Effect of cement/lime additive on clogging of vacuum-drain consolidation

Hongtao Fu*, and Jinchun Chai1

1 Departments of civil engineering and architecture, Saga university, Saga city, 840-8502, Japan

*Corresponding author’s e-mail: fuhongtao2012@163.com

Abstract. All addressing dredged slurry with high water content has become a critical issue for sea channel dredging and maintenance. Vacuum consolidation is an effective method for improving dredged slurry. Conventionally, the effect of a cement/lime additive on the clogging of vacuum-drain consolidation has been elusive and unclear. To study the effect of vacuum consolidation for soft clayey soils with high initial water content, a set of oedometer tests and vacuum consolidation tests were conducted by adding small amounts of cement/lime. The results indicated that the structural yield stresses of soils with cement and lime were 94.29% and 48.57% higher, respectively, than those of untreated soils. The final surface settlement of lime additive at the end of the tests was 59.68 mm less than that of soil without any additives, and that of the cement additive was close to that of soil without any additives. However, the dissipation of pore water pressure with cement/lime was more than that with untreated soil. The water content was higher when using cement/lime additive, but the vane shear strengths were greater than that of untreated soil at the end of the tests. In addition, the values of $k_{e2}/k_{e1}$ ($k_{e2}$ is a permeability at approximately 0.2 m away from the drain, and $k_{e1}$ is a permeability close to the drain) above and below the drain were less than those for untreated soil. This clearly indicated that the presence of lime/cement was effective in alleviating the problem of clogging around the drain. This study could provide a good technical guideline for vacuum-drain consolidation projects combined with the use of cement/lime.

1. Introduction
In general, dredged slurry comprises very soft soils that have very low shear strengths, and natural water content is higher than their liquid limit. Therefore, it is challenging to deal with these types of soils, and soil improvement is imperative for upper structure construction [1-3]. In particular, addressing this issue is critical for sea channel dredging and maintenance. A vacuum preloading method was altered to become an available method to improve these types of soils in geotechnical engineering.

The vacuum preloading method has evolved into a mature and efficient technique since the time of its development in the early 1950s[4]. Further, it has become an effective soil improvement method for soft clay, worldwide[5-11]. As compared to traditional preloading, vacuum preloading has several advantages: no heavy machinery is required, no fill material is needed, and the construction period is generally shorter[12]. Prefabricated vertical drains (PVDs) are usually installed in soil to accelerate the consolidation rate of dredged slurry under a vacuum pressure. In recent years, prefabricated horizontal drains (PHDs) have been used to accelerate the consolidation of embankments with clayey backfills[13-14] and the self-weight and vacuum pressure-induced consolidation of dredged mud[15]. There has also been research proposing an approximate consolidation theory for a surface layer under vacuum pres-
sure applied at a shallow depth through PHDs\textsuperscript{[16]}. The vacuum preloading methods with PVDs or PHDs are referred to as physical treatments for dredged slurry, based on the underlying principles of these techniques. During physical treatments, the pore water is drained out when a vacuum pressure is applied to the soft clayey soils. However, clogging problems caused by a gathering of the soil particles and a non-uniform consolidation\textsuperscript{[17]} in the PVDs or PHDs remain critical challenges for improving such techniques. Therefore, researchers have paid attention to the chemical pretreatment of dredged slurry, and have combined this with vacuum preloading to improve its consolidated behaviour\textsuperscript{[18-19]}.

