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# Experimental Investigation on Settling Behavior of Hong Kong Marine Deposits in Settling Column Condition

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**ABSTRACT:** Due to the high water content and viscous property, the Hong Kong marine deposit (HKMD) frequently poses challenge to geotechnical practice, particularly to the reclamation in Hong Kong. Formation of HKMD typically goes through sedimentation and self-weight consolidation. These behaviors are time-dependent and hard to describe. A series of one-dimensional settling column consolidation tests were conducted on the HKMD. Results reveal that the settling curve and settling rate are significantly affected by the sediment concentration in self-weight consolidation. Soils even exhibit viscosity in self-weight consolidation. After the primary self-weight consolidation, the settling rate is reduced in the “secondary consolidation”. Related parameters from sketches are evaluated for a better interpretation of the consolidation behavior.

## 1. INTRODUCTION

Marine clays in the seabed have typically high water content and high compressibility. Most marine deposits encountered in Hong Kong are silt and sand with different clay contents and have low undrained shear strength (Yin, 1999). The thickness of Hong Kong marine deposits (HKMD) is between a few meters to more than 20m. Currently more and more reclamation works have been done in Hong Kong, most of which are on HKMDs. Behaviors of HKMD are particularly time-dependent, requiring reliable soil description and understandings. Reasonable prediction of the behavior from both field tests and laboratory tests can improve design, construction, and maintenance of the structures.

Sedimentary soils are formed initially through a process of sedimentation either in sea or in fresh water. Three main sedimentation stages are defined with respect to the concentration degree as Clarification regime, Zone-settling regime, and Compression regime (Fitch, 1983). Study on self-weight settling of incompressible solids was initiated by Kynch (1952), and subsequently developed by McRoberts and Nixon (1976). The term “self-weight consolidation” was in literature proposed by Been and Sills (1981). In consolidation settling, no discrete floc is formed and the interface settles at a constant rate. Due to the low settling rate, soil layers gradually form on the bottom of the water. The bottom sediment consolidates due to its self-weight lying above. Finally this consolidation results in very soft sediments at the top with density increasing till bottom. Actually the settling rate of dredged clay is viewed as a key factor in land reclamation projects. The settling types of the dredged clay in seawater are mainly of zone settling and consolidation settling (Imai, 1980).

The self-weight consolidation recently becomes a major challenge in many engineering problems on the seashore (Mcvay et al., 1986). The settling type of marine clays in seawater shifts from zone settling to self-weight consolidation when the initial water content decreases to less than about 1000% (Imai, 1980). The zone settling rate of a clay suspension with high water content is so rapid that it is difficult to estimate. However, it has been commonly accepted that the traditional consolidation theories are inadequate to explain the self-weight consolidation (Sills, 1981).

In the formation of the soil layer, there is no sharp transition time from sedimentation to consolidation (Li and Williams, 1995). When the interface of sediment rises in consolidation, sedimentation occurs in the suspension slurry. Major argument focused on the interface/boundary was discussed by Been and Sills (1981). Based on Been and Sills’ results, the typical void ratio of the sediment surface at the end of settling lies between 6~7, and the value reaches 9~10 when the effective stress is first measured.

Secondary compression or creep effect, on the other hand, can be important for high water content soils, especially those that contain organic matter. The soil may continue to settle after the “linear region” with slower rate. This creep-like behavior has been attributed to biodegradation and gas generation. However recent studies have illustrated that treatment of the specimens for bacterial activity does not entirely remove the effect. Similar to creep in oedometer condition, it is assumed that the *e-logt* curve in self-weight consolidation becomes a horizontal line eventually.

Consolidation testing of marine clays using traditional geotechnical testing methods is not practically possible. Among the conventional consolidation apparatuses, the settling column test is suitable for 1D settling analysis. The prime objective in this paper is to examine the sedimentation and self-weight consolidation behavior of Hong Kong marine deposits. Proper interpretation of consolidation of HKMD is critical to the modeling of this type of soils and to field applications.

## 2. PHYSICAL PROPERTIES OF HKMD

Two typical viscous marine clays (19 and 17) are adopted from around Nam Sang Wai and Mai Po, Hong Kong to conduct settling column tests. Before each settling test, basic properties such as water content and specific gravity are determined and shown in Table 1 and Table 2.

