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Laboratory Study on Vallejo and Scovazzo's Methods in Estimating the Rheology Parameters of Bentonite and Kaolinite Muds

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Abstract

The mud undrained shear strength and viscosity are the essential parameters in understanding the behavior of mudflow. One of the laboratory test methods to estimate the undrained shear strength and viscosity is Vallejo and Scovazzo's cylinder strength meter test (CSMT) and flume channel test, respectively. This paper compares the undrained shear strength of kaolin and bentonite muds obtained from the CMST to those obtained using the fall cone and mini vane shear tests and also studies the scale effects in the flume channel test in measuring the mud viscosity at a 20^o to 40^o slope angles and at various liquidity indexes. The results exhibit that CMST could estimate the undrained strength of mud as low as 0.45 kN/m^2 with a liquidity index of up to 5.93. Then, the reduction of the size of the flume channel by half resulted in a mud viscosity of about 2.3 times higher.

Introduction

Mudflow is a very rapid to extremely rapid downslope movement of a soil-water mixture in which at least 50% of the mixture is characterized by fine-grained soil whose water content is equal to or exceeds the liquid limit (LL) [1–3]. The common trigger for mudflows is high-intensity rainfall that destabilizes a steep slope (20^o – 45^o) [2,4]. During rainfall, mudflow is generally initiated as a slide [2,4]. As the water content of the soil (w) increases up to its LL value due to the increase in the amount of rainwater that seeps through soil pores, the soil phase changes from a plastic state to a viscous liquid state followed by the decreased soil frictional strength. The soil in this state begins to change shape and flows rapidly down the slope like a flowing liquid because the yield stress of the soil-water mixture, or so-called mud (τ_y) is smaller than the gravitational force-induced shear stress (τ) [2,3]. Figure 1 illustrates the mechanism of mudflow drawn by Widjaja & Lee [2].

In a rheological approach, mud, especially during a mudflow, often behaves in a manner akin to a Bingham plastic material considering the mechanics of mudflow in Figure 1 [3,5]. Bingham plastic material is a type of non-Newtonian fluid that has a τ_y value, which means that it does not flow until a certain threshold stress, namely τ_y in this case, is exceeded. Once the τ_y value is surpassed, the material begins to flow like a viscous fluid [6]. However, it is also worth noting that the viscosity (η) of the mud also provides some resistance to the mud flow. Viscosity in a nutshell describes how easily a fluid can deform or move in response to an applied force or stress. Figure 2 illustrates how the Bingham plastic material models the real material shear stress-shear rate behavior. As shown in Figure 2, the shear stress-shear rate behavior of the Bingham plastic material is depicted by a linear relationship with the intersection at the ordinate indicating the τ_y value ($\tau_{y,B}$) and the slope indicating the η value. Bingham plastic material expresses the relationship between τ, τ_y, η, and the shear rate ($\dot{\gamma}$) in a linear relationship as follows:

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$$\tau = \tau_y + \eta\dot{\gamma} = c_u + \eta\left(\frac{dv}{dy}\right) \quad (1)$$

where dv/dy is the velocity gradient of the mud with a certain thickness, denoted as y, and c_u is the undrained shear strength of the mud. Note that τ_y of mud is often assumed to be equal to the undrained shear strength of the mud (c_u) because the main component of mud is fine-grained soil [7,8]. Therefore, as described in Equation (1), it is important to obtain the c_u and η values of mud for understanding and simulating the mudflow behavior, which can later be used in designing infrastructure in a mudflow-prone area and implementing strategies to reduce the risks associated with mudflow events.

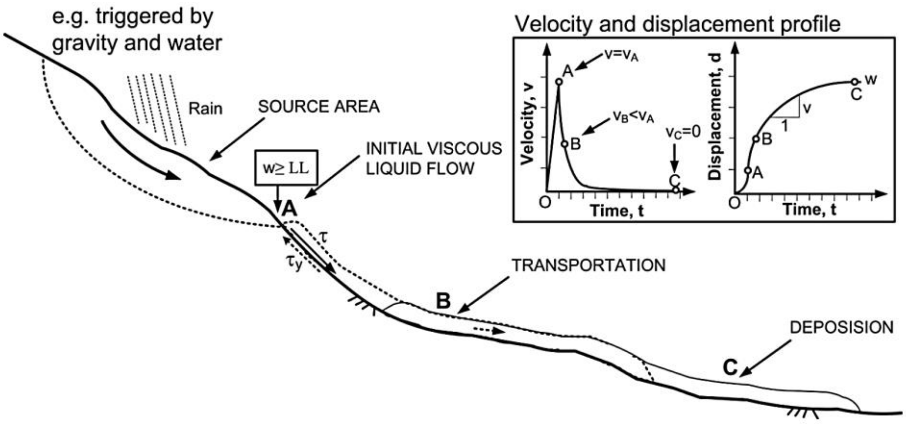

Figure 1. Mechanics of Mudflow [2]

Despite the importance of the c_u and η parameters, measuring the magnitudes of c_u and η of mud samples is fairly challenging. Soil that is under the viscous liquid state has a very low shear strength causing the inability to stand on its own weight. This results in difficulties in obtaining the undisturbed mud samples and/or reconstituting the mud sample [8]. Thus, conventional tests such as triaxial and direct shear tests cannot be conducted. Then, other laboratory tests, such as the fall cone penetrometer test (FCT) and vane shear test (VST) are used to estimate the c_u value of mud. However, those tests are also still facing some limitations, for instance, the maximum cone penetration in conventional FCT is 25 mm, while in viscous liquid soil, the required penetration may exceed 25 mm. Vallejo & Scovazzo in [8] introduce a relatively simple procedure to measure the mud c_u value, that is the cylinder strength meter test (CSMT). According to test results in [8], CSMT could measure the c_u value of kaolin mud with a liquidity index (LI) ranging from 0.7 to 1.7 at which the state of the kaolin mud was between the plastic state and the viscous liquid state. Despite having an advantage in measuring the c_u value of soil under a viscous liquid state, limited research studies the application of CSMT. Thus, the first objective of this research is to compare the c_u value of kaolin and bentonite muds obtained from the CSMT to the c_u value obtained from the FCT and laboratory miniature vane shear test (mVST).

