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STUDY ON RHEOLOGICAL CONSTITUTIVE MODEL OF CILILIN VOLCANIC CLAY, INDONESIA IN RELATION TO LONG-TERM SLOPE STABILITY

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ABSTRACT: Ground movements do not remain constant over time but it is a time-dependent phenomenon in the form of long-term deformations due to in-situ stresses acting all the time on the slope body. Therefore, the time-dependent behavior of soil mass is an important factor that influences long-term slope stability. To understand the long-term slope stability, research on soil rheological characteristics is required so that the time-dependent soil strength reduction can be known. The result of the research will be very useful for the policymaker in disaster mitigation and regional planning. To determine the rheological characteristics of Cililin Volcanic Clay, a laboratory shear creep test was carried out on soil samples taken from the Cililin area, West Java, Indonesia, which is well known as landslide-prone areas. Fifteen undisturbed soil samples were prepared for laboratory shear creep tests. The level of shear stress applied to the shear creep test is 50% - 95% of the peak strength. Then, the soil rheology constitutive models were established which were used to understand the creep behavior of the soil, determine the soil long-term strength, estimate the reduction of soil parameters as a function of time and calculate long-term slope stability. The result shows that the long-term strength of soil is reduced 48.80% from its peak strength after 16 years. The cohesion and internal friction angle reduced from 44.72 kPa and 30.34^o become 21.82 kPa and 15.94^o, respectively, and the slope safety factor is reduced from 2.16 to 1.04. Rheological modeling also shows that slope stability is a function of time.

Keywords: Rheology, Creep, Long-Term Strength, Time Dependent Behavior, Long-term Slope Stability

1. INTRODUCTION

As it is generally known that slope stability does not remain constant over time [1], but it is a long-term deformation phenomenon that causes a time-dependent reduction in the shear strength of the soil mass [2,3]. Long-term deformation moves very slowly even up to mm/year that it is usually not observed [4]. Slow slope movement which is caused by constant stress is known as creep. Vyalov [5] states that creep is a long-term deformation that occurs permanently due to constant stress acting all the time on the slope body, but it has not yet caused landslides. The constant stress in the form of in-situ stress will not have a significant effect on the short-term slope stability, however, this will cause a long-term deformation which will influence the long-term slope stability [2]. Due to the small degree of deformation caused by in-situ stress, this condition is often ignored [6,7]. However, in densely populated areas, it is very important to know the long-term stability of slopes and understanding its behavior in relation with time. Therefore, it is important to conduct research on long-term slope stability and its time dependent behavior so that the results of the research will be very useful for the policymaker in disaster mitigation and regional planning.

In relation to time-dependent slope stability, several studies have been conducted. Din et al., [8] researched the high slope at the Shuibuya Damsite. The compressive creep tests were carried out on the soft rock and hard rock and the results concluded that in the long-term, changes in stress in the field due to creep can cause a plastic zone in the rock mass so that in the long-term the slope becomes unstable. Li et al. [9] conducted a laboratory shear creep on soft-rock taken from coal mine in Xinjiang China and concluded that there was a decrease in slope stability over a period of time due to the constant stress acting on the slope body. Tianghong et al. [10] researched creeping behavior with a shear creep test on rocks taken from the coal mines in China and propose a failure criterion to determine the long-term strength of the rock mass. Chang et al., [11] conducted a geomechanical-based numerical modeling study to qualitatively explain the causes and evolution of slope creeping behavior in the Lusan Formation, Taiwan.Wu et al. [12] conducted experimentally and modeling of shear rheology on sandstones with non-persistent joints. Creep test on cube-shaped sandstones was carried out using a rock shear rheometer and the results concluded that the long-term shear strength is influenced by the long-term cohesion and internal friction angle of the rock, and reduced cohesion is an important factor in long-term strength due to constant stress. Zao et al. [13] studied slope creep law on a mudstone from open-pit mine in Xinjiang China. The results concluded that based on the results of the compressive shear test, the long-term strength of the mudstone is decreases over time and the stable creep occurs at low shear stress levels.