Table 1 Water content and density of HKMD samples

| Sample | Water Content (%) | Bulk Density (g/cm$^3$) | Specific Gravity (g/cm$^3$) |
|---|---|---|---|
| **19** | 71.0 | 1.75 | 2.52 |
| **17** | 68.8 | 2.01 | 2.62 |

Table 2 Basic properties of HKMD samples

| Sample ID | $W_P$ (%) | $W_L$ (%) | $I_p$ (%) | Particle size distribution (%) | | | |
|---|---|---|---|---|---|---|---|
| | | | | Clay | Silt | Sand | Gravel |
| **19** | 30.5 | 59.3 | 28.8 | 30 | 43 | 25 | 2 |
| **17** | 29.3 | 57.5 | 28.2 | 33 | 35 | 29 | 3 |

## 3. TESTING APPARATUS AND PROCEDURE

The soils were artificially sedimented at different initial water contents in order to obtain both segregated and homogeneous sediments. In settling column consolidation, initial water content of the soil-water suspension is one of the important factors controlling the settling behavior of soils (Imai 1980, Sridharan and Prakash 1999). The soil slurry is prepared with water content sufficiently high, yet maintaining the homogeneity (Sheeran and Krizek, 1971). At very high initial water contents, grain size sorting takes place, resulting in segregated sediments. In contrast, when the initial water content is below a limiting value, mutual mechanical interaction among the settling soil particles/flocs dominates, which results in homogeneous sediments.

The settling column setup by Delft Hydraulics (2007) is adopted to establish total six perspex columns with internal diameter of 130mm and height 1 m (volume=3.14×0.065×1= 0.01327 m$^3$). In this setup as shown in Figure 1, scale length is nearly 100 cm; distance between two ruler ticks is 1 cm with readable resolution 0.1 cm.

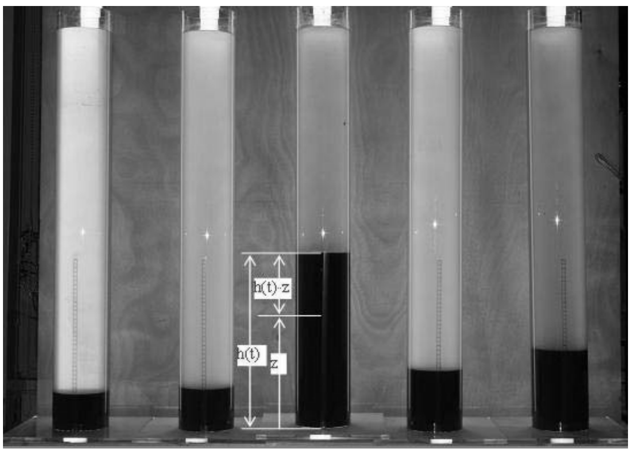



Figure 1. Column setup to for settling column tests. (advised by Delft Hydraulics, 2007)

For a soil with given concentration (weight of dry mass of the soil over 0.001 m$^3$ of soil), and for a container of a given volume of 0.015 m$^3$, the soil dry mass $M_s$ will be 1500g. The mass of water is:

$$M_w = \rho_w V_w = \rho_w (V_{total} - V_{soild}) = \rho_w (V_{total} - M_s / G_s) \quad (1)$$

where $\rho_w$ is the density of water; $G_s$ is the specific gravity of soil particles.

In order to avoid destruction of organic materials in soil during drying, the original wet soil is used to find the required dry mass. The original soil in a box is thoroughly mixed first to make the soil uniform. After this, two soil specimens are taken to find the initial water content $w_i$, of the soil. The dry density and the dry mass are calculated as follows:

$$\rho_d = \frac{\rho}{1+w} = \frac{M_s}{V} \qquad M_s = V\frac{\rho}{1+w_i} = \frac{M}{1+w_i} \qquad (2)$$
$$M = M_s(1+w_i)$$

According to the procedure illustrated in Figure 2, once the weight of wet soil is determined, the raw soil of known water content and weight was mixed with the water to form slurry, and then the water content was increased to desired value. A high-speed rotary mixer is used to prepare the slurry. The soil-water mixture was thoroughly mixed and transferred to the test jar of 0.95 m for free settling. Then, the sedimentation and subsequent consolidation starts. The level of the interface between the clear water and sediment is recorded manually at a series of time intervals: 15 sec, 30 sec, 1 min, 2 min, 4 min, 8 min, 16 min…, until the completion of the whole consolidation.

