Physical Model Study on the Clay-Sand Interface without and with Geotextile Separator

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ABSTRACT:

In most marine reclamation projects, sand fills are directly placed on soft marine soils in the seabed. The sand particles can easily penetrate into the soft marine soils, and soft soils can also move into the pores inside sand through the initial contact interface between the sand and the soft soils. In this case, the permeability and the volume of the sand above the initial surface are reduced. To avoid this problem, a geotextile separator may be placed on the surface of the soft soils in the seabed before placing the sand. In this study, a two-dimensional (2-D) physical model is utilized to study the geotextile separator effects. The initial conditions of clayey soil, filled sand, and surcharge loading were kept the same in the physical model test with the only difference in that a geotextile separator was placed on the clay surface or not. The settlements of the initial interface were recorded and compared for the two cases without or with the geotextile separator. Particle size distributions of soils taken across the interface for different time durations were also measured, analyzed, and compared. Based on the result analysis, the sand percolation depth was 40 mm and fine particle suffusion was apparent when sand was directly placed on the marine slurry surface without using the geotextile separator. Comparatively, the sand percolation was avoided, and the fine particle suffusion was effectively diminished when the geotextile separator was utilized. A relative fine particle fraction is defined to illustrate the immigrated fine particles from clay to sand soils. The fine particle percentages of the HKMD-sand mixtures are calculated for the two cases without and with a geotextile separator for a better analysis of the geotextile separator effects in practice.

Keywords: physical model, percolation, suffusion, interface, geotextile

1. INTRODUCTION

Many reclamation projects have been conducted in Hong Kong, Singapore, Australia, Malaysia, USA, *etc*. [1]. For new marine reclamation projects in Hong Kong, soft marine soils in seabed are not allowed to be dredged and then dumped at another marine site due to environmental concerns [2, 3]. In Hong Kong, sand fill is normally placed on the marine soils in seabed under sea table by a large barge with bottom openings [4,5]. When the sand fill has reached at sea level of 6 to 7 meters, vertical drains will be installed through the sand fill into the soft marine soils with a surcharge applied by additional fill. In this way, the soft marine soils are kept in place and improved with post-construction settlements reduced and shear strength increased. When the sands are filled directly on the soft marine soils, sand percolation may happen. Sand percolation is the phenomenon that the sand particles move into the clayey soils along the depth. Based on the engineering experiences, some sand particles would percolate into the soft marine slurry in the interface between sand and marine slurry. This sand percolation can result in the loss of sand above the initial contact interface and the marine soils near the interface are usually a clay-sand mixture [6]. It is necessary to consider the sand percolation in the design of a reclamation project.

For clay-sand mixtures, many researchers focused on the experimental study on the influence of sand contents on the shear strength [7, 8, 9] and consolidation properties [9, 10, 11]. Monkul and Ozden [12] conducted oedometer tests on the compression characteristics of kaolinite-sand mixtures. Peters and Berney [13] investigated the influence of fine fraction on the threshold behavior and stable force chains of clay-sand mixtures. Simpson and Evans [14] examined the behavioral thresholds of the clay-sand mixtures with fine contents ranging from 0% to 100%. Choo *et al.* [15] studied the compressibility and small strain stiffness of kaolinite-sand soils consisting high contents of clay particles, and proposed a porosity function of small particles. Park and Santamarina [16] summarized the basic properties and mechanical behavior of coarse-fine mixtures from previous studies and proposed a revised soil classification system to capture the mechanical and hydraulic properties. Shi and Herle [17,18] proposed a general procedure for the mechanical evaluation of inhomogeneous soils with stiff inclusions. This approach was further generalized by Shi and Yin [11] for the consolidation behaviour of marine-sand mixtures based on series of oedometer tests. The model has only three parameters with clear physical meaning, which may have potential application in practice. It is found that researches on the clay-sand mixtures are mainly by element tests including oedometric and triaxial compression tests, and the clay-sand mixtures are manually prepared. These provide the experimental evidence of clay particle content to investigate the fundamental behaviour of soil mixtures. However, there are seldom experimental tests to directly study the percolation amount of clay-sand interface.

