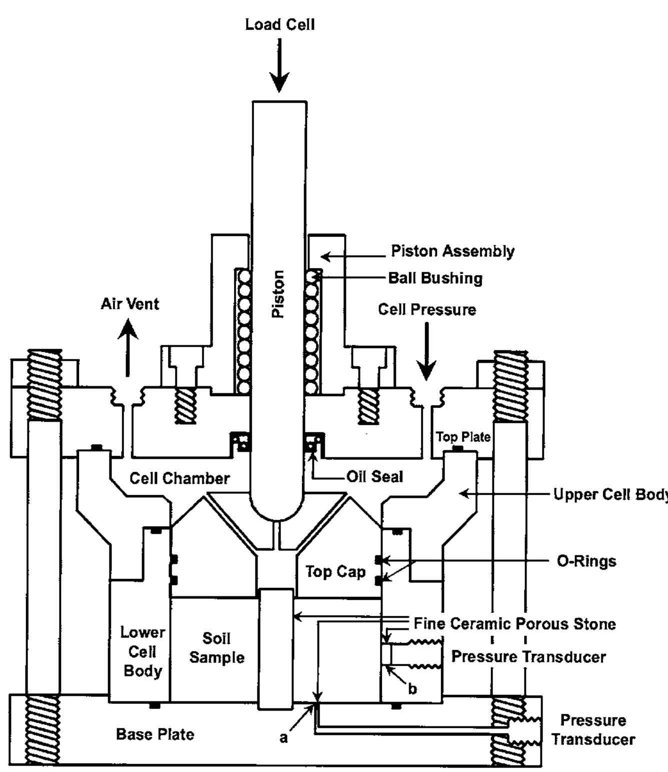

FIG. 1—*CRS consolidometer with radial drainage.*

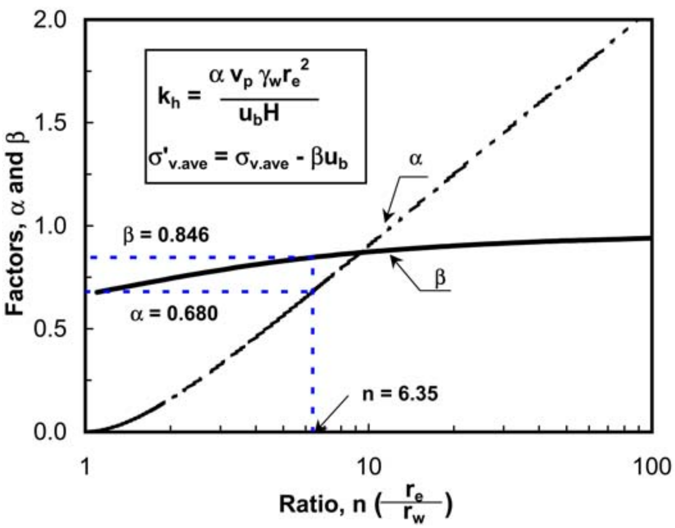

FIG. 2—*Relationship among α, β, and n.*

(1948) to determine the horizontal coefficients of consolidation and permeability under radial flow condition. For CRS tests with radial drainage, the horizontal coefficient of permeability can be estimated from the following equation:

$$k_h = \alpha \frac{v_p \gamma_w r_e^2}{u_b H} \tag{9}$$

The horizontal coefficient of consolidation is given by:

$$c_h = \frac{k_h}{m_v \gamma_w} = \frac{\alpha v_p r_e^2}{u_b H m_v} \tag{10}$$

By assuming the pore water pressure distribution based on Barron's theory, the average effective vertical stress can be expressed as:

$$\sigma'_{v.ave} = \frac{P}{A} - \beta u_b \tag{11}$$

In this study, the r_w and r_e values are 5 mm and 31.75 mm, respectively, giving α value of 0.68 and β value of 0.846 from the relationships given in Fig. 2. Based on the above equations, the consolidation parameters can be estimated accordingly.

Equipment and Testing Procedure of CRS-R Test

The major components of the constant rate of strain consolidometer with radial drainage are shown in Fig. 1. The soil sample preparation procedure has been described by Seah and Juirnarongrit (2003). The equipment consists of a base plate, two parts of the cell body (i.e., upper and lower cell bodies), top platen, top cap, and a loading piston. The lower cell body has an inner diameter of 63.5 mm that is used to hold the specimen, and the upper cell body is used for producing the space for placement of top cap. A hole in the middle of the soil is made by inserting a piece of piano wire through the center with guiding plates at both ends of the trimmed soil specimen. The wire is rotated around the guided holes of the plates until the soil in the middle hole is detached. The excess soil at the two ends is trimmed using the wire saw to give smooth surfaces. A cylindrical fine porous stone is then gently inserted in the cut hole, which will be used as the only drainage boundary of the soil specimen during consolidation. After trimming is completed, the lower cell body is fixed to the base plate with a 1-mm-diameter hole located 13 mm from the center (Location *a*). This small hole is flushed with a fine porous stone at one end and connected to a pressure transducer at the other end. Pore water pressure is measured at the outer boundary with another pressure transducer (Location *b*) attached to the wall of the lower cell body. Excess pore pressure distributions within the specimen can be determined using these two pore pressure measurements. The upper cell body acts as a water chamber to assist in specimen saturation. The top cap transfers load from the piston to the soil specimen. A 10-mm-diameter hole with a depth of 15 mm at the center of the top cap allows the cylindrical porous stone to slide freely during consolidation. Two (2) greased O-rings on the top cap are used to prevent leakage through the gap between the top cap and cell body. Previously, measured friction induced by the O-rings is deducted from the vertical force measurement to provide an estimate of the vertical load on the soil specimen. The pore water pressures are measured at two locations across the specimen during consolidation for verifying the pore water pressure distribution. Once the equipment is assembled, a backpressure of 200 kPa is applied to the soil specimen for saturation over a period of 24 h. The soil is loaded by a gear driven loading frame that forces the loading piston to move downwards at a constant speed. The vertical load, pore water pressures, and displacement are measured by means of a load cell, pressure transducers, and displacement transducer, which are recorded automatically by a data acquisition system.

