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Vacuum-drain consolidation induced pressure distribution and ground deformation

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ABSTRACT

The method of using a surface or subsurface soil layer as an air-sealing layer to carry out vacuum consolidation is referred to as the vacuum-drain method. A method of determining the vacuum-drain consolidation induced vacuum pressure distribution in the ground has been proposed based on unit cell finite element analysis results. With the calculated vacuum pressure distribution, a method for calculating the settlement–time curve and the lateral displacement profile at the edge of the vacuum consolidation area has been established. Finally, the proposed methods were applied to a project using the vacuum-drain method at Tokyo Bay in Japan. Comparing the calculated results with the measured field values indicates that the methods can be a useful tool for designing vacuum consolidation project.

1. Introduction

Vacuum consolidation is an effective and environmentally friendly soft ground improvement method (e.g. Chai et al., 2006; Saowapakpiboon et al., 2009). To shorten the consolidation period, vacuum pressure is normally combined with prefabricated geosynthetic vertical drains (PVDs). There are two ways to apply vacuum pressure to the ground (Chai et al., 2008). One is to place an air-tighten sheet or membrane on the ground surface and apply a vacuum pressure below it; and another is to use an existing surface or subsurface clayey soil layer where present as a sealing layer and apply vacuum pressure to each PVD with a geosynthetic cap (CPVD) (referred to as the vacuum-drain method here). To design a vacuum consolidation project, the final vacuum pressure distribution and the ground deformation need to be evaluated. In design the vacuum-drain improvement, currently it is assumed that the final (i.e. at 100% consolidation) vacuum pressure distributions in the ground are as shown in Figs. 1b and 2b for one-way and two-way drainage deposits respectively (e.g. Chai et al., 2008; Miyakoshi et al., 2007b). Such distributions are referred as "current" method. However, since the vacuum pressure is applied at the cap location of each CPVD, the vacuum pressure at around the end(s) of a CPVD will propagate in close to spherical radial direction as illustrated in Figs. 1a and 2a (assume a point load in an infinite space). Therefore, with the distribution of Figs. 1b and 2b the final vacuum pressure will be over-estimated. The possible actual vacuum pressure distributions are illustrated in Figs. 1c and 2c, but there is no rational method to determine this kind of vacuum pressure distributions.

For PVDs improved deposit, the consolidation is mainly controlled by radial drainage due to the PVDs, and the degree of consolidation can be evaluated by the consolidation theory for vertical drain (e.g. Barron, 1948; Hansbo, 1981). For vacuum-drain method, except the soil layer with CPVDs, there is a surface sealing layer and a bottom layer (in case of two-way drainage deposit) without the drains. There is a need to investigate what kind of consolidation model is suitable for the vacuum-drain method and especially whether the Terzaghi's one-dimensional (1D) consolidation theory is applicable for the layer(s) without the drains by using the field measured settlement data.

In this paper, the vacuum-drain consolidation induced average vacuum pressure (p_{va}) variation with depth was investigated first by finite element analysis (FEA) using a CPVD unit cell model (a CPVD and its improvement area). Based on the FEA results, a simple method has been proposed to determine the final (i.e. at 100% consolidation) p_{va} distribution in the ground. Then, a method for calculating the vacuum-drain consolidation induced ground

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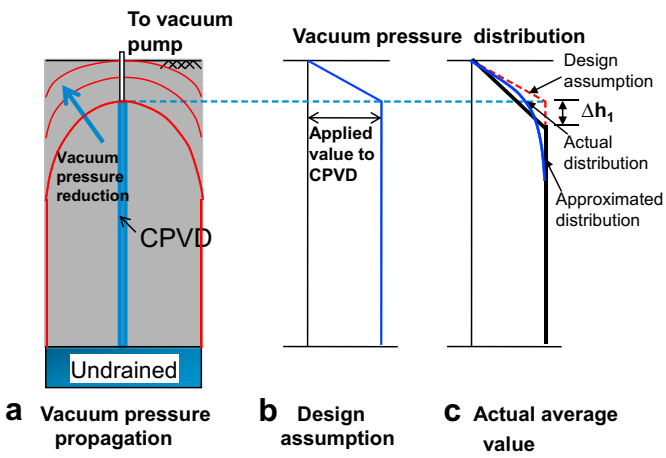

Fig. 1. Illustration of vacuum pressure distribution for one-way drainage case.

deformation is presented. Finally, the method was applied to a vacuum consolidation project with the vacuum-drain method at Tokyo Bay, Japan (Miyakoshi et al., 2007a, 2007b).

2. Vacuum pressure distribution induced by the vacuum-drain consolidation

2.1. Assumed conditions for FEA

A CPVD unit cell model (axisymmetric) is adopted to investigate p_{va} distribution with depth by FEA. The finite element code used is a modified version (M-CRISP) of the original CRISP (Britto and Gunn, 1987). Eight (8) nodes quadrilateral elements with full integration were used to represent the soils and one-dimensional three (3) nodes drainage elements (Chai et al., 1995) were adopted to simulate the effect of CPVD. To ensure that the true stress–strain law is followed closely in the numerical model, the Newton–Raphson method with an explicit sub-stepping technique that includes error control (Sloan, 1987) has been incorporated into the program to integrate the stresses in the elastoplastic range. The analyses conducted were coupled ones. Referring the soft clayey deposit in Saga, Japan, the assumed soil profile is shown in Fig. 3. A 15 m thick clayey layer underlain by a sand layer forms a two-way drainage deposit. It is assumed that the vacuum pressure distribution around the cap location of this model ground can be applied for one-way drainage condition too. The example FEA mesh (unit cell diameter, $D_e = 2.26$ m) and the boundary conditions are given in Fig. 4. The Modified

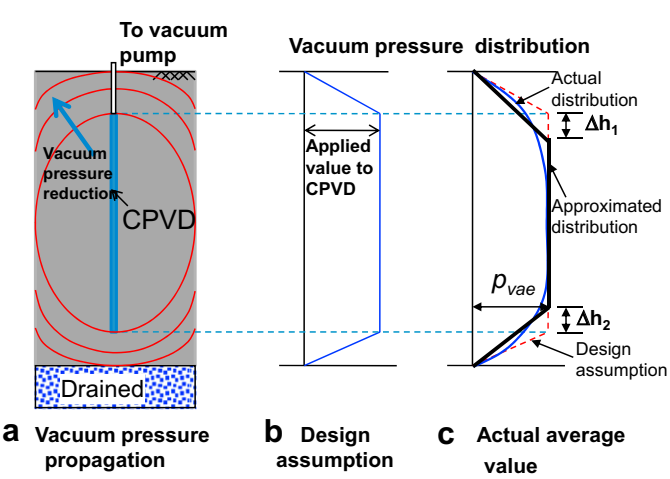

Fig. 2. Illustration of vacuum pressure distribution for two-way drainage case.

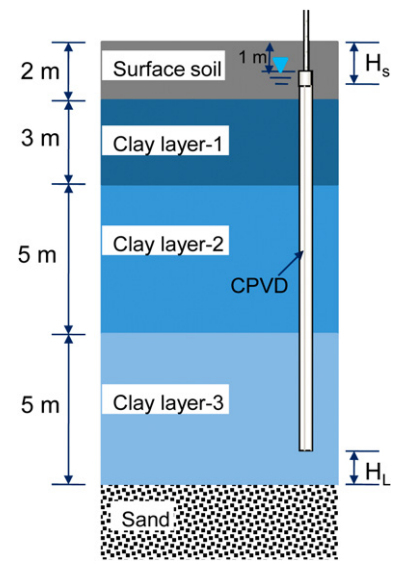

Fig. 3. Assumed soil strata.

