Compressibility and Flow Parameters from PVD Improved Soft Bangkok Clay

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COMPRESSIBILITY AND FLOW PARAMETERS FROM PVD IMPROVED SOFT BANGKOK CLAY

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ABSTRACT: The utilization of prefabricated vertical drain (PVD) has been increasingly popular in the improvement of soft Bangkok clay. Recently, three full scale test embankments were constructed on PVD improved very soft to soft Bangkok clay at Nong Ngu Hao site. Both vertical and lateral deformations as well as the excess pore pressures were monitored. Different PVD spacings at 1.0 m, 1.2 m and 1.5 m were used corresponding to each test embankment. Based on Asaoka's method, the relationships between the coefficient of horizontal consolidation, c_h, and the discharge capacity, q_w, can be established assuming the ratios of k_h/k_s and d_s/d_w. Thus, the flow and compressibility parameters can be derived. The back-calculated c_h values compared very well with the results of piezocone tests. The q_w values were also confirmed from the results of other investigators.

INTRODUCTION

The consolidation settlement of soft clay subsoil creates numerous problems in foundation engineering. To shorten the consolidation time, prefabricated vertical drains (PVD) are installed together with preloading by surcharge loads. Vertical drains are artificially-created drainage paths which can be installed by one of several methods and which can have a variety of physical characteristics. In this method, pore water squeezed out during consolidation of the clay due to the hydraulic gradients created by preloading, can flow a lot faster in the horizontal direction towards the drain and then flow freely along the drains vertically into the permeable drainage layers. Thus, the installation of the vertical drains in the clay reduces the length of drainage paths and, thereby, reduce the time required to complete the consolidation process.

CONSOLIDATION WITH PVD

Barron (1948) presented the first exhaustive solution to the problem of consolidation of a soil cylinder containing a central sand drain. Barron's theory enable one to solve the problem of consolidation under two conditions, namely: (i) free vertical strain assuming that the vertical surface stress remains constant and the surface displacements are non-uniform during the consolidation process; ii) equal vertical strain assuming that

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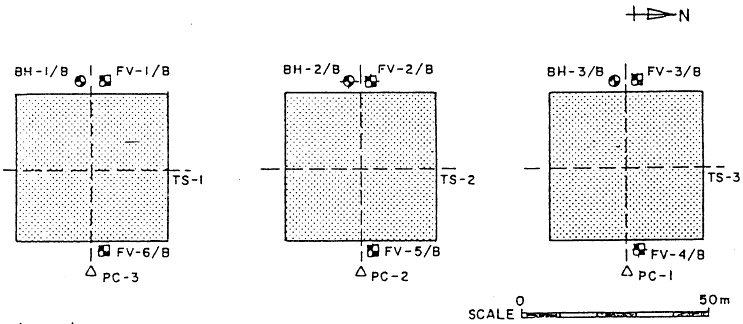
FULL SCALE TEST EMBANKMENT AND PVD IMPROVEMENT

Three full scale test embankments were constructed in stages on PVD improved soft Bangkok clay at Nong Ngu Hao site up to maximum height of 4.2 m with 3:1 side slope. Later on berms were added. The test embankments were square in plan with base dimensions of 40 m by 40 m. The site is located in Samutprakan Province, about 30 km east of Bangkok, Metropolis in Thailand. The site plan of the three test embankments designated as TS1, TS2, and TS3 is shown in Fig. 1. The generalized soil profile and soil properties are shown in Fig. 2. The soil profile is relatively uniform consisting of a thin weathered crust (2 m thick) overlying very soft to soft Bangkok clay approximately 10 m thick. Underlying this soft clay layer is a medium clay layer of about 4 m thickness. A stiff clay layer underlies the medium clay and extends to a depth of 22 m below the ground surface. The profiles of strength and compressibility parameters are shown in Fig.3. Measurements of piezometric drawdown at the site were made and the values are presented in Fig. 4.



Legend :
BH-1/B : Soil Boring Sampling O 3"-15 m, Before Improvement
BH-2/B : Soil Boring Sampling O 10"-11 m, Before Improvement
BH-3/B : Soil Boring Sampling O 3"- 30 m, Before Improvement
FV-1/B, FV-2/B, FV-3/B, FV-4/B, FV-5/B, FV-6/B -- Field Vane Shear Test, Before Improvement, △Piezocone Test Locations

Fig. 1 Locations of Boreholes and Test Embankments.

The PVD's were installed to a depth of 12 m and were cut off so that each PVD protruded 0.25 m above the sand blanket located on top of the ground surface. The sand blanket has thickness of 1.0 m. The PVD spacing were 1.0 m, 1.2 m, and 1.5 m at TS-3, TS-2, and TS-1, respectively. The respective drains were Mebra 7007, Castle Board CS-1, and Flodrain FD4-EX. The mandrel was rectangular in cross-section with thickness of 6 mm and with outside dimensions of 120 mm by 45 mm. The rectangular shaped anchoring shoe has dimensions of 150 mm by 45 mm. The section view of PVD installed at TS-3 is shown in Fig. 5.

the vertical surface stress is non-uniform. For the case of equal vertical strain, the differential equation governing the consolidation process is given as:

$$\frac{\partial u}{\partial t} = c_h \left[\left(\frac{\partial^2 u}{\partial r^2} \right) + \frac{1}{r} \left(\frac{\partial u}{\partial r} \right) \right] \tag{1}$$

where u is the average pore pressure at any point and any given time; r is the radial distance of the considered point form the center of the drained soil cylinder; t is the time after instantaneous increase in the total vertical stress, and c_h is the horizontal coefficient of consolidation. For the case of radial drainage only, the solution of Eq. 1 assuming ideal conditions (no smear and no well-resistance) was given by Barron (1948).

