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Improving consolidation of dredged slurry by vacuum preloading using PVDs with varying filter pore sizes

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Abstract: Prefabricated vertical drains (PVDs) have been used extensively to accelerate the consolidation rate of dredged slurry. While some fine particles from dredged slurry can easily squeeze through the filter into the drainage channel, many cannot. As such, these soil particles deposit on the filter surface causing partial clogging of the drainage path. Although the pore size of filter is recognized as an important factor that influences PVD clogging, the standards for determining the pore size of the filter are lacking. To this end, the traditional gradient ratio tests with four different filter pore sizes were conducted, and the results show that the permeability of the filter at a given head increases with the increase in the pore size of filter. To remove the effect of the difference between static hydraulic gradient and vacuum pressure, the vacuum preloading tests with varying pore sizes of filters were further conducted. Through these vacuum preloading test, the degree of vacuum, settlement, pore water pressure, water content, vane shear strength and other parameters of PVDs with various filter pore sizes were obtained, and the optimal pore size of filter was determined.

Keyword: gradient ratio; vacuum preloading; dredged slurry; filter; pore size
Introduction

Many coastal regions are experiencing land scarcity owing to rapid growth in population and economic development. Land reclamation has been widely used to overcome land shortages in coastal cities (Sun et al. 2018). The main fill material for land reclamation is mainly consisted of dredged slurry. Unfortunately, dredged slurry is characterized by high water content, high compressibility, and low permeability (Chu et al. 2000; Wang et al. 2016), and hardly possesses any bearing capacity. Therefore, it is imperative to treat dredged slurry to meet the requirements of infrastructure construction (Shen et al. 2012).

Vacuum preloading, which was originally proposed by Kjellman (1952), is an effective and economic method for improving soft soil (Chai et al. 2006). Meanwhile, sand drains are often utilized for drainage in early vacuum preloading projects. Nowadays, prefabricated vertical drains (PVDs) are preferred to shorten the drainage path and accelerate the dissipation of pore water pressure (Cai et al. 2018; Wang et al. 2018b and 2019). Furthermore, PVDs are commonly used for improving the consolidation process of soft soil owing to their quick installation process and low cost (Chen et al. 2012).

In general, PVD consists of a core and a filter sleeve. The particles of dredged slurry are very fine and under vacuum suction, they are forced to enter the drainage path of the PVDs, thereby clogging these PVDs. Therefore, the filter sleeve of PVDs has a significant effect on the consolidation rate of subsoil (Wang and Li 2016). Chai
and Miura (1999) conducted discharge capacity tests on two types of PVDs using a triaxial device and found that the discharge capacity decreased considerably with elapsed time. It was also concluded that the deformation of the PVD filter contributed less than 20% toward reducing the discharge capacity, and more than 80% of the reduction was attributed to filter clogging due to fine soil particles entering the PVD drainage channels. Le et al. (2014) conducted a set of gradient ratio tests and found that a new type of PVD, known as NPVD, has higher permeability compared to that of conventional PVD (referred to herein as CPVD) owing to the use of a hydrophilic filter in the NPVD. Further, Ali et al. (1993) investigated the effects of filter jacket and core geometry on the longitudinal permeability of PVDs. The results obtained show that the discharge capacity was influenced by the flexibility of the filter jacket and core, as well as the geometrical structure of the core. Furthermore, the influence is time dependent. Xu et al. (2014) also investigated the effect of vacuum loading gradient on the drainage capacity of PVD. Their results showed that for a vacuum load with a large loading gradient, the effects of the vacuum consolidation method on the reinforcement of newly dredged mud were undesirable, and the filter was easily clogged under this condition.

