# Undrained Shear Strength of Very Soft to Medium Stiff Bangkok Clay from Various Laboratory Tests

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**ABSTRACT:** The values of undrained shear strength of undisturbed Bangkok Clay specimens are investigated by various laboratory tests, i.e. (i) unconfined compression test, (ii) unconsolidated undrained triaxial test, (iii) isotropically consolidated undrained triaxial compression/extension tests, (iv) $K_o$-consolidated undrained triaxial compression/extension tests, (v) direct shear test, (vi) direct simple shear test, (vii) laboratory vane test, and (viii) triaxial vane test. The experimental data are obtained from laboratory tests performed in this study and collected from previous published studies. The soil layers of interest in this study are very soft to medium stiff clays which are located between 2-14 m depth below ground surface. The variations of undrained shear strength with depth and their degree of scattering are presented and discussed. The interrelationships among undrained shear strengths from various laboratory tests are analyzed and relevant discussions are made. The validity of various empirical and theoretical relationships for predicting undrained shear strength of Bangkok Clay are evaluated. Various approaches for estimation of mobilized undrained shear strength for embankment stability analysis are investigated and the most suitable approach for embankment stability analysis and design on Bangkok subsoils is finally suggested.

**Keywords:** Laboratory tests; Shear strength; Clays; Embankments

## 1. INTRODUCTION

Embankments are among the most common and relevant geotechnical engineering structures. They are required in the construction of most infrastructures, e.g. highway and railway networks, hydroelectric structures (retention dykes and dams), irrigation and flood control structures (regulation dams and banks), harbor facilities (quays, breakwaters, and seawall), and airport runways. The establishment of these infrastructures in regions of dense population often necessitates their constructions on unfavorable subsoils, such as soft clays. These embankments must be designed such that there is no risk of any failures in the subsoil which can lead to catastrophic effects, e.g. for water-retaining structures, or for failures close to structures which can be carried along by large ground movements. More often, these failures can be very costly, though not very disastrous, because of the resulting delay in the construction progress due to the required improvements to the design. In embankment construction on soft clay, the Factor of Safety (FOS) is a minimum at the end of construction before the clay becomes stronger after consolidation. Therefore, the shear strength to be introduced in the stability analysis to find this lowest FOS is the undrained shear strength of clay.

The undrained shear strength ($s_u$) is an important parameter for describing the consistency of cohesive soils. It is a measured response of clay during undrained loading with an assumption of zero volume change. However, $s_u$ is not a fundamental soil property because it is affected by mode of shearing, boundary conditions, rate of loading, confining stress level, initial stress state, and other variables (e.g. Wroth, 1984; and Kulhawy & Mayne, 1990). In measuring $s_u$, various laboratory tests are used in practice; therefore, it is expected that different test types should produce different results of measured $s_u$. Because of this variation, correlations of $s_u$ measured from various laboratory tests can conveniently help the comparison of results among them. In this research, the values of $s_u$ of very soft to medium stiff Bangkok Clay are measured by various laboratory tests together with a review of additional published data to assess their interrelationships. The thorough appraisal of the undrained shear strength characteristics of Bangkok Clay can avoid much of the present empiricism to yield safe design which can avoid overconservatism and allow greater control over the design.

## 2. BACKGROUND OF RESEARCHES ON UNDRAINED SHEAR STRENGTH

The undrained shear strength ($s_u$) from laboratory tests normalized with effective overburden stress ($\sigma_{vo}'$) or maximum past pressure ($\sigma_{v,max}'$) is an important index in evaluating the shear strength and stress history of in-situ clay. In the field, different elements of soil are subjected to different boundary conditions and loading stress paths, i.e. compression mode under central part of the embankment, direct simple shear mode where the failure surface is close to horizontal, and extension mode near the toe of the embankment. Therefore, the undrained shear strengths at various points are different because natural clays are anisotropic. Various undrained shear strengths can be measured by a number of different laboratory tests which subjects the soil specimens to different boundary conditions, loading stress paths, and strain rates. However, undertaking various tests pertinent to particular field conditions is likely to be an excessive requirement for common and routine design cases. Therefore, the correlations among the $s_u$ measured from various tests can be useful for comparison and design purposes.

Skempton (1957) suggested the general correlation for undrained shear strength ratio ($s_u/\sigma_{vo}'$) of normally consolidated (NC) clay determined from a field vane shear test (FV) as a function of the plasticity index (PI) as shown in Eq. (1) which has also been corroborated by e.g. Ladd & Edgers (1972). Bjerrum & Simons (1960) showed that, for NC sensitive clays, the undrained shear strength ratio from triaxial compression test (CIUC) can be correlated with the plasticity index (PI) and liquidity index (LI), as shown in Eqs. (2) and (3), respectively. However, some researchers did not find the relationship between undrained strength ratio with PI (e.g. Hanzawa, 1983; and Nakase & Kamei, 1988).

$$\left(\frac{s_u}{\sigma_{vo}'}\right)_{\mathrm{FV}} = 0.11 + 0.0037\mathrm{PI} \qquad \text{(PI in \%)} \tag{1}$$

$$\left(\frac{s_u}{\sigma_{vo}'}\right)_{\mathrm{CIUC}} = 0.045\mathrm{PI}^{0.5} \qquad \text{(PI in \%)} \tag{2}$$

$$\left(\frac{s_u}{\sigma_{vo}'}\right)_{\mathrm{CIUC}} = \frac{0.18}{\mathrm{LI}^{0.5}} \qquad \text{(LI in decimal)} \tag{3}$$

The undrained shear strength ratio increases with increasing overconsolidation, as measured by the overconsolidation ratio (OCR = $\sigma_{v,max}'/\sigma_{vo}'$). Jamiolkowski et al. (1985) reported the experimental observation by direct simple shear (DSS) test on low to moderate PI soils, as shown in Eq. (4). Mesri (1975) reported that the mobilized undrained shear strength at failure ($(s_u)_{\text{mob}}$) under an embankment can be written using the maximum past pressure ($\sigma_{v,max}'$) as shown in Eq. (5). Chandler (1988) suggested that this ratio should increase with PI as shown in Eq. (6).

$$\left(\frac{s_u}{\sigma_{vo}'}\right)_{\text{DSS}} = (0.23 \pm 0.04)\text{OCR}^{0.8} \tag{4}$$

$$\left(\frac{s_u}{\sigma_{v,\max}'}\right)_{\text{mob}} = 0.22 \tag{5}$$

$$\left(\frac{s_u}{\sigma_{v,\max}'}\right)_{\text{FV}} = 0.11 + 0.0037\text{PI} \qquad \text{(PI in \%)} \tag{6}$$

In laboratory testing, sampling process inevitably introduces sample disturbance to the soil structure, a major source of which is the stress relief involved in taking a sample from the ground. This causes a reduction in negative pore pressure which causes a reduction in the effective stress in the sample and results in a decrease in $s_u$, the reduction of which typically ranges from 20-50% of the "perfect sample" strength (Ladd & Lambe, 1968). Bjerrum (1973) suggested the Recompression technique which tries to recover the in-situ stress of the specimen prior to shearing. Ladd & Foott (1974) later suggested the SHANSEP technique to minimize the effects of sample disturbance on the strength measured with the Recompression technique. In SHANSEP technique, the specimens are first consolidated well beyond their maximum past pressures to achieve NC state before rebounded to varying OCRs and being subjected to undrained compression, extension, and direct simple shear tests. The results can generally be written as Eq. (7) where $m$ is a parameter close to 0.8.

$$\left(\frac{s_u}{\sigma_{vo}'}\right)_{\text{OC}} = \left(\frac{s_u}{\sigma_{vo}'}\right)_{\text{NC}} \text{OCR}^m \tag{7}$$