Chemical pretreatment can be interpreted as altering the microcosmic structure of soils by adding chemical additives, to influence specific engineering properties further. For example, chemical pretreatment may alter the arrangement of particles, particle groups, and pore spaces in a soil, generally termed as the fabric\textsuperscript{[20]}. The fabric of soils varies greatly depending on the mineralogy of the soil, especially while interacting with various stabilising agents, such as chemical waste stabilisers\textsuperscript{[21-23]}. When it is utilised in combination with a vacuum preloading method, chemical pretreatment mainly acts as a catalyst for flocculation. Because of the effects of flocculation, the arrangement of soil particles is altered, thereby forming bigger flocs so that soil particles can settle faster, and in favour of forming of a flocs net-frame structure\textsuperscript{[24]}. The effect of FeCl\textsubscript{3} on the consolidation property of sewage sludge from a lagoon was reported by Lin et al.\textsuperscript{[25]}, and the body volume of the sewage sludge treated by the vacuum preloading method was reduced significantly. A vacuum preloading method combined with polyacrylamide (PAM), based on the functions of PAM, was also investigated\textsuperscript{[26-27]}. In this regard, the two most widely-used methods today involve lime and cement additives. For example, the 4th runway of Haneda airport in Japan was built by mixing cement with slurry\textsuperscript{[28]}. This leads to an improvement of the soil workability and mechanical properties through the development of chemical processes, and involves four distinct processes: cation exchange, flocculation and agglomeration, cementitious hydration, and pozzolanic reaction. Cement provides the compounds and chemistry necessary to achieve all four processes. Lime can accomplish all the processes except for cementitious hydration\textsuperscript{[29]}. A combination of vacuum preloading and lime treatment for improvement of dredged fill has been proposed\textsuperscript{[30]}. With the presence of lime, small particles aggregate into larger clusters. Notable results have been obtained using additive agents that significantly mitigate the risk of clogging around drains and increase the soil permeability, thereby improving the efficiency of vacuum consolidation.

The focus point of these studies is frequently confined to the treatment phase of such soft clay soils, before these materials are sufficiently stiff for project utilisation. The profound impact of the presence of cement/lime with small proportions in dredged slurry (combined with vacuum preloading) have not yet been assessed. Therefore, in this study, lime and cement were used as a stabilising additive and mixed with soft clay. The effects on consolidation properties are first determined by an oedometer test. Then, the combination is consolidated by vacuum preloading with a PHD to determine the effects on the vacuum consolidation properties of soft clay. Further, the effect of the cement and lime on clogging around the PHD is determined by analysis of the dissipation of pore water pressure and the distribution of water content.

2. Tested materials
The dredger slurry sample used in this study was obtained from the Kasegawa river bed at Kubota of Saga, a city located in the northwestern region of Kyushu, Japan. Here, sea water from Ariake Sea can enter the river. The dredger slurry sample has a high natural water content (154%), liquid limit (109.2%), plastic limit (44.3%), low strength, and high compressibility. Based on the USDA classification system\textsuperscript{[31]}, the dredger slurry sample of this area is classified as soft clay in this study.

3. Specimen preparation
A predetermined amount of additive (cement or lime) was directly added into the dredged slurry, and then the specimen was mixed thoroughly by a laboratory mixing machine to obtain a cement-soil-water mixture or lime-soil-water mixture. The chemical compositions of the cement and quicklime used are listed in Table 2. The cement used is called US10, a commonly-used cement in Japan for soft
ground improvement. The dredger slurry sample, cement-soil-water mixture, and lime-soil-water mixture will be termed DS, CM, and LM soil, respectively.

The basic characteristics of the DS, CM, and LM soils are listed in Table 1. As shown in Table 1, a small amount of cement or lime was added into the soil and the specific gravity and void ratio relatively decreased, but the hydraulic conductivity increased. The particle size distributions of the DS, CM, and LM soils are shown in Fig. 1. It can be seen that, with a limited quantity of cement or lime, the silt particles increase from 32.1% to 34% or 38.3% as a result of flocculation-agglomeration. That is, after the reduction in water layer thickness, the soil particles become closer to each other, causing the soil texture to change and to form larger aggregates.\[32-33\].

Table 1. Basic characteristics of the dredged slurry (DS), cement-soil-water mixture (CM) and lime-soil-water mixture (LM) soils

| Soil samples | DS   | CM   | LM   |
|--------------|------|------|------|
| Amounts of additive (by dry weight), c (%) | 0    | 2% cement | 2% lime |
| Specific gravity, Gs | 2.542 | 2.49 | 2.453 |
| Void ratio, e_0 | 3.116 | 3.304 | 3.354 |
| Compression index, Cc | 0.9   | 1.03  | 1.07 |
| Hydraulic conductivity, k (×10⁻⁹m/s) | 1.93 | 3.04 | 3.09 |

Table 2. The chemical compositions of the cement and quicklime used

| Additive      | CaO% | SiO₂% | Al₂O₃% | Fe₂O₃% | MgO% | K₂O% | Na₂O% | SO₃% |
|---------------|------|-------|--------|--------|------|------|-------|------|
| Cement        | 60.7 | 19.2  | 4.8    | 2.5    | 1.2  | -    | < 0.1 | < 0.1 |
| Quicklime     | 92   | 1.4   | 0.6    | 0.3    | 1    | < 0.1| < 0.1 | < 0.1 |

