

Laboratory Tests on the Determination of the Undrained Shear Strength of Kaolin and Bentonite Mud

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Abstract— Mudflow is a type of ground movement that occurs rapidly when the water content of the soil increases, exceeding its liquid limit. The rheological approach is commonly used to study the stress-strain behavior of a mudflow. Two rheological-based parameters used to determine the flow velocity of the mudflow are yield stress and viscosity. In this study, yield stress was assumed to be equal to the undrained shear strength of the cohesive soils. Three laboratory test methods (i.e., the fall cone penetration test, the vane shear test, and the cylinder strength meter) were adopted to estimate the undrained shear strength of kaolin and bentonite soils. The results indicated that the cylinder strength test method had benefits against the other conventional methods in measuring the undrained shear strength of the soils with high water content. The experimental results also show that increasing the water content decreased the undrained shear strength and increased the flow velocity.

Keywords—mudflow, rheology, yield stress, undrained shear strength

I. Introduction

Researchers in [1] classified the form of ground movements based on two categories, namely the type of transported material (i.e., rock, earth, soil, mud, and debris) and the movement type (i.e., fall, topple, slide, spread, and flow). One of the forms of ground movement is mudflow. A mudflow is a form of ground movement involving a rapid and sudden flow of highly saturated soil, exceeding its liquid limit, forming a viscous liquid material with components consisting of sand, silt, and clay particles, later denoted as mud. As the mud travels down a slope, the occurred shear stress (τ) due to gravitational force was resisted by the combined effects of the viscosity (η) and the undrained shear strength (c_u) of the mud [2], [3].

The soil c_u value is commonly obtained directly from the laboratory strength tests, for instance, the triaxial, direct shear, fall cone penetration, and vane shear tests, and from empirical correlations to the field or laboratory test results. However, performing conventional laboratory strength tests such as triaxial and direct shear tests on mud samples is relatively challenging because of the difficulty in obtaining and forming the mud samples that fit the tests. As a consequence, the c_u of mud is sometimes assumed as nil in practice. However,

according to the references in [4] and [5], the c_u value at the liquid limit was about 3.6 kN/m² to 1.7 kN/m². Thus, it is still required to determine the c_u value of mud to understand the mudflow stress-strain behavior.

Vallejo and Scovazzo developed the cylinder strength meter test, a laboratory test to measure the c_u of mud [3]. Researchers in [3] used the cylinder strength meter test to estimate the c_u of kaolin mud. The advantage of using this method is that the c_u value measurement can be carried out without significantly disturbing the mud soil-water structure [3]. However, limited studies were found carrying out the cylinder strength meter test to find the c_u of mud. Thus, this research aims to study the c_u value of kaolin and bentonite measured using the cylinder strength meter test and compare it with the measurements obtained from the fall cone and miniature laboratory vane shear tests. The water content of the mud samples in this study also varied to a value greater than the liquid limit to understand the change of c_u with the water content value in the mud samples. The structure of this paper is then structured as follows. First, the theoretical background of the laboratory tests used in this study is briefly reviewed. Second, the research methodology is described. Eventually, the results of tests are presented and discussed.

II. Laboratory Tests

A. Cylinder Strength Meter Test

The c_u measurement using the cylinder strength meter was developed based on a force balance acting on a smooth cylinder with a known dimension and weight penetrating a mud sample. The cylinder is first slowly lowered into the mud sample. As the cylinder slowly penetrates the mud sample, an upward force which is a function of c_u and buoyancy force acts on the cylinder as the reaction forces to the acting force (i.e., the weight of the cylinder). The following expresses the equation to measure the weight of the cylinder:

$$W = \pi R^2 L \gamma_c \quad (1)$$

where W is the weight of the cylinder, R is the cylinder radii, L is the cylinder length, and γ_c is the unit weight of the cylinder. The cylinder used in this study was made of acrylic glass with $\gamma_c = 1.2 \text{ g/cm}^3$ and a dimension similar to the one used in [3], whose diameter and height were 3.5 cm and 7 cm, respectively. Fig. 1 shows the acrylic cylinder used in the cylinder strength meter test.

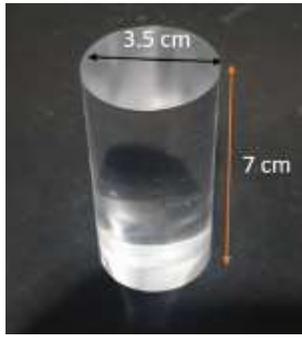


Figure 1. Acrylic Cylinder

Then, the magnitude of the upward force (P_s) is a function of the soil shear strength (i.e., c_u) [3]. Equation (2) shows the P_s equation as follows:

$$P_s = 2c_u RL \left[(\pi + 2) \sin \alpha + 2(1 - \cos \alpha - \alpha \sin \alpha) \right] \quad (2)$$

and the buoyancy force can be computed as follows:

$$P_b = R^2 L (\alpha - \sin \alpha \cos \alpha) \gamma_f \quad (3)$$

where γ_f is the mud unit weight, and α can be estimated using the following equation:

$$\alpha = \cos^{-1} \left[1 - \frac{h}{R} \right] \quad (4)$$

with h is the cylinder penetration depth in the mud sample.