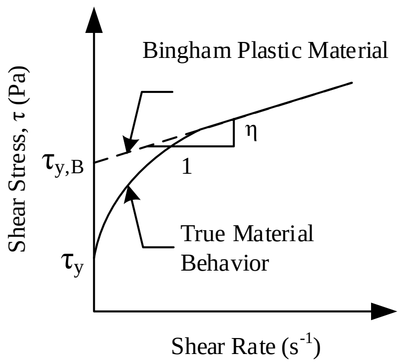

Figure 2. Comparison between Bingham Plastic Model and Real Material Behavior (After [7])

Similar issues are also encountered when measuring the η value of mud. The conventional laboratory test to measure η value is a viscometer. The viscometer is however used for measuring the η value of viscous liquid materials with LI greater than 2, but not for materials under plastic conditions [3,7]. Some previous studies ([3,6,8,9]) proposed different test methods to measure η of mud, for instance by using flume channel test, FCT, and flow box test (FBT). Vallejo & Scovazzo [8] used a transparent Plexiglass® flume to simulate a mudflow traveling down a slope with a certain degree of inclination. Then, the η value of mud is estimated based on the measured velocity and displacement of the mud. Compared to the other tests, one of the advantages of Vallejo & Scovazzo's method is that the test mimics the actual mudflow shear failure. However, the results obtained from this method are comparatively high for initial viscosity [3]. Moreover, few studies discuss Vallejo & Scovazzo's method in measuring the η of mud. Therefore, another objective of this research is to study the scale effects on the η value of kaolin and bentonite muds obtained from Vallejo & Scovazzo's flume channel test.

This research then follows the following structure. The test procedures of mVST, FCT, CSMT, and Vallejo & Scovazzo's flume channel test were first reviewed. Second, the laboratory tests on the index properties of soils and Atterberg's limits tests were conducted on two types of soil, which are kaolin and bentonite clays. Third, the mVST, FCT, and CSMT were performed to find the c_u values of kaolin and bentonite muds with various LI values. Fourth, flume channel tests were conducted on the kaolin and bentonite mud under different LI values and slope inclination to vary the shear stress acting on the mud sample. Eventually, all of the test's results were compared to the previous studies and discussed.

Laboratory Test on Mud Undrained Shear Strength and Viscosity

Fall Cone Penetrometer Test

The fall cone penetrometer test (FCT) is a well-known method used to assess the consistency by estimating the soil liquid limit (LL) and plastic limit (PL), and also the c_u value of fine-grained soils, especially cohesive soils such as clays and silts. The main component of the apparatus is a cone-shaped penetrometer with a standardized geometry of 30° apex angle and weight of 0.79 N or 80 g. The cone is attached to a rod, and the entire assembly is allowed to fall freely for 5 s into the soil sample [10]. Figure 3 shows the fall cone apparatus used during the test. The LL and PL values are defined as the water content of the soil sample at a 20 mm and 2 mm 30° cone penetration depth for 5 ± 1 s, respectively [10,11]. Then, the c_u value of fine-grained soils can be estimated using the following equation proposed by Hansbo [12]:

$$c_u = k\frac{W}{d^2} \tag{2}$$

where k is the cone factor, W is the weight of the fall cone, and d is the cone penetration depth. Note that the k value used in this study was 1.33 for a standard cone apex angle of 30° with a cone weight of 0.79 N, and the test followed a standard code of BS 1377:1990 [10].

Figure 3. Fall Cone Apparatus

Laboratory Miniature Vane Shear Test

The laboratory miniature vane shear test, denoted as mVST is a rapid method used to determine the c_u value of undisturbed or remolded fine-grained clayey soils. This test is especially applicable to soils with c_u value less than 100 kN/m^2 [13]. The main apparatus includes a cylindrical soil specimen container, a miniature vane device, and a torque measurement system. The testing mechanism of mVST is similar to the field vane shear test (VST) where the miniature vane is inserted into the soil specimen, and rotational force (torque) at a constant rate is applied to generate shear in the soil specimen. In this study, a 12.7 mm × 12.7 mm four-bladed vane was rotated at constant rotation of about 6° to 12° per minute. Figure 4 depicts the mVST apparatus used in this study. Then, the c_u value of the soil sample is calculated based on the torque applied and the dimensions of the vane by using the provided standard formulas as follows:

$$c_u = \frac{1000M}{K} = \frac{1000M}{\pi D_v^2\left(\frac{H}{2}+\frac{D_v}{6}\right)} \tag{3}$$

where M is the result of multiplying the calibration factor in Nmm per degree by the maximum angular rotation in degrees, K is a constant, D_v is the overall vane width measured to 0.1 mm, and H is the vane length measured to 0.1 mm. The M value used in this study is 0.786 Nmm and the c_u produced from Equation (3) is in kN/m^2.