Figure 1. The particle size distributions of the dredged slurry (DS), cement-soil-water mixture (CM) and lime-soil-water mixture (LM) soils

4. Oedometer test
Oedometer tests were performed on the dredged clay samples (CM, LM, and DS soil) at the engineering laboratory of Saga University in Japan. The consolidation rings containing the samples were placed in the oedometer cell with filter paper and porous stone on both ends of the samples. The loading schedule consisted of mainly doubling the normal vertical stress with each loading (load increment ratio = 1). Under this condition, the samples experienced an incrementally higher pressure. Incremental consolidation pressures were applied to the specimens based on the consolidation test (incremental loading test). The incremental consolidation pressures were 4.9, 9.8, 19.6, 39.2, 78.5, 157, 314, 628, and 1256 kPa.
4.1. End of primary consolidation
The end-of-primary (EOP) consolidation curves of all samples are shown in Fig. 2. It is apparent that the consolidation curve from the DS sample was below the consolidation curves of the DM and LM samples, and the compression curves of the DM and LM samples are steeper than that of the DS soil. These indicate that adding cement or lime has not only produced cementitious compounds to induce the preconsolidation pressure, but also changed the nature of the dredged slurry.

4.2. Structural Yield Stress
Structural yield stress ($P_y$) is a critical parameter for evaluating the structural properties of soil. The structural characters of the cement-soil-water mixture and lime-soil-water mixture were predicted considering the aggregation mechanism induced by cement or lime. Therefore, $P_y$ was obtained based on Butterfield’s double logarithm coordinates method (Fig. 3). The results indicated that an overall increase in $P_y$ occurred because of the addition of cement or lime (Fig. 3). A sample without any additives presented a lower value of $P_y$, and the value of $P_y$ increased from 10.5 to 15.6 kPa for LM soils, and to 20.4 kPa for DM soils. The structural behaviour was explained by the cation exchange in the short term, as bivalent calcium ions ($Ca^{2+}$) are replaced by monovalent cations, thereby reducing the repulsion forces and the thickness of the diffused water layer, and finally serving as bonding hinges among the dredged clay particles. The data demonstrated the predictions for the effects of cement/lime on the structural behaviour, and may be considered in a technique to rapidly form a crust for ultraclay preliminary treatment.

![Figure 2](image1.png)

Figure 2. The end-of-primary (EOP) consolidation curves of all samples

![Figure 3](image2.png)

Figure 3. The $\ln(e+1)$-$\log P$ and structural yield stress curves
5 Vacuum consolidation test

5.1 Testing equipment
The model test setup is illustrated in Fig. 4. The inner dimensions of the model box are: 0.60 m in length, 0.30 m in width, and 1.0 m in height. The model box is made of steel. The technique adopted for applying vacuum pressure is called the vacuum-drain method, which applies vacuum pressure to the PHD and uses the plastic membrane as an airtight layer. The PHD was installed horizontally into the middle of test model to accelerate the rate of consolidation. The PHD used in the test was made of non-woven geotextile with a cross section of 150 mm × 5.5 mm. The PHD included a rubber cap and 6 mm diameter geosynthetic tube on the side of the rubber cap, and is designated as capped PHD. The geosynthetic tubes of the capped PHD were connected together by plastic T-couplings, and then to a vacuum generation system.

![Diagram of the model test set-up](image)

5.2 Testing procedure
The remoulded soil samples (DS, CM, and LM soil) with a desired content of approximately 200% were placed into the model, layer by layer. The PHD was installed horizontally at 0.5 m from the bottom of the model, and the piezometers, P1 and P2, were installed at 0.75 m and 0.25 m from the bottom of the model, respectively, as shown in Fig. 4. After the thickness of the model ground reached approximately 1.0 m, 2 layers of geotextiles were placed on the top of the model as the airtight layer. During the test, a settlement gauge was set on the plastic membrane to measure the settlements, and the piezometers P1 and P2 were installed to monitor the excess pore water pressure variations. When the test stopped, the water content and vane shear strength were measured, and a total of 18 soil samples along horizontal and longitudinal directions were obtained by a cutting ring (height: 60 mm, dimension: 75 mm). A custom-made vane shear apparatus was utilised to measure the vane shear strength value of the model.[34]

5.3 Case tested
For testing the CM and LM soils, a vacuum pressure of −60 kPa was applied and maintained for 34 days. However, for testing the DS, a vacuum pressure of −60 kPa was applied and maintained for 70 days.