Hansbo (1979) modified the solution developed by Barron (1948) for PVD applications. The modifications dealt mainly with simplifying assumptions due to the physical dimensions, characteristics of prefabricated vertical drains, and the effects of PVD installation. The modified general expression for average degree of horizontal consolidation, U_h, is given as:

$$U_h = 1 - \exp\left(\frac{-8T_h}{F} \right) \tag{2}$$

$$T_h = \frac{c_h t}{De^2} \tag{3}$$

and

$$F = F_n + F_s + F_r \tag{4}$$

where F is the factor which expresses the additive effect due to the PVD spacing, F_n; smear effects, F_s; and well-resistance, F_r. The components of F are defined as follows:

$$F_n = \log_e (D_e/d_w) - 0.75 \tag{5}$$

$$F_s = (k_h/k_s - 1) \log_e (d_s/d_w) \tag{6}$$

$$F_r = \pi z (2L - z) k_h/q_w \tag{7}$$

where D_e is the equivalent diameter of a unit PVD influence zone, d_w is the equivalent diameter of PVD, k_h is the horizontal permeability in the undisturbed soil, k_s is the horizontal permeability of the smeared zone, z is the distance from the drainage end of the drain, L is the length of PVD for one-way drainage and is half of drain length for drainage boundary at both ends of PVD, and q_w is the discharge capacity of the PVD at hydraulic gradient of 1. Neglecting the vertical consolidation, the settlement due to radial consolidation at time t, S_t is given as:

$$S_t = S_{fc} . U_t \tag{8}$$

where S_{fc} is the final consolidation settlement.

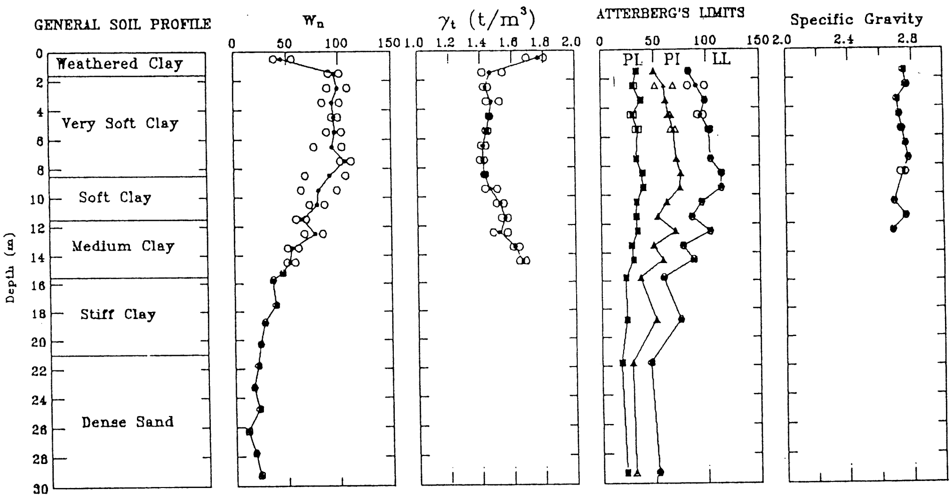


Fig. 2 Generalised Soil Profile and Properties.

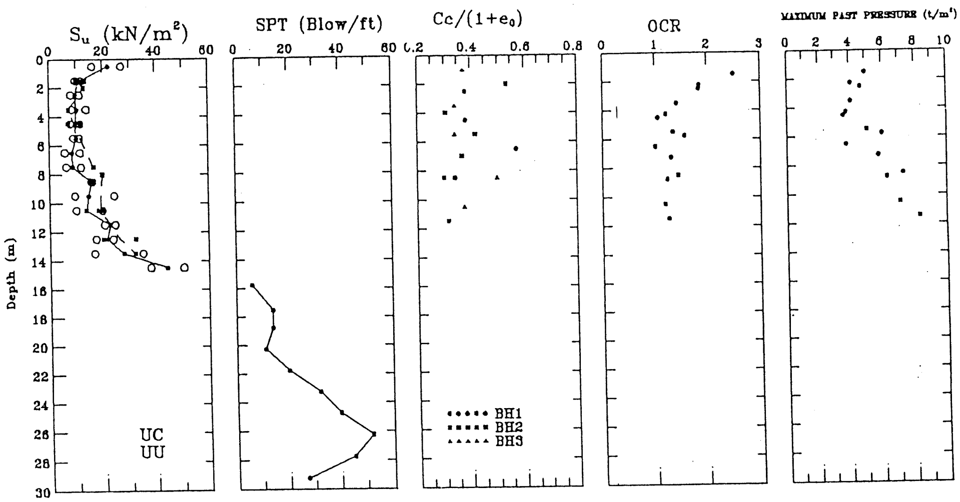


Fig. 3 Strength and Compressibility Parameters with Depth.

FIELD PERFORMANCES OF TEST EMBANKMENTS

Measurements were made on the surface and subsurface settlements, lateral movements, and excess pore pressures in the three test embankments by settlement plates, inclinometers, and piezometers, respectively. A clear trend of settlement magnitudes emerged in Fig. 6 taken from test embankment TS-3, at depth intervals 0-2 m, 0-8 m, 0-12 m and 0-16 m. A comparison of surface settlements of the three test embankments is given in Fig. 7. It is shown that the test embankment TS-3, with the

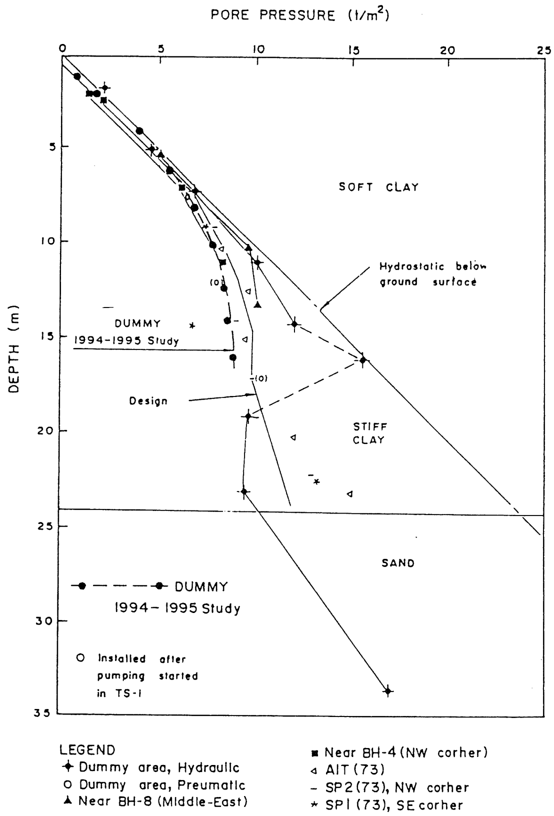


Fig. 4 Variation of Piezometric Pressures with Depth.