Comparatively, fine particle suffusion, as one type of internal erosions, is used to describe the fine soil particles transport through the pore domain in a coarse layer by seepage flow [19]. Fine particle transport can induce the internal instability such as piping and sinkholes [20, 21], and nonneglected amount of clayey soils are lost in the process of suffusion. It is difficult to quantitively evaluate the loss amount of clayey soils and instability of clay layer in reclamation projects. However, the fine particles amount is usually measured in the laboratory tests [22, 23]. Based on the mass balance equation, the fine particle suffusion can result in the change of particle size distribution of the original soils. As a result, a comparison of particle size distributions of soils measured before and after fine particle movement is an effective means for evaluating the amount of fine particle transport and fine particle suffusion.

A geotextile sheet has been suggested as a geotextile separator to be placed on the marine soil surface before filling sand in order to minimize the sand percolation effects in marine reclamation projects in Hong Kong. The geotextile separator also plays a significant role in minimizing the generation of mud waves, confining the marine mud under the seawater, and stabilizing the filled area. This approach has been used in the projects such as Pak Shek Kong reclamation in Hong Kong [24], Changi east reclamation in Singapore, New Kita-Kyushu airport in Japan *etc.* [5]. However, it is still unclear about the efficiency of the geotextile separator in the clay-sand interface. In Hong Kong, marine reclamation for the construction of the third-runway system is about to start. Clean sand will be placed on the surface of Hong Kong Marine Deposits (HKMD) in seabed under the seawater. However, whether or not a geotextile separator shall be placed on the surface of HKMD has not been decided yet with different views, especially questions on the effects and efficiency of a geotextile separator. This is main motivation of our research project.

In the field, the real marine and geotechnical conditions are usually very complicated. A proper physical model shall represent the typical field conditions with initial controllable conditions. The main goal of the physical model is to study the sand percolation and fine particle suffusion during the filling and consolidation process of HKMD. The HKMD and sand used in the physical model were taken from a real marine reclamation project in Hong Kong. This physical model with the same HKMD was divided into two parts: one part without a geotextile separator; while the other part with a geotextile separator. The same sand was filled on the surface of the HKMD slurry in the model and the same multi-stepped vertical loading was applied. In this way, the effects of the geotextile separator could be investigated and accessed.

2. PHYSICAL MODEL AND EXPERIMENTAL PROCEDURES

2.1 Physical Model and Materials

A plane strain physical model, designed by Yin and Fang [25], was adopted for the study in this paper. This model has the dimensions of 900 mm (in length), 300 mm (in width), and 900 mm (in depth). This physical model has two transparent sides of 25.4 mm thickness with adhered rulers and marked horizontal lines (as shown in Figure 1), which were used to monitor the real time-settlement of the HKMD-sand interface. In order to investigate the effects of a geotextile separator, one third (300 mm) of the length (900 mm) of the physical model space was placed with a geotextile separator directly the HMKD in a slurry state.

The HKMD was taken from the East Coast of Lantau Island in Hong Kong. The initial water content of the HKMD was 44%~52%. This HKMD was mixed with water using a miniature motorized mixer [25]. The prepared water content of HKMD slurry was controlled in the range of 105% ~110%, which was around 2 times of the liquid limit of the HKMD. Clean sand, which was used in the field as the sand fill, was adopted in this physical model study. Special sand layer with orange color was placed on the HKMD surface nearby the transparent sides. The geotextile separator used in the physical model test was the same type of geotextile to be used in the site. The basic properties of geotextile separator are listed in Table 1. The initial particle size distribution (PSD) curves of the sand fill and HKMD are shown in Figure 2.

2.2 Test Procedures Description

Detailed test procedures are described as follows:

(a) Pour the marine clay slurry of HKMD with an initial water content of 105% ~110% into the physical model of full length (900mm). Initial heights of the slurries in both left and right parts were 750 mm.

(b) After one night's initial consolidation of the marine clay slurry under its self-weight, an approximate 5 mm settlement at the clay surface and some clean water were observed. The initial undrained shear strength (C_u) of the marine slurry was measured by a mini-shear vane, which varies between 0.1 kPa and 0.2 kPa due to its high initial water content.

(c) Then, the sand layers were uniformly sprayed on the marine clay surface for both two parts. To spray the sand uniformly on the surface of marine slurry, a container with length of 630 mm and width of 300mm was made and used. The sand was uniformly sprayed on the container with 10mm thickness and 30 mm marked interval, as shown in Figure 3. The sand layer with a marked length of 30 mm was carefully scraped off from the container to the soft marine soil. This procedure was done stepwise until the marine slurry surface was totally covered by sand.

