Measurement of shear strength for marine clay

Abstract

In this study a series of fall cone tests were carried out to investigate the undrained shear strength of soil involved in submarine landslides. Two different remoulded soils, Kaolin Clay and a natural submarine soft clay collected from the Red Sea (WND), were measured, using three different cones with masses of 80 g, 20 g, and 13.6 g. To eliminate the buoyant effect, all the data has been converted into modified shear strength. The relationship between the modified shear strength and the liquidity index can be described by a unique power law function for a given soil. On the other hand, when the liquidity index is above 1.5, the modified shear strength begins to deviate from the original data. The maximum difference is about 20% for Kaolin Clay and 30% for WND. Hence, the buoyant effect must be considered when using the fall cone test to measure extremely soft clay. Furthermore, the effects of salt were well captured by the control group experiment, and the result shows that adding 3.5% salt into water increased the shear strength for both clay samples, but WND is more sensitive to changes in salinity.

Keywords:

Shear strength; Laboratory tests; Clay; Offshore engineering

Notation

a , b material constants, describing the variation of s_u as a function of LI

h penetration depth of fall cone

K fall cone factor

LI liquidity index

Q weight of the cone

R_{MW} material constant

R^2 coefficient of determination

S_u shear strength at the strain rate of $\dot{\gamma}$

$S_{u.l}$ undrained shear strength at the liquid limit;

$S_{u,ref}$ reference shear strength at the reference strain rate of $\dot{\gamma}_{ref}$

$\dot{\gamma}$ strain rate

$\dot{\gamma}_{ref}$ reference strain rate

μ parameter describing the variation of s_u as function of $\dot{\gamma}$

δ inclination of the heave surface in degrees

1. Introduction

Due to ever increasing human activity in offshore areas, submarine landslides have attracted increasing attention. The causes of submarine landslides vary according to both the geomechanical attributes of landslide material, and transient environmental changes affecting the submarine environment. Some submarine landslides occur due to gas hydrate dissociation, which causes an increase in pore water pressure over a short period of time, and meanwhile decreases the resisting shear strength. Some other triggering events of submarine landslides include the rapid accumulation of sedimentary deposits, submarine earthquakes, and loading from tsunamis, all of which lead to a sharp increase of the downward driving stress. Common to all of these cases is a change in the downslope driving stress so that it exceeds the resisting strength of the marine clay which forms the seafloor slope material. Therefore, the undrained shear strength of marine clay plays an essential role in submarine landslides, and is of great concern for geotechnical and submarine engineering. The shear strength of marine clay near the seabed is significantly small, probably due to its high water content. This paper aims to estimate the undrained shear strength of remoulded marine clays, based on shear strength measurements conducted on two different types of clay, which varied the conditions relevant to soil with solid behaviour to those relevant to soils with fluid-like behaviour, (especially for soils with a liquidity limit greater than 1) which corresponds to the whole process of submarine landslides.

Muir Wood (1990) provided a basis for the widely accepted and quoted framework of the undrained shear strength model:

$$\mathrm{S_u} = \mathrm{S_{u.l}} \mathrm{R_{MW}}^{(1-\mathrm{LI})} \tag{1}$$

where $\mathrm{S_{u.l}}$ is the undrained shear strength at the liquid limit, $\mathrm{R_{MW}}$ is a material constant, and LI is the liquidity index, which can be defined as:

$$\mathrm{LI} = \frac{w - w_p}{w_l - w_p} \quad (2)$$

where w is the water content, w_p is the plastic limit, and w_l is the liquid limit.