Most of these studies discuss the creeping behavior in relation to long-term slope stability but have not discussed the long-term slope stability reduction and its behavior in relation to time and formulate this into a mathematical equation. With the mathematical equations obtained, it will be possible to predict the failure time of a slope due to constant stress. Several studies have also been conducted to predict the failure of a slope. Saito and Uezawa [14] conducted a series of studies to predict the landslide time of a slope. The research was carried out by conducted direct observations in the field by recording daily slope movements in relation with time and the result shows that the slope failure has close relation with the deformation rate. Fukuzono [15] conducted a direct slope monitoring and predict the slope failure time using the Inverse Number of Velocity of Surface Displacement. Adriansyah [16] conducted a prediction of slope failure time on mine slopes in West Nusa Tenggara Indonesia based on daily slope movement monitoring and proposed the sliding constant value for that site. However, direct observation of landslides in the field will take a long time and relatively expensive, so it is necessary to propose a method to estimate the time-dependent reduction of soil strength parameters to obtain long-term slope stability and estimate the time of slope failure. This paper discusses the time-dependent reduction in soil strength parameters based on the rheological constitutive model obtained from laboratory shear creep tests to calculate long-term slope stability and predict the time failure of the slope. A series of shear creep tests were carried out on undisturbed soil samples taken from the research site. The rheological equations established from the shear creep test were used to determine the time required for the soil samples to fail; then the long-term stability of the slopes and their behavior with respect to time will be determined.

2. MATERIALS AND METHODOLOGY

2.1 Research Location

The research location was carried out at the southern slope of Lembang Village, Cililin, West Java, Indonesia. The Cililin area and its surroundings are known as landslide-prone areas. Almost every year, especially in the rainy season, large and small landslides often occur in this area [17]. In this location in 2013, there was a big landslide that killed 18 people, damaged 10 houses and evacuated 23 heads of families. Landslide materials moving as far as 500m hit densely populated settlements below.

Geologically, the research area is a Miocene age of weathering soil of volcanic material composed of unconsolidated tuff and pyroclastic fall monomic Breccia [18]. XRD test performed on the soil sample showed the content of halloysite which is classified as Sensitive Clay [19].

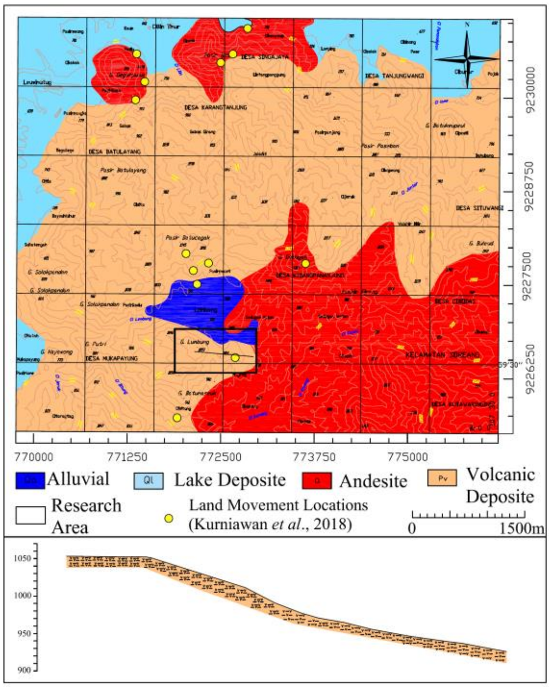

Fig.1 Geological Map and Section [17,18]

2.2 Soil Creep

Creep of soil is the tendency of a soil mass to deform slowly and permanently due to the constant stress in the form of in-situ stress that works continuously [5]. To understand the time-dependent behavior of soil creep due to constant stress, shear creep tests were carried out on fifteen (15) undisturbed soil samples at GeoACE's soil mechanics laboratory in Bandung, Indonesia, so that the rheological parameters will be obtained and the rheological equation can be established. The rheological equation is an equation that describes the amount of deformation as a function of time, so this equation can be used to determine the time it takes for a soil sample to reach deformation where failure begins.