Under the equilibrium condition, the weight of the cylinder is equal to the upward force and the buoyancy force. Based on this force balance, the c_u value of the mud is obtained by using the following equation:

$$c_u = \frac{R \left[\pi \gamma_c - (\alpha - \sin \alpha \cos \alpha) \gamma_f \right]}{2 \left[(\pi + 2) \sin \alpha + 2(1 - \cos \alpha - \alpha \sin \alpha) \right]} \quad (5)$$

B. Laboratory Miniature Vane Shear Test

The miniature laboratory vane shear test later denoted mini vane shear, is used to measure the c_u of a remolded soil sample. In this study, the mini vane shear used in the experiment was the hand operator vane. Like the field vane shear test, the laboratory miniature vane shear test was carried out by rotating the four-bladed vane in the soil sample at a constant rate of about 6°-12° per minute. Fig. 2 shows the mini vane shear used in this study. The vane blades' dimensions are 12.7 mm wide and 12.7 mm long.



Figure 2. Laboratory Miniature Vane Shear

The vane undrained shear strength of the soil can then be calculated using the following equations:

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

where M is a product of maximum angular rotation in degrees and the calibration factor in Nmm per degree, and in this study, it is equal to 0.786 Nmm, K is a constant, D_v is the overall width of the vane measured to 0.1 mm, and H is the length of the vane measured to 0.1 mm. Note that the unit of the c_u in (5) is in kN/m^2 .

C. Fall Cone Penetrometer Test

The fall cone penetrometer test is a widely used laboratory test to measure the liquid limit and the plastic limit of cohesive soil. For a standard cone with a 30° apex angle and 0.79 N weight, the water content of the soil that allows a 20 mm cone penetration for a period of 5 s is defined as the soil liquid limit (LL). Meanwhile, the water content at a 2 mm cone penetration depth for 5 s is defined as the cohesive soil's plastic limit (PL).

However, applying the fall cone test is wider than estimating the Atterberg limits. Researchers in [6] proposed an equation for computing the c_u using the fall cone test as shown as follows:

$$c_u = k \frac{W_c}{d^2} \quad (7)$$

where k is the cone factor depending on the angle of the cone, W_c is the weight of the fall cone, and d is the cone penetration depth in mm. Note that the c_u in (7) is in kN/m^2 . In this study, the k value was equal to 1.33 with a standard cone apex angle of 30° and a cone weight of 80 g or 0.79 N [7]. Note that during the test for obtaining the c_u of the mud samples in this study, the cone was also released for only 5 s. Fig. 3 shows the fall cone penetrometer used in this study.



Figure 3. Fall Cone Penetrometer

III. Methodology

This study used kaolin and bentonite soil mixed with water until reaching a particular liquidity index (LI). The index properties, sieve analysis, and Atterberg limits tests were first carried out to identify the physical properties and behavior of the soils. Based on the sieve analysis, the percentage of passing sieve No. 200 for the kaolin soil was 99.9%, and the clay-sized particles dominated at 42.9%. Meanwhile, for the bentonite soil, the percent passing No. 200 was 89.8% which consisted of 32.8% of clay-sized particles. Then, Table 1 shows the results of the Atterberg limits tests for kaolin and bentonite, and Fig.4 displays the plotting of the Atterberg limits tests in the Casagrande plasticity chart. Note that the Atterberg limits in this study were obtained using the fall cone penetration test (FCT). As shown in Fig. 4, it was indicated that the soil behavior of both the kaolin and bentonite soil samples was classified as silt with high plasticity (MH). In addition, the activity value, denoted as A (i.e., $A = PI/\%Clay$ where %Clay is the percentage of the clay fraction), of the kaolin and bentonite soils was 1.16 and 1.02, respectively. According to [8], the activity of kaolin and bentonite soils was classified as normal.

TABLE I. ATTERBERG LIMITS AND PHYSICAL PROPERTIES OF THE KAOLIN AND BENTONITE SOILS

Parameters	Symbol	Samples	
		Kaolin	Bentonite
Plastic Limit	PL	49.6	57.1
Liquid Limit	LL	73.1	90.4
Plasticity Index	PI	23.5	33.3
Specific Gravity	G_s	2.64	2.55
USCS	-	MH	MH

The test was continued by preparing dry samples of kaolin and bentonite as much as ± 6.5 kg. The dry samples were then mixed with distilled water to the targeted water content until they became homogeneous. The targeted water content ranged from 0.8LL to 3.6LL. Subsequently, the mud samples were left for 24 hours in an air-tight container to avoid water

evaporation. The purpose of this step was to let the water seep into and fill the pores. Next, the samples were placed into a glass container with a size of 30 cm \times 30 cm \times 20 cm. Fig. 5 shows the glass container used during the cylinder strength meter tests (CSMT). Note that the glass container was filled with mud samples approximately only up to half of its height.