Much research has been conducted to estimate $\mathrm{R_{MW}}$ for different kinds of soils. Muir Wood (1990) postulated that $\mathrm{R_{MW}}$ is a function of clay mineralogy, based on data from Dumbleton & West (1970) in which $\mathrm{R_{MW}}$ is close to 100 for montmorillonitic soils and 30 for kaolinitic soils. Whyte (1982) suggested that for particular Swedish clays the ratio was about 70, while Karlsson (1977) indicated that this ratio is between 50 and 100. Using the fall cone test, Koumoto & Houlsby (2001) obtained a value of approximately 44 based on data from only six clays. O'Kelly (2013) reported that $\mathrm{R_{MW}}$ can vary widely between different types of soils. Based on vane shear strengths, he presented $\mathrm{R_{MW}}$=43–128 for 14 mineral soils, and $\mathrm{R_{MW}}$=10–27 for four organic sediments. Recently, based on a database of 641 fall cone tests on 101 soil samples, Vardanega & Haigh (2014) found an average ratio of 35, instead of the 100-fold factor that was considered to overestimate the measured data of the soil strength.

Some other researchers have proposed that the evolution of the undrained shear strength can be written as a power function of the liquidity index:

$$\mathrm{S_u} = \mathrm{aLI^{-b}} \quad (3)$$

where the parameters a and b are material constants. In addition to fall cone tests, the viscometer test is often used to test the strength of high water content clay soils. The viscosity of sensitive clays was first related to remoulded undrained shear strength by Eden and Kubota (1962), who used a rotating coaxial viscometer to measure the remoulded shear strength. By using the fall cone test and viscometer test, Locat & Demers (1988) developed the shear strengths of soils with LI in the range 1.5–6 within this framework, using parameters $a = 1.46\,\text{kPa}$ and $b = 2.44$. Jeong et al. (2009) studied the shear strengths of soils of different origins and characteristics, and the results showed that the power law applies to all these soils, but the power law index varies for different kinds of soil. For a clay sample from eastern Canada, Jeong et al. (2009) proposed values of $a = 0.90\,\text{kPa}$ and $b = 3.4$. In order to measure the strength of fine-grained soils at the solid–fluid transition, Boukpeti et al. (2012) carried out a series of intrusive tests, including fall cone tests, vane shear tests, T-bar, and ball penetrometer tests. The results showed that the power function fits the test data quite well for Kaolin when $a = 1.71\,\text{kPa}$ and $b = 2.64$, and for Burswood (a natural soft clay collected from the Burswood site) when $a = 1.34\,\text{kPa}$ and $b = 4.03$. Another similar relationship was reported by Leroueil et al. (1983) from fall cone measurements on a variety of clays from Canada and other places, which showed a general equation for values of LI between 0.4 and 3 of

$$S_u = 1/(\text{LI} - 0.21)^2 \tag{4}$$

To summarize the relationships between the undrained strength and the liquidity index, Table 1 shows the shear strength (S_u) models and the range of applications of liquidity index (LI) from the literature.

Table 1. Published strength-liquidity index correlations for remoulded soils

Table 1 clearly shows that most published relationships based on exponential functions apply to clays with a water content below or around the liquidity index ($LI \leq 1.8$), and other data with a high liquidity index can be fitted using power functions. Therefore, the water content is a critical criterion in choosing the shear strength model. Besides the water content, there are other factors that play important roles in the shear strength of marine clay, as discussed below.

● Buoyant effect

One of the important roles is probably the buoyant effect. It may account for only about a 1.5% decrease in the fall cone factor K (Koumoto & Houlsby, 2001). K can be calculated by the cone angle β ($K = \frac{g}{\pi}\cos^2\frac{\beta}{2}\cot\frac{\beta}{2}$), where g is acceleration due to gravity. This conclusion applies in the standard fall cone test, in which the soil shear strength is large enough to render the buoyant force negligible. However, the clay in this research is very soft, and hence the buoyant effect may be much larger than in the other traditional fall cone tests.