Considering different mode of shearing, Kulhawy & Mayne (1990) suggested the empirical relationships between undrained shear strength ratio and friction angle ($\phi'$) of CIUC, $\text{CK}_\text{o}\text{UC}$, and DSS tests for NC clay as shown in Eqs. (8), (9), (10), respectively.

$$\left(\frac{s_u}{\sigma_{vo}'}\right)_{\text{CIUC}} = 0.0120\phi_{\text{TC}}' \tag{8}$$

$$\left(\frac{s_u}{\sigma_{vo}'}\right)_{\text{CK}_\text{o}\text{UC}} = 0.0117\phi_{\text{TC}}' \tag{9}$$

$$\left(\frac{s_u}{\sigma_{vo}'}\right)_{\text{DSS}} = \frac{\sin\phi_{\text{PSC}}'}{(1+\sin\phi_{\text{PSC}}')^2} \qquad (\text{with } \phi_{\text{PSC}}' \approx 1.1\phi_{\text{TC}}') \tag{10}$$

Kulhawy & Mayne (1990) also suggested the empirical relationships between DSS and other tests as shown in Eqs. (11) to (13). For homogeneous clay, it is typically found that $s_u$ from plane strain compression (PSC) tests is greater than the value from direct simple shear (DSS) tests, which in turn is greater than the plane strain extension (PSE) tests. Triaxial compression (TC) strengths are generally very close to PSC values while triaxial extension (TE) strengths are 10-25% lower than those of PSE tests.

$$\left(\frac{s_u}{\sigma_{vo}'}\right)_{\text{DSS}} = 0.67\left(\frac{s_u}{\sigma_{vo}'}\right)_{\text{CK}_\text{o}\text{UC}} \tag{11}$$

$$\left(\frac{s_u}{\sigma_{vo}'}\right)_{\text{DSS}} = 0.40\left[\left(\frac{s_u}{\sigma_{vo}'}\right)_{\text{PSC}} + \left(\frac{s_u}{\sigma_{vo}'}\right)_{\text{PSE}}\right] \tag{12}$$

$$\left(\frac{s_u}{\sigma_{vo}'}\right)_{\text{DSS}} = 0.45\left[\left(\frac{s_u}{\sigma_{vo}'}\right)_{\text{CK}_\text{o}\text{UC}} + \left(\frac{s_u}{\sigma_{vo}'}\right)_{\text{CK}_\text{o}\text{UE}}\right] \tag{13}$$

Previous works (e.g. Ladd & Foott, 1974; and Graham et al., 1983) have shown that the strength obtained from laboratory tests varies with the strain rate used. In general, each log cycle of an increase in strain rate is accompanied by a 10±5% increase in $s_u$. The effect is due to undrained creep in the sample which occurs during shear, giving increased pore pressures, decreased effective stresses, and decreased strengths. The slower the strain rate, the more time there is for this creep to occur and the lower the $s_u$ obtained. If a testing rate of 1%/hr is considered as the standard reference rate, the result from other rate can be approximated as shown in Eq. (14).

$$\frac{s_u}{s_u \text{ for } \dot{\varepsilon} = 0.1\%/\text{hr}} = 1 + 0.1\log\dot{\varepsilon} \tag{14}$$

## 3. BACKGROUND OF RESEARCH ON BANGKOK CLAY

Bangkok is situated on the delta of Chao Praya River in the Lower Central Plain of Thailand. The Bangkok subsoils typically consist of the upper layer about 2-5 m of backfill (very loose to medium dense silty sand) and weathered crust (medium to stiff silty clay) which is light to yellowish grey in color. The SPT $N$-value varies from 2-21 and the water content is 10-35%. At depth of 3-12 m, the very soft to soft clay layer is observed which is medium to dark grey in color. Its undrained shear strength is 10-30 kPa and the water content is 60-105%. At depth of 15-35 m, the medium stiff to very stiff clay layer is observed which is dark grey to brownish grey in color. Its undrained shear strength is 26-160 kPa and water content is 15-60%. These clay layers described above, generally called "Bangkok Clay", are followed by a first sand layer. Below this first sand layer, there are alternate layers of clay and sand/gravel. The underlying profile of the bedrock is still undetermined, but its level is known to be between 550 and 2000 m below ground surface. Tanaka et al. (2001) reported that the mineralogy of the clay fraction of Bangkok Clay in order of decreasing abundance are smectite, illite, kaolinite, chlorite, and some mixed-layer minerals. The overall soil also contains primary minerals such as quartz. Bangkok Clay has an activity of about unity and its relationship between LL and PI is located considerably above the A line in plasticity chart. The microstructure of Bangkok Clay is characterized by the frequent presence of pellets filled with pyrite. The aggregates are quite compact and consist of an assemblage of clay-size particle providing a flocculated structure. The pore space consists primarily of the inter-aggregate pore family. Microfossils, such as diatoms or foraminifera, are rare except near the surface.

The properties of Bangkok Clay at MRT Sutthisan Station are shown in Table 1 and Figure 1. Bangkok Clay has the maximum past pressure ($\sigma_{v,max}'$) slightly larger than the in-situ effective overburden stress ($\sigma_{vo}'$). However, at depths larger than 12 m, $\sigma_{v,max}'$ values are significantly greater than $\sigma_{vo}'$. This is due to the deep well pumping undertaken since 1970s which has caused the piezometric pore water pressure of Bangkok subsoils to drawdown from the upper soft clay and the stiff clay layers to the first sand layer (Yudhbir & Honjo, 1991). Bangkok Clay has OCR of between 1.1 and 1.6 (normally consolidated to lightly overconsolidated state) which gives this soil a low shear strength, with resulting in problems of stability in the construction of embankments and significant settlement under most civil engineering works. (Note: NC = normally consolidated (OCR = 1.0-1.3), LOC = lightly overconsolidated (OCR = 1.3-3), MOC = moderately overconsolidated (OCR = 3-10), and HOC = heavily overconsolidated (OCR>10)).

This research performs laboratory tests and reviews published data of $s_u$ of Bangkok Clay from 5 sites, i.e. (i) Chulalongkorn University (CU), (ii) Kasetsart University (KU), (iii) Asian Institute of Technology (AIT), (iv) MRT Sutthisan Station, and (v) Suvarnabhumi Airport, the locations of which are shown in Figure 2. The soil profiles of all sites are summarized in Table 2. The consistency of clays is classified according to $s_u$ from UC test as recommended by Terzaghi & Peck (1967). The soils of interest in this research are only very soft, soft, and medium stiff clays which are located between 2-14 m depth BGL. The index properties ($w$, LL, PL, PI, and LI) of all sites are shown in Figure 3. Their average values are summarized in Table 3. The results shown that the soil profiles of Bangkok Clay of all sites are relatively uniform with LL = 80-90%, PL = 30-33%, and PI = 52-57%. The average water contents of very soft to soft clay layers (2 – 11 m BGL) are 70-80% which give LI = 0.75-0.83, whereas the average water content of medium stiff clay (11-14 m BGL) is 65% which gives LI = 0.66.