Figure 4. Laboratory Miniature Vane Shear Apparatus

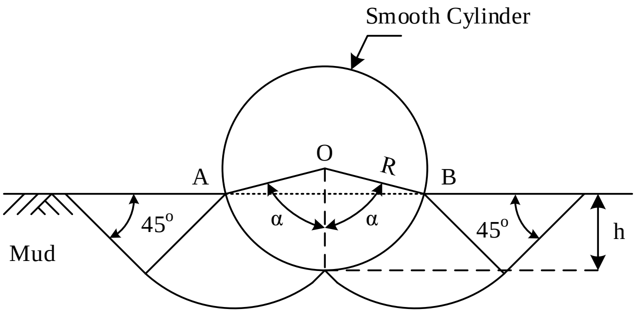

Figure 5. Illustration of Shear Development in CSMT (After [8])

Cylinder Strength Meter Test

The cylinder strength meter test or CMST was developed by Vallejo & Scovazzo [8] based on a force equilibrium between the weight of a smooth cylinder with a known height and diameter as the active force and the c_u-dependent upward force and the buoyancy acting on the cylinder as the reaction forces. The test procedure of CMST is started by slowly putting a cylinder with a smooth surface area and known dimension on the mud sample surface, and letting it sink by gravity. After the cylinder reaches equilibrium or the cylinder does not sink any further, the penetration depth of the cylinder is measured. The c_u value of the mud sample can eventually be estimated by using the following equation:

$$c_u = \frac{R[\pi\gamma_c-(\alpha-\sin\alpha\cos\alpha)\gamma_f]}{2[(\pi+2)\sin\alpha+2(1-\cos\alpha-\alpha\sin\alpha)]} \tag{4}$$

$$\alpha = \cos^{-1}\left(1-\frac{h}{R}\right) \tag{5}$$

where R is the radius of the cylinder, γ_c is the cylinder unit weight, γ_f is the mud unit weight, and h is the penetration depth of the cylinder into the mud sample. Figure 5 illustrates the parameters used in Equations (4) and (5), and also the development of failure surface in CSMT. Note that Equations (4) and (5) can be used when α is equal to or smaller than 90° or half of the cylinder is submerged. Thus, based on the CSMT mechanism, it can be inferred that the softer the soil, the deeper the cylinder penetration depth and the lower the c_u value. In addition, one advantage of CSMT is that the c_u of the mud can be measured without significantly disturbing the mud soil-water structure [8].

Vallejo and Scovazzo's Flume Channel Test

Vallejo and Scovazzo's method used a flume channel made of transparent Plexiglass® which was 80 cm in length, 15 cm in height, and 20 cm in width to measure the η value of kaolin mud [8]. The test procedure is initiated by placing the mud sample with a designed water content behind the gate in the flume channel, called the mud chamber. However, note that before placing the mud sample into the flume, some strings shall be loosely attached to the side walls of the mud chamber, but firmly attached to the base of the flume so that no displacement is recorded. The strings are used to record the displacement at different depths which later is translated into the velocity of mudflow. In addition, the side walls of the flume are greased to reduce friction between the mud sample and the side walls, but the base of the flume is not greased. The mud-filled flume channel is then placed on an inclined plane with a 39° inclination [8] and the gate is opened to let the mud sample flow down gravitationally. The time and displacement of the mudflow are recorded until there is no mud movement observed. Then, assuming that the τ-$\dot{\gamma}$ behavior of the mud follows the Bingham plastic material model in Equation (1), the mud viscosity can eventually be computed using the equation in [1,8]:

$$\eta = \frac{\gamma_f h_m^2 \sin\beta - 2c_u h_m}{2(V_t - V_b)} = \frac{\gamma_f h_m^2 \sin\beta - 2c_u h_m}{2V_t} \tag{6}$$

where h_m is the mud sample depth, β is the slope inclination in degrees, V_t is the velocity of the free surface of mudflow, and V_b is the velocity of the mudflow at the base of the flume which is equal to zero in this case. The c_u value in Equation (6) can be obtained from FCT, mVST, VST, or CSMT. However, as noted by Widjaja & Pratama [1], the η obtained from Equation (6) could be unreasonable when the "$\gamma_f h_m^2 \sin\beta$" term was smaller than the "$2c_u h_m$" term at a particular w value. Also, Vallejo & Scovazzo's [8] method was more suitable for obtaining the η value at a relatively low shear strain rate level.

Experimental Procedure

This study was started by conducting some laboratory basic soil properties tests such as sieve analysis, FCT, and specific gravity tests on both soil samples used in this study which are kaolin and bentonite soils to obtain the basic soil parameters and soil classification.

Table **1** lists the basic soil parameters including specific gravity (G_s), plastic limit (PL), liquid limit (LL), plasticity index (PI), average particle diameter (D_{50}), percentage of fine-grained soil and clay fraction, and also soil classification of the kaolin and bentonite soil samples. It was found that according to the Unified Soil Classification System (USCS) [14], the behavior of both soil samples was categorized as silt with high plasticity (MH). The D_{50} of the kaolin sample was 0.003 mm, whereas for bentonite sample, D_{50} = 0.025 mm. Moreover, based on the sieve analysis on the kaolin sample, the percentage of the soil sample passing sieve No. 200 was 99.93% with 42.9% clay-sized particles. Meanwhile, for the bentonite sample, the percentage of fine-grained soil was 89.80% with 33.3% clay-sized particles. This indicates that both samples tended to behave as fine-grained soils concerning soil response to load.

The CSMT was then carried out to estimate the c_u of the kaolin and bentonite muds. First, the dry kaolin and bentonite soil samples in the form of powder with a weight of ±6.5 kg were prepared and mixed with a certain amount of distilled water to reach a targeted water content. The targeted water content in this study ranged from 0.8LL to 3.6LL with an interval of 0.2LL. After the soil samples and water were mixed thoroughly, the mud samples were put into the glass box with a dimension of 30 cm × 30 cm × 20 cm in layers. Note that the height of the mud sample in the glass box was maintained at 10 cm and the surface of the mud sample was level. Later, a smooth acrylic cylinder measuring 3.5 cm in diameter and 7 cm in height was lowered slowly onto the mud surface at the center of the glass box to avoid boundary effects. The cylinder at this state was allowed to penetrate the mud sample until it reached equilibrium. After that, the cylinder penetration depth was recorded, and the c_u value of the mud sample was estimated by using Equations (4) and (5). The test procedure was repeated for both kaolin and bentonite mud samples with different targeted water content.

