

Strength and Deformation Characteristics of Bandung Lacustrine Clay Under Consolidated-Undrained True Triaxial Testing

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ABSTRACT: The effects of intermediate and principal stress rotation are commonly ignored when determining soil stress-deformation behavior under two-dimensional or plane-strain conditions. In actual field conditions, soil experiences three-dimensional loading and undergoes stress rotation to reach a stable condition. Therefore, a stress-deformation test that accounts for the effects of intermediate- and principal-stress rotation to capture real soil behavior was conducted. Bandung lacustrine soft clay is formed by the deposition of clay in a giant ancient lake, estimated to have a depth of about 200 m below ground surface. Undisturbed soil samples from this soft clay were taken and investigated in the laboratory by using the Consolidated-Undrained True Triaxial Test. The experiment shows that effective shear strength (ϕ') increases with increasing stress variation (b) up to 10% with an increment of intermediate principal stress 0.5 times the major principal stress. Increasing the principal stress rotation from 0 (zero) degrees to 90 (ninety) degrees decreases undrained shear strength (S_u/σ_c) by up to 15%. The stiffness modulus and the Skempton A_f parameter also increase with increasing stress variation. The results obtained from this study are important for evaluating a suitable soil constitutive model for finite-element load-deformation analysis that accounts for anisotropic behavior.

Keywords: True Triaxial, Stress Variation, Stress Rotation, Strength-Deformation Characteristic, and Bandung Lacustrine Clay.

1. INTRODUCTION

It is known that field ground conditions, when responding to loads, are highly stress-path-dependent. Currently, conventional triaxial testing is widely used to obtain soil shear strength parameters and stress-strain behavior. However, this test only describes soil elements loaded under axisymmetric conditions, where only two types of stresses occur: σ_1 and σ_3 . In fact, the soil element experiences stress in three directions, which are called the principal stresses. All three are σ_1 called the major principal stress, σ_2 is called the intermediate principal stress, and σ_3 is called the minor principal stress. Laboratory equipment capable of involving these three stresses is a True Triaxial Cell (TTC). This tool can independently apply the main stress and model the soil under compression and extension conditions, with loading in the axial and lateral directions.

Bandung clay is a lacustrine soil type formed by deposition in a giant ancient lake in the southern part of the Bandung Basin. Based on the Bandung geological map, the southern and eastern parts of Bandung City are dominantly deposited with soft lacustrine and alluvial soils, as shown in Figure 1.

Figure 2 shows the North-South geological cross-section of the Bandung Basin, which clearly shows the deposition of Bandung lacustrine clays.

Clayey soil has very low permeability, so it is strongly influenced by interactions between water in the soil pores and the soil particles. When loading occurs faster than the dissipation of excess pore water pressure, the direct stress from the increase in load is first carried by the pore water. This phenomenon is referred to as undrained behavior of saturated clay soil. The collapse of the geotechnical material is determined by the effective soil strength parameters, which are obtained by subtracting pore water pressure from total stress. Studies on the influence of σ_2 , or intermediate principal strain, by Prashant and Dayakar (2004) indicate that this stress plays a significant role in the stress-strain behavior and pore water pressure response of clay.

Studies on the behavior of Bandung lacustrine clay remain far from complete, especially regarding the influence of major stresses

on modeling of actual field conditions. The study of the stress-strain behavior of Bandung lacustrine soft clay was first conducted by Lumbantoruan (2013) using a true triaxial cell. In this study, the effects of various stress paths in unconsolidated undrained (UU) and consolidated drained (CD) testing have been examined to assess variations in the shear and tensile strengths of this clay soil. However, the effects of major principal stress (σ_1) and intermediate principal stress (σ_2) in obtaining the behavior of deformation stress in the consolidated undrained (CU) test of Bandung lacustrine clay have not been studied further.