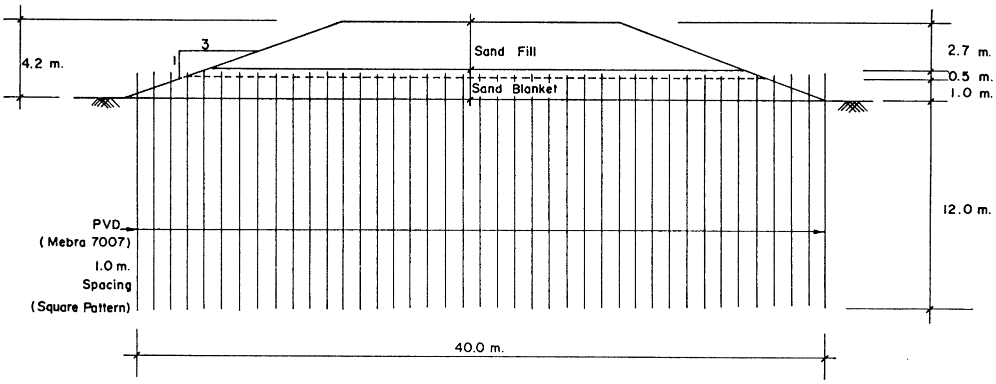


Fig. 5 Test Embankment TS3 (4.2 m. Height).

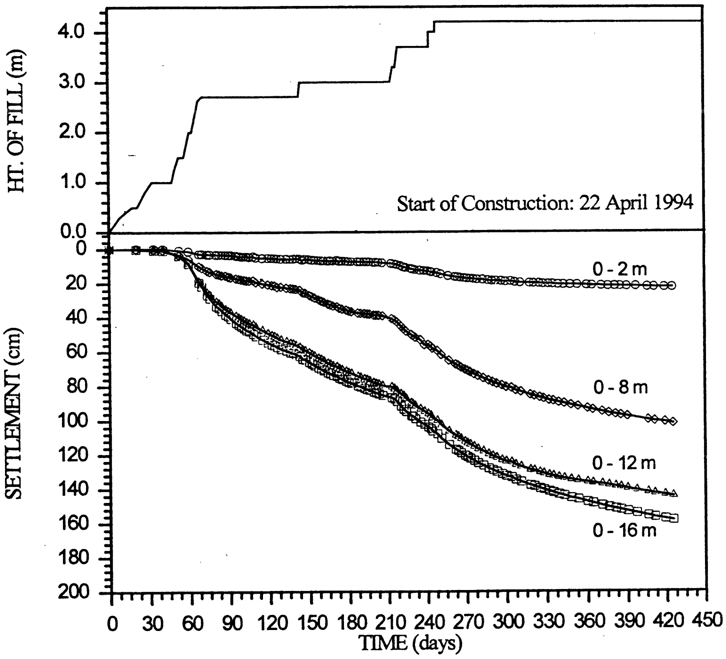


Fig. 6 Settlement of Layers of Increasing Thickness from the Ground Surface.

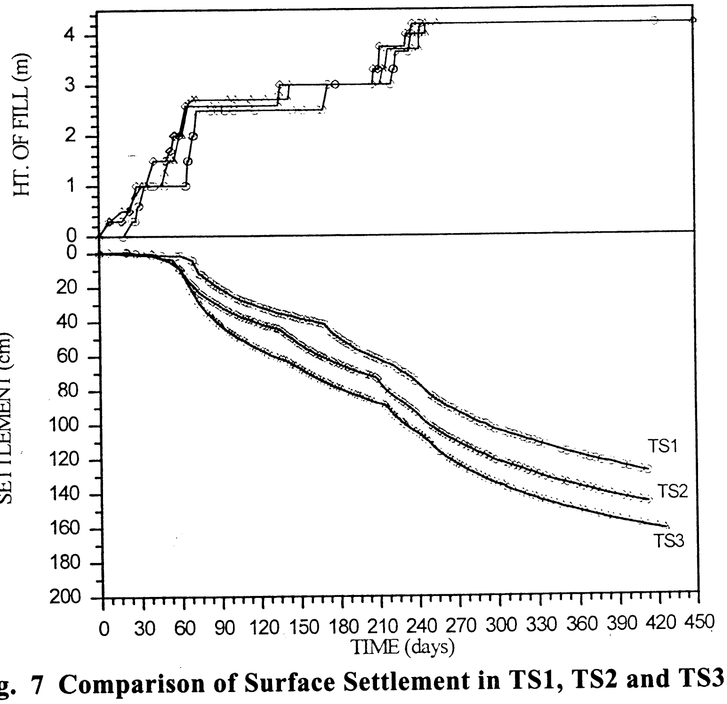


Fig. 7 Comparison of Surface Settlement in TS1, TS2 and TS3.

closest spacing of PVD, indicates the largest settlements. The next larger settlements occurred at test embankment TS-2 with 1.2 m spacing while TS-1 with 1.5 m spacing have the lowest settlement.

Two slope indicators labelled I1 and I2 were installed in each of the three test embankments. The inclinometer I1 was located at the outermost edge of the embankment at a distance of 20 m from the center, while inclinometer I2 was installed at the shoulders of maximum fill height where the side slope of the test fill begins. The results of measurement for inclinometer I2 at TS-2 is shown in Fig. 8.