## 4. UNDRAINED SHEAR STRENGTH FROM LABORATORY TESTS

### 4.1 Theoretical consideration

The undrained shear strength ratio for triaxial compression (CIUC) as a function of index properties can be determined from Critical State Soil Mechanics (CSSM) using modified Cam Clay model (e.g. Wroth & Wood, 1978; Wroth & Houlsby, 1985). For NC clay, this relationship is given by Eqs. (15) and (16)

$$\left(\frac{s_u}{\sigma_{vo}'}\right)_{\mathrm{CIUC}} = 0.129 + 0.00435\mathrm{PI} \qquad \text{(PI in \%)} \qquad (15)$$

$$\left(\frac{s_u}{p_a}\right)_{\mathrm{CIUC}} = 1.7e^{-4.6\mathrm{LI}} \qquad \text{(LI in decimal)} \qquad (16)$$

from $\ln S = (1 - LI)\ln R$ in which $S$ = $s_u$/($s_u$ at LL) and $R$ = ($s_u$ at PL)/($s_u$ at LL). Typically $R \approx 100$ and ($s_u$ at LL) $\approx 0.017p_a$, where $p_a$ = atmospheric pressure (100 kPa).

The undrained shear strength ratio in triaxial compression can be expressed by analysis of the Mohr-Coulomb failure envelope (Kulhawy & Mayne, 1990). For $K_o$-consolidation, the undrained strength ratio is given as Eq. (17). For isotropic consolidation ($K_o$ = 1), Eq. (17) reduces to Eq. (18).

Table 1 Properties of Bangkok Clay at MRT Sutthisan Station (Shibuya et al., 2001)

| Depth (m) | $\gamma_t$ (kN/m$^3$) | $w$ (%) | LL (%) | PL (%) | PI (%) | $\sigma_{vo}$ (kPa) | $u$ (kPa) | $\sigma_{vo}'$ (kPa) | $\sigma_{v,max}'$ (kPa) | OCR |
|---|---|---|---|---|---|---|---|---|---|---|
| 3.5 | 17.3 | 48.5 | 60.4 | 25.2 | 35.2 | 62.9 | 20.9 | 42.0 | 114.8 | 2.7 |
| 4.5 | 16.9 | 54.3 | 67.5 | 22.2 | 45.3 | 79.8 | 29.2 | 50.5 | 56.9 | 1.1 |
| 5.7 | 16.5 | 68.5 | 73.7 | 25.7 | 48.0 | 99.6 | 37.6 | 62.0 | 94.4 | 1.5 |
| 6.7 | 16.3 | 48.8 | 45.8 | 19.4 | 26.4 | 115.9 | 45.9 | 69.9 | 92.7 | 1.3 |
| 7.5 | 16.2 | 51.7 | 62.4 | 20.5 | 41.9 | 128.8 | 54.1 | 74.7 | 92.7 | 1.2 |
| 8.8 | 16.0 | 77.3 | 74.0 | 23.5 | 50.5 | 149.6 | 62.1 | 87.5 | 103.6 | 1.2 |
| 9.8 | 15.9 | 63.9 | 67.0 | 21.4 | 45.6 | 164.8 | 70.1 | 94.7 | 118.2 | 1.2 |
| 10.7 | 15.9 | 68.8 | 94.5 | 26.5 | 68.0 | 179.9 | 78.1 | 101.8 | 124.8 | 1.2 |
| 11.7 | 15.8 | 69.2 | 97.3 | 27.2 | 70.1 | 195.7 | 80.0 | 115.7 | 124.8 | 1.1 |
| 12.7 | 15.8 | 60.0 | 84.8 | 23.9 | 60.9 | 211.4 | 85.2 | 126.2 | 184.3 | 1.5 |
| 13.7 | 16.0 | 64.7 | 87.7 | 24.6 | 63.1 | 227.5 | 87.3 | 140.2 | 221.9 | 1.6 |

Table 2 Soil profiles of studied sites

| Soil profile | CU | KU | AIT | MRT Sutthisan Station | Suvarnabhumi Airport |
|---|---|---|---|---|---|
| Top soil | 0-3 m | 0-2 m | 0-2 m | 0-4 m | 0-1 m |
| Very soft clay | 3-6 m | 2-5 m | Not found | Not found | 1-8 m |
| Soft clay | 6-11 m | 5-11 m | 2-10 m | 4-9 m | 8-13 m |
| Medium stiff clay | 11-14 m | 11-13 m | Not found | 9-13 m | 13-15 m |

Table 3 Average index properties of Bangkok Clay at studied sites

| Consistency | Approx. depth BGL (m) | $w$ (%) | LL (%) | PL (%) | PI (%) | LI |
|---|---|---|---|---|---|---|
| Very soft clay | 2 – 6 | 81.6 | 91.4 | 33.5 | 57.9 | 0.83 |
| Soft clay | 6 – 11 | 72.0 | 84.9 | 33.2 | 51.7 | 0.75 |
| Medium stiff clay | 11 – 14 | 64.6 | 82.6 | 29.4 | 53.2 | 0.66 |