Creep is classified into 4 stages, namely: instantaneous elastic deformation (ε_o), primary creep (ε_1), secondary creep (ε_2), and tertiary creep (ε_3) as can be seen in Fig.2 [5,7,20].

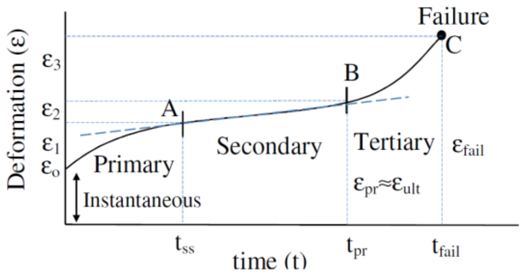

Fig.2 Creep Curve [5,7,20]

The stages of creep as time-dependent deformation refer to Fig.2, can be formulated:

$$\boldsymbol{\varepsilon_{(t)} = \varepsilon_0 + \varepsilon_1|_{0}^{t_{ss}} + \varepsilon_2|_{t_{ss}}^{t_{pr}} + \varepsilon_3|_{t_{pr}}^{t_{fail}}} \quad (1)$$

From Fig.2 and Eq. (1) it can be illustrated that immediately after the loading is applied, the test sample will experience initial acceleration in the form of instantaneous deformation (ε_o) that occurs at time t_0. The test sample will react to the initial loading and attempt to achieve stability. This sample reaction will slow down the deformation until it reaches a stable condition. The creeping process is then started by primary creep (ε_1). The ε_1 is the attenuation deformation that develops at time intervals 0 - t_{ss}. ε_1 occurs after ε_o in the form of exponential displacement and will stabilize at a certain time. After the stability is achieved, the test sample will experience a linear displacement at a constant rate (ε_2). The ε_2 occurs in the time range t_{ss} - t_{pr}. In ε_2, the shearing between the shear planes will cause the surface of the shear plane to become smoother. This reduces the strength of the sample to withstand given shear stress. When the resistance of the test sample is exceeded, the displacement acceleration will occur as the beginning of the tertiary creep stage (ε_3). ε_3 is a creep in which progressive deformation (a condition where the deformation rate begins to increase, ε_{pr}) begins to occur until the test sample collapses. ε_3 occurs in the time range t_{pr} - t_{fail}.

In the creep test, two types of creep may occur, which are a non-attenuating creep and attenuating creep [5]. Non-attenuating creep is a creeping process in which all four stages of creep (ε_o, ε_1, ε_2, ε_3) are fulfilled. Non-attenuating creep occurs when the applied shear stress level is large enough so that the soil sample failure. Refer to Fig.1, non-attenuating creep is drawn in segments O, A, B and C. While attenuating creep is a creeping process where during the observation time only instantaneous deformation (ε_0), primary (ε_1) and secondary creep (ε_2) occur. The ε_3 is not achieved in this creeping process. This condition is caused due to the small level of shear stress applied so that the creep process does not fail. Refer to Fig. 1, attenuating creep is drawn in segments O, A and B. In this condition, the secondary creep looks stable, tends to decrease and shows long-lasting creep [5].

In some tests, to reach the four stages of creep where the material is subjected to failure, sometimes it takes a very long time (weeks, months, or even years) [5]. Haefeli [21] conducted the creep test in a shear ring apparatus under constant stress for two months and the material had not yet collapsed. He was then destroyed some specimens by a steeped loading. Bishop and Lovenbury [22] conducted a triaxial creep test on undisturbed clay up to more than 1000 days and the strain rate still falling at 1250 days. For research purposes, shortening the creep test duration is allowed under various conditions. To obtain rheological parameters for a short period of time, [5] suggested to do a creep test for at least 10 days. Tran et al, [4] conducted a triaxial creep test on halloysite clay for 10^4 minutes. The author believes that the creep test can at least be terminated if the secondary creep curve has formed perfectly indicated by the strain rate shows a steady-state condition. In this research, the authors conducted a maximum creep test for ±7 days for each test sample or maybe faster if the sample failed. Vyalov [5] states that in long-lasting creep, failure does not occur at that time but will occur sometime time where the deformation is a function of time. In this condition, tertiary creep stage (ε_3) will be reached for a very long time. Therefore, to simplify the analysis, tertiary creep (ε_3) can be excluded from the analysis [23, 24], so that Eq. (1) can be rewritten as:

$$\boldsymbol{\varepsilon_{(t)} = \varepsilon_0 + \varepsilon_1|_{0}^{t_{ss}} + \varepsilon_2|_{t_{ss}}^{t_{pr}}} \quad (2)$$