Descriptions of Piezoprobe for Dissipation Test

A piezoprobe was designed and fabricated for execution of the dissipation test as shown in Fig. 3. The size of the piezoprobe used is smaller in diameter than the standard size so that the shorter testing duration can be achieved, since the dissipation response is proportioned to the square of cone diameter (refer to Eq 13). The piezoprobe used has a diameter of 20 mm, with a 60° apex cone and a 5-mm-thick porous stone located just behind the cone tip. A pressure transducer with a maximum capacity of 700 kPa is connected to the porous element for pressure measurement. The resolution of the pressure measurement is within ±0.1 kPa.

Equipment and Testing Procedure of Dissipation Test

Before testing, the pressure transducer was calibrated in the laboratory by the pressure calibrator. To achieve proper pore water pressure response, the porous stone and the cone have to be deaired and saturated. After cleaning the porous stone in the ultrasonic cleaner followed by drying in the oven, the porous stone is then placed in the desiccator where vacuum can be applied to the desiccator through a vacuum pump for at least 3 h. Deaired water is then flushed through the valve into the desiccator to saturate the porous stone. After the porous stone is saturated, the piezoprobe is assembled underwater and stored in the water before use. For testing in the field, the cone element is connected to a 1-m-long extension rod with the electrical cable of the pressure transducer passing through the rod. The rod is pushed continuously into the ground at a steady rate of about 20 mm/s until the required depth is reached. Once the cone has reached the required testing depth, the cone together with rod is held in position by clamping the rod to the ground surface. The pore water pressure measured by pressure transducer is recorded periodically until it is almost fully dissipated, which may take up to 20 h.

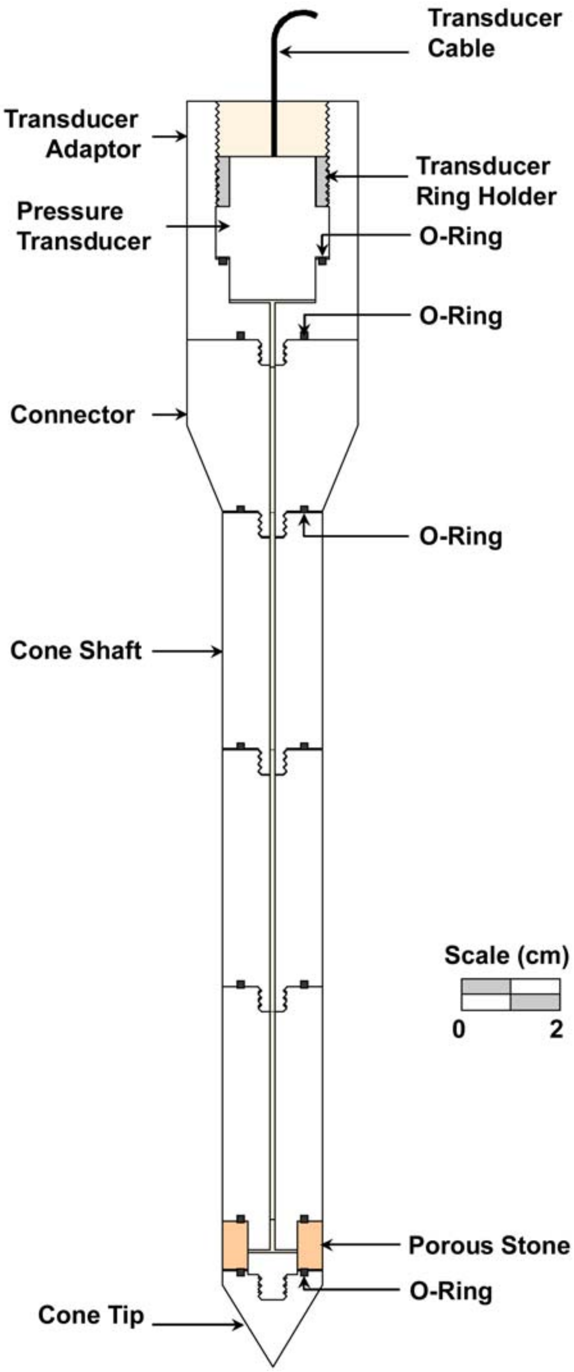

FIG. 3—*Schematic of piezoprobe.*

Interpretation of Dissipation Test Results

To interpret the dissipation data, normalization of pore pressure relative to the initial (or maximum) pore pressure at the beginning of dissipation, u_{max}, and the equilibrium in situ pore pressure, u_o, is needed. The normalized excess pore pressure, $\overline{U}$, at a given time, is expressed as:

$$\overline{U} = \frac{u_t - u_o}{u_{\max} - u_o} \tag{12}$$

Teh and Houlsby (1991) proposed a relationship between a dimensionless time factor and c_h value based on numerical analysis of dissipation pore water pressure with consideration of soil rigidity index, I_r. The dimensionless time factor, T^*, is defined as:

$$T^* = \frac{c_h t}{r_o^2 \sqrt{I_r}} \tag{13}$$

Robertson et al. (1992) reviewed some dissipation data from piezocone tests, and concluded that the predicted coefficient of consolidation by Teh and Houlsby's (1991) solution compared well with laboratory test results. Therefore, this theoretical solution has been adopted in this study for estimation of the coefficient of consolidation.

The horizontal coefficient of permeability can also be computed based on the following equation:

$$k_h = \gamma_w m_v c_h \approx \frac{\gamma_w (RR) c_h}{2.3 \sigma'_{vo}} \tag{14}$$

The recompression ratio, *RR*, can be obtained from the consolidation tests at the corresponding stress level.

Testing Material and Test Program

The soft clay for laboratory testing was collected from the runway section of the airport site before filling. The general physical properties and stress history of the subsoil down to 22 m are presented in Fig. 4. The laboratory and field tests were mainly concentrated on the very soft to soft clay layer where the prefabricated vertical drains were installed.

Undisturbed samples were collected by means of a stationary piston sampler before embankment construction at 1 m interval between 4 and 9 m. Once the Shelby tube was withdrawn from the borehole, the soil sample at the end of the tube was excavated for waxing at both ends. The collected samples were stored at the site temporarily before transporting back to the laboratory for testing. A total of six CRS-R consolidation tests was performed at a strain rate of 1×10^{-6}/s. Standard oedometer tests were also performed to yield the consolidation properties in the vertical direction. A summary of oedometer test data is given in Table 1.