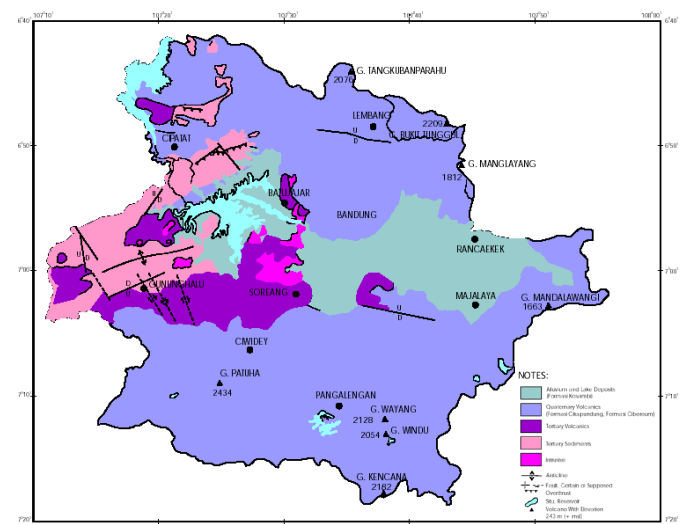


Figure 1 Geological map of Bandung (Dept of Energy and Mineral Resources West Java & ITB-Institute for Research and Community Services, 2002)

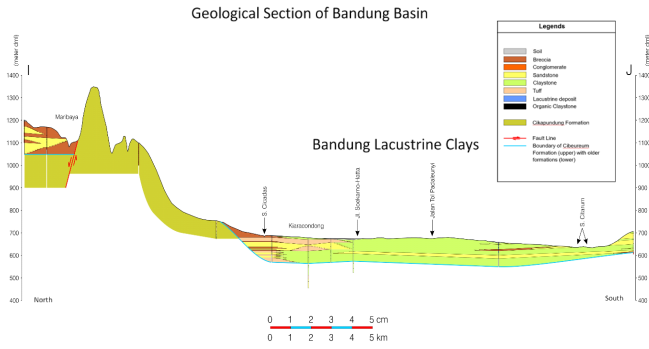


Figure 2 North-South geological cross section of Bandung Basin (Dept of Energy and Mineral Resources West Java & ITB-Institute for Research and Community Services, 2002)

2. EXPERIMENTAL WORKS

Soil samples used in this work were taken at Gedebage sub-district, Bandung City. The soil sampling process for the TTC test generally differs from that for the triaxial test because the sample is cube-shaped, measuring $8.0 \times 8.0 \times 8.0 \text{ cm}^3$. Therefore, the sampling process in this study was carried out using a hand auger and a 15 cm tube sample to minimize disturbance. After the soil samples taken from the field are brought to the laboratory, the next step is to cut each sample to $8.0 \times 8.0 \times 8.0 \text{ cm}^3$. The cutting process should be performed slowly and carefully to avoid damaging the soil sample. Then the soil sample, sorted by size, is stored in the desiccator to maintain its moisture content. Prior to testing with the True Triaxial Cell (TTC), the mechanical and physical properties of the soil material to be tested must first be known. Hence, soil samples need to be tested for grain size analysis to determine the specific soil type, specific gravity (G_s), water content, Atterberg Limits, and compressibility parameters.

Table 1 Physical parameters of Bandung lacustrine soft clays

No	Parameters	Magnitude
1	Soil Classification (USCS)	Silty clay (CH)
2	Water content (w)	55.90 – 58.80 %
3	Volume Weight (γ_m)	1.625 – 1.678 g/cm ³
4	Liquid Limit (LL)	79.626 – 91.400 %
5	Plastic Limit (PL)	19.300 – 38.730 %
6	Plasticity Index (PI)	40.896 – 72.100 %
7	Specific gravity (G_s)	2.501 – 2.622
8	Void ratio (e_o)	1.360 – 1.590
9	Compression index (C_c)	0.360 – 0.414
10	Swelling index (C_s)	0.095 – 0.115
11	Preconsolidation pressure (P'_c)	0.6 – 0.69 kg/cm ²

To calibrate the LVDT, a micrometer with a maximum reading of 2.50 cm is used, while the pressure transducer is calibrated using a manometer. Basically, the principle is the same: calibration of the two instruments, namely the search for a common input line to be included in the datalogger. The calibration of the LVDT is carried out with a precision of 0.5 mm over a range of 2.5 cm. Calibration of the pressure transducer is done with an accuracy of 5.0 psi, up to 100 psi. The linear regression line equation is then entered into the Data Logger program, and select Fresh Calibration in the Data Logger command. Then the program will be read in accordance with the conditions that occur in the sample in the cell.