(d) It was observed that the first 20 mm height filled sand was nearly infiltrated into the marine clay for the part of HKMD-sand interface without geotextile separator. After the sand was filled up to 70 mm, it was kept for 1 hour to record the interface location.

(e) The sand was filled up to 150 mm and kept for 1 hour to record the interface location.

(f) A rigid plate with 30mm × 30mm with evenly distributed holes was put on the filled sand surface and a 5 kPa vertical pressure was applied by the dead weight for 1 hour.

(g) Afterwards, the pressure was increased to 10 kPa by the dead weight and kept for 12 hours. Settlement values at 1 hour, 4 hours, and 12 hours were recorded. After this, the pressure was increased to 15 kPa by the dead weight and kept for 12 hours. Settlements at 1 hour, 12 hours were recorded.

(h) Finally, the vertical pressure was increased to 20kPa and kept for 819 hours. Figure 4 shows the process of adding vertical stresses for the physical model test. Relation of applied vertical stress versus time is presented in Figure 5.

2.3 Vane Shear Tests

After 10 days of preloading, the dead weights applying 20 kPa of vertical stress was removed for the first vane shear test to investigate the undrained shear strength of soils. Afterwards, the vertical stress was reloaded to 20 kPa to continue the test. After 10 days of reloading, second vane shear test was conducted. Then, the dead weights were reloaded after second vane shear test. Lastly, the third vane shear test was measured after another 10 days of the second vane shear test. Figure 6 shows the locations of the first, second, and third vane shear tests, in which the side friction effect and soil disturbance of vane shear tests were considered. In this study, the handy mini vane tester with 33 mm in diameter and 55 mm in height was utilized to quickly and accurately determine the undrained shear strength of the soils. The vane shear tests were conducted at the interval of 40 mm along the depth. For accuracy concerns, each vane shear test was measured at two or three different locations with different depths as shown in Figure 6. For the HKMD-sand interface with geotextile separator, the undrained shear strength was not measured because the handy mini vane tester could not go through the geotextile separator easily.

2.4 Particle Size Distribution Analysis

To directly quantify the sand percolation amount, one approach is to analyze the particle size distributions (PSD) of soils cross the initial interface between the sand and the HKMD slurry. The PSD of HKMD soil could change due to the sand percolation in the interface zone. Similarly, the PSD of sand above the initial interface might vary due to fine particle suffusion in the interface zone above. Fine particles are referred as the soil particles passing though a standard sieve with opening size of 0.063 mm. Supposing that HKMD has a fine content by weight of M_{clay} and the original sand has a fine content by weight of M_{sand}, the fine content of HKMD-sand mixture is $M_{mixture}$ at the HKMD-sand interface. A relative fine particle fraction (denoted as Θ) is the ratio of immigrated fine particles of clay-sand mixtures to the fine particles of clayey soils, expressed as:

$$\Theta = \frac{M_{mixture} - M_{sand}}{M_{clay}} \tag{1}$$

In this study, there are very less fine particles in original sand fills, $M_{sand} = 0.1\%$. It can be expected that the value of Θ is in the range of *0~1*, which is the indicator of the immigrated fine particle percentages due to the sand percolation and fine particle suffusion effects.

To analyze the PSDs varying with depth through the HKMD-sand interface, a square hole of 50 mm×50 mm was excavated from sandy soil surface. Soil samples at different depths were taken out for particle size tests and analysis in steps and at depths of 0~40 mm, 40~80 mm, 80~100 mm, 100~120 mm, 120~140 mm, and 140~160 mm, as illustrated in Figure 7. Details of the soils at each depth were photographed (see part of photos in Figure 7). It is worthwhile to note that a thin layer of colored sand placed at the interface before sand filling was scattered on the surface of HKMD, which was detected at the depth of 100~120 mm. Afterwards, the sandy soils taken from different depths were sieved by dry sieving method (from 0.063 mm to 2 mm). After sieving process, the samples were put in the oven to dry for one day. Finally, the dry mass of the samples was measured and the PSD was determined. The hydrometer method was utilized to analyze the particle size distribution of the HKMD [26]. For the HKMD-sand interface with geotextile, a similar approach was used for obtaining PSDs for the sandy soil and HKMD.