Table 1. Parameters of Kaolin and Bentonite Soils Samples

Soil Parameters	Symbol	Unit	Soil Samples	
			Kaolin	Bentonite
Specific Gravity	G_s	-	2.64	2.55
Plastic Limit	PL	-	49.6	57.1
Liquid Limit	LL	-	73.1	90.4
Plasticity Index	PI	-	23.5	33.3
Average Particle Diameter	D_{50}	mm	0.003	0.025
Fine-Grained Soil	-	%	99.9	89.8
Clay Fraction	%Clay	%	42.9	32.8
USCS	-	-	MH	MH

A similar sample preparation procedure was also carried out for the FCT and mVST where mud samples were first conditioned to reach the targeted water content. The FCT and mVST were repeated for the kaolin and bentonite mud samples at different targeted water content. It was worth noting that in this study, the CMST, FCT, and mVST were initiated when a reading from the apparatus could be obtained and stopped when readings could no longer be continued. For instance, when the cylinder in CMST could not penetrate or the penetration of the cylinder was deeper than the radius of the cylinder, the test was stopped and the results were omitted.

The experiment was continued by carrying out Vallejo & Scovazzo's flume channel test, later simply called the flume test to estimate the viscosity of the mud samples. To study the scale effects in the flume test, two flumes were used with different sizes in this study. One was a half-sized flume channel (HS-FC) with a respective size of 40 cm x 10 cm x 12.5 cm and the other was a quarter-sized flume channel (QS-FC) with a respective size of 20 cm x 5 cm x 5 cm. The mud sample was placed in the half-sized flume with a height of 7.5 cm and a length of 20 cm, whereas for the quarter-sized flume, the mud sample was 3.25 cm in height and 10 cm in length. Strings were loosely attached to the side walls of the flume with 5 cm spacing for the half-sized flume and 2 cm for the quarter-sized flume, meanwhile, the bottom of the strings was fixed at the base of the flume channel. The strings used in this study were more than the strings used by Vallejo & Scovazzo [8] to obtain a more detailed reading of the mud displacement. Figure 6 shows the flume dimension and string configuration used in this experiment.

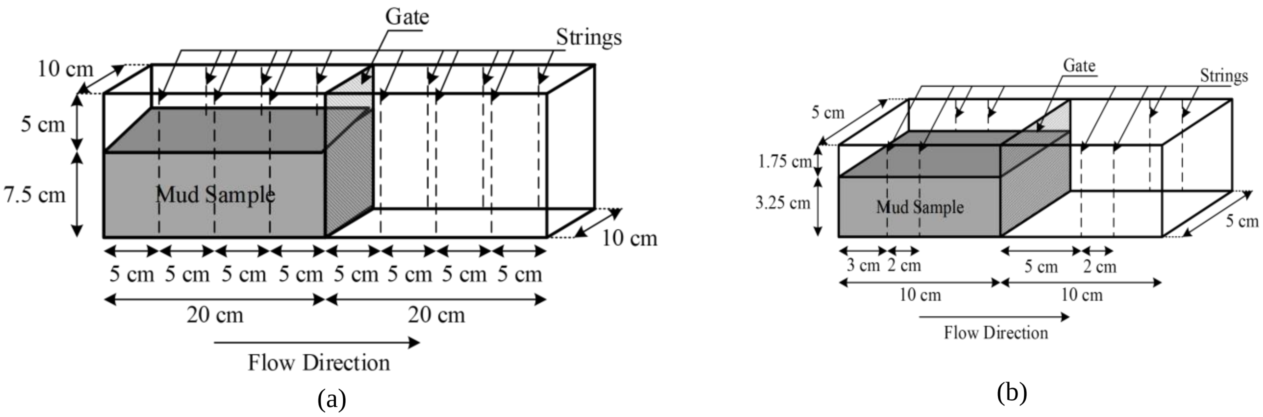

Figure 6. (a) Half-sized (HS-FC) and (b) Quarter-sized (QS-FC) Flume Channels

The flume tests for kaolin and bentonite mud samples were started at the w and β values where the mud began to flow. The first trial w value was at w = LL, meanwhile, the β value was first varied from 20° to a maximum slope angle of 40° to vary the τ acting on the mud sample. After the w and the β values at the first observed flow were obtained, the w value was then increased with an interval of 0.1LL, whereas the β values were increased with an interval of 5° to 10°. The flume test was stopped when β = 40° and no mud displacement was detected. The variations of w and β values for the kaolin and bentonite mud samples in the tests using HS-FC and QS-FC are shown in Table 2. Then, during the tests, the mud displacement and travel time were recorded to later be used in calculating the mud flow velocity. According to Vallejo & Scovazzo [8], the τ and $\dot{\gamma}$ values of mudflow could then be estimated based on the forces acting on the mud sample during the flume test using the following equations:

$$\tau = \gamma_f h_m \sin \beta \tag{7}$$

$$\dot{\gamma} = \frac{dv}{dy} = \frac{V_t - V_b}{h_m} \tag{8}$$

Table 2. The w and β Values Variations for the Flume Test

Soil Samples	Target Water Content, w_{tgt} (%)	Slope Angle, β	
		HS-FC	QS-FC
Kaolin	1.5LL	40°	-
	1.6LL	40°	25°, 30°, 35°, 40°
	1.7LL	20°, 30°, 40°	25°, 30°, 35°, 40°
	1.8LL	20°, 30°, 40°	25°, 30°, 35°, 40°
	1.9LL	20°, 25°, 30°, 35°, 40°	
	2.0LL	20°, 25°, 30°, 35°, 40°	
Bentonite	LL	20°, 25°, 30°, 35°, 40°	
	1.1LL	20°, 25°, 30°, 35°, 40°	
	1.2LL	20°, 25°, 30°, 35°, 40°	