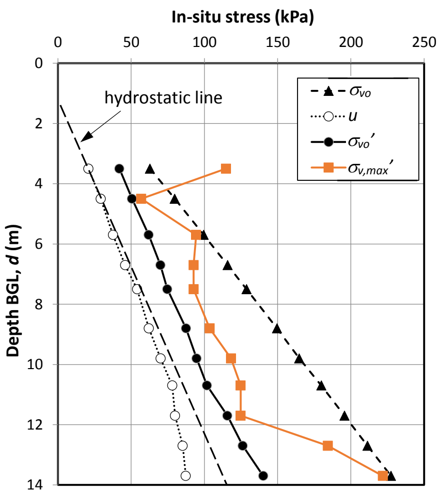



Figure 1 Properties of Bangkok Clay at MRT Sutthisan Station

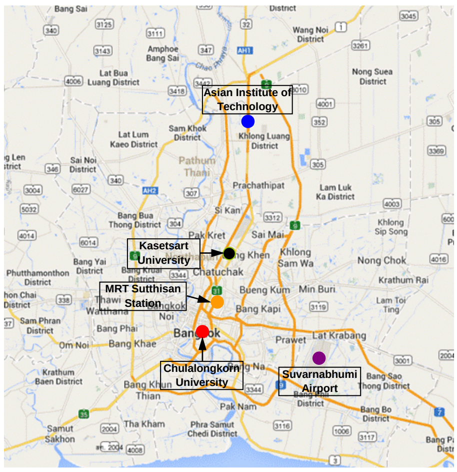
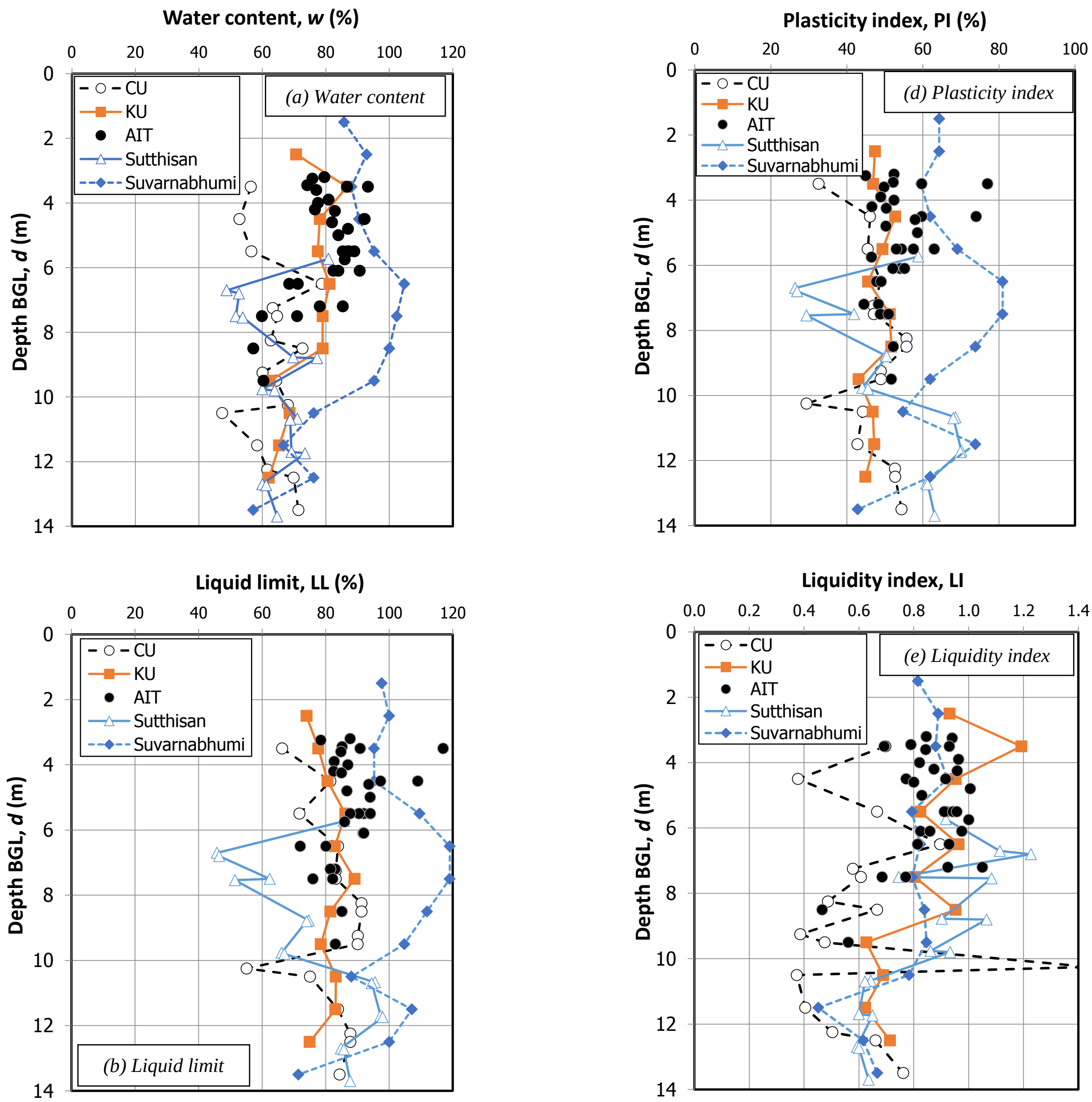



Figure 2 Locations of studied sites



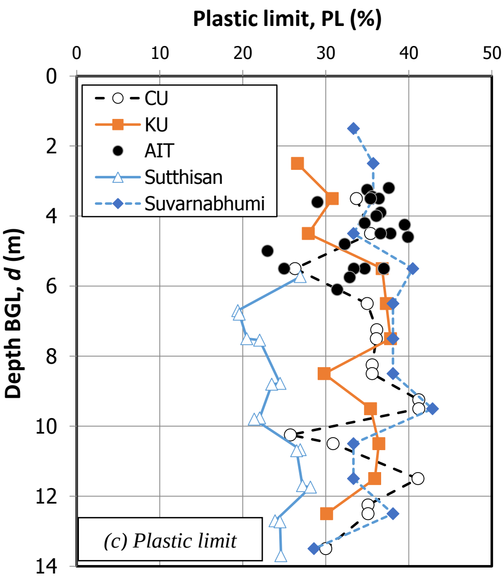




Figure 3 Index properties of studied sites

$$\left(\frac{s_u}{\sigma_{vo}'}\right)_{\mathrm{CK_oUC}} = \frac{[K_o + A_f(1-K_o)]\sin\phi_{\mathrm{TC}}'}{1+(2A_f-1)\sin\phi_{\mathrm{TC}}'} \tag{17}$$

$$\left(\frac{s_u}{\sigma_{vo}'}\right)_{\mathrm{CIUC}} = \frac{\sin\phi_{\mathrm{TC}}'}{1+(2A_f-1)\sin\phi_{\mathrm{TC}}'} \tag{18}$$

in which $K_o$ = coefficient of earth pressure at rest and $A_f$ = Skempton's pore water pressure parameter at failure = $\left(\frac{\Delta u - \Delta\sigma_3}{\Delta\sigma_1 - \Delta\sigma_3}\right)_f$.

The undrained shear strength ratio of NC clays in triaxial compression can also be analyzed from modified Cam Clay model (e.g. Wroth, 1984) for isotropic consolidation and $K_o$-consolidation as shown in Eqs. (19) and (20), respectively.

$$\left(\frac{s_u}{\sigma_{vo}'}\right)_{\mathrm{CIUC}} = 0.5M(0.5)^{\Lambda} \tag{19}$$

$$\left(\frac{s_u}{\sigma_{vo}'}\right)_{\mathrm{CK_oUC}} = \frac{\sin\phi_{\mathrm{TC}}'}{2a}\left(\frac{a^2+1}{2}\right)^{\Lambda} \tag{20}$$

in which $M$ = 6sin $\phi_{TC}$'/(3-sin $\phi_{TC}$'), $\Lambda = (\lambda-\kappa)/\lambda = 1-C_s/C_c$ (typically ≈ 0.8), and $a = \dfrac{3-\sin\phi_{TC}'}{2(3-2\sin\phi_{TC}')}$.

The influence of OCR on $(s_u)_{CIUC}$ can also be analyzed by modified Cam Clay model, as shown in Eq. (21) (e.g. Wroth & Houlsby, 1985) which is identical to SHANSEP equation (Eq. 7).

$$\frac{(s_u/\sigma_{vo}')_{OC}}{(s_u/\sigma_{vo}')_{NC}} = \mathrm{OCR}^{\Lambda} \tag{21}$$

The influence of the intermediate principal stress between plane strain and triaxial tests can be analyzed by modified Cam Clay model (e.g. Wroth, 1984), the undrained shear strength ratio of NC clay in plane strain compression can be expressed as Eq. (22).

$$\left(\frac{s_u}{\sigma_{vo}'}\right)_{PSC} = \frac{\sin\phi_{PSC}'}{2d}\left(\frac{d^2+1}{2}\right)^{\Lambda} \tag{22}$$

in which $d = \dfrac{1}{2-\sin\phi_{PSC}'}$ and $\phi_{PSC}' \approx 1.1\phi_{TC}'$.

Chen & Kulhawy (1993) theoretically approximated the interrelationships among CIUC, UU, and UC tests by a simple stress path evaluation. This procedure requires Skempton's $A$ parameter, $\sigma_s'/\sigma_{ps}'$, $\sigma_{ps}'/\sigma_{vo}'$, OCR, $K_o$, and $\phi_{TC}'$, where $\sigma_s'$ = effective stress for actual sampling, $\sigma_{ps}'$ = effective stress for perfect sampling. Their results showed that, for $\phi'$ = 30° and OCR < 2, $(s_u)_{UU\text{ or }UC}/(s_u)_{CIUC}$ can approximately be 0.40-0.75 and decreasing with increasing OCR.

### 4.2 Experimental observation

The values of $s_u$ of undisturbed Bangkok Clay specimens are investigated by various laboratory tests, i.e. (i) unconfined compression test (UC), (ii) unconsolidated undrained triaxial test (UU), (iii) isotropically consolidated undrained triaxial compression/extension tests (CIUC/CIUE), (iv) $K_o$-consolidated undrained triaxial compression/extension tests (CK$_o$UC/CK$_o$UE), (v) direct shear test (DS), (vi) direct simple shear test (DSS), (vii) laboratory vane test (lab vane), and (viii) triaxial vane test (TX vane). The $s_u$ from UC, CIUC, and DSS tests are performed in this research. Additional data of $s_u$ from other laboratory tests are summarized from published experimental data. The details of experimental data are summarized in Table 4. These tests are performed at various strain rates; therefore, all data are adjusted to the reference strain rates set in Table 5 using Eq. (14).