Cam Clay model (Roscoe and Burland, 1968) was used to model the behavior of the soft clayey soils and for the sand layer, the linear elastic model was used. The assumed model parameters are listed in Table 1. The deposit was in a lightly over consolidated state with an over-consolidation ratio (OCR) of 2–5 for the surface crust (about 2.0 m thick) and about 1.2 for the rest of the layers. It will be presented later that the final (i.e. at 100% consolidation) p_{va} distribution is mostly influenced by the relative value of hydraulic conductivities (k) and other parameters have no considerable influence. As for k, the values given in the table are initial ones, and during the consolidation process, k was allowed to vary (decrease) with void ratio (e) according to Taylor's equation (Taylor, 1948)

$$k = k_0 10^{(e-e_o)/C_k} \tag{1}$$

in which k_0 is the initial k value corresponding to initial void ratio (e_0), and C_k is a constant and the adopted value was $0.4e_0$. Further,

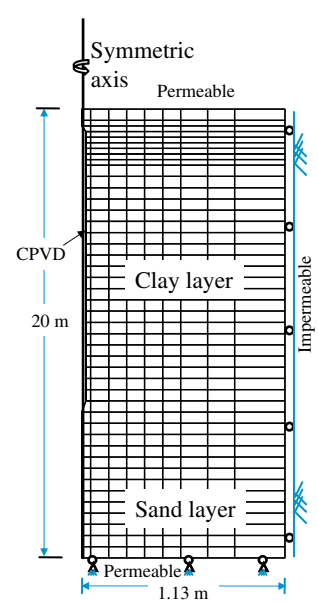

Fig. 4. Example FEA mesh and the boundary conditions.

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Table 1
Assumed model parameters.

Depth (m)	Soil layer	Young's modulus, E (kPa)	Poisson's ratio, ν	κ	λ	M	e_0	e_{cs}	γ_t (kN/m^3)	k_h	k_v
										(10^{-8} m/s)	
0.0–2.0	Surface soil		0.30	0.052	0.52	1.2	3.30	4.31	13.6	3.0	2.0
2.0–4.0	Clay-1		0.30	0.065	0.65	1.2	3.30	5.30	13.6	3.0	2.0
4.0–8.0	Clay-2		0.30	0.065	0.65	1.2	3.10	5.33	13.8	2.1	1.4
8.0–12.0	Clay-3		0.30	0.052	0.52	1.2	2.60	4.84	14.4	1.1	0.7
12.0–15.0	Sand	30,000	0.25	–	–	–	–	–	18.0	290	290

Note: λ, slope of consolidation line in e:ln p' plot (p' is effective mean stress); κ, slope of unloading–reloading line in e–ln p' plot; M, slope of critical state line in p'–q plot (q is deviator stress); e_0, initial void ratio; e_{cs}, void ratio on critical state line at $p' = 1$ kPa in compression plane; γ_t, total unit weight; k_v, and k_h are hydraulic conductivities in vertical and horizontal directions, respectively.

to investigate the effect of k on the distribution of p_{va} with depth, the following two series analyses were conducted.

(1) Analyses with k values of 1/10 and 10 times of the values in Table 1.
(2) Analyses with $k_h/k_v = 1$, 5 and 10.

The parameters related to CPVD consolidation and the geometry variables considered are given in Table 2. Except the values for H_s and H_L, other values were based on the recommendations of Chai and Miura (1999). The applied vacuum pressure at the location of the cap of the CPVD (p_{v0}) was −75 kPa. Although for most cases, primary consolidation was finished in a period of about half a year, all analyses were conducted for a period of 2 years to get a steady vacuum pressure distribution within the unit cell.

2.2. General discussion

The results of FEA are summarized focusing on p_{va} distribution with depth. As illustrated in Figs. 1 and 2, for simplicity, the actual curved distribution of p_{va} is approximated by a bi-linear or a tri-linear shape (thick solid line in Figs. 1c and 2c) under the equal distributed area assumption, and the values of Δh (Δh_1 and Δh_2) can be evaluated from the FEA results.

For a CPVD unit cell model, Δh depends on the relative contribution of cylindrical radial drainage due to the CPVD and vertical drainage of the natural ground. In case of cylindrical radial drainage with a no flow periphery boundary condition, the steady state is a uniform vacuum pressure within the unit cell. However, in case of vertical and spherical radial drainage around two ends of a CPVD, the final state is a steady flow toward the cap of the CPVD where the vacuum pressure is applied. Therefore, for a given deposit, the main factors influencing the value of Δh are (a) the factors affect CPVD consolidation, that include the unit cell diameter (D_e); smear zone parameters, especially the ratio of k_h/k_s; and the ratio of k_h/k_v, and (b) the thickness of the sealing layer, H_s, and the thickness of the soft clayey layer without CPVD at the bottom of the deposit (in case of two-way drainage), H_L (Fig. 3). As for the diameter of smear zone (d_s), it is generally agreed that $d_s = (2–3)d_m$ (where d_m is the equivalent diameter of a mandrel) (Chai and Miura, 1999). In engineering practice, d_m is about 0.1 m, and in this study $d_s = 0.3$ m was adopted. Then conceptually, Δh can be expressed as follows:

$$\Delta h = \Delta h_0 f_1(D_e) f_2(k_h/k_s) f_3(k_h/k_v) \quad (2)$$

where Δh_0 is a constant and is assumed to depend on H_s and H_L, and f_1, f_2 and f_3 are unknown functions. Generally, the larger the D_e and k_h/k_s and the smaller the k_h/k_v, the larger the Δh will be. There is no theoretical base for determining the forms of the functions f_1, f_2 and f_3. Power function is a commonly used form in establishing an empirical relationship and it has been chosen as the form for f_1, f_2 and f_3. The adopted base values of the variables are $D_e = 1.36$ m (square pattern, spacing of 1.2 m); $k_h/k_s = 1.0$; and $k_h/k_v = 1.5$ (a value adopted by Chai and Miura (1999) for Ariake clay deposit in Saga, Japan). Then Δh can further be expressed as follows:

$$\Delta h = \Delta h_0 \left(\frac{D_e}{1.36}\right)^{n1} \left(\frac{k_h}{k_s}\right)^{n2} \left(\frac{k_h/k_v}{1.5}\right)^{n3} \left(D_e \text{ in meters}\right) \quad (3)$$

The values of Δh_0, n_1, n_2 and n_3 will be determined based on the FEA results.

2.3. FEA results

Firstly, using the base values of $D_e = 1.36$, $k_h/k_s = 1.0$, $k_h/k_v = 1.5$ as well as $H_s = 1.0$ m, FEA results give $\Delta h_0 = 1.0$ m. In practice, H_s is normally within 1–2 m (Fujii et al., 2002; Chai et al., 2008), and FEA

Table 2
CPVD parameters.

Parameter	Symbol	Unit	Basic value	Range
Unit cell diameter	D_e	m	1.36	0.9–2.26
Drain diameter	d_w	mm	50.0	–
Smear zone diameter	d_s	m	0.3	–
Discharge capacity	q_w	m^3/day	0.27	–
Hydraulic conductivity ratio	k_h/k_s[a]	–	1	1–10
Sealing layer thickness	H_s	m	1.0	1.0–2.0
Thickness of the layer without CPVD	H_L	m	2.0	1.0–2.0

[a] k_s is hydraulic conductivity of smear zone.