In terms of the effect of filter pore size on the consolidation rate of dredged slurry, the pore size of filters should satisfy the following two requirements. On the one hand, the pore size of filter should be sufficiently small to prevent fine soil particles from entering the filter and drain. On the other hand, the pore size of the
filter should not be too small as the filter must have sufficient permeability. A suitable pore size of filter can aid in the formation of an anti-filtration layer on the surface of the filter and to alleviate PVD clogging in the process of vacuum preloading. The pore size of a filter is generally maintained within a certain range to optimize the performance of PVDs. Some criteria to determine the pore size of filter have been proposed by several researchers (Christopher and Fischer 1992; Luetich et al. 1992). Palmeira et al. (2002) conducted different types of tests to determine the pore size of geotextiles. The authors also evaluated the accuracy of the permeability estimation formula of geotextiles. Koerner et al. (1994) reported that filters can excessively clog when permeated with leachate over long periods of time and a design equation for geotextile filters or graded granular soils used in landfill leachate collection systems was proposed. Koerner et al. (1982) also conducted gradient ratio test to study the long-term drainage capability of geotextiles, and concluded that it takes approximately 200 h to stabilize the gradient ratio for dredged sludge with high clay content. However, these standards are not uniform. A commonly used criterion proposed by Carroll (1983) is $O_{95} \leq (2 - 3)D_{85}$, where $O_{95}$ is an average size that is larger than 95% of the filter pores and $D_{85}$ is the size of the soil particle that makes up 85% of the mass of the soil sample. However, Chu et al. (2004) concluded that the above criterion for pore size of filter is too conservative for Singapore marine clay. They proposed a more relaxed criterion as $O_{95} \leq (4 - 7.5)D_{85}$. Thus, it is
imperative to select a suitable pore size of filter to achieve the goals of reinforcement effect of dredger slurry.

In this study, the effect of filter pore size on the drainage characteristics of PVDs was investigated. First, a series of gradient ratio tests were conducted at different hydraulic gradients using various filter pore sizes to determine the permeability of the filter sleeve. Then, a set of vacuum preloading tests were conducted to evaluate the effect of PVD filter on the dredged slurry improvement. During the vacuum consolidation test, vacuum pressure, pore water pressure, settlement, and discharged water volume were monitored. The water content and vane shear strength were measured at the end of the test (Sun et al. 2018). Finally, the optimum pore size of PVD filter for improving consolidation of dredged slurry was determined based on the gradient ratio and vacuum preloading tests.

Test materials

Properties of test soil

The dredged slurry utilized in this study was obtained from Oufei tideland reclamation site in Lingkun, Wenzhou, China at a depth of approximately 1.0 m from the ground surface. It mainly consists of silt and clay with characteristics of high water content, low shear strength, and high compressibility. Detailed properties of the test soil are listed in Table 1. The initial void ratio, water content, and liquid and plastic limits of the soil samples were 2.32, 88, 49.1, and 28.4, respectively. The test
soil was classified as clay with high plasticity. In addition, the soil was extremely soft with virtually no shear strength.

Figure 1 shows the particle size distribution curve of the dredged slurry. It can be observed from the figure that clay particles with diameters less than 5 µm constitute 50% of the total mass of soil samples, and the size of 85% of soil particles ($D_{85}$) was approximately 23 µm. The size of 95% of soil particles was smaller than 75 µm. This indicates that the soil mainly consists of fine particles, which can easily cause clogging of PVDs during vacuum preloading.

**PVDs used in the study**

In this study, NPVD as shown in Fig. 2(a) was used. NPVDs usually consist of a core made of polypropylene and a filter sleeve made of non-woven polypropylene geotextile, in which the filter sleeve is bonded to the core by heat melting to form an integrated body. At present, NPVDs are widely used in land reclamation projects in China.

NPVDs are totally different from CPVDs, which is shown in Fig. 2(b). The filter sleeve of CPVD is separated from the inner core, and is characterized by an uneven surface. The convex surface is permeable, whereas the other concave surface is impermeable. In addition, the CPVD has only one drainage channel; therefore, clogging occurs easily in this type of PVD. The filter of an NPVD is equipped with hydrophilic fibres, and has double drainage directions. Figure 3 shows scanning
electron microscope (SEM) images of an NPVD filter with different pore sizes of filters. The weight per unit area of the $O_{95}$ 80-μm, 120-μm, 160-μm, and 320-μm filters are 105 g, 100 g, 95 g, and 90 g, respectively. It can be observed that it has a large filter diameter to ensure a higher drainage rate. In addition, the NPVDs is easier to degrade to eliminate post-construction settlement.