The TTC test types used in this research are referenced to the procedure in ASTM D4764 for the consolidated undrained (CU) Triaxial test, with the first sample made perfectly saturated. Before applying stresses to shear the soil sample in the cell, isotropic stresses are applied to each side of the cell to consolidate the soil sample. Testing is done under axial and lateral compression conditions. The

primary stress is regulated by the parameter b , which has the following equation:

$$b = \frac{\sigma_2 - \sigma_3}{\sigma_1 - \sigma_3} \quad (1)$$

with σ_3 is kept constant, then:

$$\sigma_2 = b\sigma_1 \quad (2)$$

The values of b applied in this research are 0, 0.25, 0.5, 0.75, and 1. The loading stages, in accordance with the above provisions, are illustrated in Table 2.

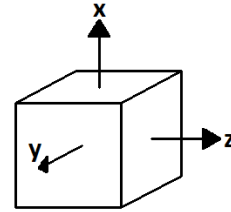


Figure 3 Loading axes at True Triaxial test

Table 2 Loading stages at True Triaxial test

α (°)	b	$\Delta\sigma$	psi / minutes	α (°)	b	$\Delta\sigma$	psi / minutes
0	0	σ_x	0.5	90	0	σ_x	0
		σ_y	0			σ_y	0
		σ_z	0			σ_z	0.5
	0.25	σ_x	0.5		0.25	σ_x	0.125
		σ_y	0			σ_y	0
		σ_z	0.125			σ_z	0.5
0	0.5	σ_x	0.5	90	0.5	σ_x	0.25
		σ_y	0			σ_y	0
		σ_z	0.25			σ_z	0.5
	0.75	σ_x	0.5		0.75	σ_x	0.375
		σ_y	0			σ_y	0
		σ_z	0.375			σ_z	0.5
1	1	σ_x	0.5	1	1	σ_x	0.5
		σ_y	0			σ_y	0
		σ_z	0.5			σ_z	0.5

Prior to the consolidation stage, ensure the sample is nearly 100% saturated. The saturation process is carried out by providing back pressure and is accompanied by isotropic side stresses. In this study, the tests performed are consolidated undrained (CU), so the K8 valve needs to be closed before deviatoric pressure is applied, as shown in Figure 4. The deviatoric pressure depends on the type of loading, and the variation of the principal stress is controlled by rotating the regulator R1 for the x-axis, the regulator R2 for the y-axis, and the regulator R3 for the z-axis. This process will finish when the sample has experienced a large deformation while the stress remains constant.

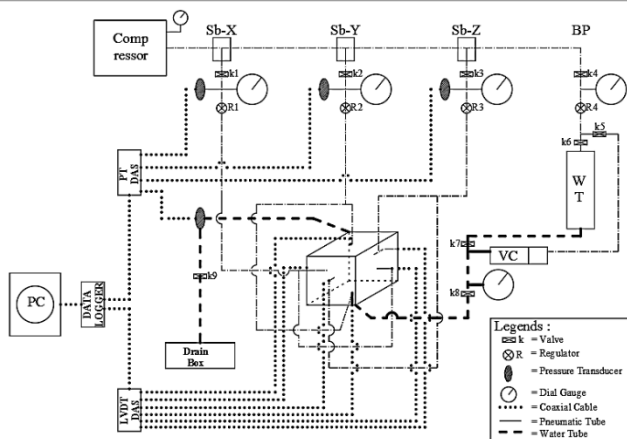


Figure 4 Schematic diagram of True Triaxial components

3. EXPERIMENTAL RESULTS AND ANALYSIS

Based on the test results, only samples with effective consolidation pressure $\sigma'_c = 110$ kPa were successfully tested under any stress variation (b) and stress rotation (α). So, the data analysis of the test results is prepared only for that condition. This research will discuss the effect of stress variation (b) and stress rotation (α) on the stress trajectory, shear strength characteristics (M and S_u/σ'_c , rigid modulus response (E_{50}), Skempton parameter A_f , and failure criteria from Bandung soft clay soil. A_f is defined as the ratio of excess pore pressure (Δu_f) to deviatoric stress (q_f) at failure. While pore pressure generation was monitored throughout the shear phase, the study prioritized quantifying the relationship between peak shear stress and the associated pore-pressure response, which governs undrained shear strength in stress-anisotropy scenarios.