Eleven piezoprobe dissipation tests were performed at depths of 4–9 m. The duration of the tests varied from 4 to 20 h, depending on the rate of dissipation. The pore water pressure was measured until very little or no change in pore water pressure had taken place. The measured data gave the dissipation response as well as the "quasi-static" or hydrostatic piezometric pressure in the ground at the testing depths.

Instrumentation on Embankment

Several types of monitoring instruments were installed to monitor the behavior of the embankment with PVD ground improvement. A typical geometry of the runway embankment along with the instrumentation is illustrated in Fig. 5.

The piezometric pressures at depths of 2, 5, 8, and 12 m were measured by means of electrical (vibrating wire type) piezometers installed in between PVDs along the center portions of the runway before and during construction of the embankment. To obtain a proper estimate of excess pore water pressure, several corrections had to be made. Since the piezometers were installed beneath the PVD embankment, any settlement or compression of soft layer would lead to settlement of piezometers as well. Hence, deep settlement gages were installed adjacent to the piezometers at the same depths for locating the actual positions of the piezometers during construction. Surface settlement plates were placed on top of the

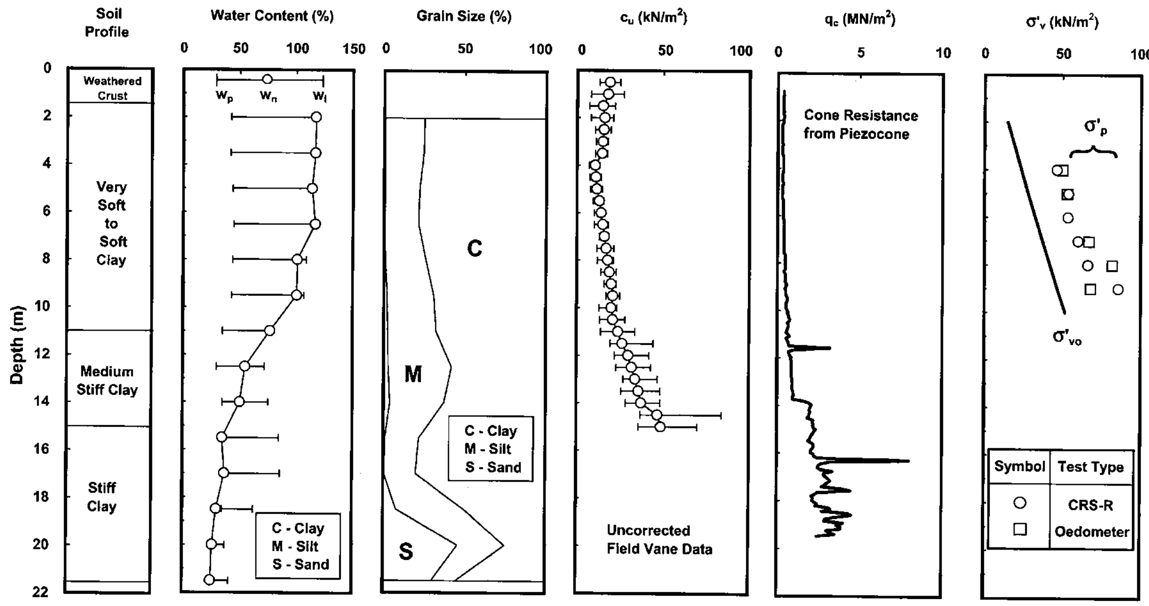

FIG. 4—*Soil profile and general soil properties.*

TABLE 1—*Summary of Coefficients of Consolidation and Permeability obtained from Different Methods.*

	At Effective Overburden Pressure, σ'_{vo}					At Final Stress Level, σ'_{vf}			
		c_h (m^2/yr)					c_h (m^2/yr)		
			Piezoprobe Test						
Depth (m)	σ'_{vo} (kPa)	CRS-R	No. 1	No. 2	c_v (m^2/yr) Oedometer	σ'_{vf} (kPa)	CRS-R	Asaoka	c_v (m^2/yr) Oedometer
4	24	11.4	–	4.6	4.3	99	0.79	0.75 ± 0.05	0.7
5	28	6.6	11.3	6.1	4.6	103	0.73	0.75 ± 0.05	0.5
6	33	6.9	7.0	7.9	3.9	108	0.76	0.75 ± 0.05	2.4
7	38	3.8	5.0	6.7	4.4	113	0.82	0.75 ± 0.05	0.4
8	44	4.5	6.3	6.5	5.0	119	0.91	0.75 ± 0.05	2.2
9	47	4.1	4.3	5.3	1.5	122	0.79	0.75 ± 0.05	0.7
	At Effective Overburden Pressure, σ'_{vo}					At Final Stress Level, σ'_{vf}			
		$k_h \times 10^{-3}$ (m/yr)					$k_h \times 10^{-3}$ (m/yr)		
			Piezoprobe Test						
Depth (m)	σ'_{vo} (kPa)	CRS-R	No. 1	No. 2	$k_v \times 10^{-3}$ (m/yr) Oedometer	σ'_{vf} (kPa)	CRS-R	Asaoka	$k_v \times 10^{-3}$ (m/yr) Oedometer
4	24	88	–	58	44	99	14	14 ± 1	16
5	28	50	154	83	35	103	13	14 ± 1	11
6	33	57	80	91	25	108	14	14 ± 1	17
7	38	47	51	67	24	113	13	14 ± 1	9
8	44	40	55	57	44	119	15	14 ± 1	28
9	47	21	20	24	17	122	19	14 ± 1	9

sand blanket (for practical reasons) along the center portions of the embankment; they were used for monitoring the surface settlement during loading.

Test Results and Discussions

CRS-R Test Results

Six constant rate of strain consolidation tests (CRS-R) with radial drainage were performed on samples collected at depths of 4–9 m. The compression curves of the CRS-R tests at various depths along with the results of horizontal coefficients of consolidation and permeability are presented in Figs. 6 and 7. The results showed a significant decrease in the horizontal coefficient of consolidation from overconsolidated range to normally consolidated range. The horizontal coefficient of permeability also decreases gradually with increasing vertical effective stress. These results are then compared with the measured field results, as well as back analysis in the later section.