In addition, the pore sizes of most CPVDs filters vary in the range of 20–40 μm and the pore sizes cannot be adjusted. However, the pore size of the filter of NPVDs is adjustable and varies within 60–320 μm. Filters with $O_{95}$ of 80 μm, 120 μm, 160 μm and 320 μm were employed in this study based on the criterion proposed by Chu et al. (2004) to determine the optimum pore size for vacuum consolidation.

**Gradient ratio test**

**Test apparatus and materials**

The gradient ratio test apparatus mainly consists of a testing cell and peripheral components, as shown in Fig. 4. The testing cell is made of polymethyl methacrylate with an internal diameter of 120 mm. The PVD filter was cut into a circular shape with a diameter of 100 mm and fixed at the bottom of the testing cell as a geofabric. Soil was placed in the testing cell and on the PVD filter.

The peripheral components mainly consist of piezometric tubes and a constant head water device. Tube 1 was used to discharge air in the testing cell. Tube 2 was connected to a water supply device to provide constant water input, whereas tube 7
was at the bottom of the testing cell and was used to discharge water. Tubes 4 and 5 on the side of the testing cell were connected to a piezometer tube to measure water pressure at varying depths.

**Test procedure**

A set of gradient ratio tests were conducted to investigate the permeability of PVD filter. The testing cell was first filled with dredged slurry to a height of 100 mm. Then, water was poured into the cell slowly to immerse the test soil. The container lid was then covered to seal the soil sample. The outlet valve above the glass container was kept open when the water valve of the constant water head device was closed. The constant head outlet supplies water until the container has been filled with water, all bubbles have been discharged, and the outlet valve has been closed. After completing the above steps, the soil sample was kept for 2 to 3 days to attain stable precipitation based on the scheduled design and arrangement. Finally, the test was conducted and the test data were recorded.

The amount of water that flows in the soil would be different depending on the value of hydraulic gradients. In this study, a uniform hydraulic gradient \((i)\) was employed for the comparison of four types of PVDs filters. The hydraulic gradient test \((i=5)\) was first carried out until a steady state was attained. Then, the hydraulic gradient was increased to \(i=10\) to continue the test until it was completed.

**Test results and data analysis**
In the gradient ratio test, the gradient ratio (GR) is defined as follows:

\[ GR = \frac{i_{sf}}{i_s} \]  

where \( i_{sf} \) is the hydraulic gradient of geotextile and soil, \( i_s \) is the hydraulic gradient in soil.

Recalling that the hydraulic gradient \( i \) is the ratio of head difference along the flow direction \( (\Delta h) \) and the length of the flow path \( (L) \) as follows:

\[ i = \frac{\Delta h}{L} \]

Therefore, the gradient ratio formula can be rewritten as follows:

\[ GR = \frac{i_{sf}}{i_s} = \frac{\Delta h_{sf}/L_{sf}}{\Delta h_s/L_s} = \frac{L_s \Delta h_{sf}}{L_{sf} \Delta h_s} \]

where \( \Delta h_{sf} \) is the head difference of geotextile and soil, \( L_{sf} \) is the length of the flow path of geotextile and soil, \( \Delta h_s \) is the head difference in soil, and \( L_s \) is the length of the flow path in soil. A higher GR value indicates the clogging around the geotextile is more serious.

The GR was determined using measured data and Eq. (3), and the variations in GR with time for different PVD filters with \( i=5 \) and \( i=10 \) (after 288 h) are shown in Fig. 5. The plots for \( i=5 \) show an increasing trend in all filters before finally achieving a steady state. Le et al. (2014) also reported similar results. This indicates that the filter is clogged; the smaller the pore size, the more obvious the clogging.

The hydraulic gradient was changed instantaneously from \( i=5 \) to \( i=10 \) at 288 h. The same instantaneous increase in the gradient ratio can be observed in all types of
filters when \( i=10 \). This indicates that soil particles were removed and accumulate on the filter surface at a high speed under a high water head, thereby inducing relatively severe clogging on the filter surfaces. After about 370 hours, the GR of the filter with \( O_{95} \) 320-\( \mu \text{m} \) was less than 3 when \( i=10 \), indicating less clogging on such a filter surface based on the gradient ratio criterion proposed by the U. S. Army Corps of Engineers (ASTM, 2006). Further, the gradient ratios (GRs) of the four filters initially decreased and then increased with time when \( i=10 \). The reasons for this interesting phenomenon are discussed as follow.