Note that in this study, the η value of the mud was not directly obtained using Equation (6), but it was estimated using the Bingham plastic material analysis. In the Bingham plastic material analysis, the τ and $\dot{\gamma}$ values are plotted based on the flume test results with various β values to predict the true material behavior. Then, according to the Bingham plastic model, the η value of the mud is defined by the slope of the linear relationship between τ and $\dot{\gamma}$, while the τ_y value is the intersection between the linear relationship and the ordinate (y-axis) as shown in Figure 2.

Results and Discussion

Comparison of c_u Values from The FCT, mVST, and CSMT

To study the performance of the CSMT in predicting the c_u of mud, the c_u values from CMST were compared to the c_u values obtained from FCT and mVST conducted in this study, and also from other studies ([8,15–19]). Leroueil, et al. [15] compiled numerous data of geotechnical data in eastern Canada, including the c_u values, and proposed the following c_u-LI relationship for LI values ranging from 0.4 to 3:

$$c_u = \frac{1}{(LI-0.21)^2} \tag{9}$$

Then, Locat & Demers [16] suggested a c_u-LI relationship based on the c_u values measured by using the Swedish fall cone as follows:

$$c_u = \left(\frac{19.8}{LI}\right)^{2.44} \tag{10}$$

This relationship was valid for LI values ranging from 1.5 to 6. Koumoto & Houlsby [17] also used FCT to estimate c_u for remolded clays and proposed the following relationship:

$$c_u = exp\left[\frac{(1.07-LI_N)}{0.217}\right] \tag{11}$$

$$LI_N = \frac{[ln(w)-ln(PL)]}{[ln(LL)-ln(PL)]} \tag{12}$$

where LI_N is a new LI with every component in the LI equation (i.e., LI = (w – PL)/(LL – PL) is expressed in the natural logarithm. The c_u-LI relationship in Equation (11) could fit the data collected by Koumoto & Houlsby [17] with LI values ranging from 0 to 1.2. Furthermore, Widjaja, et al. [18,19] also suggested an empirical correlation for estimating MH soils in West Java as follows:

$$c_u = 2.01 \times 28^{(1-LI)} \tag{13}$$

Equation (13) is valid for LI values ranging from 0 to 1.

Figure 7 compares the c_u value of kaolin and bentonite mud samples obtained from the FCT, mVST, and CSMT in this study to the c_u values collected from the previous studies in [3,8,18] and computed by using Equations (9) to (13). In Figure 7, the c_u values estimated by other test methods, such as the moving ball test developed by Lee et al. [20] are also shown for reference. The dashed lines in Figure 7 indicate the c_u data obtained from the published c_u values or the empirical formulas. Meanwhile, the c_u values obtained from the FCT, mVST, and CSMT in this study are depicted by the scatter with a continuous smooth line type. In Figure 7, LI ≥ 1 indicates that the soil is in the viscous liquid state, whereas the soil is in the plastic state when 0 ≤ LI < 1. According to the FCT, mVST, and CSMT results in this study, the range of kaolin mud c_u values in Figure 7b was from 9.76 kN/m^2 to 0.45 kN/m^2 for LI = 0.47 – 5.93. Meanwhile, for the bentonite mud (Figure 7a), the c_u values ranged from 9.38 kN/m^2 to 0.96 kN/m^2 for LI = 0.49 – 2.66.

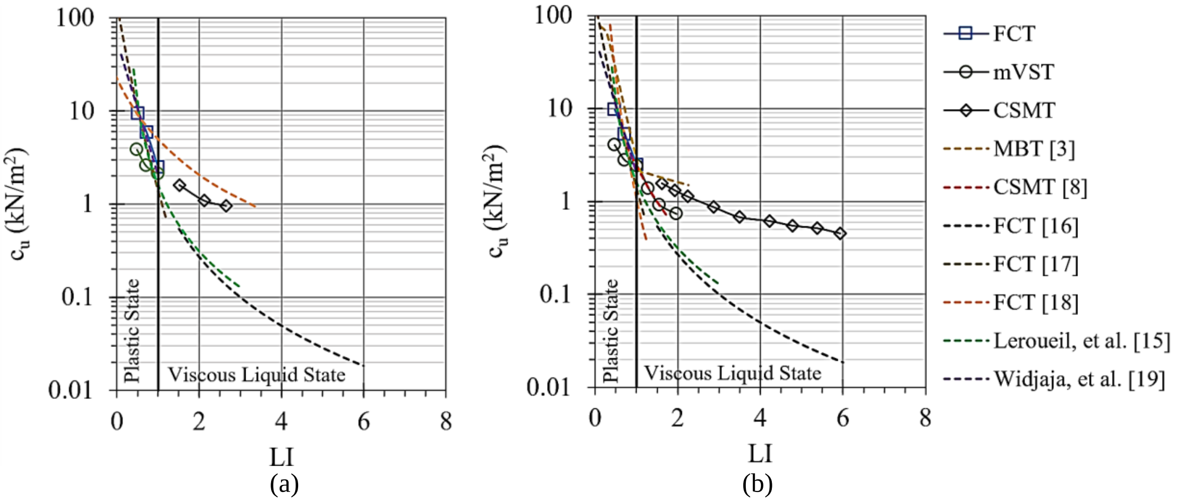

Figure 7. Comparison (a) Bentonite and (b) Kaolin c_u and LI Relationships in This Study with Previous Studies