● Strain rate

It is a widely accepted view that the strain rate has a significant effect on the soil strength (Berre & Bjerrum, 1973; Vaid & Campanella, 1977; Biscontin & Pestana, 2001; Einav & Randolph, 2005; Boukpeti, 2012). A commonly used relationship between the shear strength and the shear strain rate is expressed as (Graham et al., 1983; Biscontin & Pestana, 2001; Lunne & Andersen, 2007；Boukpeti, 2012):

$$S_u = S_{u,ref} \times (1.0 + \mu \log \frac{\dot{\gamma}}{\dot{\gamma}_{ref}}) \quad (5)$$

where S_u is the shear strength at the strain rate of $\dot{\gamma}$, $S_{u,ref}$ is the reference shear strength at the reference strain rate of $\dot{\gamma}_{ref}$, and μ is a parameter ranging in value from 0.1 to 0.2, according to the results published by Graham et al. (1983), Lefebvre & Leboeuf (1987), and Lunne & Andersen (2007).

● Test methodology

Different test methodologies tend to produce differences in the measured soil shear strength. Chandler (1988) used the vane and viscometer tests as an explanatory example. A viscometer instrument is initially used to measure the viscosity of a fluid. Viscometers only measure under a flow condition. The device is first pushed into the soil, leading to local remoulding and changes in stress, after which a waiting period is allowed before the vane is rotated at a given rate. The strength calculated from the measured torque is influenced by the details of the test, including the waiting time and the rotation rate. Boukpeti (2012) measured the shear strength of remoulded samples of Kaolin and Burswood Clay by a series of fall cone, vane shear and viscometer, T-bar and ball penetrometer tests. This paper highlighted the wide range of strain rates involved in different test methodologies, and showed the reliability and consistency of data after adopting a material model that captures rate dependence.

● Temperature Condition

Tests run at NGI (2009) showed that a lower temperature results in higher shear strengths. The results showed that, for three of the clays, the undrained shear strength was 23–31% higher when the tests were run at the in-situ temperature, +0.5℃, compared to tests run at room temperature (20℃).

● Salt concentration

The geotechnical behaviour of fine-grained soils depends on the chemistry of the pore fluid (Ajalloeian et al., 2013). According to laboratory tests performed by Warkentin & Yong (1962), differences in the inter-particle forces in the clay-water-ion system are reflected in differences in the shear strength. The shear strength parameters have a different meaning for different clays, but it is a commonly accepted concept that they can be related to inter-particle forces.

● Soil composition and structure

Soil composition refers to the mineralogy, grain size, grain size distribution, and the shape of soil particles, as well as the pore fluid type and content, and ions present in the grains and in the pore fluid. Soil structure refers to the arrangement of particles within the soil mass, including the way in which the soil particles are packed or distributed. Features such as layers, joints, fissures, slickensides, voids, pockets, cementation, etc., are part of the soil structure. However, in the fall cone test, most of these aspects of soil structure will be eliminated by the remoulding of the soil.

2. Soils tested and sample preparation

2.1 Fall cone tests

The shear strengths of two different types of clay sample, Kaolin Clay and WND, were measured in the laboratory. Kaolin Clay was prepared by mixing kaolin powder with fresh water to achieve a slurry with a water content of 64% (LI: 0.94). Subsequent samples with higher water contents were obtained by successively adding water to the base sample. Following this procedure, samples with higher water contents were prepared by successively adding water to the base sample to obtain new samples with water contents of up to 166% (LI: 2.54). All clay slurries were mixed by hand until the mixture achieved a uniform consistency. The environmental temperature was controlled at 21°C during the fall cone test, in accordance with BSI (1990), to eliminate the effect of temperature.

In order to avoid errors introduced by using different test methodologies, only one intrusive test method (fall cone test) was used to investigate the undrained shear strength of all the soil samples. Index properties of the two types of soils, such as the liquid limit and plastic limit, were determined by the fall cone test and rolling test (Table 2).

Table 2. Soil index properties

In this research, a conventional 80 g cone with a cone angle $\alpha = 30^0$ was used. However, the lower strength limit of the fall cone was about 400 Pa, as at this strength the cone will penetrate to the base of the cup. This value is still too high for soft marine clays with a high water content. Accordingly, two light cones with the same cone angle $\alpha = 30^0$ were developed and made from aluminium, with masses of 20 g and 13.6 g, as shown in Figure 1. The constant geometric shape of the different cones avoided differences in the remoulding effects during the test.