Table 3 Test results of Bandung lacustrine soft clay for pressure $\sigma'_c = 110$ kPa

α (°)	b	Code	ϕ' (°)	M	S_u/σ'_c (kPa)	E_{50} (kPa)	A_f	w (%)
0	0	A00B000	21.6	0.839	0.310	4762.145	0.640	51.500
	0.25	A00B025	22.14	0.728	0.309	5297.873	0.668	51.500
	0.5	A00B050	23.64	0.700	0.321	16425.54	0.674	51.680
	0.75	A00B075	23.86	0.687	0.315	21887.57	0.707	51.170
	1	A00B100	23.98	0.716	0.327	26653.88	0.664	51.750
90	0	A90B000	21.67	0.842	0.284	7287.759	0.795	51.480
	0.25	A90B025	22.8	0.751	0.286	8314.384	0.816	51.690
	0.5	A90B050	23.23	0.688	0.291	9719.157	0.803	51.700
	0.75	A90B075	23.51	0.678	0.284	12124.83	0.861	51.130
	1	A90B100	23.87	0.713	0.280	25012.74	0.919	51.470

Table 4 shows the parameters of the test results. Each of these parameters will be compared to previous studies.

In the soil shear testing under consolidated undrained conditions, two shear strength parameters can be obtained, namely effective shear strength and total shear strength. Both shear strength parameters are obtained from the tangent line of the Mohr-Coulomb circle, which is a function of cohesion (c) and internal friction angle (ϕ). Consequently, the consolidation effect is greater than the pre-consolidation stress of Bandung lacustrine clay, so that the result shows the normally consolidated (NC) behavior with the cohesion value approaching 0 (zero). Thus, the effective shear strength parameters yielded only the effective internal friction angle (ϕ') and the total shear strength parameter is undrained shear strength (S_u).

Table 4 List of Previous Studies

No	Researcher	Year	Soil Type	Test
1	Broms, B. B., and Casbarian, A. O.	1965	Remolded Kaolinite Clay	CIU TSHC
2	Shibata, T., and Karube, D.	1965	Remolded Osaka Aluvial Clay	CIU TTC
3	Seah, T. H.	1990	Resedimented Low Plasticity Boston Blue Clay	CKoU DSC
4	Prashant, and Dayakar	2004	OC Kaolinite Clay	CIU TTC
5	Yimsiri, et. al.	2013	Undisturbed Bangkok Clay	CIU TSHC
6	Anantanasakul, et. al.	2012	Kaolinite Clay	CID TTC

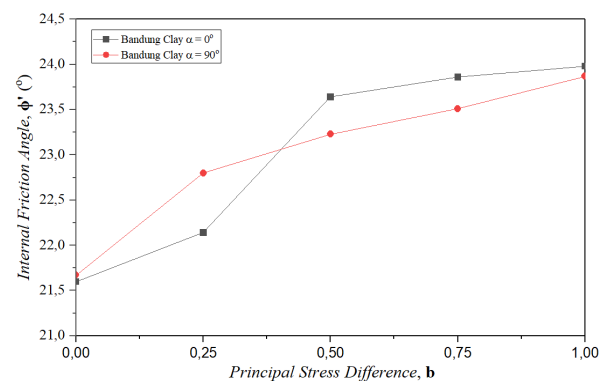


Figure 5 Effective angle change in (ϕ') test result to respond the increase of b

From Figure 5, it is seen that the increase in the stress variation (b) affects the value of the shear angle in the effective (ϕ'). In the two different stress rotations (α), the higher the value of b , the larger the effective shear angle value (ϕ') is generated. For α equal to 0 degrees (axial compression), an increase of b from 0 to 0.5 gives a change in the value of 10% shear angle; at b greater than 0.5, there is a change that is not too large but increases about 1% each increment of b . Whereas, for α equal to 90 degrees (lateral compression), the difference in change is quite large, seen in the increase of the value of b from 0 to 0.25, which is about 5%, and the rest of the difference is relatively constant at 2.5%. If we see the change in the value of α in Figure 4, for the value of b equal to 0 and 0.25, there is an increase along with the increase in the value of α , but for the value of b greater than 0.25, there is a decrease along with the increase in the value of α .