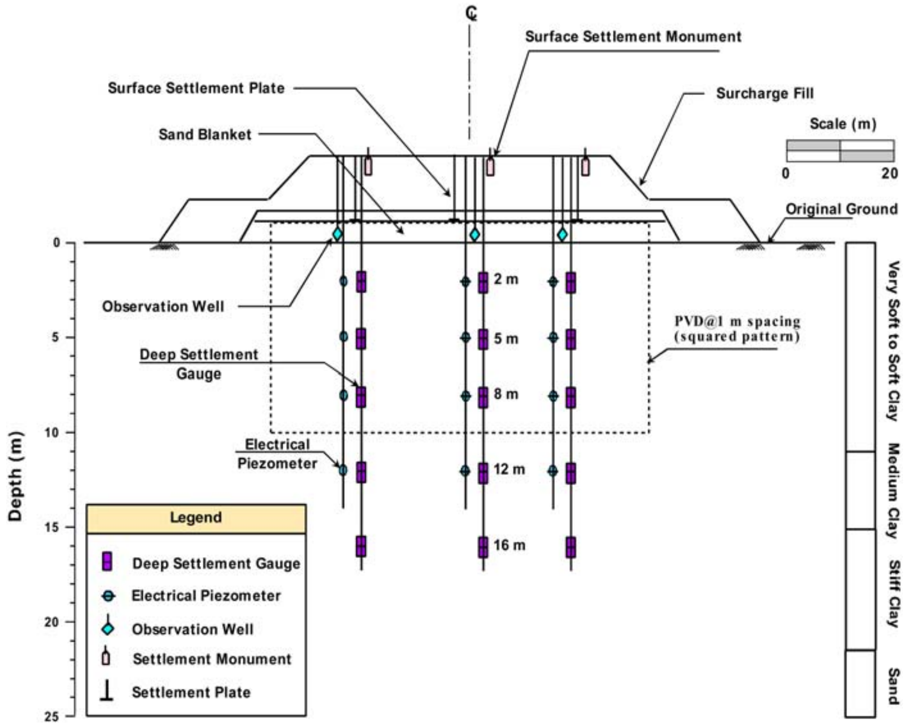

FIG. 5—*Embankment configuration and instrumentation.*

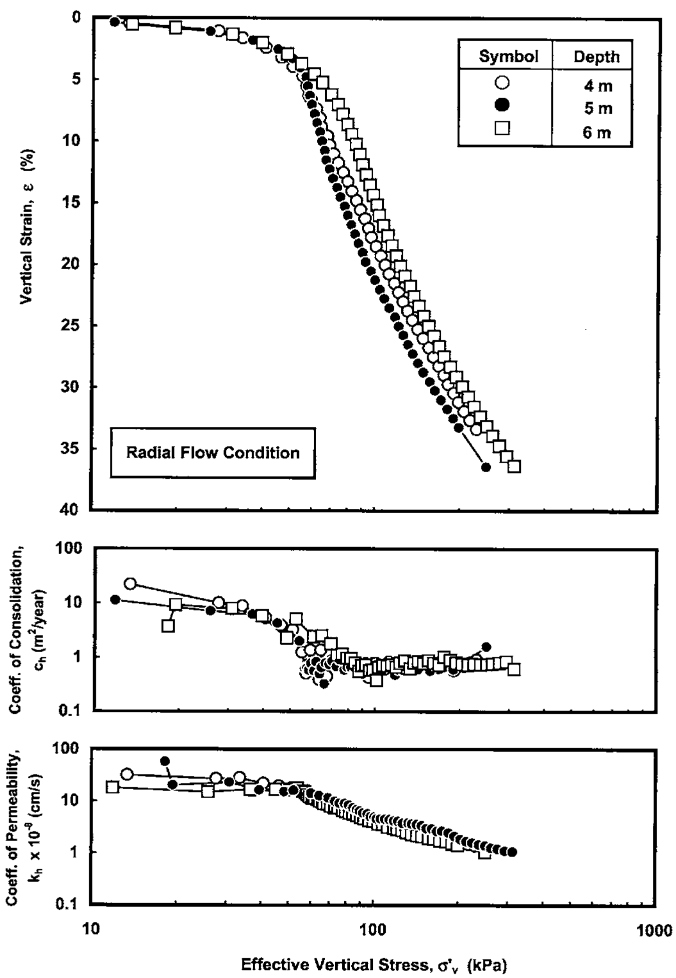

FIG. 6—*Consolidation characteristics of samples at depths of 4–6 m.*

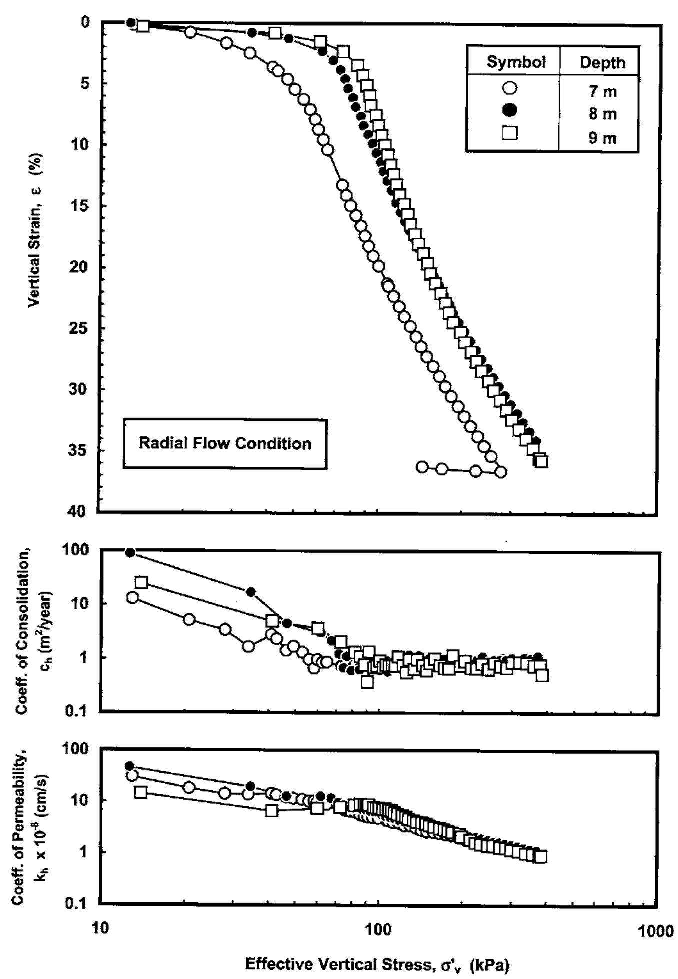

FIG. 7—*Consolidation characteristics of samples at depths of 7–9 m.*

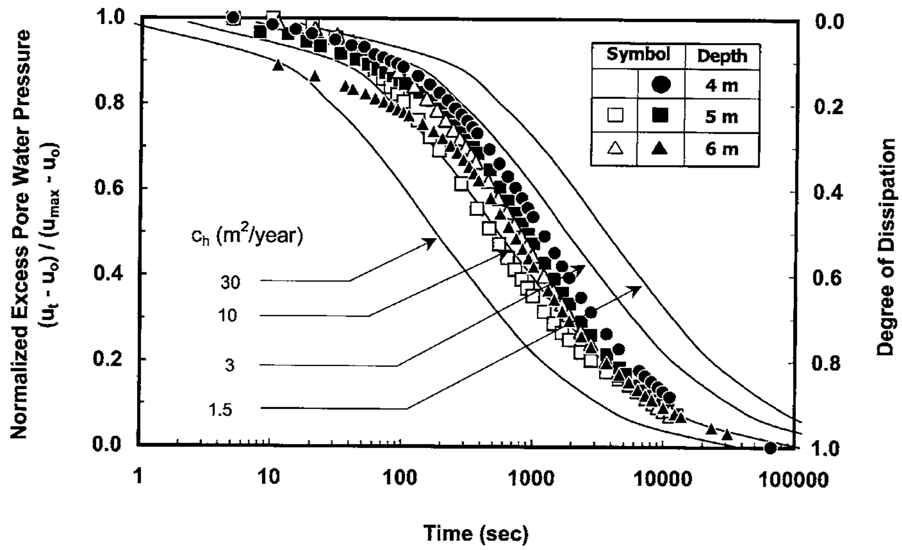

FIG. 8—*Dissipation curves at depths of 4–6 m.*

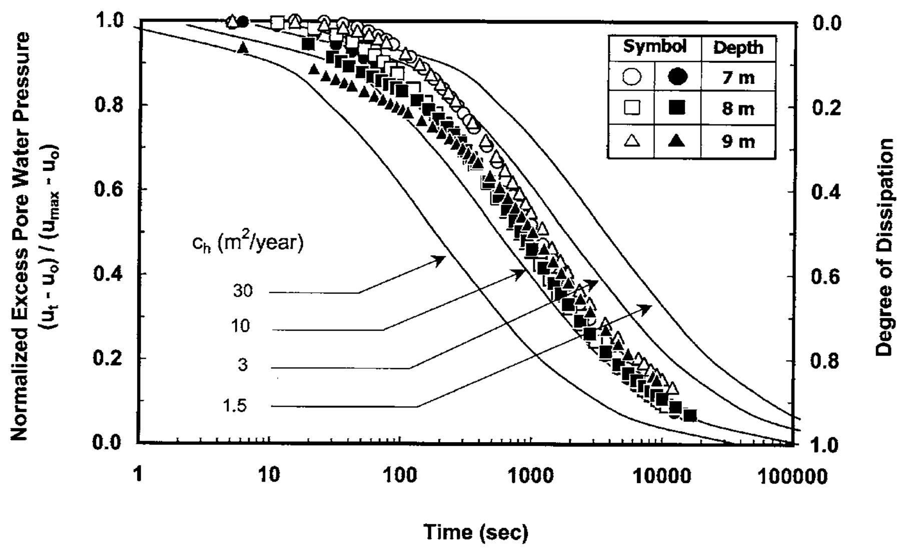

FIG. 9—*Dissipation curves at depths of 7–9 m.*

Dissipation Test Results

The coefficients of consolidation in the horizontal direction at different depths are estimated from the dissipation curves. The normalized excess pore water pressures from the tests are superimposed with the theoretical solution proposed by Teh and Houlsby (1991), as shown in Figs. 8 and 9. The rigidity index (I_r) used to calculate the c_h value is assumed to be 50 based on consolidated undrained triaxial tests on soft Bangkok clay conducted by Seah and Lai (2003). The dissipation curves showed large deviations at the initial portions of the dissipation process, and better agreement at later portions of the curves. It should be noted that 50 % of dissipation occurs within 15 min. To evaluate the variation in estimated c_h values with the degree of dissipation, the ratios of c_h values computed from different