The c_u-LI relationship obtained from the FCT, mVST, and CSMT depicted in Figure 7 shows a consistent trend that c_u decreased with increasing LI due to decreasing the interparticle contract forces. This general trend agreed well with the c_u-LI trends in previous studies. However, it was found that even though the c_u values obtained from the FCT and mVST for LI ≤ 1 were close to those obtained in the previous studies, the CSMT's c_u values for LI > 1 in this study were relatively higher than those predicted by using Equations (9) and (10). This difference was attributed to the different applied shear stress conditions and directions for each test which resulted in different developed shear failure planes. The cone in FCT penetrated a soil body generating a local or punching shear around the cone. This was similar to the concept of bearing capacity failure for shallow foundations on loose soil. Then, the vane blade in mVST was rotated to produce a cylindrical shear failure plane in the soil sample. Meanwhile, in the CSMT test, the c_u value was derived based on the force equilibrium acting on the cylinder with an assumed failure plane as shown in Figure 1. In addition, different mineralogy, soil classification, and sensitivity of the soils used in the previous studies could also be the reason for the difference in the c_u values. The soil type of the kaolin and bentonite soils in this study was classified as MH. Meanwhile, Leroueil, et al. [15] and Locat & Demers [16] focused their study on the CL and CH soil types and noted that Locat & Demers [16] used sensitive clays with sensitivity ranging from 8 to 82.

In this study, the FCT, mVST, and CSMT were conducted at an LI value where the measurement by the apparatus could be taken. This resulted in a different range of LI values for each test and soil sample. For instance, as shown in Figure 7b, the FCT for kaolin samples was stopped at LI = 1.02 because the cone penetration in FCT could not exceed 25 mm penetration depth. It was then interesting that the CSMT could measure the c_u value with LI values up to 6 indicating that CSMT could be used to estimate the c_u value of a viscous liquid material. However, as depicted in Figure 7, CSMT could not estimate the c_u values of the kaolin mud samples with LI < 1.61 and LI < 1.52 for bentonite soil samples. In addition, the CSMT for bentonite mud was stopped at LI =1.96. This was because the penetration depth of the cylinder could not be obtained for the samples that were still relatively plastic or too liquid (i.e., water-like substance). Therefore, heavier or lighter cylinders were recommended for measuring mud c_u with relatively low and high LI values, respectively using CSMT. It was also worth noting that the accuracy of CSMT was deemed operator-dependent. Several measurements were recommended to find a consistent c_u value.

This study also found that the c_u values at LI = 1 or w = LL were relatively consistent compared to the previous studies in Figure 7. Previous studies ([3,8,17–19]) found that c_u at LI = 1 were equal to 1.09 kN/m^2 – 5.08 kN/m^2. Meanwhile, according to the FCT and mVST results, the c_u value at LI = 1 for kaolin mud was 2.47 kN/m^2 – 2.38 kN/m^2, whereas for the bentonite mud, c_u = 2.49 kN/m^2 – 2.11 kN/m^2. This signified that regardless of the type of the main clay mineral (i.e., kaolinite and bentonite), the c_u value of mud at the onset of mudflow (i.e., w = LL) was approximately as low as 2 kN/m^2 on average.

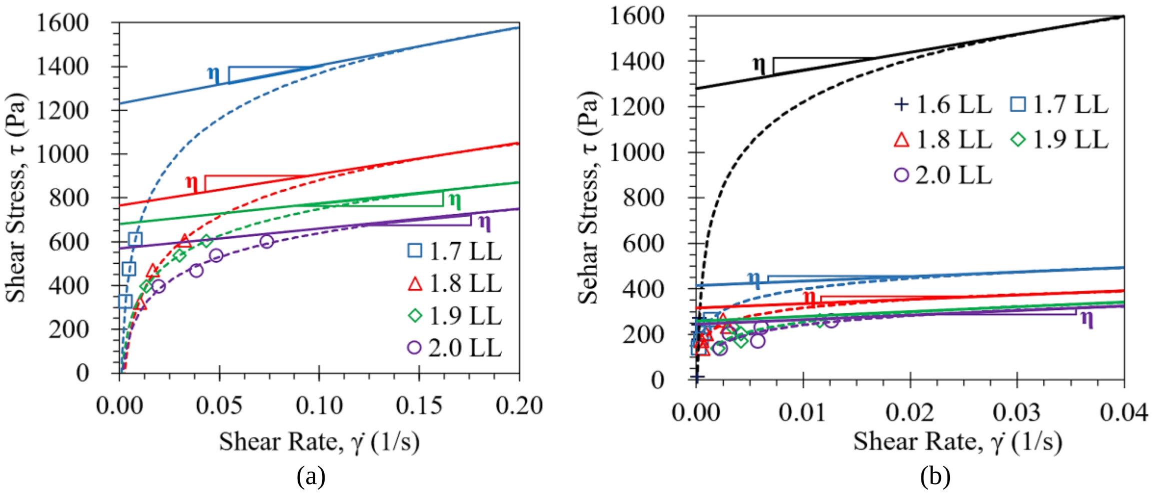

Figure 8. Bingham Plastic Material Analysis of Kaolin Mud Samples on (a) the Half-Sized Flume Channel (HS-FC) and (b) the Quarter-Sized Flume Channel (QS-FC)

Scale Effects on The Mud Viscosity

Figure 8 and Figure 9 showcase the Bingham plastic material analysis for kaolin and bentonite soil samples, respectively on the half-sized flume (HS-FC) and the quarter-sized (QS-FC). The dashed line in Figure 8 and Figure 9 represents the estimated real material behavior and the continuous line depicts the Bingham plastic material behavior, while the dots are the test results. Note that the Bingham plastic material analysis was only performed for the mud samples where the τ and $\dot{\gamma}$ values could be obtained for at least three (3) different slope angles. Then, as indicated in Equation (7), increasing the slope angle increased the τ values in Figure 8 and Figure 9. Meanwhile, the $\dot{\gamma}$ values were computed by using Equation (8) based on the estimated V_t and V_b in the flume tests. As shown in Figure 8 and Figure 9, the $\dot{\gamma}$ increased due to increasing τ and size of the flume indicating a higher mud flow velocity in the larger mud sample volume and flume size.