As the hydraulic gradient \( i \) increases from 5 to 10 abruptly, the soil particles rapidly accumulated and concentrated in the filter hole that increases the head pressure per unit area on the filter. As time increases, water gradually passes through the filter that reduces the filter clogging. However, for a constant hydraulic gradient, the force per unit area of filter hole decreases, soil particles gradually move into filter holes and then continue to cause different levels of filter clogging. Finally, the gradient ratio reaches the stable state at about 580 h.

The results of the above tests show that the larger pore size of the filter, the better permeability behaviour the filter has. The optimum pore size of the filter cannot be determined by the gradient ratio test. As the vacuum pressure played a dominant role during vacuum consolidation, the performance of the filter during vacuum consolidation would be distinct from the ones during the gradient test. The hydraulic gradient varied with the vacuum pressure cannot reflect the clogging effect of fine
particles entering the drainage path of the PVDs. The influence of different pore sizes of filters on vacuum consolidation will be further described in the following section.

**Vacuum preloading model test**

*Test apparatus and procedure*

The apparatus used in this test mainly consists of testing cylinders and an improved vacuum preloading system. Four types of NPVDs with different pore sizes of filters (80 μm, 120 μm, 160 μm, and 320 μm) were used in this test. The testing cylinders were made of polyethylene and their inner diameter and height were 500 mm and 900 mm, respectively; furthermore, their inner walls were polished to reduce friction. A PVD specimen was preinstalled at the centre of the cylinders, and the depth of the PVD was up to 900 mm. Two pore pressure transducers were mounted on an iron shelf to monitor the pore water pressure at the depths of 30 cm and 60 cm, respectively. A syringe needle inserted into the core of the PVDs and connected to a vacuum gauge through a vacuum pipe which was used as a vacuum pressure probe. Two sets of syringe needles were used; one set was positioned along with the central PVDs to evaluate the vacuum pressure in the PVDs, whereas the other set was distributed along with the undrained PVDs to evaluate the vacuum pressure in the soil. Figure 6 shows the layout of the iron rack in detail.

After placing the slurry, the iron rack was positioned at the bottom of the model box, and the PVDs were connected to the vacuum pump through vacuum tubes.
Subsequently, one layer of geotextile and two layers of geomembrane were placed on the surface of the slurry to seal it (Chai et al. 2006). The monitoring points of the vacuum pressure and pore water pressure are shown in Fig. 6. Vacuum pressure was applied on the slurries as soon as the vacuum pump starts working, and consolidation commenced. During vacuum consolidation, the pore water pressure, vacuum suction, volume of extracted water, and surface settlement were measured. The vacuum was stopped when the settlement tended to be constant. When the test has been completed, the vane shear strength and water content of the test soil were measured at different depths.

*Evaluation of anti-clogging performance of filter*

*Vacuum pressure*

The vacuum pressure in the PVDs were measured at different depths and plotted as shown in Fig. 7. It can be observed that the vacuum pressure remained stable until the end of the test. The pore size of the filter did not affect the vacuum pressure in the PVDs. In addition, the loss in vacuum pressure along the depth was estimated as 10 kPa/m. Loss in vacuum pressure occurs because the PVDs not only transfer vacuum pressure but also discharge water in the vacuum preloading process (Chu et al. 2000); furthermore, the PVDs bend with consolidation of the soil. Therefore, vacuum pressure loss along the depth occurs frequently.