degrees of dissipation to the c_h values at 50 % dissipation are calculated and presented in Fig. 10. The results show that at 20–80 % dissipation range, the c_h ratios vary between 0.8 and 1.2; in other words, the computed c_h value increases with increased degree of dissipation. It is generally recommended to use

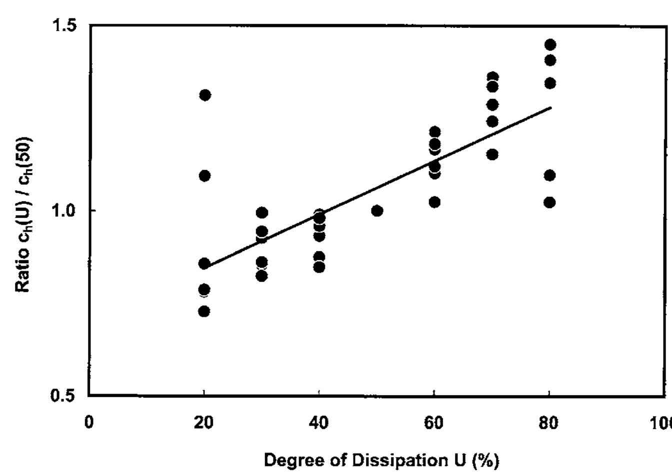

FIG. 10—*Ratio of c_h (U) to c_h (50 %) versus degree of dissipation.*

the c_h value at 50 % dissipation as a reference; hence, the values presented are with reference to 50 % dissipation. The computed c_h values at depths of 4–9 m range between 4.6 and 11.3 m^2/year.

The horizontal coefficients of permeability are calculated based on Eq 14 with recompression ratio obtained from the CRS-R tests at effective overburden stress. The k_h profiles with depth are tabulated in Table 1.

Piezometric Pressure

It should be mentioned that this site has a severe piezometric pressure drawdown problem; therefore, it is important to understand the change in the piezometric pressure profile due to PVD installation. An area adjacent to the embankment was installed with PVDs down to a depth of 10 m without any filling to determine the effect of PVDs on the piezometric pressure. Piezometric pressure in this area was measured and compared with readings in zone without any PVD installation.