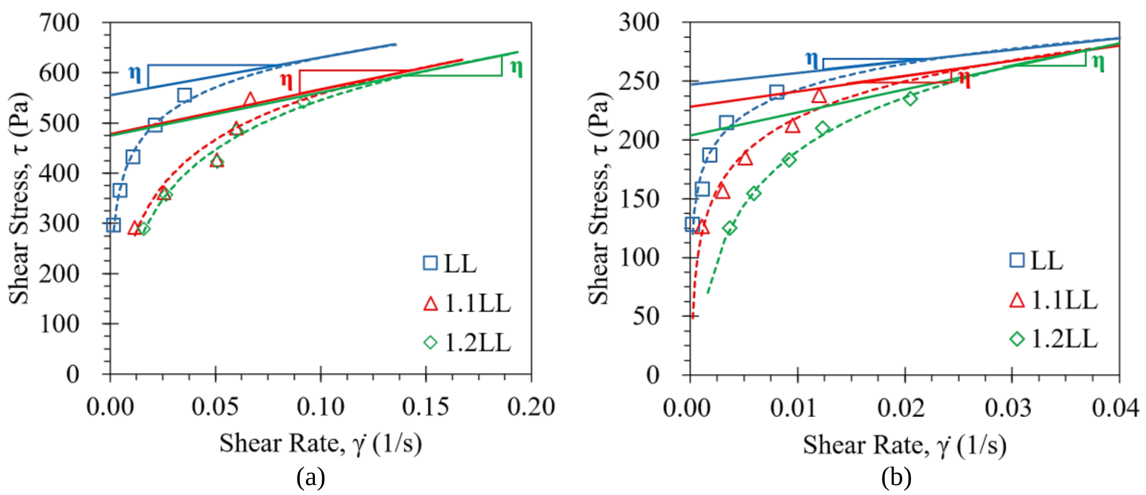

Figure 9. Bingham Plastic Material Analysis of Bentonite Mud Samples on (a) the Half-Sized Flume Channel (HS-FC) and (b) the Quarter-Sized Flume Channel (QS-FC)

Table 3 lists the τ_y and η values of the Kaolin and Bentonite mud samples obtained from the flume test using HS-FC and QS-FC. It was found that the kaolin η values varied from 905.2 Pa•s to 1752.7 Pa•s with τ_y = 0.57 kN/m^2 – 1.23 kN/m^2 in the flume tests using HS-FC, while it was about 1930.5 Pa•s to 7985.2 Pa•s with τ_y = 0.24 kN/m^2 – 1.28 kN/m^{2}for the tests using QS-FC. For the bentonite mud sample, the η values varied from 754.1 Pa•s to 881.3 Pa•s and 2063.9 Pa•s to 2549.1 Pa•s for the tests using HS-FC and QS-FC, respectively. Meanwhile, the τ_y values of the bentonite mud sample were about 0.47 kN/m^2 – 0.55 kN/m^2 for the tests using HS-FC and 0.19 kN/m^2 – 0.23 kN/m^2 for the tests using QS-FC. The results for kaolin mud samples show a consistent trend that the τ_y and η values decreased with increasing LI values. This indicates that as more water was introduced to the kaolin mud sample, the mud sample became less viscous (more liquid) and lost its shear strength due to decreasing attraction and repulsion forces between clay particles. However, for the bentonite mud sample, even though the τ_y still decreased with increasing LI, the η value exhibited an unclear relation with LI. The change of η with LI of the bentonite mud sample was relatively constant. This could be arguably caused by the unique characteristics of bentonite as one type of Montmorillonite clay that could absorb a large quantity of water. This resulted in different fluid statics (i.e., fluids at rest condition) and fluid dynamics (i.e., fluids in motion) behaviors of the bentonite mud.

Table 3. The η Values of Kaolin and Bentonite Mud Samples

Soil Samples	HS-FC			QS-FC		
	LI	τ_y (kN/m^2)	η (Pa•s)	LI	τ_y (kN/m^2)	η (Pa•s)
Kaolin	-	-	-	2.99	1.28	7985.2
	3.28	1.23	1752.7	3.13	0.41	1977.7
	3.59	0.76	1430.1	3.65	0.31	1930.5
	3.69	0.68	946.7	3.73	0.25	2200.4
	4.10	0.57	905.2	4.19	0.24	2025.5
Bentonite	1.44	0.55	754.1	0.99	0.25	985.5
	1.41	0.48	881.3	1.40	0.23	1298.6
	1.68	0.47	861.1	1.64	0.20	1960.9

Furthermore, the results in Table 3 also indicate that decreasing the flume size increased the η values at the same LI value up to 1.1 to 2.3 times the η values obtained using a larger flume. In addition, the τ_y values obtained using QS-FC were also smaller by 0.52 to 0.66 times the τ_y values obtained using HS-FC. This indicates that the measurement of mud's η and τ_y values using Vallejo & Scovazzo's [8] method was highly affected by the size of the flume due to the existence of scale effects. As shown in Figure 8 and Figure 9, smaller τ in the smaller flume would produce a low shear rate level and true material τ-$\dot{\gamma}$ behavior at the initial of the mud flow. This resulted in a higher η value of the mud. Moreover, the size of the flume also affected the mud displacement reading using the strings. The recorded mud displacement using QS-FC might have been the mud displacement at the initial low, not at the steady flow condition. Thus, a larger size of the flume was recommended in Vallejo & Scovazzo's [8] method to avoid scale effects and allow mud displacement readings under steady flow conditions.