5.4 Calculated method
A calculated method for analysing the vacuum consolidation induced by PHDs, as proposed by Chai et al. [16] was used for this study. It assumed that the vertical spacing between adjacent PHDs \(S_v = 0.5\) m, and the horizontal spacing \(S_h = 0.15\) m. Here, \(f_p < f_a\), where \(f_p\) is a ratio of the time corresponding to the average degree of consolidation \((U) = 50\%\) obtained from a finite element analysis (FEA) and from the theory of axisymmetric unit cell consolidation. Further, \(f_a\) is a ratio of the time corresponding to U
50% obtained from the FEA and from the theory of the plane strain unit cell consolidation. Thus, the study was suitable for applying the plain strain unit cell consolidation theory \(^{35-36}\). In their solution for a plain strain unit cell, \(U\) is given by the following:

\[
U = 1 - \exp\left(-\frac{8T}{\mu}\right)
\]

(1)

\[
T = \frac{ct}{4B^2}
\]

(2)

\[
\mu = \frac{2}{3} + 2\left(\frac{k}{k_s} - 1\right)(b_s - b_w^2 + \frac{b_w^3}{3}) + \frac{4l^2 \cdot k}{3q_{wp}B}
\]

(3)

where \(c\) is the coefficient of consolidation of the soil, \(t\) is time, and \(k\) and \(k_s\) are the hydraulic conductivities of the clayey soil and smear zone, respectively. Term \(l\) is the drainage length of the drain, \(B\) is half the width between the plane strain drain sheets, \(b_s\) is the half width of the smear zone, \(b_w\) is half the width of the drain, and \(q_{wp}\) is the discharge capacity of the drain in the plane strain unit cell, which is calculated as follows\(^{[37]}\):

\[
q_{wp} = \left(2B / \pi r_e^3\right)q_w
\]

(4)

In (4), \(q_w\) is the discharge capacity of the drain in the axisymmetric unit cell, and \(r_e\) is the radius of the axisymmetric unit cell. In the case of a plane strain unit cell, the distribution of \(U\) with vertical distance \((z)\) from the centre of the drain is given by:

\[
u = (2Bz - z^2)\frac{3(1-U)u_0}{2B^2}
\]

(5)

Meanwhile, the settlement \((S)\) used in following section can be calculated by:

\[
S = H \cdot \frac{c^*}{1+e_0} \cdot \log \frac{P}{P_0}
\]

(6)

In the above equation, \(H\) is the height of soil layer, \(c^*\) is the equivalent coefficient of consolidation\(^{[16]}\), \(P\) is effective stress, and \(P_0\) is initial effective stress.

The settlement and pore water pressure from the model test were calculated to evaluate the effectiveness of vacuum consolidation with PHD performance. The evaluated/back-analysed parameters for PHD consolidation are given in Table 3. The value of DS is two (2) times the equivalent width of the cross-sectional area of the model mandrel.

Table 3. The evaluated/back-analysed parameters for prefabricated horizontal drain (PHD) consolidation

| Soil samples | CM  | LM  | DS  |
|--------------|-----|-----|-----|
| Coefficient of consolidation, \(C^*\)(\(\times10^{-7}\) m\(^2\)/min) | 8.89 | 17.33 | 6.56 |
| half width of the smear zone in plane strain unit cell, \(b_s\)(\(\times10^{-2}\)m) | 7.78 | 7.78 | 15.55 |
| hydraulic conductivity, \(k\)(\(\times10^4\) m/s) | 2.0 | 1.72 | 1.37 |
| Initial efficient stress, \(P_0\) (kPa) | 1 | 5.1 | 0.5 |
| half width of a plane strain unit cell, \(B\)(m) | 0.25 | | |
| half width of the drain in a plane strain unit cell, \(b_w\)(\(\times10^{-2}\)m) | 3.89 | | |
| drainage length, \(l\)(m) | 0.5 | | |
| discharge capacity of the drain, \(q_{wp}\)(\(\times10^{-6}\) m\(^3\)/s) | 5.3 | | |
| \(k/k_s\) | 10 | | |