Figure 8 shows variation of the vacuum pressure with depth in soil. In Fig. 8(a), vacuum pressure was not observed in the soil at the beginning of the experiment,
indicating that the soil was in a saturated state at that time. As the test continued, the vacuum pressure in the soil became measurable when the soil changed from saturated state to unsaturated state (Ming and Zhao 2008). In particular, the vacuum pressure in the soil that installed PVD with a $O_{95} 320$-$\mu$m filter was first measurable after 30 h, indicating that the soil reached an unsaturated state (Qiu et al. 2007). Then, the vacuum pressure was measured in sequence for PVDs with $O_{95} 160$-$\mu$m, $120$-$\mu$m, and $80$-$\mu$m filters. It is possible that the drainage channel is unblocked at the beginning of the experiment, and the $O_{95} 320$-$\mu$m filter yielded fast drainage. At approximately 150 h, the degree of vacuum pressure for the PVD with $O_{95} 120$-$\mu$m filter was higher than those of the PVDs with $O_{95} 160$-$\mu$m and $320$-$\mu$m filters. Figure 8(a) also shows a similar upward trend for each curve. The vacuum pressure in the soil gradually increased with time, and finally reached stable state. However, the vacuum pressure for the PVD with $O_{95} 80$-$\mu$m filter was the lowest, followed by those of the PVDs with $O_{95} 320$-$\mu$m and $160$-$\mu$m filters, whereas the vacuum pressure for the PVD with $O_{95} 120$-$\mu$m filter was the highest.

The low vacuum pressure for the PVD with $80$-$\mu$m filter is probably due to the small pore size of the filter. A large amount of soil particles accumulated on the surface of the PVD filter, causing clogging and eventually resulting in low transmission of vacuum pressure. The highest values occurred when PVD with $O_{95} 120$-$\mu$m filter owing to the reduced clogging of the PVD by using appropriate pore size of filter and formation of a better anti-clogging system (Wang et al. 2018a).
However, because of the PVDs with large pore size of the 160-μm and 320-μm filters, slight clogging of the holes occurred in the early stage, which is beneficial to drainage. Later, soil particles gradually entered into the core of the PVDs and blocked the drainage channels. In addition, soil particles were partly displaced into the filter surface and blocked the holes of the filter. The soil around the PVDs cannot form a good anti-clogging layer owing to significant loss of soil particles, thereby causing serious clogging in the PVDs and hindering the transmission of vacuum pressure.

From Figs. 8(a) and (b), it can be observed that vacuum transfer in the soil layer worsens as the soil depth increases. The vacuum pressure decreased by 4 kPa as the soil depth increased. However, the vacuum pressure in the soil did not develop at the same rate as vacuum transmission at different depths, which is similar to the result reported by Khan and Mesri (2014).

*Volume of extracted water*

The water discharged during the test was collected and measured using an air–water separation flask. The change in the amount of water discharged with respect to time was then plotted. Variations in the volume of drainage over time for PVDs with different pore sizes are shown in Fig. 9. It can be observed that a large amount of drainage was collected at the early stage, and the drainage volume for the early 48 h reached 60.9% of the total drainage volume. At the early stage, the drainage increased with the increase in the pore size of the filter. Then, the drainage when PVD with $O_{95}$ 120-μm filter was used was higher than when $O_{95}$ 160-μm and 320-μm filters were
used at the middle stage. Finally, the drainage volume of the $O_{95}$ 120-µm filter was significantly higher than those of other filters. The mass of water discharged through the $O_{95}$ 120-µm filter was 37830 g, which is 16.3%, 4.5%, and 9.4% higher than those of $O_{95}$ 80-µm, 160-µm, and 320-µm filters, respectively. Besides, the permeability of the PVD filters gradually decreased, the soil porosity became smaller, and the drainage rate decreased with time (Chai and Miura 1999).

**Settlement**

Data of surface settlement at four locations in the model boxes were recorded during vacuum preloading, and the variations of the settlement over time were plotted as shown in Fig. 10. The curves show a sharp increase at the initial stage, then the corresponding settling rates began to fall between 100 h and 250 h. After 250 h, the surface settlement remained almost unchanged, revealing that further settlement cannot be achieved (Chu et al. 2000). At the end of the test, the final settlements for PVDs with $O_{95}$ 80-µm, 120-µm, 160-µm, and 320-µm filters were approximately 15.2 cm, 17.6 cm, 17.1 cm, and 16.3 cm, respectively.