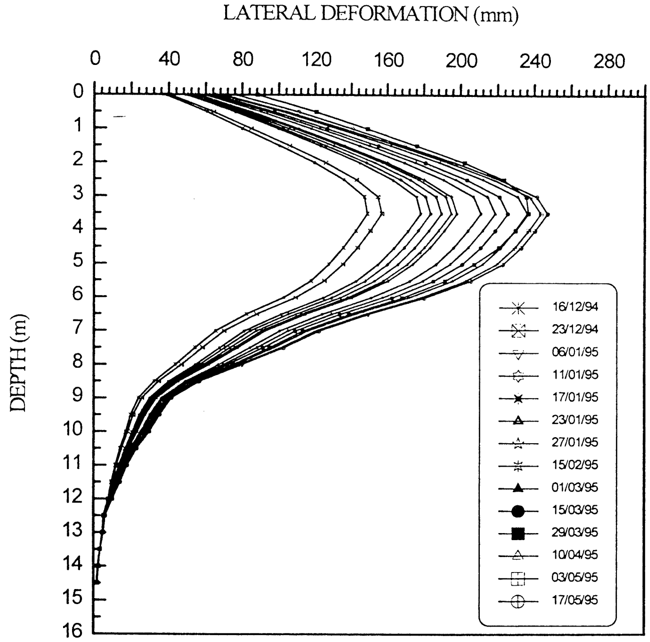


Fig. 8 Profile of Lateral Deformation with Depth for Section TS2-I2.

The total pore pressures where measured redundantly by pneumatic, hydraulic and open standpipe piezometers. All three piezometers indicate similar pattern of responses. The readings were corrected for the respective settlements of the piezometer tips. Figure 9 shows the measurements from hydraulic piezometers at test embankment TS-2.

CALCULATIONS OF COMPRESSIBILITY AND FLOW PARAMETERS

Final Settlement and Coefficient of Compressibility

Asaoka (1978) proposed a graphical method to determine the final settlements based on observational procedures. The observed time-settlement curves plotted to an

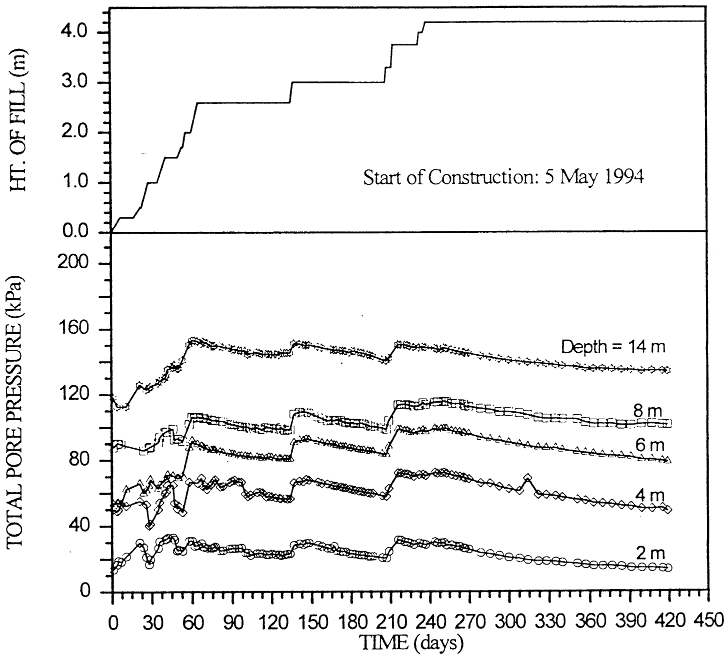


Fig. 9 Pore Pressures from Hydraulic Piezometers Corrected for Settlements (TS2).

arithmetic scale were divided into equal time intervals, Δt. The settlements S_i corresponding to t_i are read off and then the relation of ($S_i \sim S_{i-1}$) is plotted in the coordinate system as shown in Fig. 10 for test embankment TS-3 where Δt is taken as 30 days. A straight line is fitted through the points. The slope of this line is β, and its intercept with the ordinate axis is β_0. The 45^o line with $S_i = S_{i-1}$ is also plotted. The

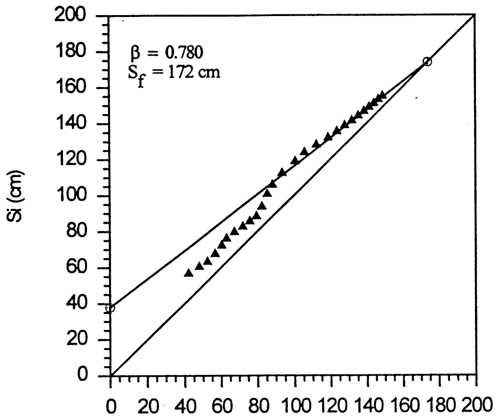


Fig. 10 Settlement Plot for Test Embankment TS3.

point where the plotted line intersect the 45^{o} line yields the final settlement, S_f. Subsequently, the final settlements for test embankments TS-1, TS-2, and TS-3 were obtained as 1.70 m, 1.70 m, and 1.72 m, respectively. The same magnitude of final settlements of three test embankments indicated the uniform soil profile at the test site. Similar conclusions on the uniformity of soft soils in Nong Ngu Hao site were made by NGI (1992)

The compression coefficient, m_v, can be back-calculated from the final settlement as follows:

$$m_v = \frac{\Delta H}{H\,(\Delta\sigma')} \tag{9}$$

where ΔH is the final settlement of the considered soil layer having thickness of H and $\Delta\sigma'$ is the increase of final effective stress. The values of ΔH can be calculated from Asaoka's method and then the corresponding values of m_v for the subsoils under test embankment TS-2 are computed and tabulated in Table 1.

The $c_h \sim q_w$ Relationship

Using Asaoka's approach for radial consolidation with the use of PVD, the horizontal coefficient of consolidation, c_h can be derived as follows:

$$c_h = \frac{(1 - \beta)\, D_c^2 F}{8\beta\Delta t} \tag{10}$$

where the terms have been defined previously. The value of c_h cannot be obtained directly from Eq. 10 because of the unknown value of k_h existing in the factor F as seen in Eqs. 4 and 7.

