GEOSYNTHETICS FOR SOFT GROUND IMPROVEMENT

-Prefabricated Vertical Drains, Vacuum/Electro-Osmotic/Heat Preloading

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ABSTRACT

Using geosynthetics in soft ground improvement is one of the main applications of geosynthetics. Firstly the recent developments on the methods for determining design parameters, and the methods for consolidation and deformation analyses in soft ground improvement using geosynthetics, especially prefabricated vertical drains (PVDs), are discussed. The current states of using electro-PVD(EPVD) and thermo-PVD(TPVD) in soft ground improvement are also reviewed. There are six (6) papers submitted to this session. The issues addressed by the papers are hybrid techniques for soft ground improvement; using suction induced by siphon to consolidate the soft clayey ground; deformation characteristics of PVD during consolidation process; a case history of an embankment on PVD improved subsoil and a case study of stabilizing a landslide using horizontal geosynthetic drains.

Keywords: Geosynthetics, ground improvement, PVD, consolidation, deformation

INTRODUCTION

The engineering properties of soft clayey ground can be improved by consolidating the deposit and cement deep mixing (CDM), normally forming soil-cement columns in the ground (Bergado et al. 1996; Chu et al., 2000; Indraratna and Chu 2005; Chai and Cater 2011). When the consolidation methods are used, for almost all cases, prefabricated geosynthetic vertical drains (PVDs) will be used, and if the consolidation pressure is a vacuum pressure with air-tight sheet method (Chai et al. 2008), geomembranes will be the air-tight sheet. Numerous projects involving PVD improvement of soft clayey deposits have been reported in the literatures (e.g. Chai and Miura 1999; Shen et al. 2005; Long et al. 2012), and large amount of PVDs has been used in soft ground improvement. For example, in Japan PVDs used were more than 40 million meters in 2000 (Japanese Association of Plastic Board Drain 2009), and in recent years, the amount is about 15 million meters per year; in 2001, Singapore installed about 20 million meters of PVDs (Karunaratne 2011); in 1998, China used more than 70 million meters, and in recent years, the amount is more than 200 million meters per year. In China, some large projects used very large amount of PVDs, e.g. in the reclamation project in Tianjin, more than 8.5billion meters has been used to improve the shore sediment (Wikipedia, 2012) and in Levee Project at the estuary of Yangtze river, more than 13 million meters has been used (Fan and Gao, 2009).

In the first part of this theme report, some recent developments on: (1) determining design parameters related to PVD performance; (2) consolidation and deformation analyses for PVD improved ground and (3) the application of electro-PVDs (EPVDs) and thermo-PVDs (TPVDS)will be briefly described. In the second part, the overview of the papers submitted to the conference is presented.

DESIGN PARAMETERS FOR PVD CONSOLIDATION

Except for geometric parameters, the parameters influence PVD consolidation are (1) discharge capacity of PVD (q_w) and (2) smear zone parameters, i.e. smear zone diameter (d_s) and hydraulic conductivity ratio (k_h/k_s,) where k_h and k_s are hydraulic conductivities of natural soil in the horizontal direction and in the smear zone.

Discharge Capacity, q_w

The discharge capacity of a PVD must be determined experimentally. An ideal discharge capacity test should simulate the drain installation, the confinement of clay on the filter sleeve of the drain, and the deformation of the drain during consolidation. For a useful laboratory discharge capacity test, the important influencing factors, such as confinement condition, must be considered. The test methods of ASTM (D4716-97) specify that a PVD sample can be confined by two stiff platens, foam rubber or clay. Figure 1 shows that confined in clay tests resulted in lowest discharge capacity (data from Karunaratne 2011).