Figure 1. Three different cones used in the research

Hansbo (1957) expressed the shear strength function of fall cone dynamic penetration depth h in mm, as:

$$\mathrm{S_u} = \frac{\mathrm{KQ}}{\mathrm{h}^2} \tag{6}$$

where Q is the weight of the cone. Through data from Skempton & Northey (1952), Wroth & Wood (1978) proposed the following assumption: (a) the shear strength of soil at the liquid limit is about 1.7 kPa; (b) the shear strength at the plastic limit is 100 times larger, at 170 kPa. In the British Standard (BS1377-2, 1990), the liquid limit tested by the fall cone is determined by an 80 g cone with a 30 ° cone angle that penetrates 20 mm into a soil sample. These assumptions, combined with the critical state relations, yield the idealized relationship:

$$\mathrm{S_u} = \left(\frac{20}{\mathrm{h}}\right)^2 \times 1.7\mathrm{kPa} \tag{7}$$

As mentioned above, it can be seen that the K value is constant for cones with the same apex angle. Therefore, combining Equation (6) into Equation (7) gives the undrained shear strength equation for cones of 20 g and 13.6 g, respectively:

for 20 g cone: $$S_u = \left(\frac{10}{h}\right)^2 \times 1.7\text{kPa} \tag{8}$$

for 13.6 g cone: $$S_u = \left(\frac{8.25}{h}\right)^2 \times 1.7\text{kPa} \tag{9}$$

2.2 Buoyant effect

A simplified schematic diagram of the fall cone test is shown in Figure 2. The geometry of the cone test is described by two variables: the cone angle β, and the penetration depth h.

Figure 2. Buoyant effect in the fall cone test: (a) simplified schematic diagram of the fall cone test; (b) ratio of buoyant force to friction

The buoyant force exerted by the pore water on the cone can be written as:

$$F_b = \frac{1}{3}\pi \cdot \tan^2(\beta/2) \cdot h^3 \cdot g \cdot \rho_{water} \tag{10}$$

where g is the gravity acceleration, and ρ_{water} is the water density.

On the other hand, the component of friction in the vertical direction is simplified as the product of the penetration surface and the undrained shear strain:

$$F_s = \pi \cdot \tan(\beta/2) \cdot h^2 \cdot S_u \tag{11}$$

In order to evaluate the buoyant effect, the ratio of buoyant force to friction is calculated as shown in Figure 2(b). It is shown that the ratio of buoyant force to friction has a linear relationship with the penetration depth and undrained shear strength. When the penetration depth reaches 30 mm and the undrained shear strength is 0.1 kPa, the magnitude of the buoyant force exerted on the cone accounts for 26% of the friction. Hence, it is necessary to consider the buoyant effect in this research. The modified shear strength can be written as:

$$\mathrm{S}_\mathrm{u}' = \mathrm{S}_\mathrm{u} - \frac{1}{3}\tan(\beta/2) \cdot \mathrm{h} \cdot \mathrm{g} \cdot \rho_{\mathrm{water}} \quad (12)$$

2.3 Effects of strain rate

Although having the same geometric shape, the three different cones would still cause different shear strain rates when measuring the same soil sample, and hence it is worth quantitatively analysing the effect of strain rate on the shear strength. Houlsby (1982), Koumoto & Houlsby (2001) estimated the strain rate in the fall cone test, $\dot{\gamma}$, as:

$$\dot{\gamma} = \frac{2\delta}{2.44}\sqrt{\frac{\mathrm{g}\sqrt{3}}{\mathrm{h}}} \quad (13)$$

where g is the gravitational acceleration (unit: m/s^2), δ is the inclination of the heave surface in degrees where the calculated value of δ for the 30^0 cone is taken as 7.87, and h is the penetration depth of the cone in mm. The reference strain rate was taken to be $1.0 \times 10^6\ \%/h$, corresponding to a penetration rate of the 30^0 apex of the fall cone (Koumoto & Houlsby, 2001). The strain rate of this series of fall cone tests is calculated using Equation 13 and is shown in Figure 3. It can be seen that using different cones can lead to a difference in the shear strain rate. When using an 80g cone, the strain rate is about $3.0\times 10^5\%/h$ – $4.0\times 10^5\%/h$ lower than other tests. On the other hand, this difference only leads to a small change in the shear strength according to Equation 5, ranging from 0% to 2%. Hence, the effect of using different cones on measuring shear strength is very limited. To eliminate this limited effect, all the shear strength data was transformed according to Equation 5, based on the reference strain rate of $1.0 \times 10^6\ \%/h$.