Assuming the compression coefficient in vertical direction, m_v as equal to that in horizontal direction, c_h, the following expression can be written:

Table 1 Calculated Values of Final Settlement and Coefficient of Compressibility

Soil layers	Weather crust	Soft clay	Very soft to soft clay	Soft to medium clay
Depth (m)	0-2	2-4	4-8	8-12
ΔH (m)	0.22	0.24	0.72	0.30
m_v ($10^{-3}m^2$/kN	1.47	1.60	2.40	1.00

$$k_s = m_v c_h \gamma_w \tag{11}$$

Substituting for k_h from Eq. 11, for F using Eqs. 5, 6, and 7 into Eq. 10, the following equation can be derived:

$$c_h = \frac{F_n + F_s}{C_1 - \dfrac{C_2}{q_w}} \tag{12}$$

where :

$$C_1 = \frac{8\beta\Delta t}{(1-\beta)D_e^2} \tag{13}$$

$$C_2 = \pi z\,(2L - z)\, m_v \gamma_w \tag{14}$$

Equation 12 consists of four unknowns: k_h/k_s, d_s/d_w, q_w, and c_h. Hence, the back-calculated values of c_h will be dependent on the assumed values of the other three unknowns. Furthermore, by assuming the diameter of the smeared zone, d_s, as twice as equivalent diameter of the mandrel as suggested by Hansbo (1987) and confirmed for soft Bangkok clay by Bergado et al (1991), the relationship between c_h and q_w can be obtained for different values of the smear ratio, k_h/k_s.

From the measured settlements, the ß values together with final settlements for the soft clay layer at depth interval from 4 m to 8 m under test embankments TS-1, TS-2, and TS-3 were obtained as illustrated in Figs. 11 to 13, respectively, in which the time interval, Δt, of 30 days was used. Using the aforementioned assumptions of d_s and k_h/k_s and the other parameters as tabulated in Table 2, the ($c_h \sim q_w$) relationships for different ratios of k_h/k_s are plotted in Figs. 14 to 16 for the corresponding embankments

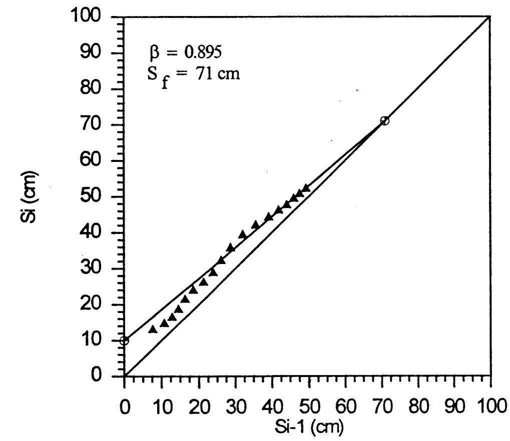


Fig. 11 Settlement Plot for 4-8 m Depth Interval of TS1.

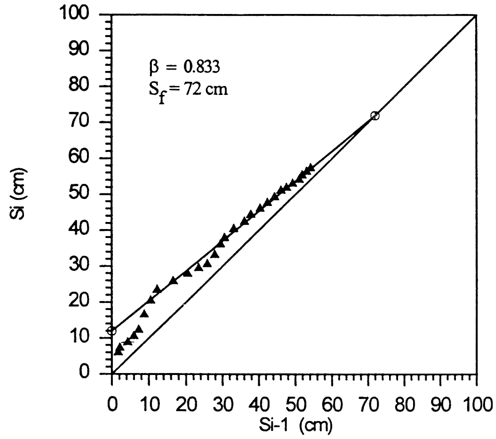


Fig. 12 Settlement Plot for 4-8 m Depth Interval of TS2.

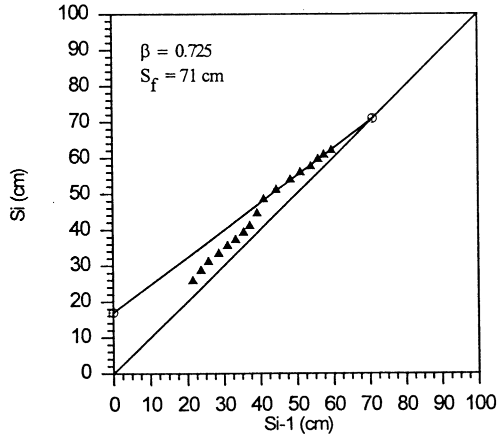


Fig. 13 Settlement Plot for 4-8 m Depth Interval of TS2.

Table 2 Data for PVD Used in Three Test Embankments

1. Dimensions of PVD	a	=	0.004 m
	b	=	0.100 m
	d_w	=	0.052 m
2. Dimensions of Mandrel	a_m	=	0.045 m
	b_m	=	0.150 m
	d_m	=	0.046 m
3. Assumed Diameter of Smear Zone	d_s	=	$2d_m$ = 0.093 m

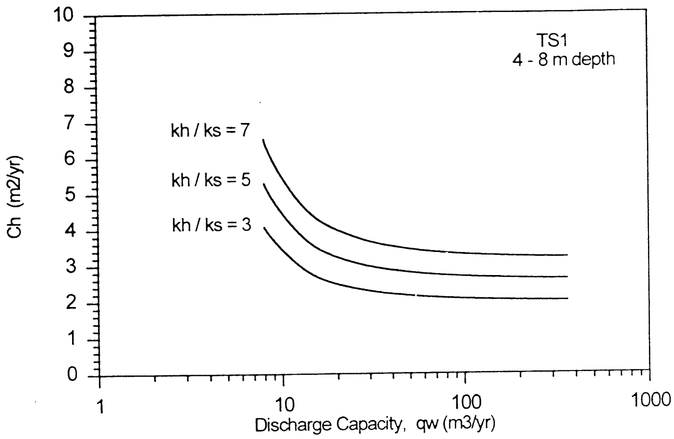


Fig. 14 Back-Calculated c_h-q_w Relations for TS1 Test Embankment.