Prior research conducted by Prashant and Dayakar (2004), Yimsiri et al. (2013), and Anantanasakul et al. (2012) also showed that with the increasing value of b , the more effective the friction angle value (ϕ'). At b equal to 0.5 is the peak; greater than 0.5 decreases slightly. Compared with the laboratory test results, the pattern of research changes reported by Yimsiri (2013) approaches the test results. However, the magnitude of changes in friction angle values in effective (ϕ') test results is not too extreme, as research on clay soil in Bangkok by Yimsiri (2013) is closer to the result of Anantanasakul's (2012) study. Against the increase of the α value, the research conducted by Yimsiri (2013) on the variation of b equal to 0, 0.5, and 1 shows a result pattern that is quite the same as the test result, which is that the bigger the α value, the smaller the effective shear angle (ϕ').

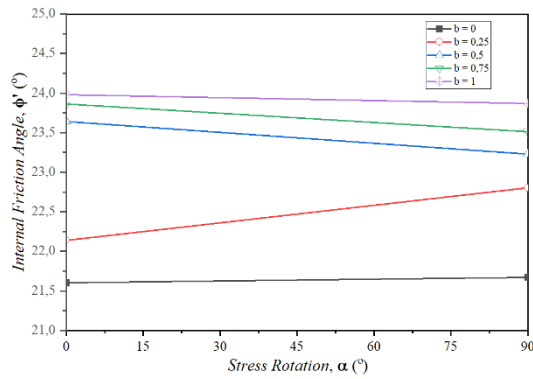


Figure 6 Change of effective friction angle (ϕ') test result towards the increase of α

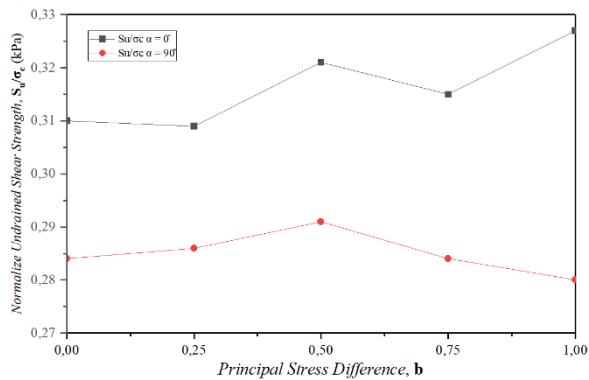


Figure 7 Changes of normalized undrained shear strength (S_u/σ_c) test results to increase of b

Figure 7 shows that the effect of an increase in the value of b causes the rise and fall of the normalized undrained shear strength value S_u/σ_c . Generally, the S_u/σ_c value at b equal to 0.5 increases by about 0.01 from the S_u/σ_c value at b equal to 0. For α equal to 0 degrees, with b equal to 0.5, the S_u/σ_c value at b equal to 1 also increases. However, for α equal to 90 degrees, it decreases. At b equal to 0.75, the S_u/σ_c value decreases. While at b equal to 0.25, the S_u/σ_c value for α equal to 0 degrees decreases and vice versa for α equal to 90 degrees. The pattern of change is close to the results of the previous test, but the value of S_u/σ_c at b equal to 1 and α equal to 0 degrees should decrease, based on previous research. This may occur due to the influence of degree of saturation (unsaturation) or Skempton B -value parameters, and of water content after consolidation, thus affecting the magnitude of the deviatoric value for the determination of undrained shear strength.