Figure 3. Strain rate in the fall cone test: (a) Kaolin; (b) WND

2.4 Effects of salinity

Considering that in reality marine clay incorporates salt water, control experiments were also conducted to test the effect of salinity on the undrained shear strength. Another group of soil samples were mixed using salt water. The salt water was prepared at a salt (sodium chloride) concentration of 3.5%, which is the average salinity of seawater.

3. Results and discussions

The shear strength data collected from all three different cones is combined in Figure 4, for Kaolin Clay and WND sediment. The modified shear strength is calculated based on the original data, using Equation 12. The data extends over a water content range between 64.47% and 142.55% for Kaolin (LI between 0.94 and 3.47), and between 93.79% and 165.92% for WND (LI between 1.02 and 2.54). For Kaolin Clay, the data collected using the 80 g cone covered a range of liquidity indexes between 0.94 and 2, the 20 g cone covered a range of liquidity indexes between 1.1 and 2.64, and the 13.6 g cone covered a range of liquidity indexes between 2.22 and 3.47. For WND the data collected using the 80 g cone covered a range of liquidity indexes between 1.02 and 1.71, the 20 g cone covered a range of liquidity indexes between 1.24 and 2.03, and the 13.6 g cone covered a range of liquidity indexes between 1.77 and 2.54.

Figure 4. Relationship between the undrained shear strength and liquidity index

It has been shown that both original and modified shear strength decrease as the water content for each soil increases. Using the liquidity index, LI, the relationship can be represented by a power function similar to Equation 3. The parameters a and b for each soil are given in Table 3. The coefficient of determination, R^2, is also listed, which indicates a very small scatter in the data obtained with the fall cone tests. As shown in Figure 4, the original and modified trend lines of the two kinds of clay almost overlap each other when the liquidity index is under 1.5, which indicates that the buoyant effect is very limited in this range. However, when liquidity index is above 1.5, the modified shear strength begins to deviate from the original data due to the ratio of buoyant force to friction increasing as the shear strength of soil decreases. The maximum difference can be about 20% for Kaolin Clay and 30% for WND. Hence, the buoyant effect must be considered when using the fall cone test to measure extremely soft clay.

Table 3. Parameter describing S_u as a function of LI

Figure 5 compares the undrained shear strength at the overlap regions of the liquidity index with different cones, in order to demonstrate the consistency of the shear strength data collected using cones of different weights.

(a) Original shear strength of Kaolin Clay

(b) Modified shear strength of WND

Figure 5. Comparison of shear strengths measured using different cones

The modified shear strengths collected using different cones agreed with each other quite well at the overlap regions of the liquidity index. Assuming that the shear strength can be approximated using a power function, as previously discussed, the correlation coefficient could be calculated to quantify the amount of scattering in the data obtained using different cones. All of these values were close to 1.0, as given in Table 3, which indicates that from a quantitative point of view there is a good match between data measured using different fall cones.

By plotting the relationships proposed by Wroth & Muir Wood, (1978) for the LI range below 1, Leroueil et al. (1983) for the LI range 0.4–3 and Locat & Demers (1988) for LI range 1.5–6, as shown in Figure 6, it was found that the original shear strength of the Kaolin Clay fits well with the relationship proposed by Leroueil et al.