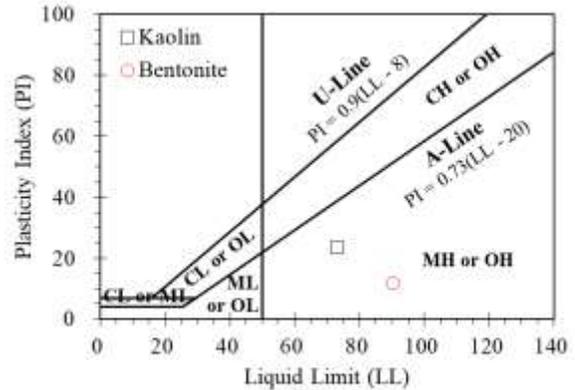


Figure 4. Casagrande Plasticity Chart

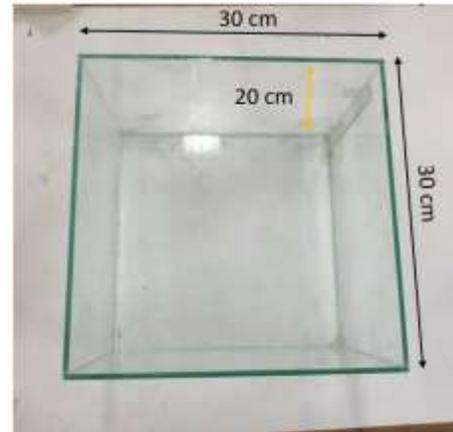


Figure 5. Glass Container Used for Placing the Mud Samples



Figure 6. Cylinder Strength Meter Test

The cylinder strength meter test was carried out by slowly lowering the cylinder on the mud surface at the center of the glass container to avoid any boundary effects. Note that the mud surface should be level during the test. Then, let the cylinder sink into the mud sample by its own weight for about 30 minutes. At this time period, the cylinder has typically reached equilibrium (i.e., no longer sinking into the mud). Fig. 6 illustrates the cylinder strength meter test performed in this study. After the cylinder reached equilibrium, the penetration

depth of the cylinder was recorded and eventually used for computing the c_u of the mud using (5).

Using the same mud samples prepared for the CSMT, the mini vane shear test (mVST) and FCT were also carried out to find the c_u value of the mud. The mVST was performed by inserting the vane blades into the mud sample that had been prepared in the metal cup. Before rotating the vane, the dial should first be ensured at zero degrees. Next, the lever was rotated to rotate the vane until it reached failure. The maximum angular rotation was recorded to later computer the mud shear strength using (6). It was worth noting that the mVST, FCT, and CSMT were repeated for mud samples with different water content ranging from LI = 0.4 to 9.1. The tests were started at an LI value when a reading (i.e., angular rotation, cone penetration, and cylinder penetration) could be obtained and ended when a reading could no longer be obtained due to the limitation of the tools. For instance, if the required cone penetration was larger than 25 mm for 5 s, the FCT for the corresponding mud sample was stopped, and the results were omitted.

iv. Results and Discussions

Fig. 7 shows the relationships between the c_u value of the kaolin mud sample obtained from the FCT, mVST, CSMT, and LI compared with the results of the previous studies [3], [9], [10]. In Fig. 7, the soil-water mixture was in the plastic state if the LI value was between 0 and 1, whereas it was in the viscous liquid state when LI > 1. Then, for kaolin soils, the FCT stopped at LI = 1 because the cone penetration exceeded 25 mm, and the instrument could only read the penetration up to 25 mm. Meanwhile, most were stopped at LI = 1.9. As shown in Fig. 7, the CSMT had a benefit in measuring the c_u of mud up to LI = 9.1. However, the CMST could not be carried out at a relatively low LI (i.e., LI < 1.6) because there was no observed cylinder settlement. The reason was the insufficiency of the weight of the cylinder to produce settlement on the mud sample. Thus, a heavier cylinder was suggested to measure the c_u of the mud with a relatively low LI value using CSMT.