At the early stage, the settlement rate increased as the pore size of the filter increased. However, it is obvious that the settlement rate of $O_{95}$ 120-µm filter gradually exceeded those of $O_{95}$ 160-µm and 320-µm filters at the middle stage. Then, at the advanced stage of the vacuum preloading, the settlement rate and the final settlement for PVD with $O_{95}$ 120-µm filter were larger than those of other pore sizes of filters.
The degree of consolidation (DOC) is an important parameter to evaluate the efficiency of vacuum preloading. Zeng et al. (1959) and Chu et al. (2000) calculated the final settlement of soil as follows:

\[ S_f = \frac{S_3(S_2 - S_1) - S_2(S_3 - S_2)}{(S_2 - S_1) - (S_3 - S_2)} \]

where \( S_f \) is the final settlement, and \( S_1, S_2, \) and \( S_3 \) are the settlements at time \( t_1, t_2, \) and \( t_3 \) respectively. This must meet the following criteria: \( t_1 < t_2 < t_3 \) and \( t_2 - t_1 = t_3 - t_2 \) (Zeng et al. (1959). The definition of DOC is given as follows:

\[ \bar{U}_i = \frac{S_i}{S_f} \]

where \( S_i \) is the settlement at time \( t \) and \( \bar{U}_i \) is the DOC.

The calculated degree of consolidation is summarized in Table 2. The degree of consolidation of the \( O_{95} \) 120-μm filter is the highest, reaching 85.22%, whereas that of \( O_{95} \) 80-μm is 70.5%, which is the lowest. It is clear that the pore water pressure of the \( O_{95} \) 120-μm filter is useful in the dissipation of pore water pressure, which improves soil consolidation.

The following equation can be derived from the equation by Hansbo (1979) for the consolidation of PVDs:

\[ \ln \left( \frac{S_f}{S_f - S_t} \right) = \alpha t \]

\[ \alpha = \frac{8C_p}{D_c^2 f} \]
where $D_e$ is the effective diameter of a unit cell of drain and $f$ is the resistance factor due to the effects of spacing, smear, and well resistance (Bergado et al. 2002).

The values of $\frac{k_h}{k_s}$ and $f$ are calculated as follows (Bergado et al. 2011):

\[
\begin{align*}
\frac{k_h}{k_s} & = \ln\left(\frac{D_e}{d_w}\right) - \frac{3}{4} \\
(8) & \\
\frac{k_h}{k_s} & = (\frac{k_h}{k_s} - 1) \ln\left(\frac{d_s}{d_w}\right) \\
(9) & \\
f & = \frac{2}{3} \pi \frac{t^2 k_h}{q_w} \\
(10) & \\
f & = f_n + f_s + f_r \\
(11) &
\end{align*}
\]

where $d_s$ is the diameter of the mandrel, $d_w$ is the equivalent diameter of the drain, $\frac{k_h}{k_s}$ is the coefficient of horizontal permeability of the undisturbed zone, $k_s$ is the coefficient of horizontal permeability of the smear zone, and $q_w$ is the discharge capacity of the drain at hydraulic gradient of 1. The estimated parameters for consolidation are listed in Table 3.

The horizontal coefficient of consolidation ($C_h$) for four PVDs with different pore sizes of filters calculated based on Eq. 5–11 is shown in Fig. 11. The back-calculated $C_h$ of 6.17 m$^2$/yr was recorded for the $O_{0.8} 80$-μm filter. The value was the lowest compared to the other pore sizes of filters. The back-calculated $C_h$ of the 120-μm filter was the highest and the maximum $C_h$ is 8.29 m$^2$/yr, indicating that it had a better compression behaviour and soil consolidation in the radial direction.

**Pore water pressure**
Figures 11(a) and (b) show variation in the pore water pressure of different pore sizes of filters at different depths. It can be observed that the dissipation of pore water pressure decreases with the increase in depth during vacuum preloading. At a depth of 30 cm, the pore water pressure of the 120-μm filter dissipated by approximately 43.4 kPa, whereas it dissipated by only 14.6 kPa at a depth of 60 cm. In contrast, the dissipation of pore water pressure in the deep soil layer was less than that in the shallow layer, which is attributed to the high level of vacuum transition in shallow layers (Cai et al. 2017; Qiu et al. 2007).