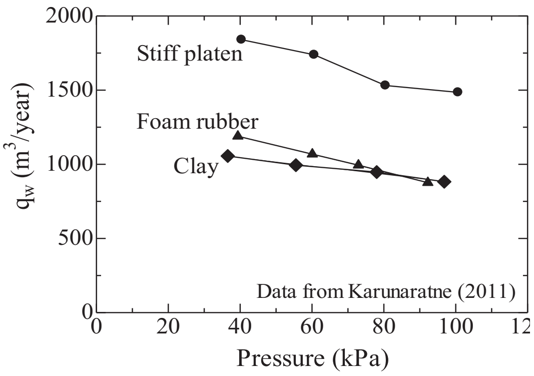


Fig. 1 Effect of confining condition on q_w value

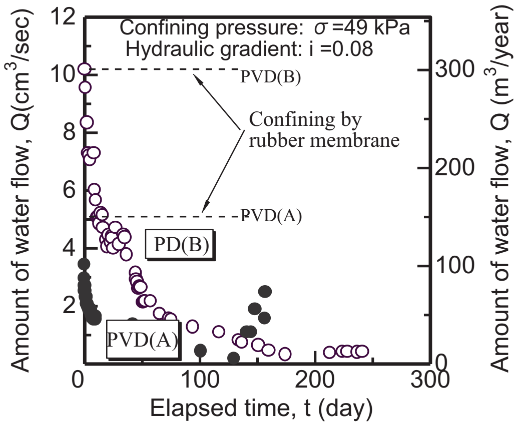


Fig. 2 Variation of water flow rate with elapsed time (Chai and Miura, 1999)

Chai and Miura (1999) conducted confined in clay discharge capacity test using a triaxial type device, and found that the discharge capacity reduced significantly with elapsed time. The results of two long-term tests with PVD(A) and PVD(B) are shown in Fig. 2. The corresponding values of confining the PVDs by rubber membrane are also indicated in the figure for comparison. The test conditions were: the confining pressure of 49 kPa and the hydraulic gradient of 0.08. For both the tests, the discharge capacities continuously reduced with elapsed time, and the lowest value was about 4% of the value of confining the PVDs by rubber membrane, respectively. For a PVD, it is normally expected to work for at least half a year. Therefore, in design, the long-term behavior of PVDs should be taken into account. When linearly converting the data in Fig. 2 to hydraulic gradient of 1.0, the lowest discharge capacity was 75 m^3/yr for PVD(A) and 126 m^3/yr for PVD(B). The factors considered for causing the discharge capacity reduction with time are: (1) the creep deformation of the filter under constant confining pressure, and (2) the clogging effect of fine particles entered the drainage path of the PVDs. For the test conducted on PVD(A), hydraulic shocks were applied after about 130 days, by firmly stepping on the inlet water flow hose, in order to examine the clogging effects. It was observed that some flocculated fine particles were forced out of the drainage channels by these pressure shocks and they were subsequently deposited on the wall of the outlet hose, i.e., some of the PVD clogging was removed by these hydraulic shocks. As shown in Fig. 2, the discharge capacities increased after the shocks.

Chung and Lee (2010) proposed a method for calculating the field mobilized discharge capacity ($q_{w(\mathrm{mob})}$) of PVDs based on the observed settlement rate fitted by a hyperbolic model and assuming a hydraulic gradient (i) in the PVDs of 0.5. For a project at Changi Airport site, Singapore, the back-estimated $q_{w(\mathrm{mob})}$ values are 6 to 18 m^3/year. Generally, i value is a function of consolidation properties of a deposit, loading and PVD installation conditions. For a given settlement rate, reducing i value will result in a higher $q_{w(\mathrm{mob})}$. Referring the available test and analysis results, it is suggested to use long-term confined in clay q_w value in design and if no test data is available, q_w = 100 m^3/year is recommended.

Smear Zone Parameters, d_s and k_h/k_s

Several investigations have been made on these factors (Jamiolkowski and Lancellotta, 1981; Jamiolkowski et al., 1983; Hansbo, 1987; Chai and Miura 1999). Regarding the value of d_s, it can be estimated as:

$$d_s = (2 \text{ to } 3)d_m \tag{1}$$

where d_m = the equivalent diameter of the cross-sectional area of a mandrel. In design, if there is no test data available for evaluating the smear zone size, the value of d_s= $3d_m$ is suggested (Chai and Miura, 1999; Chung and Lee 2010). There are many uncertainties regarding the value of k_h/k_s. Since for most natural deposits, the hydraulic conductivity in the horizontal direction is higher than that in the vertical direction, Hansbo (1987) proposed that k_s can be the same as the hydraulic conductivity of natural soil in the vertical direction (k_v). The value of k_h/k_v can vary from 1 to 15 (Jamiokowski et al. 1983). It is argued by Chai and Miura (1999) that the assumption of k_s = k_v is mainly based on laboratory test results. Laboratory tests may be a correct way for determining the value of k_s, but it generally under-estimates the hydraulic conductivity of a field deposits because of a 20mm thick odometer test soil sample cannot consider the effect of stratification of a natural deposit. It is suggested that k_h/k_s can be expressed as:

$$\frac{k_h}{k_s} = \left(\frac{k_h}{k_s}\right)_l \cdot C_f \tag{2}$$

where subscript *l* represents the value determined in laboratory, and C_f is the hydraulic conductivity ratio between field and laboratory values. It is considered that the most important factor affecting the value of C_f is the deposit stratifications. For a homogeneous deposit, the C_f value can be close to 1.0, but for stratified deposits, even those with thin sand layers and sand seams which cannot be clearly identified from the borehole record, the C_f value can be much larger than 1.0.

SOME NEW DEVELOPMENTS ON CONSOLIDATION AND DEFORMATION ANALYSES

Degree of Consolidation of PVD Partially Improved Deposit

In practice, the degree of consolidation of PVD improved clayey subsoil is calculated by Barron's (1948) or Hansbo's (1981) solutions under equal vertical strain assumption. However, there are cases that PVDs are not installed into the whole soil layer and the followings are some typical examples.

(1) Soft deposit is thick and for economic consideration only part of it is improved with PVDs.
(2) Conducting vacuum consolidation in a two-way drainage deposit, to avoid vacuum leakage through the bottom drainage boundary, PVDs have to be partially penetrated into the deposit (Fig. 3).
(3) Using vacuum-drain method (Chai et al. 2008), a surface soil layer is used as air-sealing layer and there is no PVDs in this layer (① in Fig. 3).

There are solutions for PVD partially improved uniform deposit (e.g. Tang and Onitsuka 1998). However, most natural deposits are not uniform but rather they are inhomogeneous and often layered. Estimating representative consolidation parameters for a layered deposit is a difficult task. For a PVD partially improved layered deposit, it is proposed that for the layers with PVD (②, ③ in Fig.3), their degree of consolidations can be calculated by considering radial drainage only and using Hansbo's (1981) solution, and for layers ① and ④ in Fig.3, only considering vertical drainage.

Ong et al. (2012) developed an empirical method for calculating the average degree of consolidation (U_1) of the layers (①and ④in Fig. 3) without PVD as:

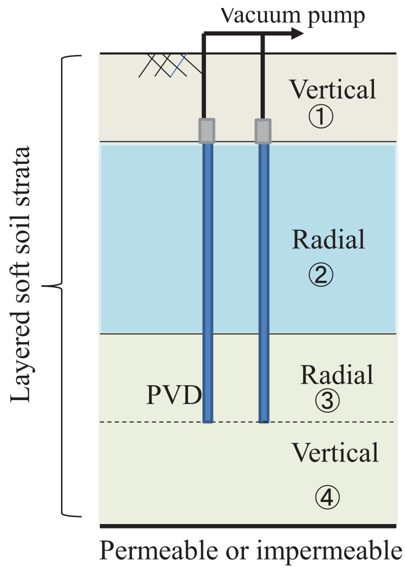


Fig. 3 Consolidation model of PVD partially improved layered deposit

$$U_1 = \alpha_2 U_T \tag{3}$$

where U_T = the degree of consolidation of the layer (without PVDs) calculated by Terzaghi'sone-dimensional(1D) consolidation theory. For layer ① in Fig.3, two-way drainage conditions should be used; and for layer ④in Fig.3, if the bottom is a permeable boundary, two-way drainage; and if the bottom is an impermeable boundary, one-way drainage condition need to be adopted. α_2 is a multiplier, which can be calculated as follows:

$$\alpha_2 = (0.05U_p^2 + 0.48U_p + 0.3)\left(\frac{k_h / k_s}{2}\right)^{0.07}$$

(Two-way drainage) (4)

$$\alpha_2 = (0.33U_p^2 + 0.20U_p + 0.1)\left(\frac{k_h / k_s}{2}\right)^{0.07}\left(\frac{D_0}{D_e}\right)^{0.3}$$

(One-way drainage) (5)

where U_p = the average degree of consolidation of the layer with PVDs, located below the surface layer, or above the bottom without PVD layer,D_e = the equivalent diameter of a single PVD-improved area (i.e., the diameter of a unit cell), and D_o = a constant (=1.5m). Equations (3) to (5) can be used whenever U_1 is smaller than the degree of the consolidation of (U_p). The criteria for judging the applicability of these equations can be found in Ong et al. (2012).