This is higher than Locat & Demers's prediction, especially in soils with a high water content. However, the modified shear strength of Kaolin Clay is closer to the data from Locat & Demers (1988). This is partly because Locat & Demers' results are from viscometric tests, on which the buoyant force has a very limited effect. Hence, it is concluded that, when considering the buoyant effect, the fall cone test results can be very close to the viscometric tests when testing the undrained shear strength of soft clay. On the other hand, the modified shear strength of WND was much lower than the Kaolin Clay with the same liquidity index, which verifies the common sense understanding that the low shear strength of marine clay is due to not only to the high water content, but also to the soil composition.

(a)

(b)

Figure 6. Comparison between different strength-liquidity index correlations: (a) original data; (b) modified data

A comparison of the salinity effect on shear strength is shown in Figure 7 and Table 4. The two data series seem to follow a similar tendency as the water content increases. However, the strength data from all the salinity samples was slightly higher than the data from the samples mixed using fresh water. A quantitative comparison is listed in Table 4. We might expect to see a 9.2%–13.4% greater enhancement for Kaolin and a 8.6%–39.1% greater enhancement for WND. In this study, it has been shown that the impact of saline water on the undrained shear strength of Kaolin Clay and WND is different, which indicates that sensitivity of shear strength on the change of salinity is material dependent. WND is more sensitive to the change of salinity. On the other hand, the coefficient of determination, R^2, is also listed in Figure 7. It is shown that R^2 decreases to 0.93 and 0.96 for Kaolin and WND with saline water, respectively, which indicates that saline water clay is a little more scattered than fresh water clay. This decrease in R^2 may be due to the smaller number of saline water clay tests. When the liquidity index is close 1, the relationship between shear strength and liquidity index seems to capture the effects related to soil type (Boukpeti, 2012). The shear strength is dominated by the friction between soil particles, and hence the salinity effect is very limited. This is why the difference in the shear strength between fresh water clay and saline water clay is very small. However as the water content increases, marine clay evolves from a soil to a viscous fluid. Accordingly, the shear strength follows the fluid mechanics framework. In this range the contact between the soil particles may have less effect on the shear strength, and instead the internal forces existing between the soil particles may have a stronger influence on the resistance. Ingles (1962) referred to this kind of force as Van der Waal's Electromagnetic Bonding. According to the test results, salinity may have no significant effect on soil strength at conventional water contents, but, as friction strength declines, it may play an important role in the internal forces between soil particles.

(a) Kaolin Clay

(b) WND

Figure 7. Comparison of results for the liquidity index and the shear strength of soil mixed with saline water and fresh water

Table 4. Shear strength increment due to increased salinity

4. Conclusion

This paper studies the undrained shear strength of marine clay using fall cone tests. Due to the effect of the strain rate on the shear strength caused by using different cones, all the data collected from the tests first had to be unified under a consistent strain rate. The main conclusions are summarized as follows:

(1) The penetration mechanism of a fall cone into clay is simple and clear, and hence the fall cone test is a useful method for determining the undrained shear strength by interpreting the penetration depth of the cone.

(2) The modified shear strength collected using different cones matched well with each other at the overlap regions of the liquidity index, which indicates that the impact of using different fall cones can be neglected.

(3) A relationship between the undrained shear strength of clay and the liquidity index is proposed in the form of a power function. The results showed that the strength degradation produced by increasing the water content is material dependent.

(4) Compared to Kaolin Clay, the marine clay (WND) is weaker, especially for soils with a higher water content. This indicates that the low shear strength of

marine clay is due not only to the high water content, but also to the soil composition.

(5) When the liquidity index is above 1.5, the modified shear strength begins to deviate from the original data. The maximum difference is about 20% for Kaolin Clay and 30% for WND. Hence, the buoyant effect must be considered when using the fall cone test to measure extremely soft clay.

(6) This paper demonstrates the influence of salinity on the undrained shear strength. A series of control tests were conducted with samples mixed using 3.5% salt water. It was found that salt water produces a slight increase of about 9.2%–13.4% in the undrained shear strength for Kaolin Clay but a large increase of 8.6%–39.1% in undrained shear strength for WND, which indicates that the impact sensitivity is material dependent.

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