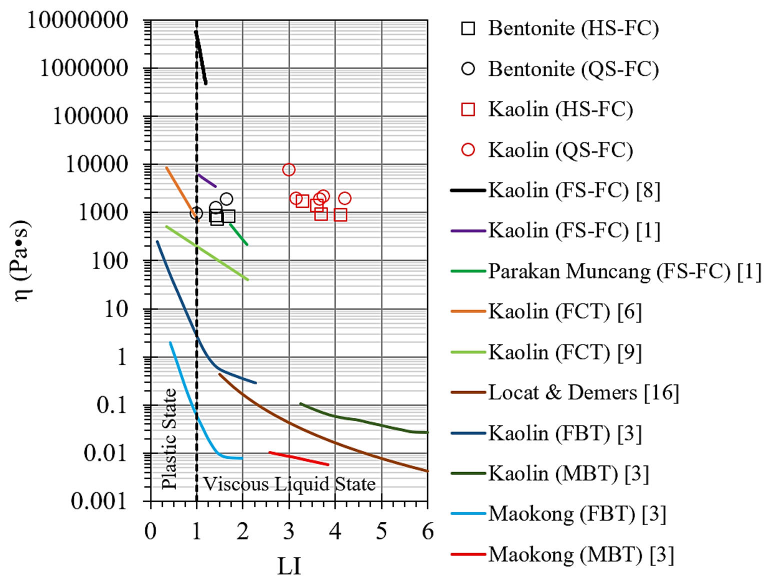

Figure 10. Comparison of η Values in This Study to the Previous Studies

Figure 10 compares the η value obtained in this study (Table 3) to the η values from the previous studies. In Figure 10, the published η values were obtained for various soil types (e.g., Parakan Muncang clay soil [1], Maokong silt soil [3], and kaolin clay [1,3,6,8,9]) using different test methods, such as FCT [6,9], MBT [3], and flow box test (FBT) [3]. Note that in Figure 10, Vallejo & Scovazzo [8] and Widjaja & Pratama [1] test results are denoted by FS-FC indicating a full-scale flume channel was used in their study. Then, according to Figure 10, the η values obtained from the tests using HS-FC and QS-FC were higher than most of the η values obtained in the previous studies but smaller than those obtained by Vallejo & Scovazzo [8] despite the same soil type being used. This was because the measurement of η through the approach introduced by Vallejo & Scovazzo [8] was obtained from the values of τ and $\dot{\gamma}$ observed at the initiation of flow or during the immediate elastic response. On the other hand, the Bingham plastic model analysis defined the η value for the material at the steady viscous response point. Similar findings were also found by Widjaja & Pratama [1]. Moreover, it was interesting that the slope of the η-LI relationship obtained in this study for kaolin mud was relatively similar to those obtained by Widjaja & Pratama [1]. This emphasizes that the size of the flume channel increased the η values of the mud due to scale effects but not the gradient of the change of η with LI.

Conclusions

This study compared the performance of Vallejo and Scovazzo's cylinder strength meter test to the fall cone and mini vane shear tests in measuring the undrained shear strength of kaolin and bentonite muds. In addition, the effects of scale on the viscosity values of kaolin and bentonite muds obtained using Vallejo and Scovazzo's flume test method were studied by carrying out a series of flume tests with half- and quarter-sized flumes. The results of the fall cone, mini vane shear, and cylinder strength meter tests showed that the undrained shear strength values for both kaolin and bentonite mud samples decreased with increasing liquidity index. It was also interesting to note when the liquidity index was equal to one indicating the initial state of mudflow, the mud's undrained shear strength was close to 2 kN/m^2. Then, it was found that the cylinder strength meter test had a benefit over the other tests in that it could measure the undrained shear strength of mud with a liquidity index greater than 1.5. However, the cylinder strength meter test was not suitable for measuring the undrained shear strength of soil with extremely high water content or close to the liquid limit because the measurement of the cylinder penetration depth could not be obtained or was too small to measure due to insufficient weight or dimension of the cylinder. This also indicated that the cylinder strength meter test's accuracy could potentially vary based on the operator's expertise and cylinder dimension. Further research was still required to verify the sensitivity of the cylinder strength meter test results concerning the dimension and/or weight of the cylinder.

Based on the flume test results, the mud viscosity was highly affected by the size of the flume channel. Smaller flume size would produce smaller shear stress and shear rate, higher viscosity, and smaller yield stress due to the scale effects. Using smaller flume sizes also limited the observation of the mud displacement to the initial flow state which resulted in a higher viscosity value. Moreover, compared to the results in the previous studies, the viscosity values obtained from the flume test, based on Vallejo and Scovazzo's method, were relatively high. This might be caused by the viscosity values being at the initial flow, not at the steady state flow. It was then recommended to estimate the viscosity value of soil at the steady viscous state, which could be obtained by using the Bingham plastic material analysis. The use of a larger flume size also allowed the measurement of the mud displacement at the steady viscous state. Then, note that the accuracy in estimating the mud viscosity value was also deemed to be operator-dependent. Thus, a more advanced instrument to record the mud displacement was recommended. However, regardless of the limitations of the flume test, this method had a major advantage in that it measured the mud viscosity based on the true mud flow shear failure mechanism. Eventually, this study could become a reference for future studies in developing a better instrument for measuring the mud viscosity and undrained shear strength considering that those parameters governed the mud flow behavior. This study was valid for homogeneous mud with properties comparable to those presented in this paper.

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