Terzaghi's 1D consolidation theory and Hansbo's (1981) solution are for the case of instantaneous loading. In order to consider the time dependent embankment construction process, Chai and Miura (2002) proposed that the application of the embankment load can be simulated as a stepwise loading. The degree of consolidation for each loading step is calculated as follows.

(i) Suppose at time t_i, the total applied load is p_i, and the degree of consolidation with respect to p_i is U_i. A load increment Δp_j is applied instantaneously at the time, t_i, so that the degree of consolidation (U_j) with respect to the loading $p_j = p_i + \Delta p_j$ at time t_i is:

$$U_j = \frac{U_i \cdot p_i}{p_j} \tag{6}$$

(ii) With U_j known, an imaginary time, t_{j0}, can be obtained from the corresponding consolidation theory.

(iii) Under the loading p_j, at time $t_i + \Delta t$, the degree of consolidation is calculated using a time of $t_{j0} + \Delta t$.

Optimum PVD Installation Depth

For a two-way drainage deposit (a sand layer underlying a soft clayey layer), even under a surcharge load full penetration of PVDs into the deposit may not be an economical choice. Leaving a thin clayey layer with a thickness of H_c near the drainage boundary without PVD improvement can result in almost the same rate of consolidation as a fully penetrated case. Chai et al.(2009) proposed an equation for calculating the value of H_c as follows:

$$H_c = \frac{1}{\sqrt{\dfrac{64}{H^2} + \dfrac{2.5}{\mu \cdot D_e^2}\dfrac{c_h}{c_v}}} \tag{7}$$

where H = total thickness of the layer, c_h and c_v = coefficients of consolidation in the horizontal and vertical directions respectively. μ is calculated as:

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

where d_s = diameter of smear zone, d_w = equivalent diameter of PVD, l = drainage length of a PVD, and q_w = discharge capacity of a PVD.

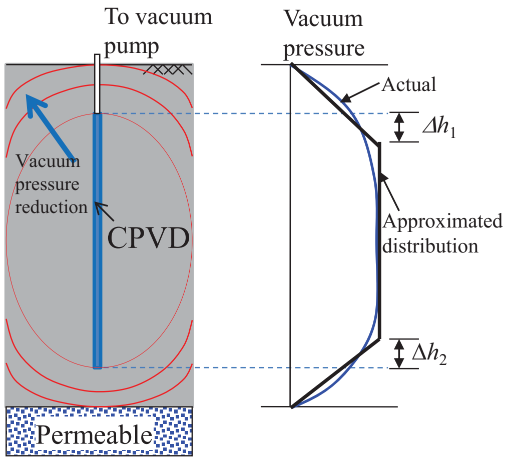
In case of vacuum consolidation in a two-way drainage deposit, a clayey soil layer has to be left near the bottom drainage boundary to avoid vacuum pressure leakage. The efficiency of the vacuum consolidation may be influenced by the thickness of this clayey soil layer without PVDs. In term of resulting in maximum consolidation settlement, there is an optimum PVD penetration depth (H_1), and Chai et al. (2006) derived the following equation for calculating H_1 value:

$$H_1 = \left(\frac{k_{v1} - \sqrt{k_{v1}k_{v2}}}{k_{v1} - k_{v2}}\right) H \tag{9}$$

where k_{v1} and k_{v2} = hydraulic conductivities of PVD-improved and unimproved layers respectively. The method proposed by Chai *et al.* (2001) can be used to calculate k_{v1} value:

$$k_{v1} = \left(1 + \frac{2.5 l^2}{\mu D_e^2}\frac{k_h}{k_v}\right) k_v \tag{10}$$

where k_v=the vertical hydraulic conductivity of the natural soil.