3. TEST RESULTS AND DISCUSSIONS

3.1 Settlement of the HKMD-Sand Interface

A side view of the settlement profile at different times is shown in Figure 8(a) and the curves of average settlement versus time of the HKMD-sand interface are shown in Figure 8(b) for two cases of without and with a geotextile separator. As stated above, the initial conditions (HKMD, sand, and loading) for the interface without and with geotextile separator were the same in the physical model, which could minimize the soil heterogeneity. The settlement difference is directly related to the effects of a geotextile separator.

It is found from Figure 8 that the rate of interface settlement at the initial stage is relatively large due to the initially large void ratio of soils near the HKMD-sand interface. Afterwards, decrease of the void ratio near the interface zone induces a decreasing permeability of the sand, which reduces drainage speed of water through the interface zone. Broadly, the settlement of the HKMD-sand interface with geotextile separator is initially the same as the one without geotextile separator as shown in Figure 8. Then, the difference of the two settlement-time curves as shown in Figure 8(b) is observed 240 hours (10 days) later. This indicates that the geotextile separator gradually took its effects in preventing the sand percolation and fine particle suffusion in the interface zone. The settlement difference between the HKMD-sand interface without geotextile separator and the one with geotextile separator increases up to 25 mm at the end of the test as shown in Figure 8. It should be noted that this settlement difference was caused by effects of geotextile separator mainly in the consolidation stage of the HKMD. In real marine reclamation projects, the sand placing method, relative density and shape of sand, and site environment factors such as tide, wave, and the marine soil conditions (heterogeneity, shear strength, permeability, etc.) may contribute mud waves and settlements of the reclamation.

3.2 Results of Vane Shear Test

The vane shear test is a relatively practical and inexpensive test, which is accurate and effective for measuring the undrained shear strength of clayey soils in geotechnical area. The value of undrained shear strength is closely related to the fine particle content in the transition zone [12]. The undrained shear strength of the soils with the depth through the HKMD-sand interface without geotextile separator was measured by a mini vane tester at 10 days, 20 days, and 30 days after the first loading respectively. Values of measured undrained shear strength along the depth at time of 10 days, 20 days, and 30 days are shown in Figure 9.

For the first vane shear test, the top surface of the mini shear vane was 20 mm beneath the surface of the HKMD. The effective middle depth of the vane corresponding to the measured undrained shear strength was 45 mm. Afterwards, the measured depths at the middle of the mini shear vane were recorded every 20 mm except for several points at the first time of measurements. It is found from Figure 9 that the undrained shear strength (USS) values at the three durations of 10 days, 20 days, and 30 days are low near the sand fill surface, but increase with depth, reach at the maximum (peak) values at locations in vicinity of the sand-HKMD interface. After peak values, USS values decrease with depth. The maximum values of undrained shear strength are 3.2 kPa, 3.9 kPa, and 4.4 kPa for durations of 10 days, 20 days, and 30 days, respectively, as shown in Figure 9. Due to consolidation, the undrained shear strength increases with time. Considering a relatively large permeability of the sand and 100 mm thickness of the sand layer, the sand above the HKMD-sand interface was close to a drain condition. Therefore, the increase of undrained shear strength in the HKMD-sand interface zone is resulted from the variation of sand content.

3.3 Observation of the Excavated Hole and PSD Analysis

As mentioned above, a square hole was excavated in the center of the physical model, and soil samples at different depths were taken after 10 days, 20 days, and 30 days, respectively for

PSD tests and analysis. It is observed from Figure 10 that the initial interface between HKMD and sandy soils is seen from the orange color sand particles. The sand above the interface has a grey color, which indicates that HKMD clayey soil particles have moved into the sand layer above. Figure 10 shows also the sand percolation evidence below the HKMD-sand interface. The orange color sand was just between the filled sand and HKMD surface. When we took out HKMD below the interface, our hands could feel sand particles, which indicates the effects of sand percolation. Therefore, a transition zone was formed near the initial HKMD-sand interface, including sand percolated into the HKMD clayey soil and the fine particles suffused into the sand layer above.