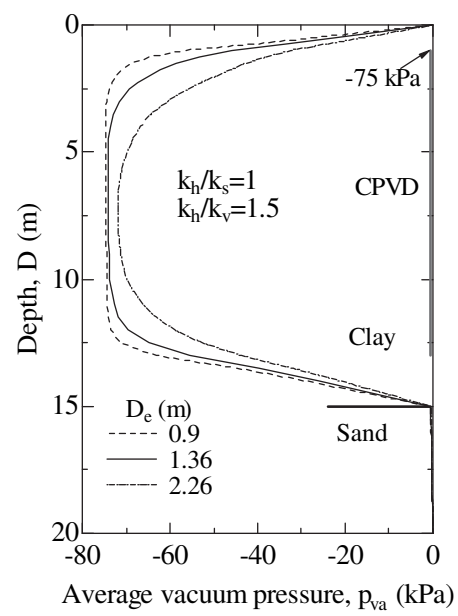

Fig. 5. Variation of p_{va} with depth for different D_e.

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Table 3
Δh values from FEA results.

No.	D_e (m)	k_h/k_s	k_h/k_v	Δh (m)	Remark
1	0.90	1	1.5	0.5	Effect of D_e
2	1.13	1	1.5	0.8	
3	1.36	1	1.5	1.0	
4	1.70	1	1.5	1.6	
5	2.26	1	1.5	2.4	
6	1.36	2	1.5	1.4	Effect of k_h/k_s
7	1.36	5	1.5	2.1	
8	1.36	10	1.5	2.9	
9	1.36	1	1	1.3	Effect of k_h/k_v
10	1.36	1	5	0.4	
11	1.36	1	10	0.3	

results indicate that within this range, the influence of H_s and/or H_L on Δh_0 is not significant and Δh_0 value has been approximated as 1.0 m.

2.3.1. Effect of the unit cell diameter (D_e) and n_1 value

With $k_h/k_s = 1.0$, and $k_h/k_v = 1.5$, the variation of p_{va} with depth for $D_e = 0.9$ m, 1.36 m, and 2.26 m is shown in Fig. 5. With the increase of D_e, the relative importance of cylindrical radial drainage is reduced, and Δh is increased. For $D_e = 2.26$ m, the maximum p_{va} value is obviously less than the applied vacuum pressure (p_{v0}). From the results in Fig. 5 as well as the results for $D_e = 1.13$ m and 1.70 m, the corresponding Δh values were determined manually under the condition that the area between the tri-linear distribution and the vertical axis is the same as between the curved distribution and the vertical axis (Figs. 1c and 2c) and listed in Table 3. The regression analysis results in $n_1 = 1.7$.

2.3.2. Effect of the ratio k_h/k_s and n_2 value

Variations of p_{va} with depth for different k_h/k_s value are depicted in Fig. 6. It shows that increasing k_h/k_s value reduces the cylindrical radial drainage effect and increases Δh value. Also, with the distributions in Fig. 6, the Δh values for each k_h/k_s value were determined (Table 3) and regression analysis results in $n_2 = 0.45$.

2.3.3. Effect of k_h/k_v and n_3 value

The larger the k_h/k_v value, the more effect of cylindrical radial drainage and the smaller Δh value. The variations of p_{va} with depth for different k_h/k_v value are given in Fig. 7. With the determined Δh values (Table 3) from the distributions in Fig. 7, the regression analysis yields $n_3 = -0.65$.

Summarizing the above results, the equation for calculating Δh is proposed as:

$$\Delta h = 1.0\left(\frac{D_e}{1.36}\right)^{1.7}\left(\frac{k_h/k_v}{1.5}\right)^{-0.65}\left(\frac{k_h}{k_s}\right)^{0.45} \quad (D_e \text{ and } \Delta h \text{ in meters}) \tag{4}$$

Eq. (4) has been developed by assuming that each influencing factor is independent, but these factors may influence each other. Additional analyses were conducted to check the validity of Eq. (4). It has been found that for $k_h/k_s \le 5$, and $D_e \le 1.70$ m, for the ranges of the parameters investigated, Eq. (4) can be used for all combinations of the parameters. For $D_e = 2.26$ m, Eq. (4) can be used for $k_h/k_s \le 2$; and for $k_h/k_s = 10$, it can be used for $D_e \le 1.5$ m. Fig. 8 shows two example analyses. As indicated in the figure, from the distribution of p_{va}, the estimated Δh values are 0.9 m and 3.3 m, and the corresponding calculated values from Eq. (4) are 0.94 m and 3.24 m.

2.3.4. Effect of hydraulic conductivity (k) on p_{va}

The variations of p_{va} with depth for different magnitude of k value are shown in Fig. 9. For the range of k value investigated, it

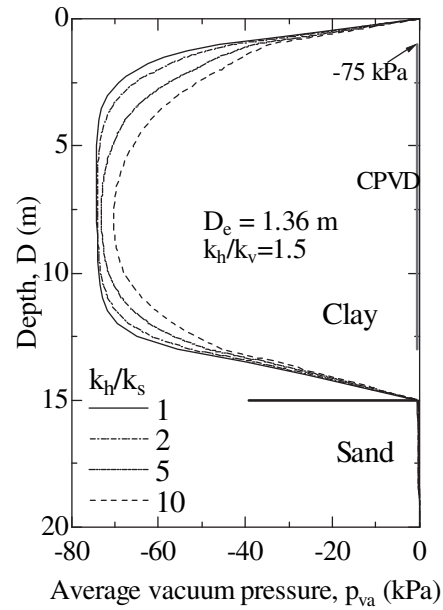

Fig. 6. Variation of p_{va} with depth for different k_h/k_s.

can be seen that the magnitude of k has no significant influence on Δh as well as the value of p_{va} at around the cap location of the CPVD where the vacuum pressure is specified. This is because we checked the final distribution pattern, and it is mostly influenced by the relative k values rather than the absolute ones. However, Fig. 9 indicates that for a two-way drainage deposit p_{va} at close to the bottom end of the CPVD (p_{vae}) is a function of the magnitude of k. With the increase of k, p_{vae} reduced, which indicates that increasing k increases the relative importance of the vertical drainage. To derive an explicit equation for calculating the value of p_{vae}, it is assumed that up to the cap location, the layers with and without CPVDs form a two-layer system. At the end of consolidation, there will be an upward steady flow in this two-layer system. Under one-

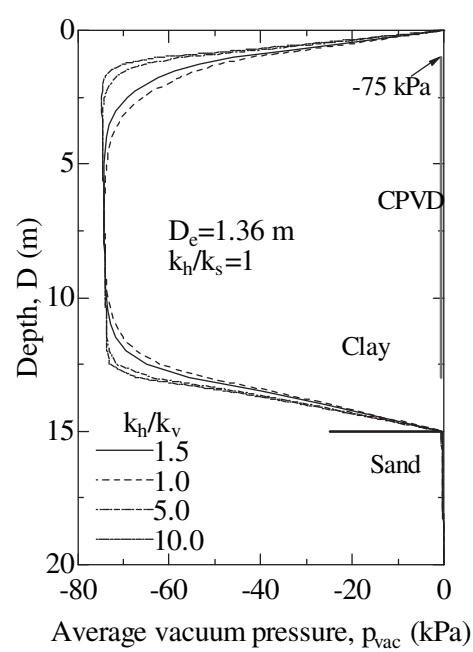

Fig. 7. Variation of p_{va} with depth for different k_h/k_v.