(a) Vacuum pressure propagation (b) Approximated distribution

Fig. 4 Vacuum pressure distributions

Vacuum Pressure Distribution for Vacuum-Drain Method

In vacuum-drain method (Chai et al. 2008), vacuum pressure is applied to the cap location of each capped prefabricated vertical drain (CPVD). At the cap location of each CPVD, the vacuum pressure propagates into the ground in a close to "spherical form" as shown in Fig. 4a. As a result, the average vacuum pressure at the level of cap location is less than the applied value to the cap. Chai et al. (2010) proposed that the final vacuum pressure distribution can be approximated as a tri-linear line given in Fig. 4b in case of a two way drainage deposit. For a one-way drainage deposit, it is a bi-linear line, i.e. without reduction in the layer near the bottom boundary. For simplicity, Δh_1 and Δh_2 (Fig.4b) can be treated as the same (Δh), and the equation for calculating Δh is as follows (Chai et al. 2010):

$$\Delta h = 1.0\left(\frac{D_e}{1.36}\right)^{1.7}\left(\frac{k_h/k_v}{1.5}\right)^{-0.65}\left(\frac{k_h}{k_s}\right)^{0.45}$$

(D_e and Δh in meters) (11)

Deformation Analysis under Vacuum Pressure

Vacuum pressure will induce settlement and inward (toward the center of the consolidated area) lateral ground deformation. Imai (2005) proposed a semi-theoretical elastic method and Chai et al. (2005) proposed a semi-theoretical elasto-plastic method for calculating vacuum consolidation induced ground deformation. Chai et al.'s method is briefly described here.

Vertical strain (ε_{vv})

$$\varepsilon_{vv} = \alpha\frac{\lambda}{1+e}\ln\left(1+\frac{|\Delta\sigma_{vac}|}{\sigma'_{v0}}\right), \tag{12}$$

where e = the voids ratio, λ = the virgin compression index in an e-lnp' plot (where p' is effective mean stress), σ'_{v0}= initial vertical effective stress, $\Delta\sigma_{vac}$= vacuum pressure, and α = a factor with a value less than or equal to unity.

$$\alpha = \alpha_{\min} + \frac{1-\alpha_{\min}}{|\Delta\sigma_{vac}|}\left(\frac{K_0\sigma'_{v0}-\sigma'_{av}}{1-K_0}\right)$$

$$\text{for}\,|\sigma_{vac}| \geq \frac{K_0\sigma'_{v0}-\sigma'_{av}}{1-K_0} \tag{13}$$

where K_0 = at-rest earth pressure coefficient, σ'_{av}= initial vertical effective stress at the bottom of tension cracks, and α_{min} = a constant.It is convenient to denote the values ofα_{min} for triaxial stress condition and plane strain condition asα_{min-T}andα_{min-P}, respectively. Based on laboratory test results, Chai et al. (2005) proposed that α_{min-T}=0.80 and that α_{min-P}=0.85.

Average horizontal strain (ε_h)

With known values of the vertical strain and further assuming that the volumetric strain is the same as the 1D consolidation volumetric strain and designated it as, ε_{vol}, expression for ε_h value of the consolidated area will be:

$$\varepsilon_h = \frac{1}{2}(\varepsilon_{vol}-\varepsilon_{vv})\text{ ,for triaxial stress conditions} \tag{14a}$$

$$\varepsilon_h = (\varepsilon_{vol}-\varepsilon_{vv})\text{ ,for plane strain conditions} \tag{14b}$$

Lateral displacement

Once ε_h is known, the lateral displacement (δ_h) can be approximated quite simply as follows:

$$\delta_h = B\cdot\varepsilon_h \tag{15}$$

where B = the half width of the area treated by vacuum consolidation.

ELECTRO-PVD AND THERMO-PVD

Electric PVD (EPVD)

When an electrical potential is applied to a wet soil mass, cations in pore water are attracted to the cathode and anions to the anode. As the hydrated ions migrate they carry water. Naturally, there are more cations in the pore water of clayey soil (clay particles carry minus charge on their surfaces), and there are net water flow towards cathode. This water flow is called electric-osmosis and its magnitude through a unit area can be expressed by the following equation:

$$q_{he} = -k_e\Delta E/L \tag{16}$$

where q_{he} =electro-osmosis flow rate (L^3/T), k_e= coefficient of electro-osmosis hydraulic conductivity (L^2/T·V), ΔE = voltage difference (V), and L = length (L). Olsen(1969, 1972) reported that for kaolink_e/k_{he} ratio is in a range of 10^2~10^3 (cm H_2O/V). k_e/k_{he} ratio means the hydraulic head difference required to balance that caused by a 1V difference in electrical potential on the opposite sides of a soil layer. Therefore, electro-osmosis is an effective way to consolidate clayey soil as well as remediate heavy metal contaminated ground.