To quantitatively estimate the percolation depth, soil samples were taken from different depths in the physical model for particle size distribution (PSD) tests. Note that the coordinate of sand surface was denoted as 0. The ones below this surface are supposed to be positive. Figure 11 presents the comparison of PSDs of soils and the fine particle percentages at different depths cross the HKMD-sand interface without geotextile separator after 30 days of 20 kPa. Fine particle percentage of original filled sand is 0.1% and fine particle percentage of original HKMD is 84.7%. After 30 days, the PSD curve at top 40 mm is close to the original sand material, and the fine particles increase slightly with the depth until the clay-sand interface (80~100 mm). The fine particle percentages of sandy soils are from 0.8% to 1.8% for the HKMD-sand interface without geotextile separator, as shown in Figure 11(b). This is because that some fine particles of HKMD can transport into the sand layer during the consolidation stage, as illustrated the fine particle suffusion in Figure 11(b). In the depth of 100~120 mm, the PSD curve changes significantly, and its fine particle percentage is 42.2%, indicating the percolation effect at the clay-sand interface. Below that, the PSD curve of 120~140 mm is slightly different from that of 140~160 mm, which is also affected by sand percolation. i.e., some fine particles of sand fill the inter-particle voids. The PSD curve of 140~160 mm is the same as the original HKMD. Therefore, the thickness of sand percolation is 40 mm in this study.

Similarly, the PSD curves of soils from different depths and fine particle percentages across clay-sand interface with geotextile separator are shown in Figure 12. It is found that the PSD curve of 0~40 mm almost overlaps that of 40~80 mm, as shown in Figure 12(a). The fine particle percentages of soils above the interface are in the range of 0.22% ~ 0.55% for the HKMD-sand interface with geotextile separator (Figure 12b). There is a bit different from those of 80~100 mm and 100~110 mm. The difference is the result of fine particle suffusion. Below the geotextile, the PSD of HKMD (110~120 mm) is the same as the original HKMD, which indicates that sand particles hardly percolate into clayey soils due to the geotextile separator.

3.4 Analysis and Discussion on Geotextile Effect on the Clay-Sand Interface

Taking the relative fine particle fraction as the indicator, Figure 13 shows the geotextile effect by comparing the results from the clay-sand interface without and with geotextile separator in this physical model.

As shown in Figure 13(a), the original interface has become a transition zone for clay-sand interface without geotextile separator. It is found that the sand particles percolate nonlinearly into the clayey soil along the depth. The relative fine particle fraction varies from 0.1 to 0.953 in the depth of 100~160 mm if without using geotextile separator, while that of 100~110 mm is 0.962 for the clay-sand interface with geotextile separator. Thus, the geotextile separator can effectively prevent the sand particles from percolating into the clay surface if the proper geotextile separator is adopted. The shadow area in Figure 13(a) illustrates the geotextile effect on the sand percolation in this physical model test. Above the clay-sand interface, Figure 13(b) shows the relative fine particle fraction along the depth. For the sands taken from 0~80 mm, the relative fine particle fraction values are 0.0072~0.0089 for the clay-sand interface without geotextile separator, whereas those of the interface with geotextile separator are 0.0014~0.0017. For the sand of 80~100 mm, a lot of fine particles transport into the sand and its value is up to 0.019 for the clay-sand interface without geotextile separator, while it is 0.005 for clay-sand interface with geotextile separator. Therefore, the geotextile separator can largely reduce the suffusion of fine particles.

Limited researches were focus on the fine particle immigration from the clay-sand interface during the consolidation stage. Sterpi [23] proposed an empirical relationship for immigrated fine particles in sand-clay mixture, hydraulic gradient, and time due to suffusion. In fact, the soils in the interface zone is a double-porosity media [27, 28, 29], Bonelli and Marot [19] explained that the soil suffusion is an interfacial process and derived the suffusion law to quantify the amount by means of the multi-scale approach, which is newly developed to describe the behaviour of clay-sand mixture in recent years [30, 31]. Afterwards, Golay and Bonelli [32] used a finite element numerical model to simulate the clay-water interface erosion process. Furthermore, Chung [33] and Chia [34] observed that the bridge network behind the geotextile separator, which helps to prevent the fine particles of clay from moving into the sand particles. The study in this paper provides the evidence for this opinion.