#### 4.2.1 Unconfined compression test

The unconfined compression tests (UC) are performed on the soil specimens from CU, KU, and AIT sites. The tests are performed on the specimens of ϕ 38 mm × ht. 76 mm and with an axial strain rate of 1%/min. Additional $(s_u)_{UC}$ data on Bangkok Clay are also available from Akrapongpisai (1970) at AIT, Shibuya et al. (2001) and Tanaka et al. (2001) at MRT Sutthisan Station, and Shibuya & Hanh (2001) at Suvarnabhumi Airport. The test conditions of all published data are identical to those performed in this research. The resulting $(s_u)_{UC}$ is summarized in Figure 4 and a best-fit straight line through the origin is assumed and superimposed. Figure 4 shows that the $(s_u)_{UC}$ with depth is very scatter.

After careful observation of Figure 4, it can be seen that the $(s_u)_{UC}$ data of AIT are much more scatter than others while the $(s_u)_{UC}$ data of Suvarnabhumi Airport are rather uniform with depth. If these data are excluded (together with some data at shallow depth of MRT Sutthisan Station), the adjusted $(s_u)_{UC}$ can be presented in Figure 5 which shows considerable reduction in scattering. This relationship is later used in subsequent analysis.

#### 4.2.2 Unconsolidated undrained triaxial test

The unconsolidated undrained triaxial test (UU) data on Bangkok Clay are available from Akrapongpisai (1970) at AIT and Shibuya & Hanh (2001) at Suvarnabhumi Airport. Both data were performed on the soil specimens of ϕ 38 mm × ht. 76 mm and with an axial strain rate of 1%/min. The resulting $(s_u)_{UU}$ results are summarized in Figure 6 and a best-fit straight line through the origin is assumed and superimposed. It can be seen that the $(s_u)_{UU}$ with depth is very scatter.

After careful observation of Figure 6, it can be seen that the $(s_u)_{UU}$ data of AIT are more scatter than those of Suvarnabhumi Airport. If only Suvarnabhumi Airport data are considered, the adjusted $(s_u)_{UU}$ can be presented in Figure 6 which shows considerable reduction in scattering. This relationship is later used in subsequent analysis.

Table 4 Summary of experimental data

| Tests | References | Locations | Test conditions |
|---|---|---|---|
| UC | This study | CU, KU, AIT | specimen of ϕ 38 mm × ht. 76 mm, $\dot{\varepsilon}_a$ = 1%/min. |
| | Tanaka et al. (2001) | MRT Sutthisan Station | |
| | Shibuya et al. (2001) | MRT Sutthisan Station | |
| | Akrapongpisai (1970) | AIT | |
| | Shibuya & Hanh (2001) | Suvarnabhumi Airport | N.A. |
| UU | Akrapongpisai (1970) | AIT | specimen of ϕ 38 mm × ht. 76 mm, $\dot{\varepsilon}_a$ = 1%/min. |
| | Shibuya & Hanh (2001) | Suvarnabhumi Airport | N.A. |
| CIUC/ CIUE | This study (CIUC) | CU, KU, AIT | specimen of ϕ 50 mm × ht. 100 mm; $\dot{\varepsilon}_a$ = 0.025%/min |
| | Tapubolon (1981) (CIUE) | AIT | specimen of ϕ 38 mm × ht. 76 mm; $\dot{\varepsilon}_a$ = 0.064%/min |
| CK$_o$UC/ CK$_o$UE | Seah & Lai (2003) (CK$_o$UC/CK$_o$UE) | AIT | specimen of ϕ 50 mm × ht. 100 mm, $\dot{\varepsilon}_a$ = 0.0067%/min. |
| | Tanaka et al. (2001) (CK$_o$UC/CK$_o$UE) | MRT Sutthisan Station | specimen of ϕ 38 mm × ht. 76 mm, $\dot{\varepsilon}_a$ = 0.1%/min. |
| | Shibuya et al. (2001) (CK$_o$UC) | MRT Sutthisan Station | specimen of ϕ 50 mm × ht. 100 mm, $\dot{\varepsilon}_a$ = 0.05%/min. |
| DS | Seah et al. (2004) | AIT | specimen of ϕ 60 mm × ht. 20 mm, speed = 1 mm/min |
| | Tanaka et al. (2001) | MRT Sutthisan Station | specimen of ϕ 60 mm × ht. 20 mm, speed = 0.25 mm/min |
| | Shibuya et al. (2001) | MRT Sutthisan Station | specimen N.A., speed = 0.1 mm/min |
| | Memon (1976) | AIT | specimen of ϕ 60 × 60 mm$^2$ × ht 50 mm, speed = 0.6 mm/min |
| DSS | This study | BTS On-Nut Station | NGI type, specimen of ϕ 67 mm × ht. 20 mm, $\dot{\gamma}$ = 0.083%/min |
| Lab vane | Akrapongpisai (1970) | AIT | specimen of ϕ 75 mm × ht. 75 mm, blade of 13 × 13 mm$^2$, speed = 10°/min |
| TX vane | Seah et al. (2004) | AIT | specimen of ϕ 50 mm × ht. 100 mm, blade of width 8.4 mm × ht. 16.8 mm, speed = 19°/min |

Table 5 Reference strain rates

| Tests | Reference strain rates |
|---|---|
| UC, UU | $\dot{\varepsilon}$ = 1%/min |
| CIUC, CIUE, CK$_o$UC, CK$_o$UE | $\dot{\varepsilon}$ = 0.025%/min |
| DS | speed = 1 mm/min |
| DSS | $\dot{\gamma}$ = 0.083%/min |