If failure does not occur while stress is applied, the maximum deformation that occurs during the test is defined as the ultimate deformation (ε_{ult}) of the soil being tested [5]. While the strength limit is the stress condition at which the material begins to fails and is described by progressive deformation (ε_{pr}). In this paper, the authors propose to use the rheological equation obtained from the shear creep test to get the value of ε_{pr}, so that in this case it is assumed that $\varepsilon_{ult} \approx \varepsilon_{pr}$ (see Fig.1). Deformation when the sample failure calculated using the formula as suggested by [5]:

$$\varepsilon_{fail} = 1.5\ \varepsilon_{pr} \quad (3)$$

The ε_{fail} value was obtained based on a laboratory direct shear test (quick test) conducted on a soil sample before the shear creep tests. Goldstein and Ter-Stepanian [25] conducted a series of creep tests on clay samples and stated that in similar soil samples and the same type of test, the soil samples will fail at different times according to the stress level applied but deformation occurs with relatively the same magnitude.

2.3 Laboratory Shear Creep Test and Long-term Strength Curve

The normal load and constant shear stress applied in the test equipment is a weight system so that the application of normal load and shear stress is expected to be constant during the test.

The important things that need to be considered in the operation of the test equipment are:

1. Able to guarantee the applied normal load and shear stress are constant during the test time.
2. Able to read the magnitude of shear displacement accurately.
3. Easy operated for convenience and easy to change the level of normal load and constant shear stress.

Before the shear creeps, the standard direct shear test (quick test) was carried out first. The result of the standard direct shear test is used as a reference to determine the level of shear stress on the shear creep test. The normal load in the shear creep test is similar to the normal load in the standard direct shear test, which are 10 kPa, 20 kPa and 30 kPa, respectively. The given shear stress level ranges from 50% - 95% from its peak shear strength.

The result of the shear creep test is in the form of a series of creep curves as shown in Fig.3a. The curve will show the shear deformation (mm) against the loading time (minutes) for each sample and the test results will show the visco-plastic for non-attenuating creep and visco-elastic for attenuating creep. In non-attenuating creep, the time at which the material fails is determined as the failure time of the material. Meanwhile, in attenuating creep, the failure time of the material will be determined based on the rheological equation established from the shear creep test. The long-term strength curve is obtained from re-plotting the shear creeps test results into one graph as suggested by [5] (Fig.3b).

2.4 Simple Rheological Constitutive Model

The basic rheological stress-strain relation can be constructed based on a simple mechanical conceptual model based on the spring and dashpot [7,20]. The Spring element represents the elastic Hookean model, while the Dashpot element represents the Newtonian viscous model (Fig.4).

The rheological constitutive model is a mathematical equation to describe the behavior of a material in terms of stress, deformation and time. The basic principle of the combination of these models is to describe the basic components of a material such as elastic elements, plastic elements and viscous elements the material. These components can be combined to form various constitutive rheological models that represent different rheological properties of materials by means of series and parallel relationships. Each component connected in series divides the stress evenly and the sum of all elements is equal to the total deformation rate. In terms of parallel relationships, the sum of all elements is equivalent to the total applied stress and the deformation rate of each element will be equal to each other [26].