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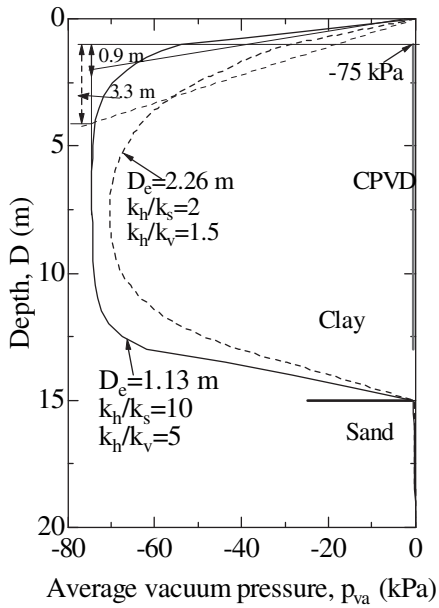

Fig. 8. Two example analyses.

dimensional (1D) assumption, the flow continuity requests the following condition to be held.

$$k_{v1} \cdot i_1 = k_{v2} \cdot i_2 \tag{5}$$

where i_1 and i_2 are the hydraulic gradients, k_{v1} and k_{v2} are the hydraulic conductivities of the layer with and without CPVDs, respectively. From Eq. (5) p_{vae} can be expressed as:

$$p_{vae} = \frac{h_2 k_{v1}}{h_2 k_{v1} + h_1 k_{v2}} p_{v0} \tag{6}$$

where h_1 and h_2 are the thicknesses of the layer with and without CPVDs, respectively, and p_{v0} is the applied vacuum pressure at the cap location. For the layer with CPVDs, an equivalent vertical hydraulic conductivity (k_{v1}) is used and it can be calculated by the method proposed by Chai et al. (2001).

$$k_{v1} = \left(1 + \frac{2.5 l^2}{\mu D_e^2} \frac{k_h}{k_v}\right) k_v \tag{7}$$

where l is the drainage length of CPVDs, and the parameter μ represents the effects of spacing, smear and the well resistance of CPVDs, and it can be expressed as follows (Hansbo, 1981):

$$\mu = \ln\frac{D_e}{d_s} + \frac{k_h}{k_s}\ln\left(\frac{d_s}{d_w}\right) - \frac{3}{4} + \pi\frac{2l^2 k_h}{3q_w} \tag{8}$$

For the unit cell model assumed, with the parameters in Table 1 and the basic values in Table 2, p_{vae} of about 71 kPa (about 95% of p_{v0}) is calculated. If the k values in Table 1 are increased by 10 times, p_{vae} value of about 60 kPa (about 80% of p_{v0}) is the result. These values are very close to the values as marked in Fig. 9 by solid dots resulting from the intersection of the bottom two lines of the tri-linear distribution. Note, the condition adopted to derive Eq. (6) ignores the following factors: (i) the location of the intersection of the lines is not at the level of the end of the CPVDs, and (ii) the flow direction near the end of the CPVDs is away from the 1D assumption.

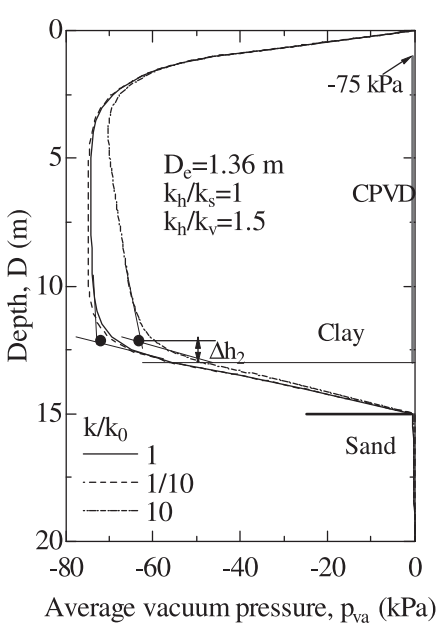

Fig. 9. Effect of the magnitude of k on p_{va}.

3. Method for consolidation and deformation calculation

3.1. Degree of consolidation

As illustrated in Fig. 10, for a deposit with two-way drainage improved using the vacuum-drain method, the sealing layer with a thickness of H_s and the bottom layer without CPVD (H_L) are mainly consolidated due to the vertical drainage, and the middle layer (H_m) with CPVDs is mainly consolidated by cylindrical radial drainage. In case of a soil deposit with one-way drainage, there is no H_L layer. It is proposed that the vertical drainage can be calculated by Terzaghi's one-dimensional (1D) consolidation theory under one-way drainage condition and the radial drainage due to CPVD can be evaluated by Hansbo's (1981) solution. Regarding the drainage condition of H_s and H_L layers, if we consider the levels at the ends of CPVDs as drainage boundaries with fixed excess pore water pressures, H_s and H_L will be two-way drainage layers with water flow toward one boundary as illustrated in Fig. 10. For this kind of situation, even the drainage length is the thickness of the layer, the degree of consolidation can be calculated by 1D consolidation theory with two-way drainage condition (Conte and Troncone, 2008). However, there are interactions between H_s, H_L and H_m

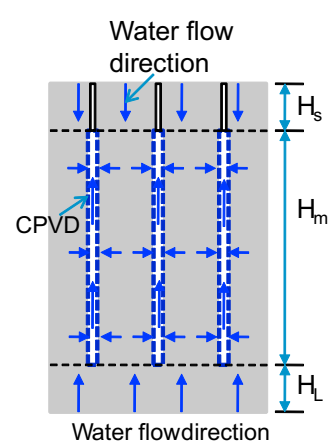

Fig. 10. Drainage condition of vacuum-drain method.

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layers, and the average excess pore water pressures at the levels of the ends of CPVDs are changing with time. In addition, the actual drainage path will be curved and toward to each CPVD and not vertical. Considering these factors, we propose that the degree of the consolidation of H_s and H_L layers can be approximately calculated using 1D consolidation theory under one-way drainage condition.

3.2. Deformation

Based on the laboratory test results and theoretical analysis, Chai et al. (2005) proposed a semi-empirical method to calculate the settlement and the lateral displacement at the end of consolidation of a deposit induced by vacuum pressure. The vacuum pressure induced settlement is calculated as a proportion of the settlement induced by an equivalent surcharge load (i.e. with the same magnitude of applied pressure) as the vacuum pressure under 1D condition. By further assuming that the vacuum pressure induced volumetric strain is the same as that induced by the surcharge load, the horizontal strain in the ground and therefore the lateral deformation of the ground at the edge of the vacuum consolidation area can be calculated. The method has been adopted in this study.

With the degree of consolidation and the settlement at the end of a project known, the settlement curve can be predicted. In both Terzaghi and Hansbo's solutions, the degree of consolidation is defined by the excess pore water pressure. For a clayey deposit, the relationship between settlement and effective stress increment is logarithm. When the degree of consolidation (U) and therefore the effective stress increment ($\Delta\sigma_v'$) are known, the corresponding settlement (S) can be calculated by the following equation.

$$S = S_e \frac{\ln\left[1 + U \cdot \Delta\sigma'_{ve}/(U_e \sigma'_{vo})\right]}{\ln\left(1 + \Delta\sigma'_{ve}/\sigma'_{vo}\right)} \tag{9}$$

where S_e is the settlement, U_e is the degree of consolidation, and $\Delta\sigma_{ve}'$ is the effective stress increment at the end of a project, respectively.