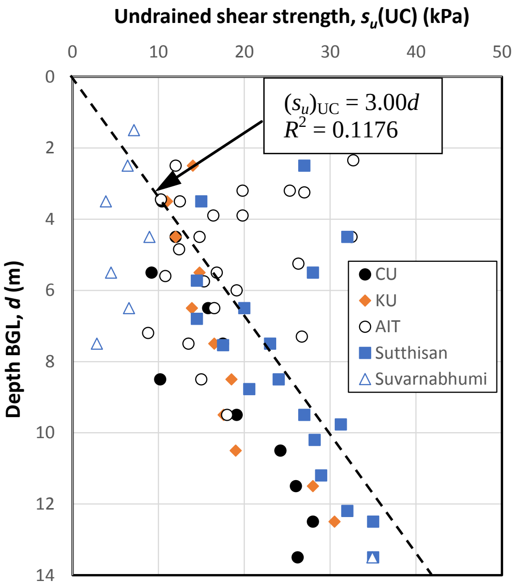



Figure 4 UC results

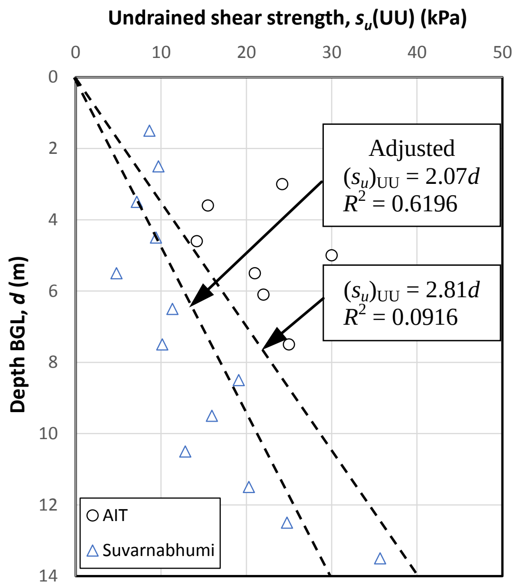



Figure 6 UU results

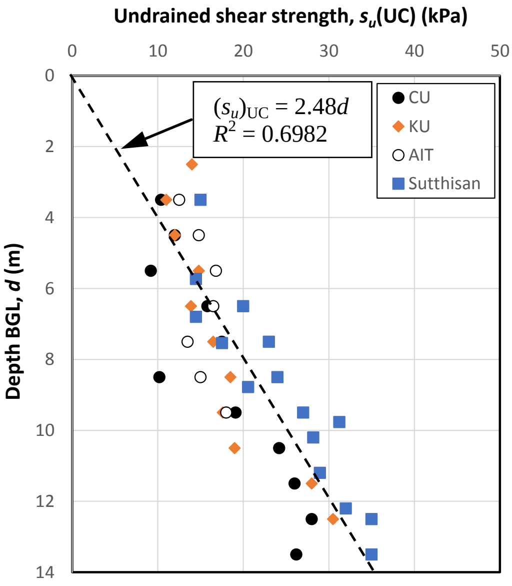



Figure 5 Adjusted UC results

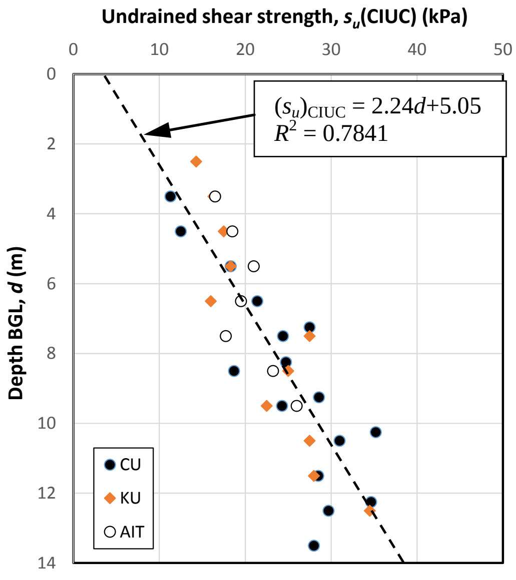



Figure 7 CIUC results

#### 4.2.3 Isotropically consolidated undrained triaxial compression/extension tests

The isotropically consolidated undrained triaxial compression tests (CIUC) are performed on the soil specimens from CU, KU, and AIT sites. The specimens are isotropically consolidated to their in-situ mean effective stresses before undrained shearing. The tests are carried out on the specimens of ϕ 50 mm × ht. 100 mm and with an axial strain rate of 0.0025%/min. The resulting $(s_u)_{CIUC}$ is summarized in Figure 7 and a best-fit straight line is superimposed. It can be seen that the $(s_u)_{CIUC}$ with depth is confined in a narrow band which is much less scatter than those of $(s_u)_{UC}$ and $(s_u)_{UU}$. The isotropically consolidated undrained triaxial extension test (CIUE) data on Bangkok Clay are available from Tapubolon (1981) at AIT.

The soil specimens were isotropically consolidated well beyond their in-situ mean effective stresses to reach NC state before swelling to OCRs = 1, 2, and 4 and being undrained sheared. The tests were carried out on the specimens of ϕ 38 mm × ht. 76 mm and an axial strain rate of 0.064%/min. The $s_u$ values are adjusted to the reference axial strain rate of 0.025%/min using Eq. (14). The adjusted data of Tapubolon (1981) is used to analyze the SHANSEP equation yielding the relationship as shown in Eq. (23). This equation together with properties of Bangkok Clay shown in Table 1 can then be used to calculate $(s_u)_{CIUE}$ variation with depth.

$$\left(\frac{s_u}{\sigma_{vo}'}\right)_{CIUE} = 0.177\mathrm{OCR}^{0.857} \tag{23}$$

#### 4.2.4 $K_o$-consolidated undrained triaxial compression/extension tests

The $K_o$-consolidated undrained triaxial compression test ($CK_oUC$) data on Bangkok Clay are available from Tanaka et al. (2001) and Shibuya et al. (2001) at MRT Sutthisan Station, and Seah & Lai (2003) at AIT. The $K_o$-consolidated undrained triaxial extension test ($CK_oUE$) data on Bangkok Clay are available from Tanaka et al. (2001) at MRT Sutthisan Station and Seah & Lai (2003) at AIT. The soil specimens were $K_o$-consolidated to their in-situ effective stresses before being undrained sheared. The tests were carried out on the specimens of both ϕ 38 mm × ht. 76 mm and ϕ 50 mm × ht. 100 mm and with different axial strain rates of 0.0067 to 0.1%/min. The $s_u$ values are adjusted to the reference axial strain rate of 0.025%/min using Eq. (14). The resulting $(s_u)_{CKoUC}$ and $(s_u)_{CKoUE}$ are summarized in Figs. 8 and 9, respectively, and best-fit straight lines are superimposed. It can be seen that the $(s_u)_{CKoUC}$ with depth are the least scatter data.

#### 4.2.5 Direct shear test

The direct shear test (DS) data of Bangkok Clay are available from Memon (1976) and Seah et al. (2004) at AIT, and Tanaka et al. (2001) and Shibuya et al. (2001) at MRT Sutthisan Station. The soil specimens were one-dimensionally consolidated to their in-situ vertical effective stresses before being constant-volume sheared. The tests were carried out on the specimens of both ϕ 60 mm × ht. 20 mm and □ 60 × 60 mm$^2$ × ht 50 mm and with different speeds of 0.1-1 mm/min. The $s_u$ values are adjusted to the reference speed of 1 mm/min using Eq. (14). The resulting $(s_u)_{DS}$ are summarized in Figure 10 and a best-fit straight line is superimposed. It can be seen that the $(s_u)_{DS}$ with depth is much less scatter than those of $(s_u)_{UC}$ and $(s_u)_{UU}$.

#### 4.2.6 Direct simple shear test

The direct simple shear test (DSS) is performed on the soil specimens from BTS On-nut station. The DSS apparatus is of NGI type (Geonor H-12) which is carried out on the specimen of ϕ 67 mm × ht. 20 mm and with a shear strain rate of 0.083%/min. The specimens are one-dimensionally consolidated well beyond their maximum past pressure to reach NC state before swelling to OCRs = 1, 1.5, 2, 3, and 4 and being constant-volume sheared. The SHANSEP equation is analyzed as shown in Eq. (24). This equation together with properties of Bangkok Clay shown in Table 1 can then be used to calculate $(s_u)_{DSS}$ variation with depth.

$$\left(\frac{s_u}{\sigma_{vo}'}\right)_{DSS} = 0.23OCR^{0.77} \tag{24}$$

#### 4.2.7 Laboratory vane test

The laboratory vane test (lab vane) data (ASTM D4648) of Bangkok Clay are available from Akrapongpisai (1970) at AIT. The tests were carried out on the specimens of ϕ 75 mm × ht. 75 mm with a blade of 13 × 13 mm$^2$ rotating at a speed of 10°/min. The results are presented in Figure 11. It can be seen that the $(s_u)_{lab\ vane}$ decreases with depth.

#### 4.2.8 Triaxial vane test

The triaxial vane test (TX vane) data of Bangkok Clay are available from Seah et al. (2004) at AIT. The tests were carried out on the specimens of ϕ 50 mm × ht. 100 mm which were $K_o$-consolidated to their in-situ effective stresses. The vane blade had the size of width 8.4 mm × ht. 16.8 mm and was rotated at a rate of 19°/min until the rotation reached 360°. The results are presented in Figure 11. It can be seen that the $(s_u)_{TX\ vane}$ decreases with depth.