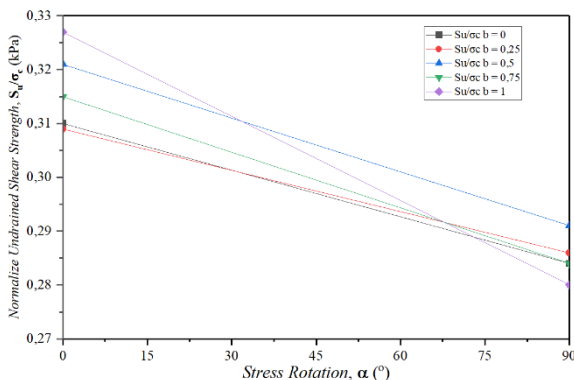


Figure 8 Changes of normalized undrained shear strength (S_u/σ_c) test results to increase of α

With the increase in α values, the normalized undrained shear strength value (S_u/σ_c) decreases, as shown in Figure 6. This is consistent with the previous study: the greater the increase in α , the

more the normalized undrained shear value (S_u/σ_c) will decrease from its initial value. In this study, the reduction was 15%, while Seah (1990) concluded with a 50% reduction.

The rated stress ratio (M) is a function of the effective shear angle and stress variations and is a criterion of the collapse parameter (Prashant & Dayakar, 2004). The following equations are used in obtaining the value of M :

$$M = \frac{6 \sin \phi' \sqrt{1 - b + b^2}}{3 + (2b - 1) \sin \phi'} \quad (3)$$

From Figure 9, the stress ratio for the Bandung lacustrine clay soil test result approaches the pattern of the test result by Prashant and Dayakar (2004) and decreases with increasing b . For both variations of α , the characteristic M reduction pattern is almost the same.

The change in deformation of a material depends on the stiffness modulus. Commonly known as the Young modulus or secant modulus, it is obtained from the gradient of 50% maximum deviatoric stress and strain. The soil test of Bandung lacustrine clay is performed in a consolidated undrained condition (CU), so the secant modulus obtained is the undrained secant modulus (E_{u50}) calculated based on major principal stress increase ($\Delta\sigma_1$) divided by major principal strain ($\Delta\epsilon_1$) (Yimsiri and Soga, 2011).

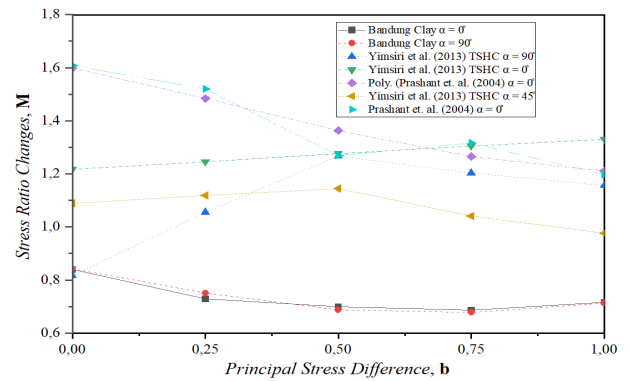


Figure 9 Comparison of stress ratio changes (M) to the increase of α in previous research and test results

Table 5 Comparison of secant modulus varied by α from test result

b	$E_{50(\alpha=0)}/E_{50(\alpha=90)}$ (kPa)
0	0.653
0.25	0.637
0.5	1.690
0.75	1.805
1	1.066

From Figure 10, it can also be concluded that for b values from 0 to 0.325, lateral compression loading ($\alpha = 90^\circ$) has a higher stiffness value than axial compression loading ($\alpha = 0^\circ$). The value of b over 0.325 in axial compression testing indicates that the restraints of the intermediate principal stress increase (σ_2) have affected the stiffness of the Bandung lacustrine clay soil. The comparison of undrained secant modulus (E_{u50}) between axial compression testing ($\alpha = 0^\circ$) and lateral compression ($\alpha = 90^\circ$) TTC of Bandung lacustrine clay is shown in Table 5. These values are in the range of the previous study, as shown in Table 6.