3.3. Calculation procedure

The procedure for calculating ground deformation as a result of the vacuum-drain consolidation can be summarized as follows:

(1) Determine the final (i.e. at 100% of consolidation) vacuum pressure distribution in the deposit with bi-linear or tri-linear pattern using Eq. (4) for Δh and Eq. (6) for p_{ave}.
(2) For a given time (t) series, calculate corresponding degrees of consolidation ($U(t)$) for each layer by Terzaghi's 1D consolidation theory for the sealing layer and the clayey layer without CPVDs at the bottom and Hansbo's solution for vertical drain consolidation for the layer with CPVDs.
(3) Determine the actual vacuum pressure distribution at the end of a project using the corresponding degree of consolidation (U_e) and the final vacuum pressure distribution from (1).
(4) Calculate the settlement (S_e) and lateral displacement profile at the edge of the vacuum consolidation area corresponding to the end of the project by Eqs. (4)–(7) and (10)–(15) of Chai et al. (2005).
(5) Calculate the settlements ($S(t)$) for the given time (t) series by Eq. (9).

The vacuum-drain method is suitable for the situations (1) combining the vacuum pressure and embankment load and (2) carrying out vacuum consolidation for a deposit below water level (Chai et al., 2008). The proposed method is only applicable for the later case (vacuum pressure alone) and not applicable for the combining vacuum pressure with embankment load case.

4. Analysis of a case history

4.1. Brief description of the case history

Two test sections were constructed with the vacuum-drain consolidation method in Tokyo Bay in Japan (Miyakoshi et al., 2007a,b). Section-A had an area of 60 m × 60 m and Section-B of 61.2 m × 61.2 m, and the two sections were almost joined together as shown in Fig. 11. The soil profile consists of a reclaimed clayey silt layer of about 12 m in thickness at the top. Below it is a thick clayey deposit of about 29 m in thickness which in turn underlain a sand layer. The majority of the reclamation was carried out between 2003 and 2005 with a rate of about 3.5 m/year. The total unit weight (γ_t), compression index (C_c) and maximum consolidation pressure (p_c) of the deposits before the vacuum consolidation are shown in Fig. 12. It can be seen that the reclaimed layer was in a state of close to normally consolidated but the original clayey deposit was in an under-consolidated state. In the original deposit, the clay (grain size less than 5 μm) content was more than 50% and for the reclaimed layer, the sum of sand and silt contents was more than 50% (Fig. 13).

Takeya et al. (2007) reported that for both the original clayey deposit and the reclaimed layer, the coefficient of consolidation (C_v) was about 0.012 m^2/day. However, from the grain size distribution and pre-consolidation pressure (p_c) information shown in Figs. 12 and 13, it can be argued that the reclaimed layer may have a higher C_v value. In the analysis, it was assumed that C_v value for the reclaimed layer is 0.024 m^2/day, i.e. twice of the value of the original clayey deposit. Considering possible stratification of the soil layers, for all the layers, the coefficient of consolidation in the horizontal direction is 2 times of the corresponding vertical values. CPVDs used at this site had a cross-section of 150 mm × 3 mm. At Section-A, CPVDs had a spacing of 2.0 m and 1.8 m for Section-B with a square pattern. For the both sections, CPVDs were installed to 30 m depth from the ground surface, and the sealing surface

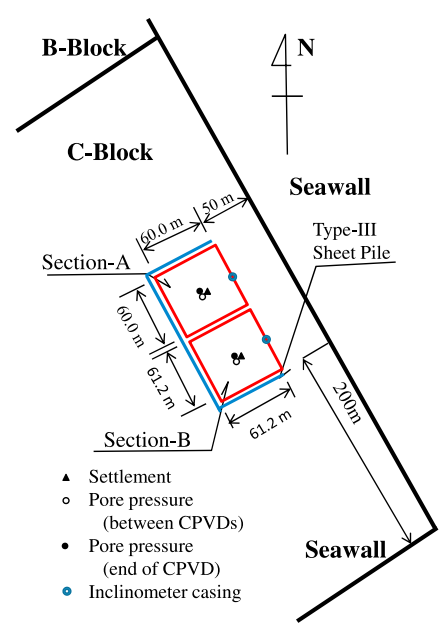

Fig. 11. Plan view of the test sections and the key instrumentation points.

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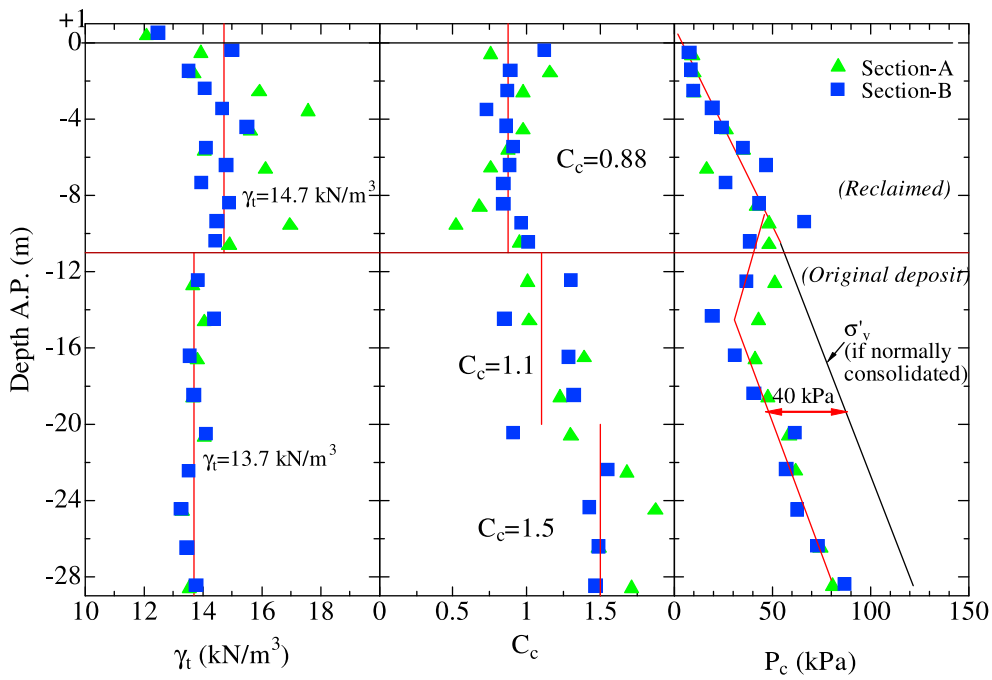

Fig. 12. Properties of γ_t, C_c and P_c of the Tokyo Bay deposit (measured data from Miyakoshi et al., 2007a).

layer had a thickness of about 1.0 m. The field installation of CPVDs started at the beginning of January 2006 for Section-A, and at the beginning of February 2006 for Section-B, and for both the sections, the duration of installation was about one month. The durations between the end of the CPVDs installation and before the application of vacuum pressure were 5 and 4 months for Section-A and -B, respectively. Considering the half of the period of the CPVDs installation as consolidation time, the partial self-weight consolidation periods before vacuum pressure application were about 165 days and 135 days for Section-A and -B, respectively. From June 30, 2006, vacuum pressure of 80–90 kPa at the vacuum pump location was applied and was kept for 204 days. Surface and subsurface settlement gauges, excess pore water pressure (vacuum) gauges, as well as inclinometer casings were installed to monitor the ground response. The key instrumentation points are given in Fig. 11.