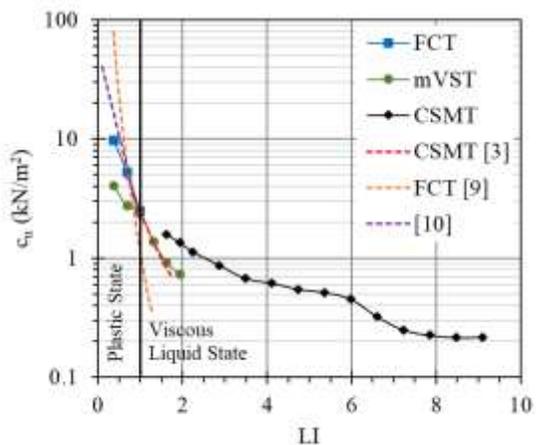


Figure 7. Comparison of Kaolin c_u vs. LI Relationships of This Study to the Previous Studies

In Fig. 7, the results of this study are depicted by the continuous line with a square, circle, or diamond shape, whereas the results of the previous studies were drawn in dash lines. Note that the c_u values obtained for [10] were estimated using the empirical correlation for computing the c_u values of MH soils proposed in [10]. The results clearly show that the c_u value decreased with increasing LI. As the LI kept increasing, the kaolin c_u value became relatively constant at about 0.2 kN/m². Furthermore, the c_u values obtained using the FCT and mVST were close to those obtained in [3] using the CSMT, where researchers in [3] also used kaolin in their tests. In addition, the FCT and mVST c_u values at LI = 1 were about 2.38 kN/m² to 2.47 kN/m². Similar findings were also presented in [9], [10], and [11]. Furthermore, the c_u values obtained from the CSMT in this study tended to follow the trends of c_u vs. LI graphs obtained for the FCT and mVST test results. It was argued that the c_u values from the CSMT were the results continuation from the FCT and mVST c_u values.

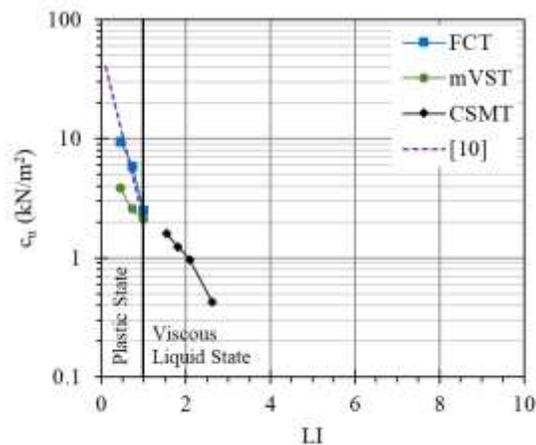


Figure 8. Comparison of Bentonite c_u vs. LI Relationships of This Study to the Previous Studies

Fig. 8 shows the change of the c_u values in the bentonite soil sample obtained from FCT, mVST, and CSMT with LI. The results of this study in Fig. 8 were also compared to the c_u values obtained from the empirical correlation for MH soil proposed in [10]. Then, the change of c_u with LI in bentonite soil was found to be relatively similar to those obtained for kaolin. Moreover, compared to the c_u values obtained from the empirical correlation in [10], the FCT c_u values were close to those obtained using the empirical correlation. Then, the c_u values at LL were also about 2.11 kN/m² to 2.49 kN/m². However, it was worth noting that the bentonite soil in this study was relatively sensitive to changes in water content compared to the kaolin mud sample. As the LI increased, the c_u value sharply decreased, causing the cylinder to sink entirely into the bentonite mud so that the penetration depth could not be measured. It showed one of the CSMT limitations is that the reading is limited to the dimension of the cylinder.

v. Conclusions

This paper studied the determination of the undrained shear strength of kaolin and bentonite muds using the fall cone

penetrometer, the miniature laboratory vane shear, and the cylindrical strength meter test. Increasing the kaolin or bentonite liquidity index decreased the undrained shear strength value. The undrained shear strength of the kaolin and bentonite soils at the liquid limit was about 2.11 kN/m² to 2.49 kN/m². These results were also in good agreement with the previous studies. Then, the cylinder strength meter test had the benefit of measuring the mud undrained shear strength over conventional tests such as the fall cone penetrometer and the miniature laboratory vane shear tests. The cylinder strength meter test could measure the undrained shear strength of mud with a liquidity index greater than 1.5. However, it was worth noting that the cylinder strength meter test also had some limitations. The measurement of the cylindrical strength meter test was deemed operator dependent and highly affected by the weight and the dimension of the cylinder. If the weight of the cylinder was too light, it did not suffice to penetrate the mud making the measurement could not be taken. In contrast, a heavier cylinder could sink the cylinder entirely into the mud, and the penetration depth could not be measured. Then, the maximum penetration depth of the cylinder was limited to its diameter. Eventually, the results of this study only applied to a homogeneous mud sample. Further research using other measuring tools were still required to verify the undrained shear strength obtained from the cylindrical strength meter test. In addition, future studies could also be carried out to study the boundary effects in the cylinder strength meter test and the effects of the weight and the dimension of the cylinder.

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