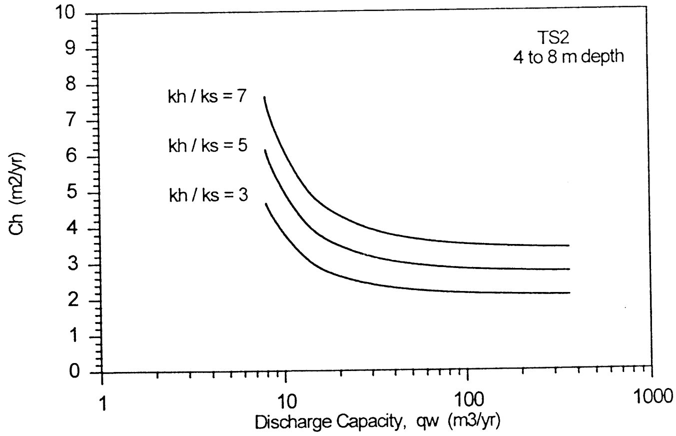


Fig. 15 Back-Calculated c_h-q_w Relations for TS2 Test Embankment.

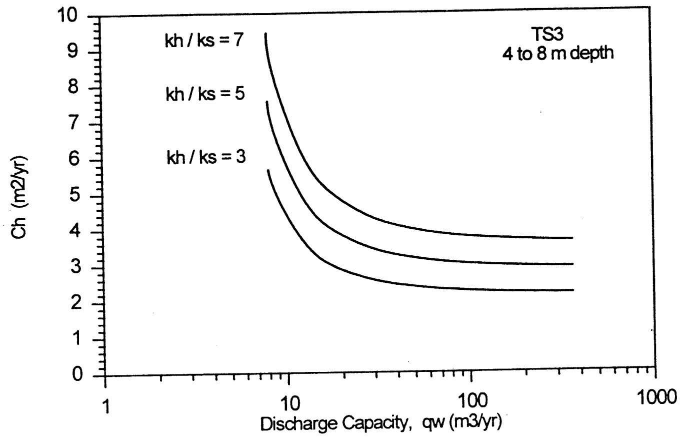


Fig. 16 Back-Calculated c_h-q_w Relation for TS3 Test Embankment.

Table 3 Back-Calculated Results of $c_h \sim q_w$ Relationship

q_w	Embankment TS1 c_h (m^2/yr)			Embankment TS2 c_h (m^2/yr)			Embankment TS3 c_h (m^2/yr)		
(m^2/yr)	$k_h/k_s = 3$	$k_h/k_s = 5$	$k_h/k_s = 7$	$k_h/k_s = 3$	$k_h/k_s = 5$	$k_h/k_s = 7$	$k_h/k_s = 3$	$k_h/k_s = 5$	$k_h/k_s = 7$
4	-66.9	-87.0	-107.1	-16.3	-21.5	-26.7	-8.8	-11.8	-14.7
5	11.2	14.6	18.0	20.4	26.9	33.4	394.4	526.7	659.0
6	6.3	8.2	10.1	8.2	10.8	13.4	12.5	16.7	20.9
7	4.8	6.3	7.7	5.7	7.5	9.4	7.4	9.9	12.4
8	4.1	5.3	6.5	4.7	6.1	7.6	5.7	7.6	9.5
15	2.7	3.5	4.4	2.9	3.8	4.8	3.2	4.3	5.4
30	2.3	3.0	3.7	2.4	3.2	3.9	2.6	3.4	4.3
45	2.2	2.8	3.5	2.3	3.0	3.7	2.4	3.2	4.0
60	2.1	2.8	3.4	2.2	2.9	3.6	2.3	3.1	3.9
90	2.1	2.7	3.3	2.1	2.8	3.5	2.3	3.0	3.8
180	2.0	2.6	3.2	2.1	2.8	3.4	2.2	2.9	3.7
360	2.0	2.6	3.2	2.1	2.7	3.4	2.2	2.9	3.6

TS-1, TS-2, and TS-3, respectively. The calculated results are also tabulated in Table 3.

Based on the physical condition that the coefficient of consolidation, c_h, cannot be negative, the minimum field values of the discharge capacity, q_w, must be greater than 4 m^3/year as seen in Table 3. Moreover, Figures 14 to 16 indicated that the calculated c_h values become little affected by the value of discharge capacity when q_w is greater than 30 m^3/year. Also seen in these figures is the minimum value of c_h cannot be smaller than 2 m^2/year if the ratio k_h/k_s is greater or equal to 3.

The back-calculated value of c_h are dependent significantly on the effects of smear. Assuming $k_h/k_s = 5$ and $d_s/d_m = 2$, the ß, F, and the average c_h values for a depth interval of 0 -12 m, have been calculated and tabulated in Table 4 for a certain assumed value of q_w of 30 m^3/year. The corresponding c_h values for the soil profile are given in Table 5. These calculated values can be compared directly with the field values measured by piezocone and piezoprobe tests as presented in Table 6 and Fig. 17. There is an excellent agreement between back-calculated and field values of c_h .

Table 4 Calculated Values of ß, F, and c_h for Depth Interval of 0-12 m

Embankment	ß	F	c_h (m^2/year)
TS1	0.865	6.24	4.2
TS2	0.800	5.98	4.1
TS3	0.725	5.77	4.2

Table 5 Back-Calculated Values of c_h for Subsoils Under TS2 Embankment

Subsoil Layer	Weathered crust	Soft clay	Very soft to soft clay	Soft to medium clay
Depth (m)	0-2	2-4	4-8	8-12
c_h (m^2/year)	4.0	3.1	3.0	7.8

Table 6 Values of c_h from Piezocone Tests (m^2/year)

Depth (m)	PC-1	PC-2	PC-3
4	3.3	4.4	3.8
8	4.3	4.7	4.2
12	8.8	7.9	7.1

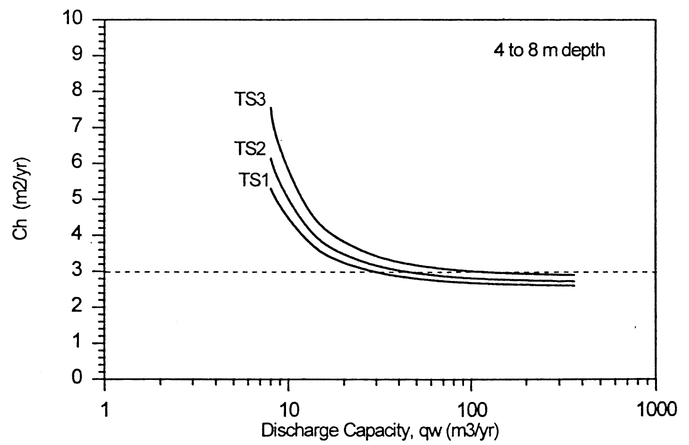


Fig. 17 Comparison of c_h-q_w Relations of Three Test Embankments TS1, TS2 and TS3 Using the Same Value of Smear Ratio, kh/ks = 5.