4.2. Analysis model and soil parameters

The analysis model and soil parameters for each layer are shown in Fig. 14. The values of the initial void ratio (e_o) shown in Fig. 14 were calculated from the total unit weights (γ_t) by assuming the soil was 100% saturated and the density of the soil particles (γ_s) is 26.5 kN/m^3. The parameters related to CPVD consolidation are: diameter of CPVDs, $d_w = 75$ mm; diameter of smear zone, $d_s = 0.3$ m; discharge capacity, $q_w = 1.37$ m^3/day (500 m^3/year); and the hydraulic conductivity ratio, $k_h/k_s = 2$. The value of q_w was assumed.

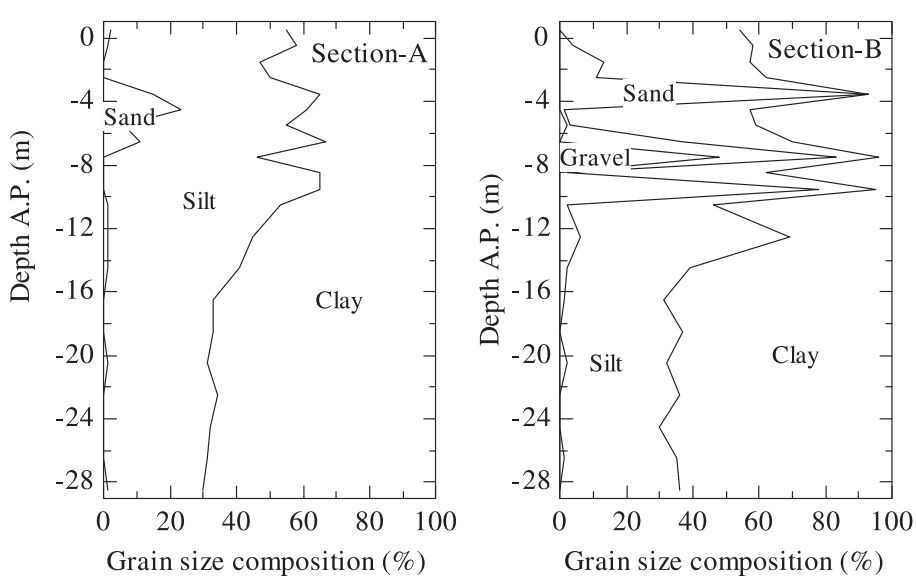

Fig. 13. Composition of the subsoil (data from Miyakoshi et al., 2007a).

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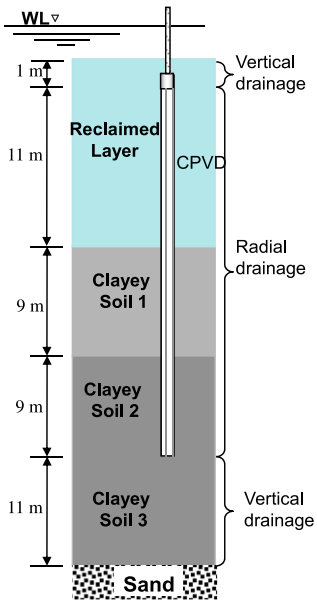

γ_t (kN/m³)	C_c	C_v (m²/day)	e_o
14.7	0.88	0.024	2.41
14.7	0.88	0.024	2.41
13.7	1.1	0.012	3.28
13.7	1.5	0.012	3.28
13.7	1.5	0.012	3.28

For all layers, $C_h = 2C_v$

Fig. 14. Consolidation analysis model.

For the value of k_h/k_s, it was back fitted to result in partial self-weight induced settlement before vacuum pressure application close to the measured value for Section-A.

4.3. Degrees of consolidation

The consolidation can be divided into 2 periods, i.e. Period-1, after CPVDs installation and before vacuum pressure application; and Period-2, during vacuum consolidation. The analyses were also divided into accordingly these two parts.

The CPVDs improved zone contains 3 soil layers (Fig. 14). Hansbo's (1981) solution is for a uniform soil deposit and cannot be directly applied to multilayer condition. To apply Hansbo's solution to a multilayer condition, it is assumed that the degree of consolidation for each layer can be calculated separately using corresponding coefficient of horizontal consolidation (C_h) of each layer, but the drainage length of CPVD was assumed as 29 m. It means that, for example, when calculating the degree of consolidation for the reclaimed layer by Hansbo's solution, it was assumed that there is a layer with the properties of the reclaimed layer and has a thickness of 29 m. Terzaghi's solution can be directly applied to the surface sealing layer (H_s) and the bottom layer without CPVD (H_L) under a one-way drainage condition. The calculated degrees of consolidation are listed in Table 4.

4.4. Settlement and lateral displacement

Based on the field measured data reported by Miyakoshi et al. (2007a), it was assumed that the final vacuum pressure at the cap location of CPVDs (1.0 m below the ground surface) was 70 kPa. From Eqs. (6)–(8), the value of vacuum pressure at the bottom end of CPVDs is calculated to be about 69.3 kPa for both sections. Then with the initial effective vertical stress (σ_{vo}'), which is equal to the p_c value in Fig. 12, and the degrees of consolidation in Table 4, the effective vertical stresses in the ground corresponding to different stages can be easily calculated. Fig. 15 shows the initial, before the vacuum pressure application and the self-weight induced normally consolidated effective vertical stresses distributions of Section-A. The end of the vacuum consolidation and the assumed final

Table 4
Calculated degrees of consolidation.

Soil layer	Thickness (m)	Degree of consolidation, U (%)			
		Part of self-weight induced (before vacuum application)		Vacuum consolidation	
		A	B	A	B
Sealing layer	1	–	–	100	100
Reclaimed	11	84.8	85.6	90.4	94.7
Clayey soil 1	9	68.9	70	76.6	83.9
Clayey soil 2	9	68.9	70	76.6	83.9
Clayey soil 3	11	17.8	15.7	20.3	20.3

(i.e. at 100% consolidation) effective vertical stress distributions are depicted in Fig. 16.

With $D_e = 2.26$ m and 2.03 m for Section-A and -B, respectively, $k_h/k_s = 2$, and $k_h/k_v = 2$, Eq. (4) results in Δh of 2.69 m and 2.25 m for Section-A and -B respectively. Fig. 16 shows that the proposed method results in a smaller effective vertical stress around the ends of the CPVDs than that from the "current" method. It is worth to mention that Eq. (4) has been developed assuming the thickness of the layer above or below the CPVDs is 1–2 m. For the case considered, the thickness of the bottom layer without CPVDs was about 11 m. As a general tendency, the thicker the CPVD unimproved layer, the smaller the Δh value. Therefore, there may be an over-estimation of Δh at the bottom end of the CPVDs. However, for both the sections, deformation analysis indicates that at the bottom of CPVDs, varying Δh from 0 to the value from Eq. (4) has no noticeable difference on the resulting settlement curves due to the higher initial effective stress around that location.