Figure 11(a) also shows the variation of pore water pressure of different pore sizes of filters at a depth of 30 cm during vacuum consolidation of soft soil. It can be observed that a similar trend of the dissipation of pore water pressure of PVDs occurred for different pore sizes of filters after vacuum activating. In particular, the dissipation of pore water pressure for the $O_{95} 120$-μm filter after 150 h was larger than those for the $O_{95} 160$-μm and 320-μm filters. The final dissipation of pore water pressure of the $O_{95} 120$-μm filter was the largest compared to those of other pore sizes of filters; that is, the pore water pressure values at the end of the test were -32.1 kPa, -40.9 kPa, -37.7 kPa, and -35.1 kPa for the $O_{95} 80$-μm, 120-μm, 160-μm, and 320-μm filters, respectively. It is known that a higher vacuum pressure always contributes to a higher rate of drainage, which increases the dissipation of pore water pressure (Indraratna et al. 2010). Thus, the above results are in good agreement with the vacuum pressure distribution of the soil (Cai et al. 2017).
*Water content*

To evaluate the effect of different pore sizes of filters on vacuum consolidation properties of soft soil, the water contents at various distances from PVDs were measured and plotted as shown in Figs. 12(a) and (b). It can be observed from Figs. 12(a) and (b) that there is an approximate tendency of the water content distributions for all PVDs with different pore sizes of filters. First, for the four different pore sizes of filters, the water content at the soil surface is essentially similar, which is mainly responsible for the relatively steady vacuum pressure at the soil surface and the excellent drain performance of the horizontal PVDs. Then, the water contents increased with the increasing depth; this behaviour is attributed to the fact that the vacuum pressure transferred to the soil decreases with the increase in the drain depth (Indraratna et al. 2005).

It can be observed from Fig. 12(b) that the average water contents for $O_{95}$ 80-μm, 120-μm, 160-μm, and 320-μm filters were of 56.24%, 53.14%, 54.06%, and 54.92%, respectively at the end of the tests. At depths of 0 cm to 60 cm, the maximum increment of water content along the depth occurred in the $O_{95}$ 80-μm filter, which reached 12.99%, whereas the minimum increment of water content along the depth occurred in the $O_{95}$ 120-μm filter, which is reached 7.7%. This indicates that the water content of the soil decreased as the $O_{95}$ 120-μm filter PVD is more uniform along the depth. Compared to the $O_{95}$ 120-μm filter, the water contents of the soil with $O_{95}$ 160-μm and 320-μm filters had increments of 1.73% and 3.35%, respectively. It can
be observed from Figs. 12 (a) and (b) that there is a distinct difference in the
distribution of water contents of soil strengthened along the radial direction using
different pore sizes of filters along the radial direction. The water content of soil close
to the PVDs is generally low.

_Vane shear strength_

Figures 13(a) and (b) show variations in the vane shear strength of soil reinforced
using PVDs with different pore sizes of filters along the drain depth. As the depth
increases, a similar decreasing trend was observed in the vane shear strength for all
PVDs with four pore sizes of filters. It is considered that a better consolidation was
achieved in shallow layers because a higher vacuum pressure was maintained.

Figure 13(b) shows plots of the shear strength of the vane at different depths and
at a distance of 15 cm from the plate. Average vane shear strengths of 8.8 kPa, 10.43
kPa, 9.97 kPa, and 9.47 kPa were observed for PVDs with $O_{95}$ 80-μm, 120-μm,
160-μm, and 320-μm filters, respectively. The vane shear strength of the $O_{95}$ 120-μm
filter was the highest, whereas that of the 80-μm filter was the lowest. The vane shear
strength of the $O_{95}$ 120-μm filter increased by 1.63 kPa compared to that of the 80-μm
filter. It can be concluded that PVD with the $O_{95}$ 120-μm filter had the best
reinforcement effect.

**Conclusions**
The effect of the filter pore size on the permeability of the filter sleeve was investigated by a series of gradient ratio tests at different hydraulic gradients, and the effect of different pore sizes of PVD filters on soil improvement was further evaluated with the vacuum preloading tests. The following conclusions are drawn by comparing the results from the two sets of experiments:

(1) The gradient test showed that the gradient ratio decreased with the increase in the pore size of the filter. However, the gradient test may not accurately reflect the clogging of the PVDs due to the fine soil particles during vacuum consolidation. Thus, further tests with the varying filter pore size during vacuum consolidation is necessary.