To conduct electro-osmosis in the field, EPVDs have been developed using conductive polymer and carbon fabric. Further the conductivity is enhanced by embedded metal wires or strips (such as copper strips) into the core of PVD (Jones et al. 2006; Karunaratne 2011). Some successful field trials have been reported (Chen and Murdoch 1999; Chew et al. 2004; Jones et al. 2006; Rittirong et al. 2008). Rittirong et al.(2008) reported a case history of treating an organic silty clay deposit by electro-osmosis for 5 days with polarity reversals. The EPVDs were installed to a depth of 6 m into the ground with spacing of 1.0 m by 1.4 m. Applied voltage was about 7 to 30 V, and measured electric currency was about 1 A/m^2, and energy consumption was about 0.7 kWh/m^3. The initial undrained shear strength (s_u) of the soil layers of 2 to 6 m depth from the ground surface was 5 to 13 kPa. After the treatment, s_u values increased to 22 to 39 kPa. The significant increase of s_u values has been attributed to the combined effects of electro-osmosis and electro-chemical reactions.

Thermo-PVD (TPVD)

Heat induced water flow in soil is called thermo-osmosis, and the water flow rate (q_{ht}) can be calculated as:

$$q_{ht} = -D_{TL} \nabla T \rho_w \tag{17}$$

where: D_{TL} = thermal liquid diffusivity (L/T/°K),
T = temperature difference,
ρ_w = density of water (M/L^3).

In a saturated clayey soil, the thermo-osmotic pressure is only few tenths centimeter water head per degree of Celsius (Mitchell 1993). Therefore, thermo-osmosis is not very effective for inducing consolidation of clayey deposit. However, increasing temperature can increase hydraulic conductivity (k) of clayey soil significantly primary due to the reduction of viscosity of water. For example, k value at T = 40°C is about 2 times of the k value at T = 10°C. Abuel-Naga et al. (2006) showed that there was significant thermally induced irreversible contraction of Bangkok clay.

Bergado et al. (2010) reported a field trial of an embankment on TPVD improved Bangkok clay deposit, and its behavior was compared with that of an embankment on conventional PVD improved ground at the same site and had the same geometry. The TPVDs used consisted of conventional PVDs and "U" shaped tubes. The U-tube was connected around edges of the PVD and hot water can be circulated through it from the ground surface (Fig. 5). The both embankments had a base dimension of 11 m by 11 m, top dimension of 3 m by 3 m, and 6 m in height. PVDs and TPVDs were installed to 8 m depth with spacing of 1.0 m (square pattern). For the TPVD improved case, hot water of 70 to 90°C (heated by solar power and/or electricity) was circulated through the TPVDs. For a loading period of about 200 days, the surface settlement of TPVD improved case was about 0.4 m compared with about 0.25 m of the PVD improved case. The results of back-calculation indicate that increasing temperature reduced k_h/k_s value from about 6.2 to 4.1 (temperatures in the smear zone was higher), and increased c_h value from about 6.7 m^2/year to about 8.5 m^2/year.

OVERVIEW OF THE PAPERS SUBMITTED TO THE CONFERENCE

There are six (6) papers submitted to this session as summarized in Table 1.These papers have following characteristics/features.

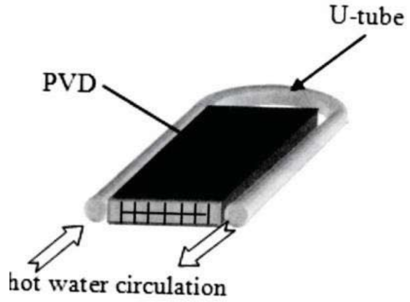


Fig. 5 Configuration of thermo-PVD (Bergado et al. 2010)