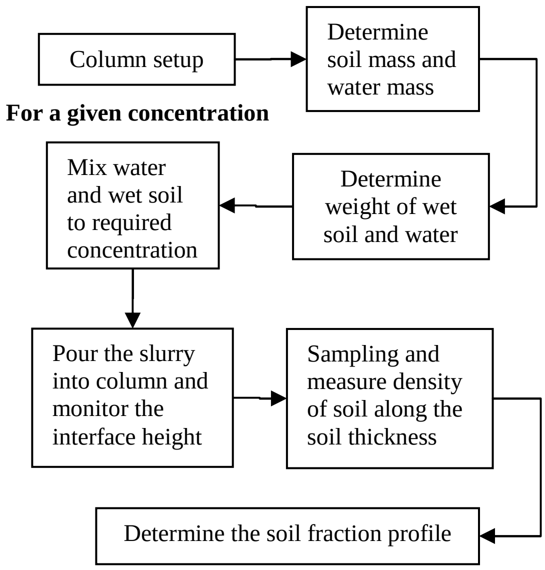



Figure 2 Flow chart of the experiment procedure

The experiments were conducted using site water with a pH of about 7.2. Two soil samples in different colors with site water were tested. By varying the initial content, segregated and homogeneous sediments were obtained for the same soil. The sediments formed were left undisturbed for a sufficiently long time to allow the self-weight consolidation to complete for all practical purposes. Results from the settling tests are presented and discussed in the following section. All the tests were carried out under the constant laboratory temperature of 20±1°C.

The time to reach full consolidation depends on the initial concentration of the sample used to fill the column. By doubling the concentration, the duration also (nearly) doubles, requiring a four times longer consolidation time. A very low initial concentration certainly results in a short experiment time, a thin final layer and a low effective stress. In these tests, the duration of time varied from about 7 days for soils with 20g/L concentration to about two months for soils with 100g/L concentration.

Delft Hydraulics (2007) used a "conductivity probe" to measure the density of the soil along the soil thickness. Using this method calibration must be done properly. In this study, a direct method is used. After free-settling for sufficient time, the water above the sediment was siphoned out and a specially made spatula (bent horizontally at end) was inserted for sampling layer by layer from the top to the bottom of the sediment. The soil is carefully transferred into glass containers. The container with the wet soil is put in an oven to dry for 24 hours at the standard temperature of 105°C and then weighed to get the dry mass. In this way, the water content distribution and hence, the void ratio profiles over the entire depth of the sediment were obtained based on equation (3).

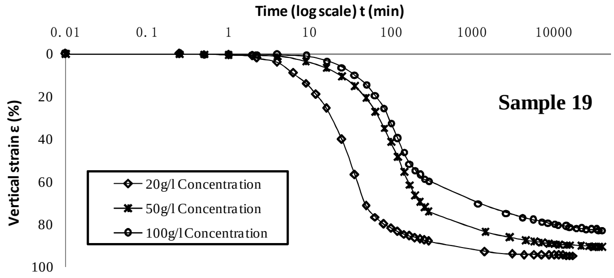
$$\begin{aligned}
&\phi = \frac{V_s}{V} \\
&\rho_d = \frac{\rho}{1+w} = \frac{M_s}{V} \Rightarrow M_s = V\frac{\rho}{1+w} \\
&\rho_s = \frac{M_s}{V_s} \Rightarrow V_s = \frac{M_s}{\rho_s}; \rho_s = G_s \rho_w \\
&\phi = \frac{V_s}{V} = \frac{M_s}{V\rho_s} = \frac{V}{V\rho_s}\frac{\rho}{1+w} = \frac{1}{\rho_s}\frac{\rho}{1+w} = \frac{1}{G_s}\frac{\rho/\rho_w}{1+w}
\end{aligned} \tag{3}$$

where $\phi$ is the soil fraction (volume of solid mass over the total volume).