(2) In the early stage of vacuuming preloading, the vacuum pressure and discharged water volume of the soil increased with the increase in the filter pore size, and the pore water pressure dissipation was more obvious. However, as the vacuum preloading continued, the drainage effect of PVD with $O_{95}$ 120-μm filter became more obvious compared to those of the other three filters of PVDs, and a higher degree of consolidation and a larger $C_h$ were finally obtained.

(3) The average water contents for $O_{95}$ 80-μm, 120-μm, 160-μm, and 320-μm filters were 56.17%, 52.21%, 53.79%, and 55.60% respectively at the end of the tests. This indicates that a better consolidation can be achieved by using the $O_{95}$ 120-μm filter.
In summary, the pore size of filter was not entirely determined by the gradient ratio test due to the difference between static hydraulic gradient and vacuum pressure. Thus, the optimal pore size of the filter was further determined by vacuum preloading test; that is, the $O_{95} 120$-$\mu$m filter was determined as equivalent to $5.22D_{85}$ and suitable for vacuum consolidation of dredged slurry. This can provide a design guide for large-scale application of tideland reclamation projects in Wenzhou, China.

Acknowledgments

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References


Figure captions

Fig. 1. Particle size distribution of the test soil.

Fig. 2. Structure of NPVDs and CPVDs.

Fig. 3. SEM images of the NPVD filter with different pore sizes (×400)

Fig. 4. Schematic diagram of inlet and outlet components in constant head test.

Fig. 5. Variations in GR for different PVD filters at two hydraulic gradients.

Fig. 6. Apparatus for vacuum preloading model test (in mm).

Fig. 7. Vacuum pressure in PVDs measured at different depths.

Fig. 8. Vacuum pressure in soil measured at different depths.

Fig. 9. Volume of drainage for PVDs with four different pore sizes.

Fig. 10. Surface settlement at four locations in the model boxes.

Fig. 11. Pore water pressure of different pore sizes of filters at different depths.

Fig. 12. Soil water content at different depths and different distances from the plate.

Fig. 13 Shear strength of slabs with different depths and different distances from the plate.
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(a) NPVDs  
(b) CPVDs

Fig. 2. Structure of NPVDs and CPVDs.
Fig. 3. SEM images of the NPVD filter with different pore sizes (×400)
Fig. 4. Schematic diagram of inlet and outlet components in constant head test.

(1) Air outlet
(2) Connection of container of constant head
(3) Transparent cylinder of 100mm diameter
(4) Connection of piezometric tube
(5) Connection of piezometric tube
(6) Geofabric
(7) Outfall

Fig. 5. Variations in GR for different PVD filters at two hydraulic gradients.
Fig. 6. Apparatus for vacuum preloading model test (in mm).

Fig. 7. Vacuum pressure in PVDs measured at different depths.
(a) 30 cm depth  
(b) 60 cm depth

Fig. 8. Vacuum pressure in soil measured at different depths.

Fig. 9. Volume of drainage for PVDs with four different pore sizes.
Fig. 10. Surface settlement at four locations in the model boxes.

(a) Depth 30 cm  
(b) Depth 60 cm

Fig. 11. Pore water pressure of different pore sizes of filters at different depths.
Fig. 12. Soil water content at different depths and different distances from the plate.

Fig. 13. Shear strength of slabs with different depths and different distances from the plate.
Table captions

Table 1. Basic physical and mechanical parameters of the test soil sample
Table 2. Degree of consolidation of filter with different pore sizes
Table 3. Parameters for consolidation
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<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
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<td>Specific gravity %</td>
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<td>Saturation %</td>
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<tr>
<td>Void ratio %</td>
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<td>Permeability coefficient cm/s</td>
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Table 2. Degree of consolidation of filter with different pore sizes

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<th>Pore Size</th>
<th>Degree of Consolidation</th>
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<td>80 µm</td>
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<td>120 µm</td>
<td>U₁₉₅ 85.22%</td>
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<td>160 µm</td>
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<td>320 µm</td>
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Table 3. Parameters for consolidation

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<td>dᵥ (m)</td>
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<td>Dₑ (m)</td>
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<tr>
<td>dₕ (m)</td>
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<td>qᵥ (m³/s)</td>
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<td>kₛ (m³/s)</td>
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