The undrained shear strengths from various laboratory tests are found to be approximately increasing linearly with depth. This is due to the uniformity of very soft to medium stiff Bangkok Clay with NC to LOC states. However, the results from lab vane and TX vane tests show opposite tendency which may be due to the fact that the soil specimens were not under their in-situ effective stress and they might not be fully saturated. Although, the soil specimens in TX vane test were reconsolidated to their in-situ effective stresses and were fully saturated, the resulting $s_u$ values are still decreasing with depth. This is quite surprising and the factors inherit in the vane-type test may be the reasons, e.g. anisotropy, progressive failure, and strain softening. However, due to the limited number of TX vane data, the conclusion of suitability of this test for Bangkok Clay is still reserved. Due to this peculiar characteristic of undrained shear strengths from lab vane and TX vane, their data are excluded from subsequent discussion.

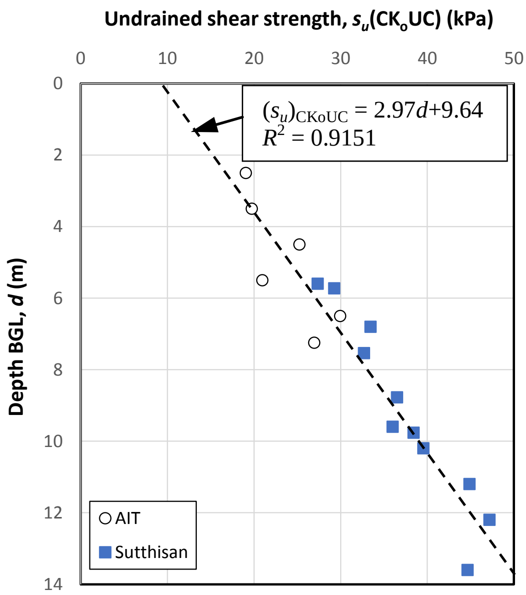



Figure 8 $CK_oUC$ results

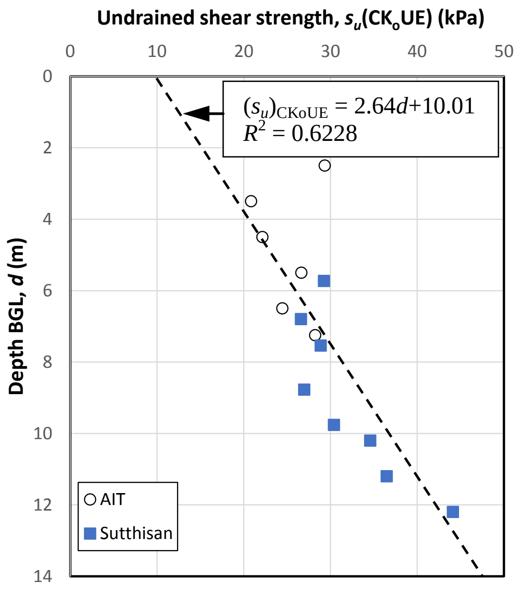



Figure 9 $CK_oUE$ results

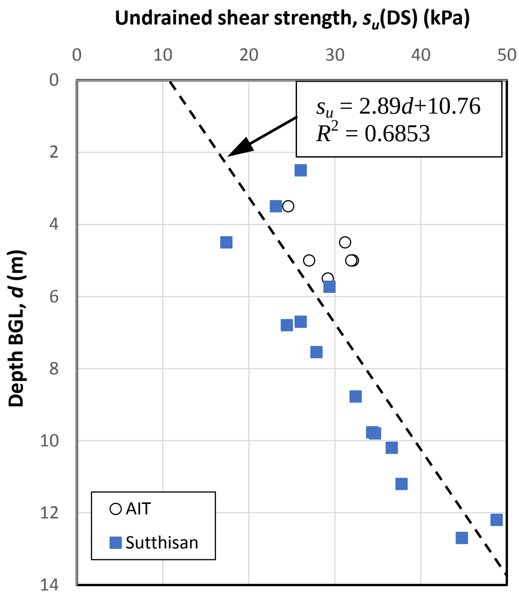



Figure 10 DS results

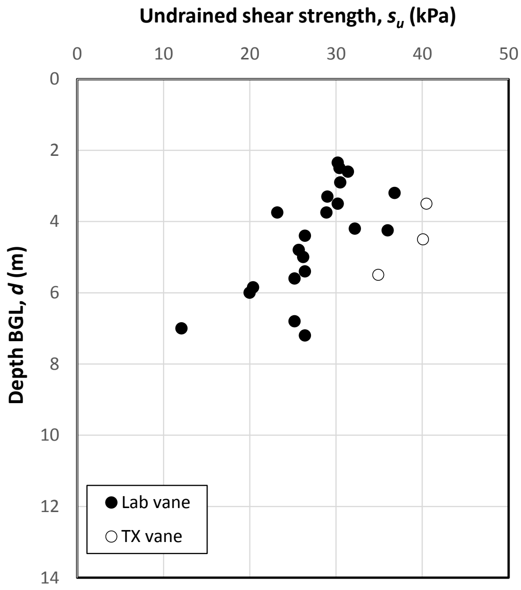



Figure 11 Lab vane and TX vane results

The straight-line relationships of $s_u$ with depth from various laboratory tests have different scattering. The degree of scattering (indicating from $R^2$) in increasing order is $CK_oUC$, CIUC, UC, DS, $CK_oUE$, and UU, respectively. There is no data of CIUE and DSS because they are analyzed by SHANSEP technique. The scattering of $s_u$ data of UC and UU tests are comparable with other tests only after the data have gone through some adjustment. The unadjusted data of UC and UU tests are very scatter because these tests do not reconsolidate the soil specimens to their in-situ effective stress and do not ensure fully saturation. Therefore, the $s_u$ from the tests with these conditions should be treated with caution for their scattering nature. It is noted that the scattering of $s_u$ data from DS is relatively low although this test is known to have several defects, e.g. the stresses and strains are not uniform, the location of the shear surface is imposed on the soil, and there are frictional losses.

## 5. CORRELATION AMONG UNDRAINED SHEAR STRENGTH FROM VARIOUS LABORATORY TESTS

Figure 12 shows undrained shear strength ($s_u$) with depth measured by various laboratory tests. The values of $s_u$ from UC, UU, CIUC, $CK_oUC$, $CK_oUE$, and DS tests are average values of the data presented earlier. The values of $s_u$ from CIUE and DSS are derived from SHANSEP equations (Eqs. (23) & (24)) together with properties of Bangkok Clay shown in Table 1. There is a certain magnitude of variation in strength measured in the laboratory depending on the test types. With few exceptions, the largest $s_u$ is obtained from $CK_oUC$ test, whereas the smallest $s_u$ is obtained from UU test.

Wroth (1984) recommended the $s_u$ from isotropically consolidated undrained triaxial compression (CIUC) test as a standard "test of reference" for convenience in comparison of $s_u$ among various tests. Figure 13 shows the comparison of $s_u$ from various laboratory tests with $s_u$ from CIUC. The linear regression lines are superimposed and their equations and $R^2$ are summarized in Eq. (25). The values of $s_u$ in order from largest to smallest are $CK_oUC$, DS, $CK_oUE$, DSS, CIUC, CIUE, UC, and UU, respectively. The DSS and CIUC yield nearly the same results and CIUE and UC also yield nearly the same results. The $R^2$ in decreasing order is $CK_oUC$, UC, DS, $CK_oUE$, DSS, CIUE, and UU, respectively. The small values of $R^2$ of UU test are due to their scattering of data as described earlier.