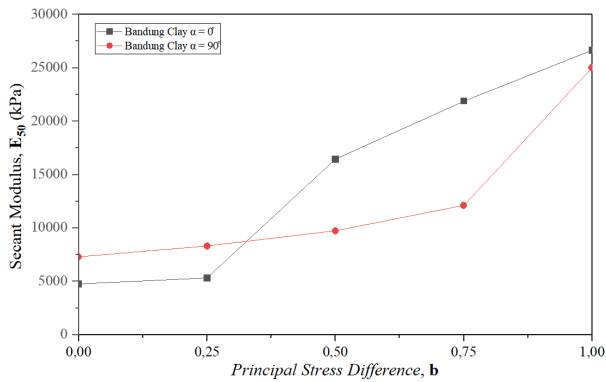


Figure 10 Secant stiffness modulus changes towards the increment of b

Table 6 Comparison of secant modulus from previous study

Soil	$E_{u50(\alpha=0)}/E_{u50(\alpha=90)}$	Strain Level	Reference
Bangkok Clay	1.5 – 1.7	Medium	Yimsiri, et. al. (2013)
	1.6	Small	
	1.7	Small	
San Francisco Sea Mud	0.7	Medium	Ratananikom, et. al. (2013)
London Clay	0.8	Medium	Yimsiri, et. al. (2013)
	0.6	Small	
Gault Clay	0.6	Medium	Kirkgard and Lade (1991)
	0.6	Small	
Gault Clay	0.6	Small	Atkinson (1975)
	0.6	Small	
Gault Clay	0.6	Small	Yimsiri and Soga (2011)
	0.6	Small	

The Skempton parameter is the ratio of the increase in excess pore water pressure to the deviatoric stress when collapsed. From previous studies conducted by Prashant and Dayakar (2004) and Yimsiri (2013), the value of A_f increases with an increase in the value of b . It is a trend similar to the test results for Bandung lacustrine clay. The effect of the increase in α produces a greater A_f value as illustrated in Figure 11.

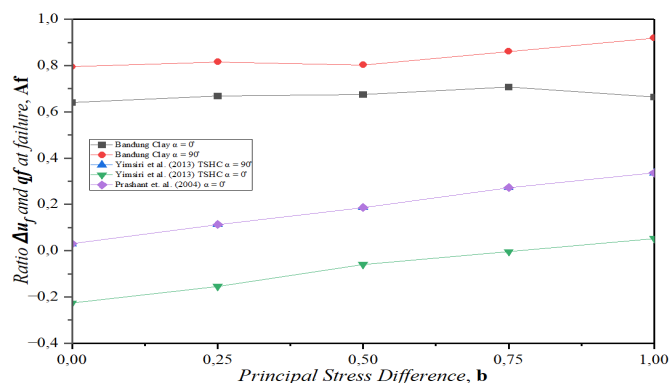


Figure 11 Skempton Parameter A_f changes due to increasing b

Failure criteria at different stress levels may be described in the octahedral (π -plane) field in polar coordinates by the Lode angle (θ). In this study, the results of the TTC test of Bandung lacustrine clay are compared with the collapse criterion model in Table 7.

Table 7 Some failure criteria models

Model	Failure criteria
Mohr – Coulomb	$f = \sqrt{3}J_2 - M(\theta)(p' + c' \cot \phi') = 0$
	$M(\theta) = \frac{3 \sin \phi'}{\sqrt{3} \cos \theta + \sin \theta \sin \phi'}$
Extended Tresca	$f = \sqrt{J_2} \cos \theta - k = 0$
	$M(\theta) = \frac{\sqrt{3}k}{p' \cos \theta}$
Drucker – Prager	$f = \sqrt{3}J_2 - M(\theta)(p' + c' \cot \phi') = 0$
	$M(\theta) = \frac{6 \sin \phi'}{3 - \sin \phi'}$
Lade – Duncan	$f = \sqrt{3}J_2 - M(\theta)(p' + c' \cot \phi') = 0$
	$2M(\theta)^3 \sin 3\theta + 9M(\theta)^2 - 10.67 = 0$
Matsuoka Nakai	$f = \sqrt{3}J_2 - M(\theta)(p' + c' \cot \phi') = 0$
	$24.25M(\theta)^3 \sin 3\theta + 82.11M(\theta)^2 - 84.32 = 0$