(1) Development of hybrid soft ground improvement techniques. The paper by Deng and Zhang (2012) reported the laboratory test results of combining vacuum preloading with electro-osmosis, and corresponding consolidation analysis theory. The results can be a good reference for further developing the technique. The paper by Kumar et al. (2012) reported the laboratory test results of applying vacuum pressure to geosynthetics encased stone column. After vacuum improvement, a stronger and stiffer model ground was formed comparing with without vacuum case.
(2) Innovative method and advanced test technique. Siphon is a well-known phenomenon but there are few researches or applications using the principle of siphon for ground improvement. Tong et al.'s (2012) paper, described laboratory test results of using suction induced by siphon to improve ultra-soft clay soils. The most attractive point of the method is that it does not consume additional energy. How PVDs will deform in the ground during consolidation process, and therefore how that deformation will influence the performance of the PVDs is an important practical issue. The laboratory test results reported by Tagashira et al. (2012) using X-rag CT technique providing useful inside on this aspect. The results may be used to develop more effective new type of PVD.
(3) Case history/study. Although there are numerous case histories reported in the literature on embankment constructed on PVD improved ground, the case history reported by Chen et al.(2012) has a unique point. The embankment construction period was about 1 year and it is unusually long for an embankment on PVD improved subsoil. The field results can be used to evaluate the long-term discharge capacity of PVDs used as well as time dependent behavior of PVD improved soft clayey soil. Sharifipower and Zareie(2012) reported a case study of using geosynthetics horizontal drain to improve the stability of a landslide. It enriched and widened the applications of geosynthetics horizontal drain.

Table 1 Summary of the papers in this session

No.	Title	Authors	Origin	Type of Geosynthetics	Main points
1	Experimental study on the consolidation behavior of ultra-soft clay with horizontal geosynthetics drainage layer by siphon method	Tong et al.	Japan	Horizontal drain	• Laboratory test • Consolidation of soft clay
2	A consolidation model of single drain driven by combining vacuum preloading and electro-osmosis	Deng and Zhang	Austral ia/Chin a	PVD	• Vacuum and electro-osmosis consolidation • Theoretical study
3	Field behavior of PVD-improved soft deposit near Jiujiang, China	Chen et al.	China	PVD	• Case history • Embankment
4	Deformation of vertical drain materials under consolidation settlement using x-ray CT	Tagashira et al	Japan	PVD/paper drain	• Deformation of PVD • Laboratory test, X-ray CT
5	Stabilization of soft clays using geosynthetics encased stone columns with vacuum application	Kumar et al.	India	Reinforce/ drainage geocomposite	• Stone column with vacuum pressure • Laboratory test
6	A case study of investigation the influence of subsurface drainage on slope stability	Sharififipour and Zareie	Iran	Horizontal drain	• Drainage (dewater) • Landslide

SUMMARIES

Using geosynthetics, especially prefabricated vertical drains (PVDs) and geomembranes in soft ground improvement is one of the main applications of geosynthetics. Recent developments in design and analysis of soft ground improvement using geosynthetics and the overview of the papers submitted to this conference under the theme of "geosynthetics for soft ground improvement" are reported in this article.

About the recent development in design and analysis methods, the issues described are: (1) methods for determining parameters related to the performance of PVD improvement; (2) method for calculating the average degree of consolidation of PVD partially improved clayey deposits; (3) optimum PVD installation depth in a two-way drainage deposit; (4) final vacuum pressure distribution in a subsoil when the vacuum consolidation is conducted using vacuum-drain method; and (5) deformation analysis of vacuum consolidation. The current states of consolidating the soft clayey deposit by electro-osmosis using electro-PVD (EPVD), and combined thermal effect and embankment loading using thermo-PVD(TPVD) are also discussed.

There are six (6) papers submitted into this theme. Two of them reported hybrid soft ground improvement techniques using geosynthetics, in which one is combination of vacuum pressure and electro-osmosis, and one is applying vacuum pressure to geosynthetic encased stone column installed in soft ground. One paper reported the laboratory test results of using siphon induced suction to consolidate the ultra-soft clay, and one paper investigates the deformation of PVD and paper drain during consolidation process of soft clay using x-ray CT technique. One paper reported a case history of an embankment on PVD improved soft clayey deposit with a relative long construction period (of about 1 year), and one paper presented the results of a case study of using geosynthetics horizontal drain to stabilize a landslide.

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