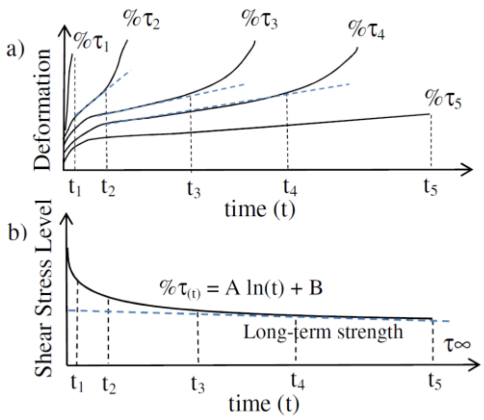

Fig.3 Long-term Strength Construction [5];
a) Creep Curve, b) Long-term Strength Curve

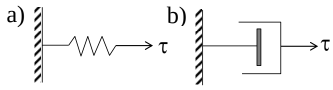

Fig.4 Basic Rheological Model;
a) Hookean (Spring), b) Newtonian (Dashpot)

The most widely used is the Burgers rheological model. There has been a lot of research on rheology that uses the burgers model with satisfactory results. Burgers model schematic is depicted in Fig.5.

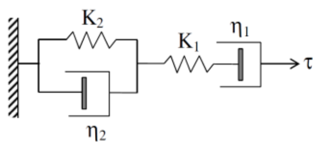

Fig.5 Burgers Rheological Model Scheme

Referring to Fig.5, it can be seen that the Burgers rheological model is divided into 3 stages, namely instantaneous deformation (ε_0), primary creep (ε_1) and tertiary creep (ε_2) which are represented in Eq. (4):

$$\boldsymbol{\varepsilon_o} = \frac{\tau}{K_1};\ \boldsymbol{\varepsilon_1} = \frac{\tau}{K_2}\left(1 - e^{-\frac{K_2}{\eta_2}t}\right);\ \boldsymbol{\varepsilon_2} = \frac{\tau}{\eta_1}t \qquad (4)$$

The Burgers rheological equation is a sum of ε_0, ε_1, and ε_2 and for shear stress applications it can be written as:

$$\boldsymbol{\varepsilon_{(t)}} = \frac{\tau}{K_1} + \frac{\tau}{K_2}\left(1 - e^{-\frac{K_2}{\eta_2}t}\right) + \frac{\tau}{\eta_1}t \qquad (5)$$

From Fig.5 and Eq. (5), it is clear that the Burgers rheology model is composed by three creep components ε_0, ε_1, and ε_2. This is consistent with the Eq. (2) suggested by [23,24] that for practical reason, tertiary creep (ε_3) can be excluded from the analysis.

2.5 Long-term Strength of a Material

Long-term strength is the stress condition under which failure begins and is a response of a material due to constant long-term stress and at a certain time, the material will experience to fail. The shear stress level reduction in relation to time is expressed by the formula [5]:

$$\%\boldsymbol{\tau_{p(t)}} = \frac{\beta}{\ln\frac{t_f+1}{T}} \qquad (6)$$

Where $\%\tau_{p(t)}$ is reduction of stress level as function of time in %, β = 1/B and T=$e^{-\beta D}$. When t = t_o, the Eq. (5) become:

$$\%\boldsymbol{\tau_{po}} = \frac{\beta}{\ln\frac{1}{T}} \qquad (7)$$

So, the long-term strength can be determined as:

$$\%\boldsymbol{\tau_{plt}} = \frac{\beta}{\ln\frac{t_{lt}}{T}} \qquad (8)$$

t_{lt} is time to reach the long-term strength where the strength reduction is less than 5% in 100 years and determined by equation:

$$\boldsymbol{t_{lt}} = (100\, T^{0.05})^{1/1.05} \qquad (9)$$

3. RESULTS AND DISCUSSION

3.1. Basic Laboratory Test

Basic laboratory tests were carried out before the shear creep test. The results of the basic laboratory test are listed in Table 1.