Since the piezoprobe dissipation tests were conducted until the pore water pressure had reached an equilibrium value, the piezometric pressure obtained could also be compared with measurement from the piezometers. The results indicated that there is little difference in the piezometric pressure measured by means of piezometer and piezoprobe before construction. From the measurement, there is a deviation in the pore water pressure from the assumed "hydrostatic" pressure line of about 27 kPa at a depth of 9 m due to the drawdown effect caused by deep pumping.

The pore water pressure or piezometric profiles obtained from the piezoprobe tests and piezometer readings before and after PVD installation are plotted and compared, as shown in Fig. 11. The results indicated that the piezometric pressure down to the PVD installation depth of 10 m had recovered fully to hydrostatic condition within one month after installation due to recharging from the ground surface. The high discharge capacity of the PVDs assists in the recovery of the piezometric pressure in the upper 10 m of soft clay. In addition, after construction of the embankment at the end of surcharging, some PVDs were recovered from the field to evaluate the discharge capacity of the PVD after use; the results indicated very little reduction in the discharge capacity.

Performance of Embankment

The construction history of the embankment is presented in Fig. 12*a*, showing the fill thickness along with the activity over a three-year period. After clearing and grubbing of the ground, a layer of geotextile and a 1-m layer of sand blanket were placed with a subdrain network as drainage layer. The PVDs were then installed at 1-m spacing with squared pattern down to a depth of 10 m with reference to original ground level. An additional 0.5 m of sand blanket was placed before placing of another layer of geotextile and crushed rock as surcharge fill. The placement of surcharge was divided into two stages, with Stage No. 1 of 1.3 m for a period of 3 months followed by Stage No. 2 of another 0.8 m for a period of about 7 months before removal of surcharge, giving a maximum load of about 75 kPa.

The measured settlements of the embankment along the center portions are plotted with time as shown in Fig. 12*b*. Using Asaoka method with radial consolidation, the horizontal coefficient of consolidation can be estimated from Eq 9.

Hansbo (1979) suggested that the factor, F, might consist of the following components with consideration of the effect of smear

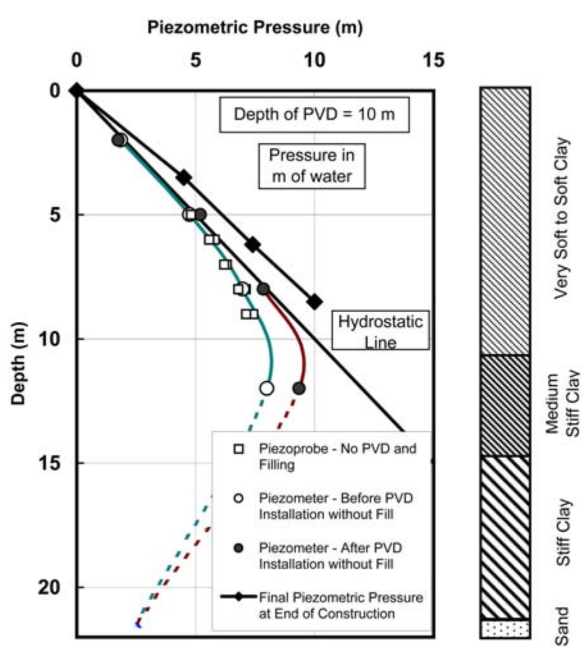

FIG. 11—*Piezometric pressure profile.*

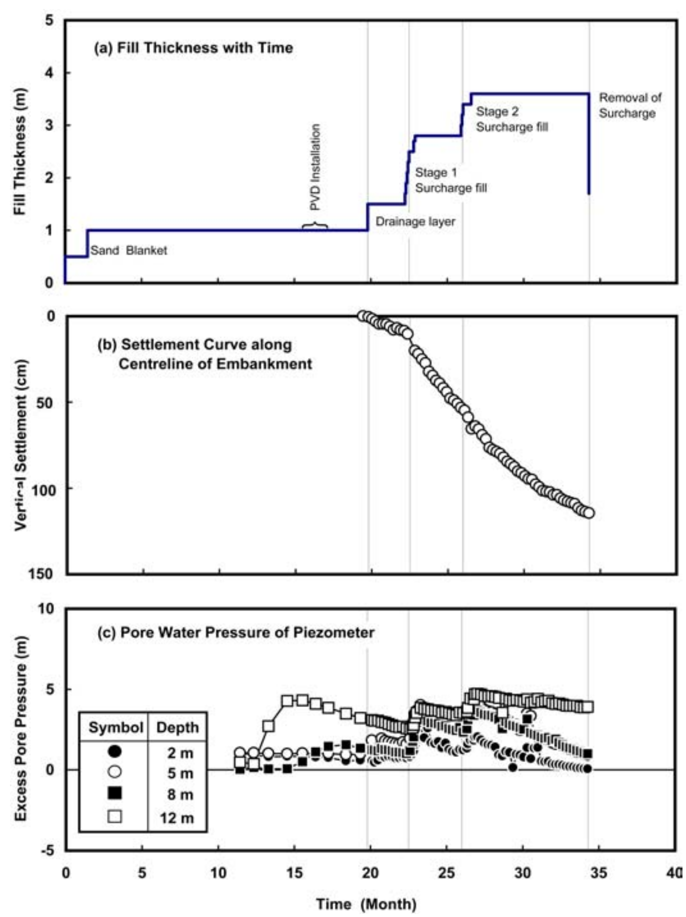

FIG. 12—*Loading condition and monitoring data.*

zone and well resistance:

$$F = F(n) + F_s + F_r \tag{15}$$

where

$$F(n) = \ln \frac{d_e}{d_w} - 0.75 = 2.09\,(d_e = 1.13,\ S = 1.13\,\text{m}$$

$$\text{and}\ d_w = 0.066\,\text{m})$$

$$F_s = \left[\frac{k_h}{k_s} - 1\right] \ln \frac{d_s}{d_w} = 0.28$$

($\frac{k_h}{k_s} \approx 1.4$ from consolidation test at final vertical stress, and $\frac{d_s}{d_w} = 2$ based on recommendation by Hansbo (1979))

$$F_r = \pi z\,(L - z)\,\frac{k_h}{q_w} = 0.001 \quad (k_h = 0.0145\text{m/year}, q_w$$

$$= 1000\,\text{m}^3/\text{year})\ \text{Giving},\ F = 2.09 + 0.28 + 0.001 = 2.37$$

It is assumed that the coefficient of permeability of the disturbed clay is similar to the vertical coefficient of permeability, giving rise to a low F_s value. For the third component, since the discharge capacity of the PVD is over 1000 m^3/year at a hydraulic gradient of 1, the F_r value is therefore negligible. In summary, the F value is dominated by the first component; the other two components make a small contribution to the computed horizontal coefficient of consolidation.