When the vacuum pressure distribution and therefore the effective stress variations in the deposit have been established (i.e. Figs. 15 and 16 for Section-A), the settlements and lateral displacements can be calculated by the method proposed by Chai et al. (2005).

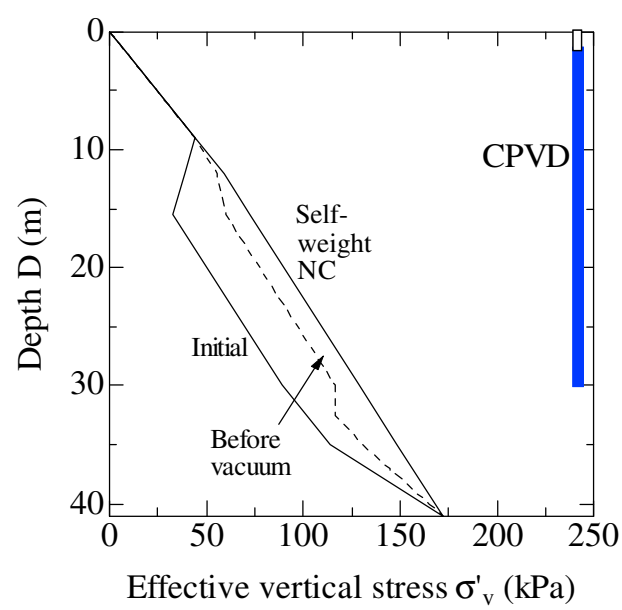

Fig. 15. Effective vertical stress distribution of Section-A (initial and before vacuum pressure application).

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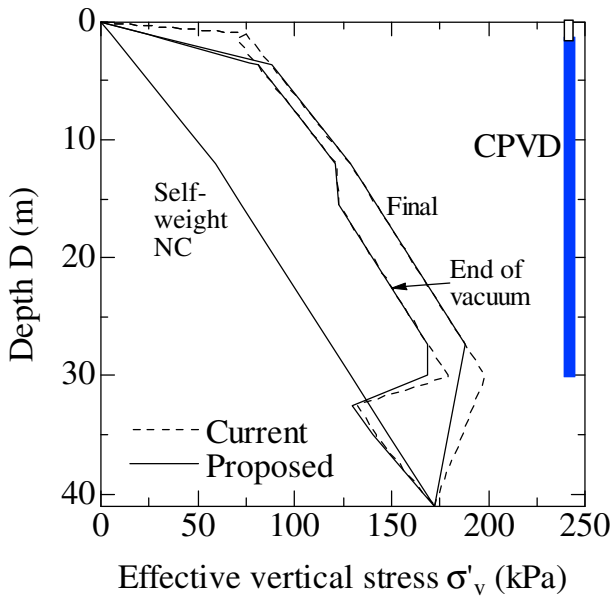

Fig. 16. Effective vertical stress distribution of Section-A (end of consolidation).

Although there was lateral displacement after CPVDs installation and before vacuum pressure application, it was assumed that the consolidation induced by self-weight was 1D and only the settlement was calculated. To calculate the settlement–time curves before vacuum consolidations using Eq. (9), for Section-A, the initial stress (σ_{vo}') is the line marked with "initial", and the stress increment at the end of this period ($\Delta\sigma_{ve}'$) is the difference between the lines of "initial" and "before vacuum" in Fig. 15.

During vacuum consolidation, both the settlement and the lateral displacement were calculated. In this case, for Section-A, the initial stress (σ_{vo}') is the line marked with "before vacuum", and the stress increment at the end of the period ($\Delta\sigma_{ve}'$) is the difference between the lines of "before vacuum" in Fig. 15 and "end of vacuum" in Fig. 16. For deformation calculation, there is a difference in assuming triaxial or plane strain deformation patterns in the ground (Chai et al., 2005). For the case considered, the improved area of each section was a square and close to triaxial state, but since the two sections were almost joined together, and there was likely some plane strain effect. In the analysis, it was assumed that the deformations are average values of triaxial and plane strain assumptions. The additional parameters adopted for calculating the ground deformation induced by vacuum pressure using the method proposed by Chai et al. (2005) are: effective stress internal friction angle, $\varphi' = 30°$, cohesion, $c' = 5$ kPa, a dimensionless constant for determining the earth pressure coefficient under vacuum consolidation area, $\beta = 1.0$, half width of the improved area, $B = 30$ m for Section-A and 30.6 m for Section-B, respectively, and the minimum ratio between the vacuum pressure induced settlement and the same magnitude surcharge load induced settlement, $\alpha_{min} = 0.8$ for triaxial condition and 0.85 for plane strain condition (Chai et al., 2005).

The results of settlement are compared in Fig. 17a and 17b for Section-A and -B respectively. With p_{va} values from the proposed method, the calculated compression of the surface soil layer is obviously smaller than that of the "current" method. This is because that the initial effective stress in the surface layer was small and the difference on the estimated vacuum pressure can induce considerable difference on the calculated compression. Section-A resulted in a larger value of Δh, and larger difference on the calculated settlements. There is no noticeable difference on the calculated compression of the original deposit by the "current" and the proposed methods. Generally, using the p_{va} values from the proposed method, the predicted values are in better agreement with the measured field data.

Comparison of the lateral displacement profiles at the edge of the improved area is depicted in Fig. 18a and 18b for Section-A and -B, respectively. Firstly, it can be seen that the proposed method yields a much better prediction for the values at near the ground surface. Secondly, both the methods predict a smaller lateral displacement for the original deposit which was in an under-consolidated state before the vacuum consolidation. Chai et al. (2008) reported the same tendency for a site in Yamaguchi, Japan, and suggested that α_{min} values proposed by Chai et al. (2005) may be only applicable for a normally consolidated deposit, and for an under-consolidated deposit, a smaller α_{min} value may be used. Further study is needed to establish a guide for selecting a suitable α_{min} value for an under-consolidated deposit.

The above comparisons indicate that with the p_{va} values from the proposed method, an improved prediction on settlement–time curves and lateral displacement profiles induced by the vacuum-drain consolidation can be obtained. The proposed method is simple and the only additional work over the "current" method is calculating Δh by Eq. (4). It is suggested that the method can be used to design ground improvement for soft clayey soils using the vacuum-drain method.

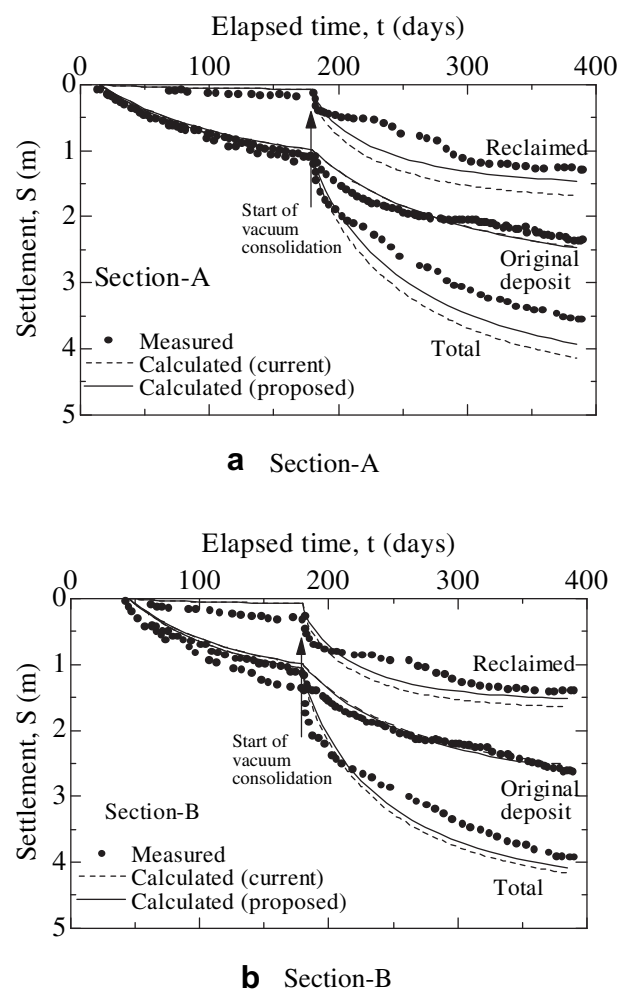

Fig. 17. Settlement versus elapsed time curves (measured data from Miyakoshi et al., 2007a).