Table 1 Basic Laboratory Test

Laboratory test	Symbol	Unit	Value
Index Properties			
Water Content	wc	%	31.95
Specific Gravity	Sg	kN/m^3	2.66
Bulk density	γn	kN/m^3	16.92
Dry density	γdry	kN/m^3	12.82
Atterberg limit			
Liquid limit	LL	%	43.01
Plastic limit	PL	%	28.87
Plastic index	PI	%	14.13
Soil class		Class	ML-CL
Grainsize distribution			
Clay		%	73.83
Silt		%	24.71
Sand		%	1.46
Direct shear test			
Cohesion	C	kPa	44.72
Internal friction angle	o	Deg.	30.34

3.2. Shear Creep Test

The normal loads applied to the shear creep test were 10 kPa, 20 kPa and 30 kPa, respectively. At each normal load, 5 samples of the shear creep test were carried out. The applied shear stress level is 50% - 95% from the peak shear strength. The shear creep curve is depicted in Fig.6.

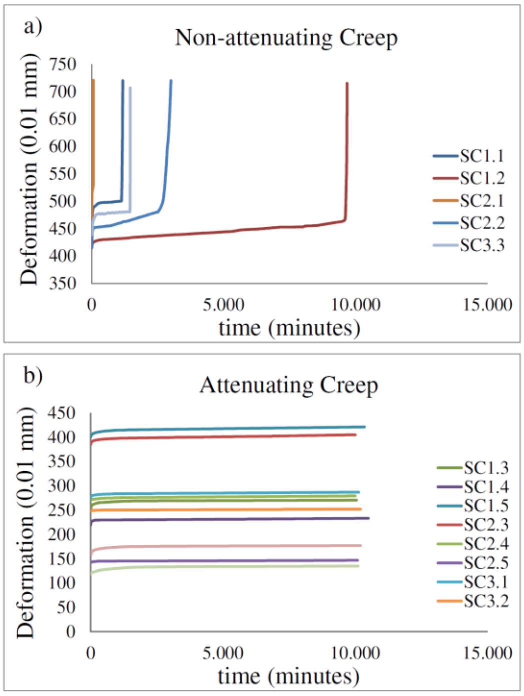

Fig.6 Shear Creep Curve

Fig.6 shows that there are 2 types of creep curves, namely the non-attenuating creep curve and attenuating creep curve. In the non-attenuating creep (Fig.6a), the instantaneous deformation (ε_o), primary creep (ε_1), secondary creep (ε_2) and primary creep (ε_3) are achieved perfectly. The shear creep test shows that the failure time ranges from 60 minutes to 6 days. The duration of failure time is closely related to the shear stress level applied. The higher of shear stress level the faster failure is achieved. Non-attenuating creep occurred in samples SC1.1, SC1.2, SC2.1, SC2.2 and SC 3.3 with a shear stress level of 92.29%, 83.06%, 95.00%, 86.52% and 88.00%, respectively.

In the attenuating creep (Fig.6b), only instantaneous deformation (ε_o), primary creep (ε_1) and secondary creep (ε_2) were observed. In this condition, failure was not achieved during the test period (±7 days). This is due to the small level of shear stress applied so that in the secondary creep stage (ε_2) the deformation rate tends to be stable and shows long-lasting creep. Attenuating creep are shown in the test samples SC1.3, SC1.4, SC1.5, SC2.3, SC2.4, SC2.5, SC3.1, SC3.2, SC3.4 and SC3.5 with shear stress levels ranging from 52.15% - 72.35%. In attenuating creep, the rheological equation obtained from the shear creep test will be used to estimate the soil failure time.

The two conditions discussed above indicate that the level of constant shear stress has a significant effect on the rheological characteristics of a material.

3.3. Rheological Constitutive Model

The rheological constitutive model is an equation that describes the physical and mechanical properties of a material based on the creep test refer to Eq. (5). In this paper, the author will predict the time required ($t_{\varepsilon pr}$) to reach progressive deformation (ε_{pr}) with the assumption that the ε_{pr} is the deformation when the test sample begins to fail. The value of ε_{pr} is determined based on Eq. (3). Laboratory direct shear test (quick test) shows that the deformation at failure (ε_{fail}) is 7.4 mm, so the value of ε_{pr} is 4.911 mm. Burgers rheological parameters and their relationship to the time required to reach ε_{pr} are presented in Table 2.