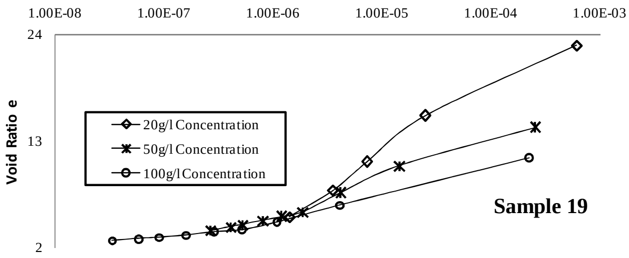
The effective stress at any depth was calculated using the effective weight of the soil mass above that depth. The segregated layers were carefully identified visually as the thin scrapped layers were taken out. Grain size analyses were carried out to confirm whether the sediments formed were homogeneous or segregated.

## 4. SETTLING BEHAVIORS AND DISCUSSIONS

Typical settling results for both samples with different concentrations (20g/l, 50g/l and 100g/l, respectively) are presented in Figure 3 and Figure 4. In Figures 3(a) and 4(a), both soils first settle quickly, and then the settling becomes slower and slower. Initially some grain size sorting takes place during settling. After the particle segregation, the self-weight consolidation begins, which makes the t-ε curve a straight line. A more flattened curve means a more homogeneous soil. The curve with higher soil concentration is obviously more flattened. Thus, initial soil concentration has critical impact on the settling behaviour. The void ratio of the sediment surface at the end of zone settling is found to be about 10.5~15.6 for sample 19, and about 8.5~13.3 for sample 17. These values are slightly larger than those reported in Been and Sills (1981).

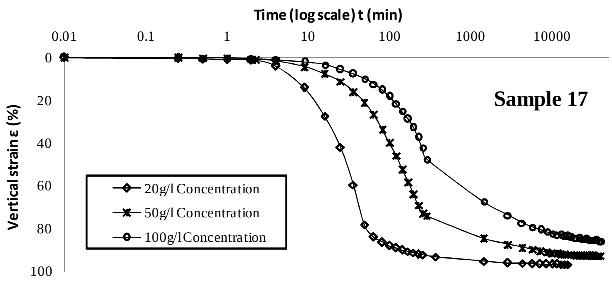



(a) Relationship between vertical strain and elapsed time

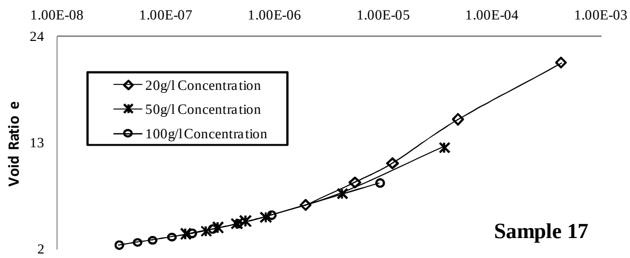



(b) Relationship between permeability and void ratio

Figure 3 Consolidation behavior of sample 19




(a) Relationship between vertical strain and elapsed time



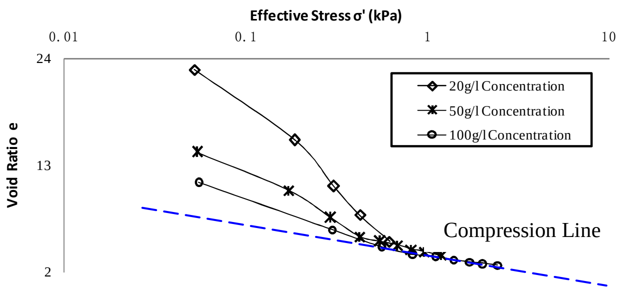

(b) Relationship between permeability and void ratio

Figure 4 Consolidation behavior of sample 17

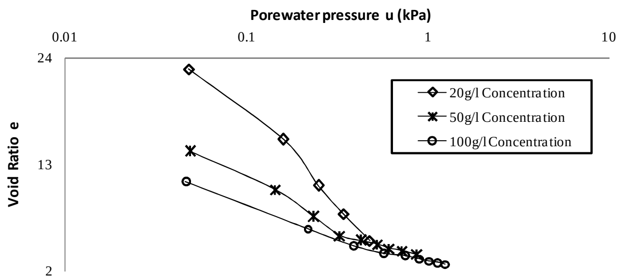
The void ratio vs. permeability curve is presented in Figures 3(b) and 4(b) in term of the void ratio over the depth of sediment. As the initial HKMD concentration is reduced (water content increased), the initial part of the curve becomes steeper. The e-k curves due to self-weight consolidation converge at a later stage, which is consistent with the findings in Sridharan and Prakash (2003).