Sterpi [23] proposed a relationship for immigrated fine particles in sand-clay mixture $M_{mixture}$, hydraulic gradient i_w, and time t due to suffusion, expressed as:

$$\Theta = 1 - \left[\exp\left\{ -\left(\frac{i_w^c}{a} \right) \left(\frac{t}{t_o} \right)^b \right\} \right] \tag{2}$$

where t_o=1 h; t is the consolidation time; a, b, and c are three fitting parameters, i_w is the hydraulic gradient, which is related to the hydraulic load and the distance of flow path:

$$i_w = \frac{1}{\gamma_w} \frac{\partial \sigma_z}{\partial x} \tag{3}$$

where σ_z is the applied stress, $\sigma_z = 20 kPa$; γ_w is the water specific weight, $\gamma_w = 10 kN/m^3$; x is the suffusion depth. In this study, consolidation time under 20 kPa is 819 hours, the hydraulic gradient of sand is a constant since flow of most soils can be considered as laminar. The hydraulic gradient is related to the hydraulic load and the distance of flow path. Because the surcharge loading is 20 kPa, the hydraulic gradient is assumed to be linear to the suffusion depth (x =0.1 m). Thus, $i_w = 20$. Substitute Eq. (2) into Eq. (1), we can obtain:

$$M_{mixture} = M_{clay} \left[1 - \exp\left\{ -\left(\frac{i_w^c}{a} \right) \left(\frac{t}{t_o} \right)^b \right\} \right] + M_{sand} \tag{4}$$

Values of Θ in Figure 13(b) are utilized to determine the exact values of a, b, and c. The back-calculation method of the least squares is used by minimizing the discrepancy between calculated results, Θ_{cal}, and experimental results, $\Theta(\mathrm{i}_w, \mathrm{t})$, due to the fine particle suffusion [23]:

$$E(a,b,c) = \sum_{\min} \left[\Theta_{cal} - \Theta(\mathrm{i}_w, \mathrm{t}) \right]^2 \tag{5}$$

The best fitted parameters with $a = 1200$, $b = 0.23$, and $c = 0.4535$ are utilized for clay-sand mixtures above the clay-sand interface without geotextile separator. As a comparison, the values of $b = 0.23$ and $c = 0.4535$,which are related to time and hydraulic gradient, should be kept the same, and $a = 5000$ is utilized in the least squares to consider the geotextile separator effect in the clay-sand mixture for the case using geotextile separator. By using Eq. (3), the fine particle percentages of the clay-sand mixture at the depth of 90 mm, which is close to the clay-sand interface, can be predicted with the consolidation time, as shown in Figure 14.

In this physical model study, some limitations should be pointed out: the factors such as mud wave, tide effect, *etc.*, which are usually generated in a reclamation project, were not considered in our physical model test. These factors perhaps induce a thicker sand percolation zone through the clay-sand interface if the geotextile separator is not used. Furthermore, the spreading of sand fills in the physical model is uniform, which is different from the field practice and would affect the sand percolation and fine particle suffusion on the HKMD-sand interface. In other words, this study makes a special effort to investigate the influence of the geotextile separator on sand percolation and fine particle suffusion in a certain condition. Further study is needed to deeply investigate and understand the effects of geotextile separator.

4. FINDINGS AND CONCLUSIONS

To get a better understanding of the geotextile effects on the HKMD-sand interface in reclamation projects, a physical model was built and one model test was performed by dividing the model into two parts: the clay-sand interface without geotextile separator in one part; while the clay-sand interface with geotextile separator in the other part. Based on observations, test data and their analysis, and discussions, main findings and conclusions can be drawn as follows:

(a) For the clay-sand interface without geotextile separator, sand would percolate into the clayey soil and the fine particles from clayey soils would suffuse into the sand.

Therefore, the actual clay-sand interface is a zone.

(b) By the particle size distribution analysis of soils retrieved from an excavated hole, the sand percolation distance was 40 mm and fine particle suffusion decreases nonlinearly along depth of clay-sand interface for the interface without a geotextile separator.

(c) The geotextile separator can prevent the sand particles from percolating into the clay surface. The geotextile separator can effectively minimize the fine particle suffusion amount due to the bridge network. By comparing the behaviour of the interface without and with geotextile separator, it is proved that fine particle suffusion is an interfacial process rather than volume process.

(d) The maximum undrained shear strength of the soils in vicinity of the HKMD-sand interface is found to increase with time in the consolidation progress due to the change of sand content in clay-sand interface zone.

(e) The relative fine particle fraction values along the depth are calculated for the clay-sand interface without and with geotextile separator, which is helpful to quantitively analyze the geotextile separator effects. Further study by using finite element simulation and theoretical analysis will be conducted later by authors.

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