Table 2 Burgers Constitutive Model and $t_{\varepsilon pr}$

Sample	$\%\tau_p$	τ/K_1	τ/K_2	K_2/η_2	τ/η_1	$t_{\varepsilon pr}$ (min)
SC1.1	92.29	4.41	0.47	0.070	1E-4	1175
SC1.2	83.06	4.19	0.09	0.014	4E-5	9681
SC1.3	60.91	2.61	0.08	0.002	2E-6	1111550
SC1.4	57.22	2.24	0.06	0.015	3E-6	872000
SC1.5	72.35	4.03	0.11	0.004	7E-6	109650
SC2.1	95.00	4.48	0.61	1.170	4E-3	60
SC2.2	86.52	4.30	0.21	0.042	1E-4	3000
SC2.3	67.86	3.90	0.07	0.004	8E-6	117788
SC2.4	61.07	2.71	0.04	0.003	4E-6	539253
SC2.5	52.59	1.42	0.03	0.009	2E-6	1730655
SC3.1	61.92	2.79	0.04	0.003	4E-6	519140
SC3.2	58.66	2.49	0.01	0.005	2E-6	1205200
SC3.3	88.00	4.42	0.33	0.023	5E-5	1459
SC3.4	53.78	1.65	0.10	0.002	2E-6	1581700
SC3.5	52.15	1.20	0.13	0.002	2E-6	1789610

Figure 7 is a long-term strength curve resulted from Table 2 which is determined by plotting the shear stress level in correlation to time ($t_{\varepsilon pr}$).

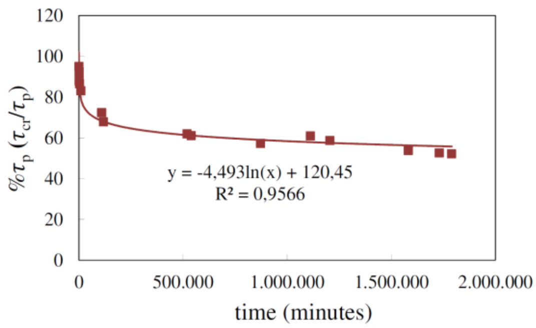

Fig.7 Long-term Shear Strength Curve

From Fig.7 it can be seen that at high shear stress levels between 80% - 95% the failure time is quite fast ranging from 60 minutes to 6 days which is marked by a sharp downward curve. The curve begins to gentle after the shear stress level is reduced to 70%. For smaller shear stress levels, the curve continues to gentle and shows a relatively stable rate. Long-term shear strength in relation to time in Fig. 7 can be represented in Eq. (10).

$$\%\boldsymbol{\tau_{p(t)}} = -\mathbf{4.493}\ \mathbf{ln}(\boldsymbol{t}) + \mathbf{120.45} \tag{10}$$

Where $\%\tau_p$ is shear stress level reduction (%) in relation to time (t) in minutes.

3.4. Long-term Strength and Time Dependent Behavior of Cililin Volcanic Clay

The reduction of shear stress level ($\%\tau_{plt}$ and $\%\tau_{p(t)}$) is calculated by Eq. (8) and (10). While the time required to reach the Long-term shear strength (t_{lt}) is determined by Eq. (9). The result is listed in Table 3.

Table 3 Long-term shear strength

t_{lt} (year)	$\%\tau_{plt}$	Peak		Long-term	
		c_p (kPa)	ϕ_p (°)	c_{lt} (kPa)	ϕ_{lt} (°)
16	48.80	44.72	30.34	21.82	15.94

Table 3 shows that the long-term shear strength is decreased by 48.80% from its peak strength after 16 years. The soil parameters c and ϕ decreased from 44.72 kPa and 30.34° to 21.82 kPa and 15.94°, respectively. The Mohr-Coulomb criterion of long-term shear strength and peak shear strength is depicted in Fig.8.

Equation (10) is used to determine the soil strength reduction (c and ϕ) in relation to time and the results are listed in Table 4.