Using the settling-column technique, the compression curve over a very low stress range can be obtained (Imai 1981; Umehara and Zen 1982; Scully 1984). Combining the void ratio profile and the effective stress variation with depth, the void ratio vs. effective stress relationship with different concentrations can be established and shown in Figure 5. Data used to plot the compression curve (i.e., $e$-log$\sigma'$ or $w$-log $\sigma'$, where $w$, $e$, and $\sigma'$ are the water content, void ratio, and the effective overburden pressure at any depth) were measured from the top of the soil sediment. In Figure 5(a), the void ratio reduces rapidly with the increase of effective stress. Also the soil concentration influences the settling curve. However when the void ratio reaches about 4, all the settling curves coincide to form a compression line.

For the HKMD slurries, water content is much higher than the liquid limit. Therefore the soil grains flocculate in water. Despite that some deformations occur due to the reduction of pore water in the settling, the pressure actually does not dissipate until the grains are in mutual contact. In Figure 5(b), variation of porewater pressure during settling is illustrated. Similarly the porewater increases slowly in the beginning, and then speeds up with the effective stress increase.



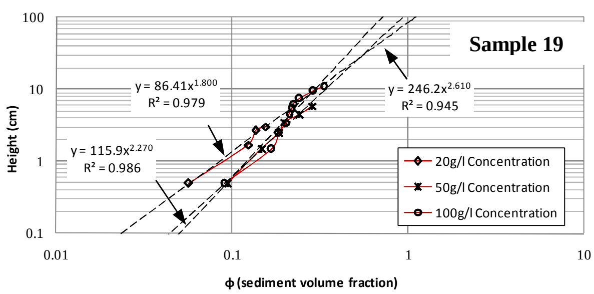

(a) Relationship between effective stress and void ratio




(b) Relationship between porewater pressure and void ratio

Figure 5 Consolidation of sample 19

The final density profiles after self-weight consolidation are exhibited in Figure 6. It can be seen in Figure 6 that the slope of soil height over volume fraction generally increases with the increase of soil concentration, which indicates more sediment at the bottom. Once fitted by power functions, straight lines are obtained in log space. All the slopes and exponents increase with the increase of soil concentration. Data in Figure 5 can be used to compute the effective stress parameter $K_\sigma$ in equation (4).



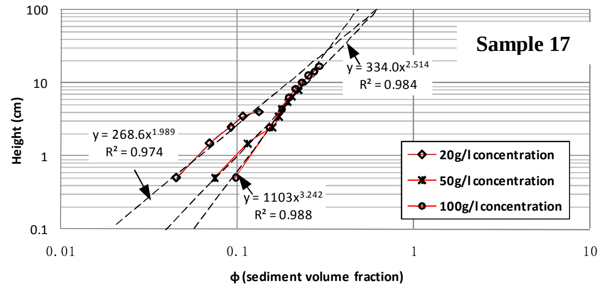




Figure 6 Relationship between sediment height and volume fraction

According to Merckelbach (2000) and Merckelbach and Kranenburg (2004a, b), the settlement of the interface *h(t)* in self-weight consolidation may be determined as:

$$\log h(t-z) = \log\left(\frac{2K_\sigma}{(D-1)(\rho_s-\rho_w)g}\right) + \frac{D-1}{3-D}\log\phi \qquad (4)$$

where $\rho_s$ is the density of the solid particles; $\rho_w$ is the density of pore water and can be taken as 1.0 Mg/m$^3$; $K_\sigma$ is stress parameter; $n$=2/(3-$D$), $D$ is the fractal dimension; $\phi$ is sediment volume fraction, namely the volume of the solid particles with respect to the total volume.