A total of nine sets of settlement data along the runway embankment was used in the back analysis by the Asaoka method; the computed c_h values fall within a small range of 0.70–0.83 m^2/year as tabulated in Table 2. An example of the method is shown in Fig. 13, showing the method of estimating c_h value and total settlement. The degrees of consolidation based on the settlement data and back analysis are also presented in Table 2, having an average value of 84 %.

By considering the factors affecting piezometric pressure, such as position of the piezometer with settlement and the recovered reference piezometric pressure profile, the excess pore water pressure is then calculated as shown in Fig. 12*c*. The excess pore water pressure responses with time at various depths for the embankment are plotted as shown in Fig. 12*c*, indicating low remaining excess pore water pressure before surcharge removal; in other words, high degree of consolidation has been achieved. The degree of consolidation computed based on final pore water pressure (shown in Fig. 11) with consideration of pore pressure distribution between

TABLE 2—*Summary of settlement data and results of back analysis.*

Settlement Data No.	Back Analysis by Asaoka Method β_0(m)	β_1	ρ_f(m)*	Measurement ρ_c(m)**	Degree of Consolidation (%)
1	0.20	0.87	1.50	1.26	84
2	0.19	0.86	1.35	1.18	88
3	0.22	0.86	1.57	1.27	81
4	0.19	0.87	1.43	1.15	80
5	0.19	0.86	1.34	1.13	84
6	0.21	0.86	1.48	1.22	83
7	0.22	0.86	1.55	1.24	80
8	0.21	0.85	1.39	1.23	89
9	0.21	0.85	1.40	1.24	88
Average	0.2	0.86	1.45	1.21	84

Note: * ρ_f = final settlement predicted by Asaoka method.
** ρ_c = settlement at end of construction.

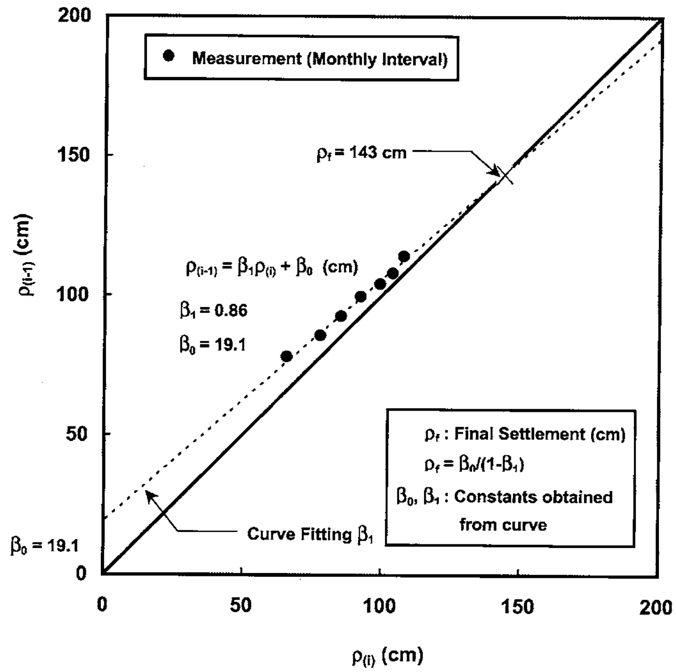

FIG. 13—*Back analysis by Asaoka method.*

PVD and piezometers, gives a value of 87 %, which is close to the estimate of 84 % obtained from the Asaoka method, indicating good agreement between degrees of consolidation computed from settlement data and pore water pressure distribution.

Comparison between Measurements and Back Analysis

The coefficients of consolidation and permeability from various methods are compared, including the results from oedometer, CRS-R, and piezoprobe dissipation tests, as well as back analysis of actual embankment as tabulated in Table 1. It is important to note that the coefficients of consolidation and permeability are greatly influenced by the stress level. Two stress levels have been selected for comparison; they are at effective overburden stress and final effective vertical stress, which have to be consistent with the stress levels of the tests and measurements.

For oedometer and CRS-R tests, the coefficients of consolidation and permeability at the two stress levels are computed and plotted in Figs. 14 and 15. For piezoprobe dissipation tests, the initial stress of the surrounding soil can be assumed to be close to the effective overburden stress; hence, the estimated c_h value has been compared with other values at this stress level. Whereas for the c_h value from back analysis of embankment, the stress level is closer to the final vertical stress; therefore, the estimated c_h value from back analysis is compared to values at that stress state. The results in Fig. 16 indicated relatively good agreement between the c_h values from CRS-R and piezoprobe dissipation tests at effective overburden stress. At the final vertical stress, the c_h values from CRS-R tests agree well with estimated c_h values from Asaoka method at the final stage loading. The results from the CRS-R and oedometer tests also indicated that the soft clay is slightly anisotropic.

By applying the relationship between k_h and c_h given in Eq 14, the horizontal coefficients of permeability of the piezoprobe tests and the back analysis can be estimated. Figure 17 presents the results of the coefficient of permeability profiles at different stress