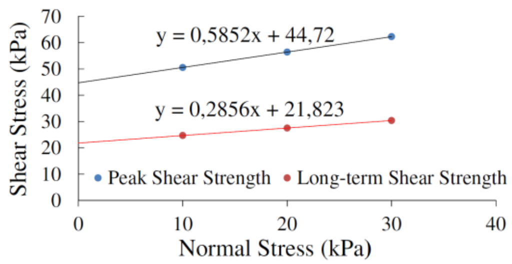

Fig.8 Mohr-Coulomb Criterion of Peak Shear Strength and Long-term Shear Strength

Tabel 4. c and ϕ in Relation to Time

Stress Level ($\%\tau_p$)	Failure Time (years)	$C_{(t)}$	$\phi_{(t)}$
48.80	16.0	21.82	15.94
55	4.03	24.59	17.84
60	1.32	26.83	19.35
65	0.44	29.07	20.83
70	0.14	31.30	22.27
75	0.047	33.54	23.69
80	0.015	35.78	25.09
85	0.005	38.01	26.45
90	0.002	40.25	27.78

Fig.9 is the reduction in soil parameters (c and ϕ)) as a function of time based on Table 4.

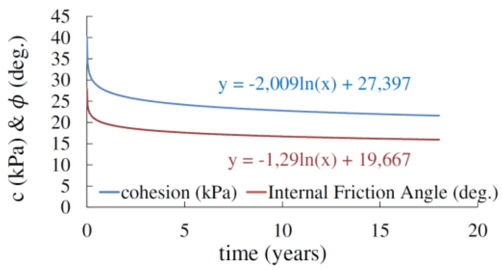

Fig.9. c and ϕ as Function of Time

Figure. 9 shows that the cohesion and internal friction angle decrease with time due to constant stress and written as:

$$c_{(t)} = -2.009\ln(t) + 27.397 \quad (11)$$
$$\phi_{(t)} = -1.290\ln(t) + 19.677 \quad (12)$$

Equations (11) and (12) show that the constant stress has a significant effect on the decrease in soil parameters (c and ϕ) as a function of time.

3.5. Long-term Slope Stability

Based on Eq. (11) and (12) we can determine the value of c and ϕ at a certain time. As we know, c and ϕ are important components in the calculation of slope stability so that we can simulate slope stability in relation to time as tabulated in Table 5.

Table 5. SF Reduction in Relation with Time

Time	Soil Parameter		SF
	$C_{(t)}$ (kPa)	$\phi_{(t)}$ (°)	
0	44.72	30,34	2.16
0.5m	33.78	23,77	1.62
1m	32.39	22,87	1.56
2m	31.00	21,98	1.49
3m	30.18	21,46	1.45
6m	28.79	20,56	1.38
10m	27.76	19,90	1.33
1y	27.40	19,67	1.32
1.5y	26.58	19,14	1.28
2y	26.00	18,77	1.25
3y	25.19	18,25	1.21
4y	24.61	17,88	1.19
5y	24.16	17,59	1.17
10y	22.77	16,70	1.10
15y	21.96	16,17	1.07
16y	21.82	15,94	1.04

Table 5 shows that the Slope Safety Factor decreased from 2.16 to 1.04 within a period of 16 years where the collapse will begin to occur. The long-term Slope Safety Factor Curve is depicted in Fig.10.

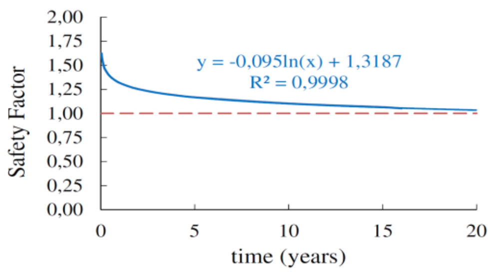

Fig.10 Long-term Slope Safety Factor Curve

From Fig. 10 it can be seen that the decrease in SF with time can be formulated by the equation:

$$SF_{(t)} = -0.095\ln(t) + 1.3187 \quad (13)$$

4. CONCLUSION

Based on the discussion above, it can be concluded that the slope stability decreases with time due to the in-situ stress acting all the time on the body of the slope. With the equation obtained from the Slope Safety Factor reduction in relation to time, we can estimate the time required for the slope to reach equilibrium so that disaster mitigation and regional planning can be carried out in a more planned manner.

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