In all the settling progress, the parameter $K_K$ is a key factor governing permeability, which can further affect settling rate.

$$k = c_v m_v \gamma_w = K_k \cdot \phi^{\frac{-2}{3-D}} \qquad (5)$$

where $k$ is the permeability; $c_v$ is the coefficient of consolidation; and $m_v$ is the compressibility.

Table 3 Summary of parameters from the settling tests

| ID | Concen. | $h$ (cm) | $\xi$ (cm) | $D$ | $K_k$ ($10^{-10}$ m/s) | $K_\sigma$ (kPa) | $C_v$ ($10^{-2}$m$^2$/yr) |
|---|---|---|---|---|---|---|---|
| **19** | 20g/L | 98.18 | 1.20 | 2.60 | 0.79 | 45.08 | 7.53 |
| | 50g/L | 96.29 | 2.99 | 2.66 | 0.27 | 82.72 | 5.54 |
| | 100g/L | 99.30 | 5.98 | 2.71 | 0.07 | 133.79 | 2.80 |
| **17** | 20g/L | 98.18 | 1.20 | 2.60 | 0.79 | 45.08 | 9.80 |
| | 50g/L | 96.40 | 2.88 | 2.62 | 4.09 | 64.72 | 6.86 |
| | 100g/L | 96.57 | 5.75 | 2.63 | 1.14 | 106.46 | 3.21 |

Using equation (4), the coefficient of consolidation $c_v$ can be determined and summarized in Table 3. The value of $c_v$ is obtained based on equation (6).

$$c_v = -\frac{k(1+e)}{\rho_w g}\frac{d\sigma'}{de} = \frac{2}{3-D}\frac{K_\sigma K_K}{\rho_w g}\frac{1+e}{e} \qquad (6)$$

It can be seen from Table 3 that the coefficient of consolidation increases with the reduction of HKMD concentration. The fractal dimension however increases with the increase of concentration. Another fact can be found is that Sample 17 consolidates “faster” than Sample 19 because Sample 17 contains more sand while Sample 19 exhibits more viscous property. The value of $c_v$ slightly varies with the void ratio and thickness of soil in column.

## 5. CONCENTRATION AND SETTLING RATE

Typical settling curves in Figure 7 are used for the analysis of settling rate and accumulated settling velocity. Within the first 10min, soil particles initially convert into loose sediments, while their settling is quite slow. After that, settling of soil particles accelerates, and a clear solid-liquid interface develops at the top of the column. This occurs because discrete flocs are formed from clay or colloid particles.

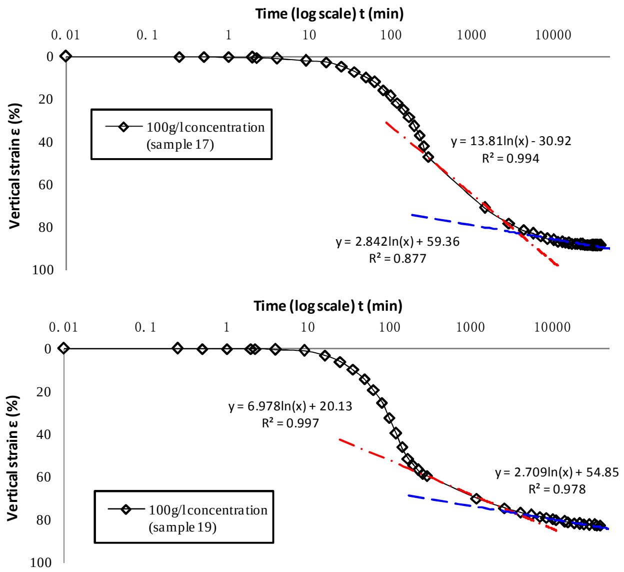



Figure 7 Curve fitting for relationship of vertical strain vs. elapsed time in self-weight consolidation

Typical curves of settling rate vs. elapsed time are demonstrated in Figure 8. It is found from the figure that the settling of sediment is closely related to soil concentration and distribution. The settling rate is dependent on soil type as well. Comparatively the soil concentration has greater influence on settling rate profile than soil type. Soil particles continuously deposit on the bottom of the column to form a bed of loose structure. Particles in sediment with high concentration may easily contact with each other, and thus the settling rate is reduced.