The final settlements for the very soft clay layer from 4 to 8 m depth were obtained to be 0.71 m, 0.72 m, and 0.71 m for TS-1, TS-2 and TS-3, respectively, as demonstrated in Figs. 11, 12, and 13. These values again confirmed the uniformity of the soil profiles under these test embankment. Thus, the same values of c_h can be assumed for all three sites. If the $k_h/k_s = 5$ is assumed, and if taking $c_h = 3$ m^2/yr as obtained from the piezoprobe tests by Moh and Woo (1987) for the subsoil at the depth interval of 4 m to 8 m, then the discharge capacities of 30, 45, and 90 m^3/yr can be obtained by Fig. 18 for PVDs at TS-1, TS-2, and TS-3, respectively.

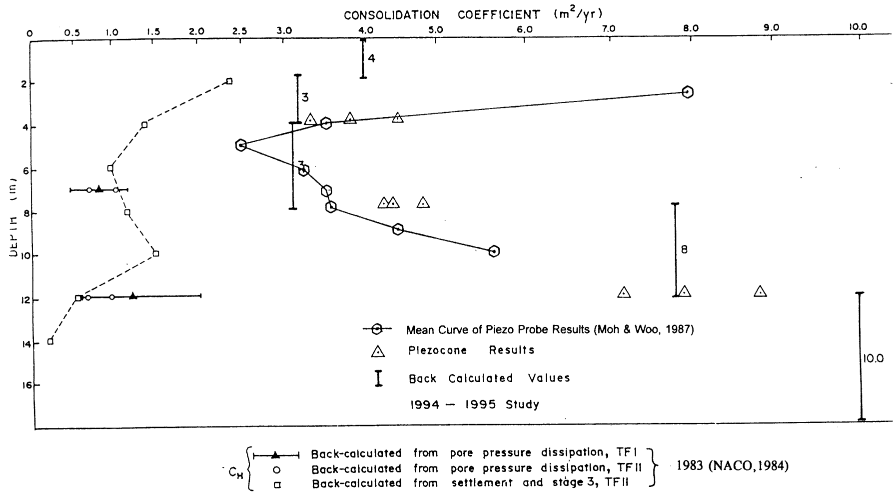


Fig. 18 Comparison of c_h Values Computed from TS2 with the values obtained from 1983 Test Embankment with Sand Drains and the 1994 Piezocone Tests.

COMPARISON OF PVD PERFORMANCE WITH SAND DRAINS IN 1983

In 1983, a field test program was carried out at Nong Ngu Hao site (NACO, 1984). The purpose of the trial test was to determine the effect of using non-displacement type of sand drains for accelerated consolidation of soft clay. The test program included three test areas, one with surcharge fill, and the others with vacuum loading as well as ground water lowering. Difficulties were encountered in maintaining vacuum loading. Thus, only the embankment with surcharge loading was reliable. The pore pressure dissipation in 1983 test fill was lower than the dissipation noted in the present study. As shown in Fig. 18, the c_h values obtained from the current study are much higher than the back-calculated c_h values in 1983 study. As demonstrated in Figs. 14 to 16, it can be seen that the underestimation of smear effects and/or overestimation of the discharge capacity will lead to the underestimation of c_h value. It is thought that the smear effects were ignored in the back-calculated values of the 1983 study.

CURRENT RECOMMENDED VALUES OF DISCHARGE CAPACITY

A wide range of discharge capacity, q_w , values have been specified for proper functioning of vertical drains. Kremer et al (1982) stated that the minimum vertical discharge capacity must be 160 m^3/yr under a hydraulic gradient of 0.625 applied across a 400 mm drain length and subjected to a confining pressure of 100 kPa. Jamiolkowski et al (1983) concluded that based on the laboratory data and actual experiences, for an acceptable quality of drain, q_w , should be at least 10 to 15 m^3/yr at a lateral stress range of 300 to 500 kPa for drains of 20 m long. Holtz et al (1989) recommended the discharge capacity to be 100 to 150 m^3/yr for 15 to 25 m long drains with horizontal permeability, k_h, of 10^{-7} cm/sec. Hansbo (1987) suggested that the proper values of q_w must be 50 to 100 m^3/yr. A summary of the recommended discharge capacities are tabulated in Table 7. In this regard, the back-calculated values of discharge capacity obtained from test embankments TS-1, TS-2 and TS-3 are at the lower bound of values in Table 6. Values of maximum flow rates estimated from field measurements vary from 7.88 m^3/yr (De Jager and Oostveen, 1990) to 52.56 m^3/yr (Lawrence and Koener, 1988). Thus, the discharge capacity values calculated in this study have been confirmed to agree within the range of other investigators. This result is confirmed by the laboratory results shown in Fig. 19 a,b wherein the PVD type

Table 7 Current Recommended Values for Specification of Discharge Capacity

Sources	Values	Lateral stress (kPa)
JAMIOLKOWSKI *et al.* (1983)	10-15	500-300
DEN HOEDT (1981)	95	50-300
KREMER *et al.* (1982)	256	100
KREMER (1983)	790	15
HANSBO (1979)	50-100	Not given
RIXNER *et al.* (1986)	100	Not given
VAN ZANTEN (1986)	790-1580	150-300
HOLTZ *et al.* (1989)	100-150	500-300
LAWRENCE & KOERNER (1988)	150	Not given
KODA *et al.* (1984)	100	50
DE JAGER *et al.* (1990)	315-1580	150-300

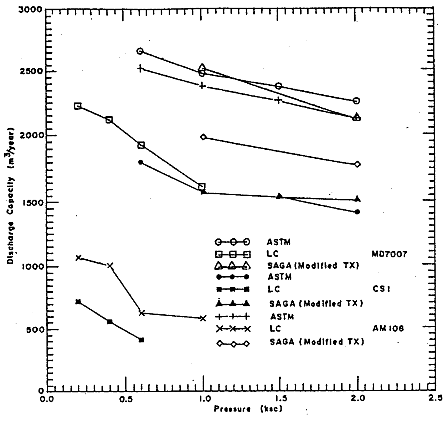


Fig. 19 (a) Discharge Capacity (with Varying Lateral Pressures).