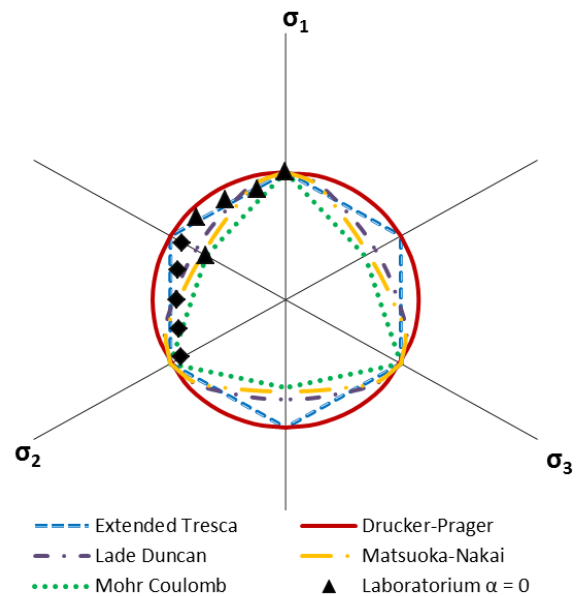


Figure 12 Failure criteria in octahedral plane

The results of TTC testing of Bandung lacustrine clay in the octahedral plane for each stress variation (b) are shown in Figure 10. For axial compression testing ($\alpha = 0^\circ$), almost all test results in each variation of b approach the Extended Tresca failure criteria, but at the value of b equal to 1 ($\sigma_1 = \sigma_2$) do not follow the model because of the uniformity of major and middle stress readings and closer to the Mohr - Coulomb model. While the results of lateral compression testing ($\alpha = 90^\circ$) are slightly higher than the test results, the Extended Tresca model predicts a slightly greater value. The Lade-Duncan model (1973) predicts lower values at two points in the TTC Bandung lacustrine clay soil test results and greater values at two points in the TTC testing of Bandung lacustrine clay.

Research conducted by Ratananikom (2013) on Bangkok clay soils is more closely related to Drucker-Prager's failure criteria. In contrast, research on the overconsolidated kaolin clay by Prashant and Dayakar (2004) more closely follows the Lade-Duncan failure criteria. The Lade-Duncan model is also closely related to research on the criteria for clay soil collapse by Kirkgard and Lade (1993), Callisto and Callabresi (1998), and Callisto and Rampello (2002). Therefore, it can be concluded that the collapse criterion that most closely approximates the behavior of clay soil is the Lade-Duncan model. Some TTC tests on lacustrine clay in Bandung also showed the same result.

4. CONCLUSIONS AND RECOMMENDATIONS

A series of true triaxial laboratory tests of Bandung lacustrine soft clay under variations in the increase of principal stress (b) and stress rotation (α) have been carried out, and some conclusions have been obtained as follows:

1. The effective friction angle value (ϕ') increases with the increase in stress variation (b) up to 10% in the intermediate principal stress increase of 0.5 times the major principal stress increase.
2. Loading with a stress rotation change of 0 (zero) degrees to 90 (ninety) degrees results in a decrease in undrained shear strength (S_u/σ_c) up to 15%.
3. The stiffness modulus and the Skempton parameters also increase with increasing stress variation. Loading in the lateral compression direction ($\alpha = 90^\circ$) yield larger Skempton A_f parameters.
4. The Extended Tresca failure criteria model predicts nearly the same value across all b values in the Bandung lacustrine clay soil testing under axial compression loading ($\alpha = 0^\circ$). However, under lateral compression loading ($\alpha = 90^\circ$), this model predicts a higher test result.

The results obtained from this study should be used to evaluate the generally used constitutive model, so that the advantages and disadvantages of each model under three-dimensional loading and different loading directions can be identified, and to inform the development of a constitutive model for finite element load-deformation analysis. In addition, in this study, minor principal stresses are maintained during the shifting time; to complement the collapse data criteria, it is desirable to conduct further research with a combination of additional stress variations.

5. ACKNOWLEDGEMENTS

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