Following the above "zone settling" (Imai, 1980), the sedimentation gradually transforms into self-weight consolidation. The transition point becomes less clear when the soil concentration increases. Compared with the zone settling, the settling rate of the solid-liquid interface becomes slow again. Moreover, self-weight consolidation starts at the bottom while the sedimentation process starts at the top.

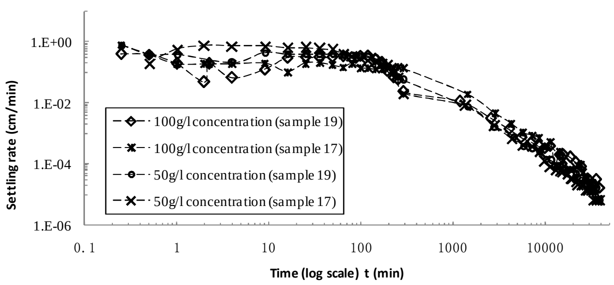



Figure 8 Relationship between settling rate and elapsed time

## 6. SECONDARY SELF-WEIGHT CONSOLIDATION

Drained creep in conventional oedometer condition has been a research interesting area with a long history (Berre and Iversen 1972; Leroueil, *et al.* 1985; Tong and Yin 2011). Mesri and Godlewski (1979) showed that $C_\alpha$ is related to the compression index $C_c$ of the soil and more precisely, that the ratio $C_\alpha/C_c$ is constant for a given soil. The $C_\alpha/C_c$ concept has been confirmed for a variety of geotechnical materials by Mesri and Godlewski (1979) and many other researchers.

For settling column condition in Figure 3(a) and 4(a), the soil continues to settle with time after the "linear region", which indicates that the structure of HKMD forms during free settling process (deposition). The above behaviour makes the void ratio smaller than that on the compression curve, which further causes creep (secondary compression) during the whole consolidation.

As illustrated in Figure 7, for sample 17 with 100g/L concentration in self-weight consolidation stage, the slope of the settling curve is determined to be 13.81 by curve fitting. Similarly to the creep behavior in oedometer condition, soils continue to settle after the self-weight consolidation. The settling rate is further reduced to 2.842 and this stage may be called the "secondary self-weight consolidation". The two corresponding slopes for sample 19 are determined to be 6.978 and 2.709, respectively.

It has been proved the $C_\alpha/C_c$ ratio for natural soils lies in a narrow range of 0.025~0.l. Once the $C_\alpha/C_c$ concept is adopted in self-weight consolidation, two ratios can be obtained: 0.21 for sample 17; and 0.39 for sample 19. More viscous soil has higher $C_\alpha/C_c$ value. The secondary compression has more significant influence on the whole self-weight consolidation.

## 7. CONCLUSIONS

The sedimentation and consolidation behaviors of Hong Kong Marine Deposits in settling column tests are investigated in the paper. The following conclusions can be drawn:

a. Typical settling curves of HKMD are obtained. The *e-k* curves due to self-weight consolidation for different soil concentrations converge at a later stage. The void ratio of the sediment surface at the end of settling is found to be 10.5~15.6 for sample 19; and 8.5~13.3 for sample 17. When void ratio reaches about 4, the *e*-log *p* curves coincide to form a compression line.

b. Soil concentration has critical influence on the settling property. In final density profile, the slope of soil height over volume fraction increases with the increase of soil concentration. The coefficient of consolidation $c_v$ is found to decrease with the soil concentration increase. For the concentration of 100g/L, the $c_v$ value is $2.80\times10^{-5}$m$^2$/yr for sample 19; and $3.21\times10^{-5}$m$^2$/yr for sample 17.

c. The settling rate profile indicates a constant settling rate at zone settling, and after reaching transition points, the settling rate decreases. Settling rate is dependent more on soil concentration than on soil type.

d. Soils exhibit viscosity in self-weight consolidation. After the primary self-weight consolidation, the settling rate is further reduced in the "secondary self-weight consolidation". Soil with more viscosity has higher $C_\alpha/C_c$ value.

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