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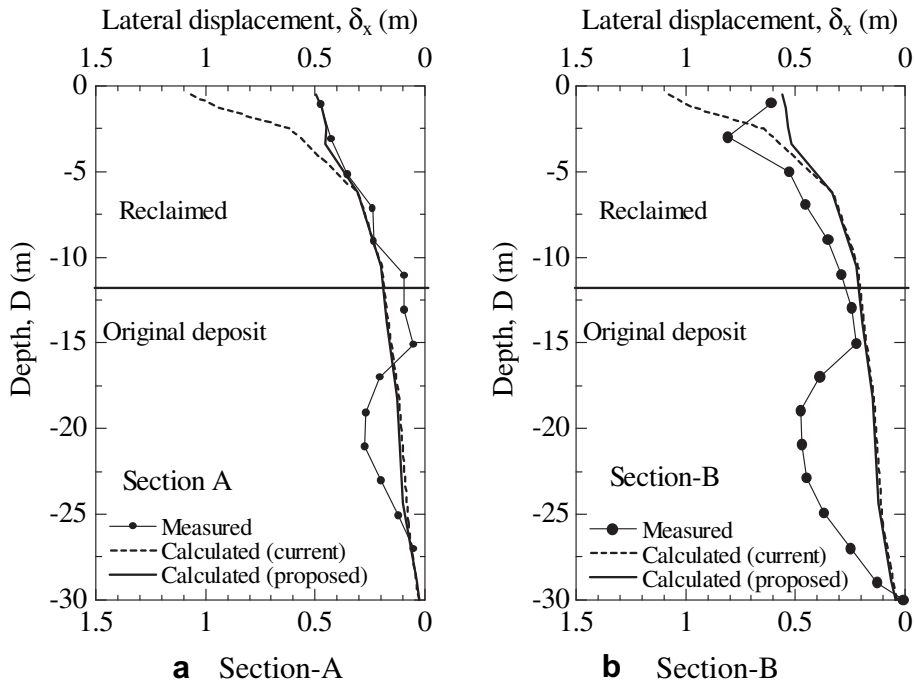

Fig. 18. Comparison of lateral displacement (measured data from Miyakoshi et al., 2007b).

5. Conclusions

For vacuum consolidation, in the case of using a surface or subsurface layer as a sealing layer and applying vacuum pressure to each prefabricated geosynthetic vertical drain (PVD) with a geosynthetic cap (CPVD) (the vacuum-drain method), a method of estimating the final vacuum pressure distribution in the subsoil has been proposed based on the results of finite element analysis using a unit cell model.

A model for consolidation analysis of the vacuum-drain method has been proposed. It is assumed that for the layer with CPVDs, the degree of consolidation can be calculated by Hansbo's solution for vertical drain consolidation. For the sealing layer and the layer at the bottom without CPVDs (if such a layer is present), the degree of consolidation can be calculated by Terzaghi's consolidation theory under one-way drainage condition. It is suggested that the ground deformation induced by vacuum pressure at the end of a project can be computed by the method proposed by Chai et al. (2005). Then an equation for calculating the vacuum-drain consolidation induced settlement–time curves has been derived.

The proposed methods were used to analyze a case history of consolidation treatment applied to an under-consolidated clayey deposit below sea level in Tokyo Bay, Japan, using the vacuum-drain method. The proposed deformation analysis method is capable of simulating the measured settlement–time curves and final lateral displacement profiles at the edge of the improved area. Using the vacuum pressure distribution from the proposed method resulted in an improved prediction of the field behavior. It is suggested that the methods may be used to design consolidation improvement of soft clayey deposits using the vacuum-drain method.

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Nomenclature

B: half width of improved area (L)
C_c: compression index (–)
CPVD: capped prefabricated geosynthetic vertical drain (–)
C_v: coefficient of consolidation in the vertical direction (L^2/T)
D_e: diameter of unit cell (L)
d_s: diameter of smear zone (L)
d_w: diameter of vertical drain (L)
k_s: hydraulic conductivity in the smear zone (L/T)
e: void ratio (–)
e_0: initial void ratio (–)
f_1: a function of D_e (–)
f_2: a function of k_h/k_s (–)
f_3: a function of k_h/k_v (–)
h_1, h_2: thicknesses of layer-1 and layer-2, respectively (L)
H_L: thickness of the bottom layer without drain (L)
H_s: thickness of surface sealing layer (L)
k: hydraulic conductivity (L/T)
k_a: active earth pressure coefficient (–)
k_{a0}: earth pressure coefficient with a value between active (k_a) and at-rest (k_0) earth pressure coefficients (–)
k_0: at-rest earth pressure coefficient (–)
k_h, k_v: hydraulic conductivities in horizontal and vertical directions, respectively (L/T)
L: length
M: mass
T: time
k_s: hydraulic conductivity of smear zone (L/T)
k_{v1}, k_{v2}: vertical hydraulic conductivities of layer-1 and layer-2, respectively (L/T)
k_{ve}: equivalent vertical hydraulic conductivity of PVD-improved subsoil (L/T)
l: drainage length of a CPVD (L)
M: slope of critical state line in p'–q plot (–)
n_1, n_2, n_3: constants (–)
OCR: over-consolidation ratio (–)
p_{va}: average vacuum pressure (ML^{-1}/T^2)
p_{vae}: average vacuum pressure at the end a CPVD (ML^{-1}/T^2)
p_{vo}: vacuum pressure at the cap location of a CPVD (ML^{-1}/T^2)
q_w: discharge capacity of a CPVD (L^3/T)
p': effective mean stress (ML^{-1}/T^2)
q: deviator stress (ML^{-1}/T^2)
S: settlement (L)
S_e: settlement at the end of consolidation (L)
U: degree of consolidation (–)
U_e: degree of consolidation at the end of consolidation (–)
α_{min}: minimum α value (–)
β: a constant (–)
Δh: a thickness for estimating vacuum pressure distribution in a ground (L)
Δh_0: base value of Δh (L)
$\Delta\sigma_{vo}'$: initial effective vertical stress increment from the end of tension crack to a point considered (ML^{-1}/T^2)
γ_t: total unit weight of soil (MT^{-2}/L^2)
λ: virgin compression index in an e–$\ln p'$ plot (p' is effective mean stress) (–)
κ: slope of unloading–reloading line in e–$\ln p'$ plot (–)
μ: a parameter represents the effect of spacing, smear and well resistance of CPVDs
σ_{v0}': initial vertical effective stress (ML^{-1}/T^2)

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