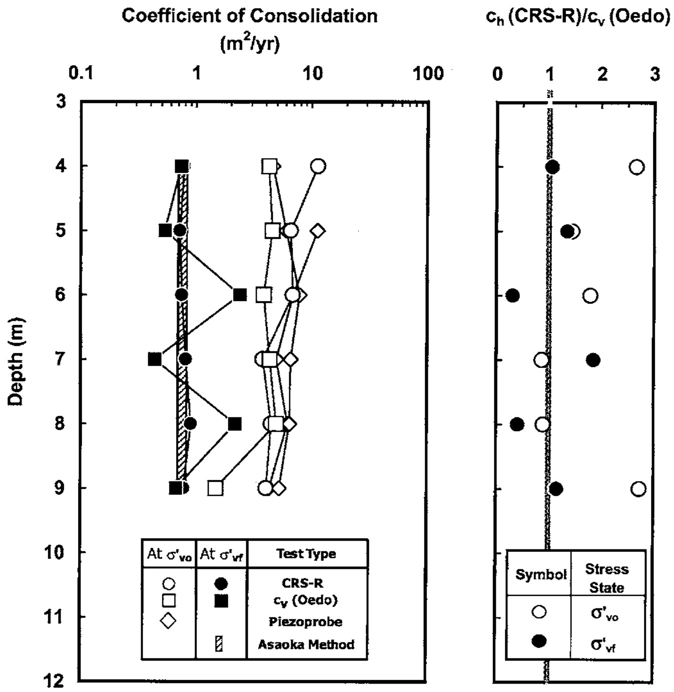

FIG. 14—*Profiles of coefficients of consolidation.*

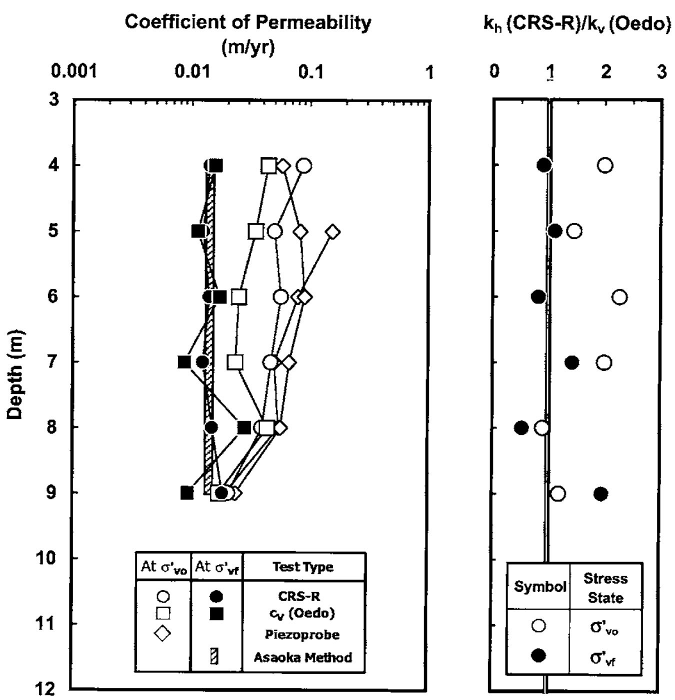

FIG. 15—*Profiles of coefficients of permeability.*

levels from various tests, indicating similar good agreement as in the coefficient of consolidation.

Summary and Conclusions

This study has demonstrated various methods of obtaining the horizontal coefficient of consolidation of soft Bangkok clay, giving relatively similar c_h values. The following conclusions can be drawn:

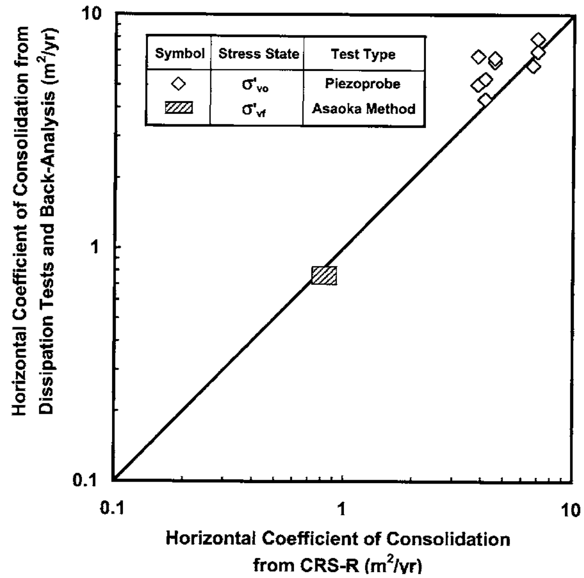

FIG. 16—*Comparison of c_h values among various methods.*

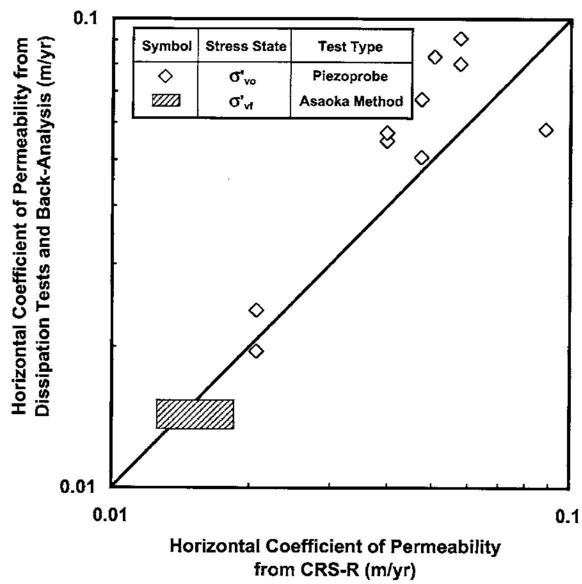

FIG. 17—*Comparison of k_h values among various methods.*

- From the results of consolidation tests, both CRS-R and Oedometer tests, the soft Bangkok clay at this site is found to be slightly anisotropic. The coefficient of consolidation decreases significantly with increasing stress; therefore, it is necessary to compare the c_h values at appropriate stress levels.
- A small piezoprobe of 20 mm diameter can reduce the testing period in the dissipation test significantly. It also gives quicker measurements of piezometric pressure compared with conventional monitoring instruments.
- Eleven piezoprobe dissipation tests were performed at the Second Bangkok International Airport, which gave reasonably consistent and repeatable results at the same depths. The in situ horizontal coefficients of consolidation were computed

based on the Teh and Houlsby (1991) solution. The estimated c_h values range from 4.6 to 11.3 m^2/year at 4–9 m depth.
- The c_h and k_h values from the CRS-R tests compared well with the values from piezoprobe dissipation tests at the effective overburden stress, validating the solution given by Teh and Houlsby (1991). At the final stress level of the embankment, the c_h and k_h values from the CRS-R tests are also compared with the values from back analysis of the constructed embankment by the Asaoka method, indicating very good agreement. The results have also validated the back analysis by the Asaoka method for PVD ground improvement scheme.
- The estimated degree of consolidation from the Asaoka method also agrees well with the degree of consolidation based on piezometric pressure from the piezometers.

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