5.5. Results and analysis

5.5.1. Settlement

The measured and calculated surface settlement curves at the centre of all tests are shown in Fig. 5. In testing CM, it is obvious that the settlement rate was faster than that in testing DS, and the surface settlement was almost constant after approximately 26 days of elapsed time. It can be seen that the surface settlement at the end of tests for CM and LM was up to 248.67 mm and 174.5 mm, respectively.
In addition, the final surface settlement of the LM test was 59.68 mm less than that of the DS test. This was attributed to long-term pozzolanic reactions. The silica and alumina that exist in the soil minerals become soluble and free from the soil. The reaction between the released soluble silica and alumina and the calcium ions from lime hydration creates cementitious materials such as calcium silicate hydrates (CSH) and calcium aluminate hydrates (CAH)\(^{38-40}\). Owing to the presence of the cementitious materials, the inter-aggregate pores begin to fill, and contribute to a decrease in compression. As these model tests can only be conducted for a limited time, the calculated method was adopted for further investigation of the long-term effect on the final settlement. It can be seen from Fig. 4 that although the calculated rates of settlement in the test DS are slightly lower than the measured values, it is generally considered that the calculated method yields an acceptable settlement curve. As for the settlement in the CM and LM tests, the calculated curves yielded a good agreement with the measured values. For the CM test, the settlement at approximately the 70th day was close to the DS test, and was attributed to the cementitious materials further filling the inter-aggregate pores.

![Figure 5](image.png)

**Figure 5.** The surface settlement curves at the centre of all tests

5.5.2. *Variations of pore water pressure*

The variations of the pore water pressures \((u)\) for P1 and P2 from all tests are presented in Figs. 6(a) and (b), respectively. The dissipation-measured \(u\) values of P1 were approximately 26.88, 42.9, and 4.8 kPa for the CM, LM, and DS tests, respectively. Meanwhile, the values of P2 were approximately 31.5, 43.2, and 13.7 kPa for CM, LM, and DS tests, respectively. With a small amount of additives, it was distinctly observed that the dissipation of the pore water pressure was greater than that of slurry without any additive. It was associated with a higher concentration of Ca\(^{2+}\) and qualitatively a possibly thinner diffusive double layer\(^{20, 41}\), and under identical microstructure conditions they may tend to exhibit a higher permeability and move freely in the pore water. For the CM test, vacuum leakage occurred at approximately 26 days of elapsed time (Fig. 7). In addition, the variations of pore water pressures were calculated closely, as compared in Fig. 6(a) and (b). For the results from the CM and LM tests at P1 and P2 for a longer elapsed time, the dissipations of pore water pressures were still greater than in that of slurry without any additives. In a sense, the presence of lime and cement in small proportions in dredged slurry can improve the vacuum consolidation, and partially relieve the influence on the PHD clogging.
5.5.3. Water content

Fig. 7 presents the measured and predicted average water content \((w)\) profiles at the centre of all tests at the desired measuring points. The values of \(w\) are also indicated in the figures. Here, \(w\) increases with increasing distance away from the PHD. The total average \(w\) reduced from the initial 200\% to 119.32\% in the CM test, and 125.23\% in the LM test. In addition, the average \(w\) at the end of the CM and LM tests were 4.09\% and 9.61\% greater than that in the DS test, respectively (without peaking). The cementitious compounds produced by the lime-slurry reactions are CSH and CAH, which are composed of the solid products of hydration and water\(^{[42-43]}\). The reactions are as follows\(^{[44]}\):

\[
\text{Ca(OH)}_2 + \text{SiO}_2 \rightarrow \text{CaO} - \text{SiO}_2 - 2\text{H}_2\text{O} \quad (7) \\
\text{Ca(OH)}_2 + \text{Al}_2\text{O}_3 \rightarrow \text{CaO} - 2\text{Al}_2\text{O}_3 - 3\text{H}_2\text{O} \quad (8)
\]

The above equations indicate that more water-containing agents are generated in a pore with cement and lime content. This is the reason why there is a higher water content in the treated soil. In addition, relatively speaking, the CM soils had a higher concentration of pore water and qualitatively a possibly thinner diffuse double layer as compared with LM soils\(^{[20, 41]}\), hence, they possess a lower final average \(w\).