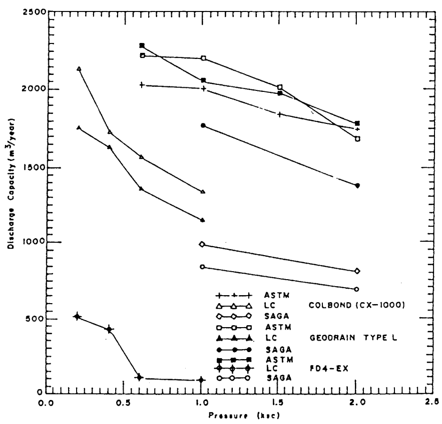


Fig. 19 (b) Discharge Capacity (with Varying Lateral Pressures).

COMPRESSIBILITY AND FLOW PARAMETERS

installed at TS-1 registered the lowest discharge capacity using large consolidometer (LC) in which clay slurry surrounded the PVD specimen. The other results shown in this figure were obtained following the procedures by ASTM and modified triaxial.

Manivannan (1995) has calculated the values of the required discharge capacity, q_{req}, using the approach recommended by Kamon et al (1984) and Pradhan et al (1991). The required discharged capacity values are plotted against the length of PVD with varying c_h values with n values equal to 20, 25, and 30 in Figs. 20 to 22, respectively, where n is the ratio of D_e and d_w. It can be observed that q_{req} decreases as drain spacing (and n) increases. This is attributed to the increase in consolidation time as drain spacing increases. The time (t_{90}) required for 90% degree of consolidation of the soft clay ground have been calculated and plotted against n in Fig. 22. Based from these figures, the specification criterion of discharge capacity can be established. For c_h value of 4 m^2/yr and PVD length of 12 m, the required discharge capacity to be used should be 35 m^3/yr and 22 m^3/yr for n equals 20 and 30, respectively.

CONCLUSIONS

The successful predictions on the behavior of PVD improved soft ground depend very much on the design parameter used. Reliable compressibility and flow parameters can be obtained from back-analyses of full scale load tests. Three full scale test embankments were constructed for preloading on PVD improved soft Bangkok clay. The PVD spacings in each embankment were 1.0, 1.2, and 1.5 m on square pattern. The PVDs were installed down to 12 m below the ground surface. The test embankments were 40 m by 40 m dimensions at the base and were constructed to 4.2 m high. There

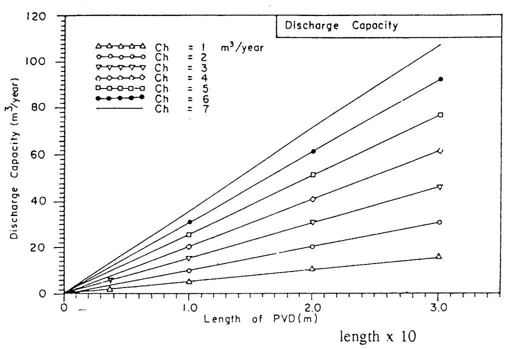


Fig. 20 Discharge Capacity, q_{req} (with Smear Effect for n = 20).

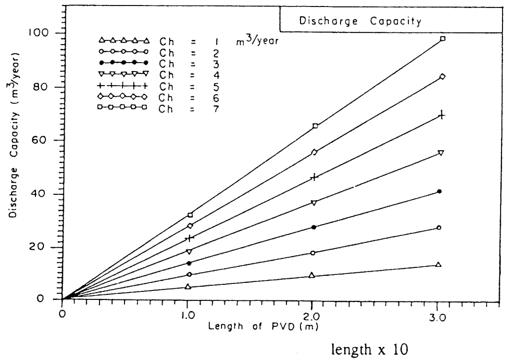


Fig. 21 Discharge Capacity, q_{req} (with Smear Effect for n = 25).

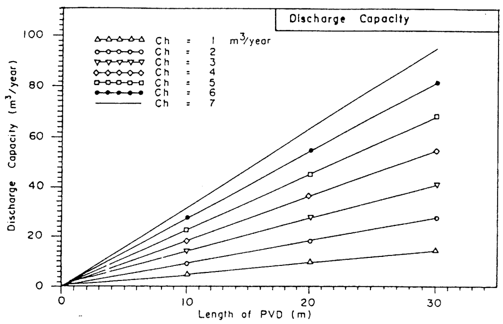


Fig. 22 Discharge Capacity, q_{req} (with Smear Effect for n = 30).

is clear trend of settlement with depth and with spacing of PVD. The observational method by Asaoka was applied for calculating the flow and compressibility parameters. The c_h values as calculated using Asaoka's method considering smear effects were found to agree well with those obtained from field piezocone tests. An average c_h value

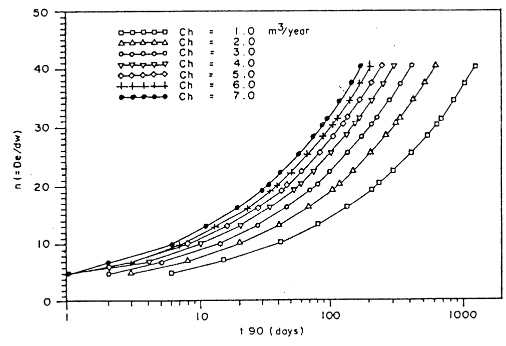


Fig. 23 Values of t_{90} versus n (different c_h Values).

of 4 m^2/year was obtained. The values of discharge capacity, q_w , were calculated corresponding to each test embankment. The required discharge capacity decreased with PVD spacing due to corresponding increase of consolidation time. For a given c_h value, length of PVD and PVD spacing, the discharge capacity was computed. This value agreed within the range obtained by the other investigators.

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