The Taylor equation\(^{[45]}\) was used to consider the permeability variation with the void ratio reduction:

\[
k = k_0 \times 10^{-(\varepsilon_0 - \varepsilon)/c_k}
\]
In the above, $e_0$ and $e$ are the initial and current void ratios, respectively, $k_0$ and $k$ are the permeabilities corresponding to void ratios $e_0$ and $e$, respectively, and $c_k$ is a constant.

Based on the results of the above oedometer tests, the variation of permeability with the void ratio can be expressed well using the Taylor equation (equation (10)) with values of $c_k = 0.28e_0$, $0.31e_0$, and $0.27e_0$ for the CM, LM, and DS soils, respectively (Fig. 8). The initial value of permeability of the model ground was extrapolated from the oedometer test results for all cases. Moreover, the final values of permeability were extrapolated and calculated based on the variation of water content. The values of $k_2/k_1$ ($k_2$ is a permeability at approximately 0.2 m away from the drain, and $k_1$ is a permeability close to the drain) above and below the PHD were obtained, and are listed in Table 4. It can be seen that the values of $k_2/k_1$ for the CM and DS soils were less than that for DS soil, i.e. $k_2/k_1 < k/k_s$, indicating that the presence of lime/cement in small proportions in dredged slurry can effectively alleviate the problem of clogging around the PHD. This exhibits good agreement with the above analysis of pore water pressure.

![Figure 7. The measured and predicted average water content (w) profiles at the centre of all tests](image)

![Figure 8. e-k relationship](image)
Table 4. The values of $k_{e2}/k_{e1}$ above and below the PHD

| Soil samples | Initial $k_s$ | $k_{e2}/k_{e1}$ above PHD | $k_{e2}/k_{e1}$ below PHD |
|--------------|--------------|---------------------------|---------------------------|
| CM           | 10           | 2.37                      | 3.47                      |
| LM           | 10           | 1.87                      | 2.32                      |
| DS           | 2.65         |                           | 4.51                      |

5.5.4. Vane shear strength

Fig. 9 presents the measured and predicted average vane shear strength ($S_u$) profiles at the centre of all tests at the designed measuring points, and after 34 days. The initial $S_u$ value of the soil was close to 0 kPa. The $S_u$ values for all tests were just opposite to those observed for the water content distribution. It can be seen that the final average $S_u$ value decreases with the distance away from the PHD. The final average $S_u$ values in the CM and LM tests are 10.57 and 15.11 kPa, respectively, which are 3.14 and 5.35 times greater than that in the DS test, respectively. In particular, the $S_u$ of the LM test is the largest, as shown in Fig. 7. This is attributed to a higher concentration of Ca\(^{2+}\) in the test LM\(^{[46]}\), and is helpful to formation of cementation products by pozzolanic reaction.

![Figure 9. The measured average vane shear strength ($S_u$) profiles at the centre of all tests](image)

6. Conclusions

In this study, a series of experimental tests have been carried out to investigate the effectiveness of lime or cement on dredger fill, as combined with a vacuum preloading method. Oedometer tests were conducted to evaluate the consolidation behaviours of dredged slurry pretreated by lime or cement. Then, to perform an analysis on the effects of cement or lime on dredger fill combined with the vacuum preloading method by using a PHD, three groups of vacuum preloading model tests were performed. The following conclusions can be drawn:

1) The structural yield stress of samples with cement or lime were 94.29% and 48.57% higher, respectively, than those of untreated samples.

2) The final surface settlement of slurry with lime additive at the end of tests was 59.68 mm less than that of slurry without additives because of pozzolanic reactions, and the settlement of slurry with cement additive at approximately the 70th day was close to that of slurry without additives. These results are attributed to the cementitious materials further filling the inter-aggregate pores.

3) With a small amount of lime or cement additives, the dissipation of the pore water pressure was greater than that without any additive, and the values of $k_{e2}/k_{e1}$ above and below the drain were less than that of untreated soil, clearly implying that the presence of lime/cement can effectively alleviate the problem of PHD clogging.

4) For all soils, the water content increases with increasing distance away from the PHD. A lower final average water content of slurry with cement additive and a higher value for slurry with no addi-
tives were obtained as compared with the slurry with lime additive. However, the vane shear strength of slurry with lime additive at the end of the test is the largest, owing to a higher concentration of Ca$^{2+}$.

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