Figure 14 shows the undrained shear strength ratios ($s_u/\sigma_{vo}'$) with depth. The $\sigma_{vo}'$ values are obtained from Table 1. The values of $s_u/\sigma_{vo}'$ tend to be larger at shallow depth (< 4 m BGL) due to their larger values of OCR. The $s_u/\sigma_{vo}'$ at NC state can be calculated from SHANSEP equation (Eq. 7) with $m$ = 0.8. The resulting $(s_u/\sigma_{vo}')_{NC}$ with depth is plotted in Figure 15 which shows more uniform values with depth. The results of $(s_u)_{CIUE}$ and $(s_u)_{DSS}$ are constant with depth because they are analyzed from SHANSEP equations (Eqs. 23 & 24) The average values of $(s_u/\sigma_{vo}')_{NC}$ of various laboratory tests are summarized in Table. 6. Table 6 also shows the values of $(s_u/\sigma_{vo}')_{field}$ of various laboratory tests which are calculated from SHANSEP technique (Eq. 7) using OCR = 1.3 and $m$ = 0.8.

$$\left.\begin{aligned}
(s_u)_{UC} &= 0.843(s_u)_{CIUC} && R^2 = 0.763\\
(s_u)_{UU} &= 0.673(s_u)_{CIUC} && R^2 = 0.414\\
(s_u)_{CIUE} &= 0.873(s_u)_{CIUC} && R^2 = 0.430\\
(s_u)_{CK_oUC} &= 1.451(s_u)_{CIUC} && R^2 = 0.870\\
(s_u)_{CK_oUE} &= 1.287(s_u)_{CIUC} && R^2 = 0.571\\
(s_u)_{DS} &= 1.364(s_u)_{CIUC} && R^2 = 0.711\\
(s_u)_{DSS} &= 1.099(s_u)_{CIUC} && R^2 = 0.511
\end{aligned}\right\} \quad (25)$$

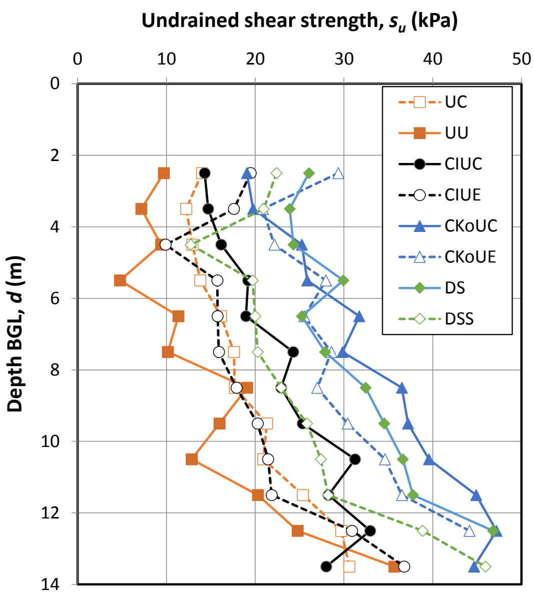



Figure 12 $s_u$ profiles with depth

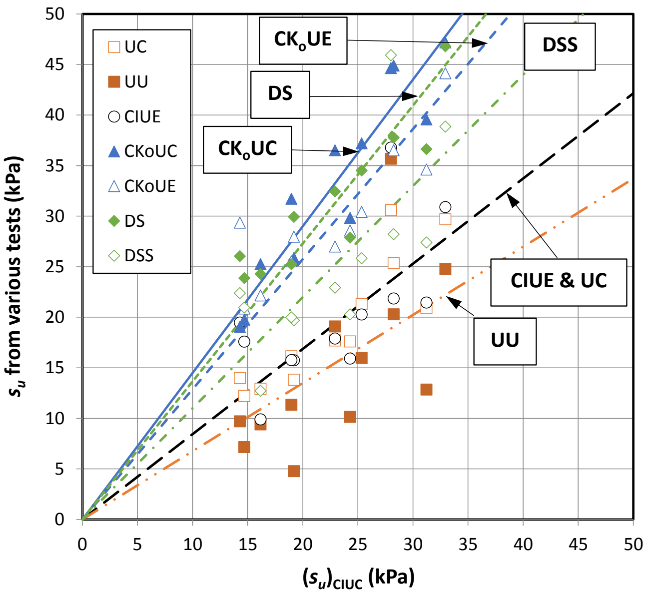



Figure 13 Comparison of $s_u$ from various laboratory tests

Table 6 Average $s_u/\sigma_{vo}'$ from various laboratory tests

| Test types | $(s_u/\sigma_{vo}')_{NC}$ | | $(s_u/\sigma_{vo}')_{field}$ |
|---|---|---|---|
| | **Bangkok Clay (this study)** | **Boston Blue Clay (Mayne et al., 2009)** | **Bangkok Clay (this study) (OCR = 1.3, *m* = 0.8)** |
| UC | 0.18 | 0.14 | 0.22 |
| UU | 0.13 | 0.19 | 0.16 |
| CIUC | 0.22 | 0.32 | 0.27 |
| CIUE | 0.18 | 0.24 | 0.22 |
| CK$_0$UC | 0.31 | 0.33 | 0.39 |
| CK$_0$UE | 0.30 | 0.16 | 0.36 |
| DS | 0.31 | NA | 0.38 |
| DSS | 0.23 | 0.20 | 0.28 |

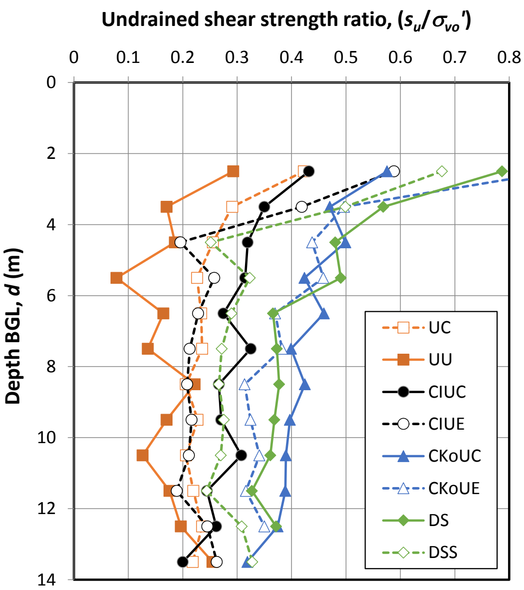



Figure 14 $s_u/\sigma_{vo}'$ profiles with depth

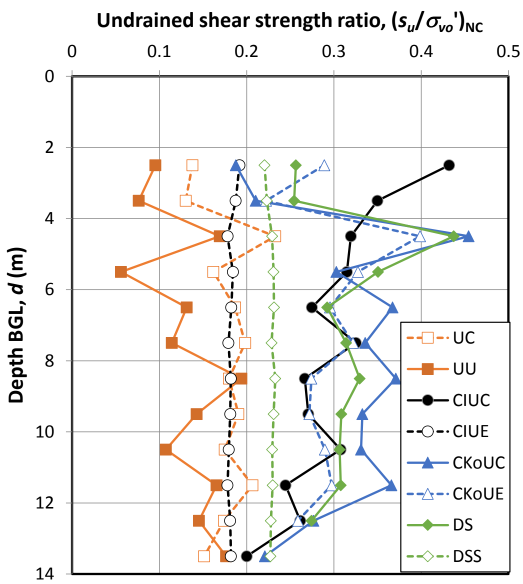



Figure 15 $(s_u/\sigma_{vo}')_{NC}$ profiles with depth

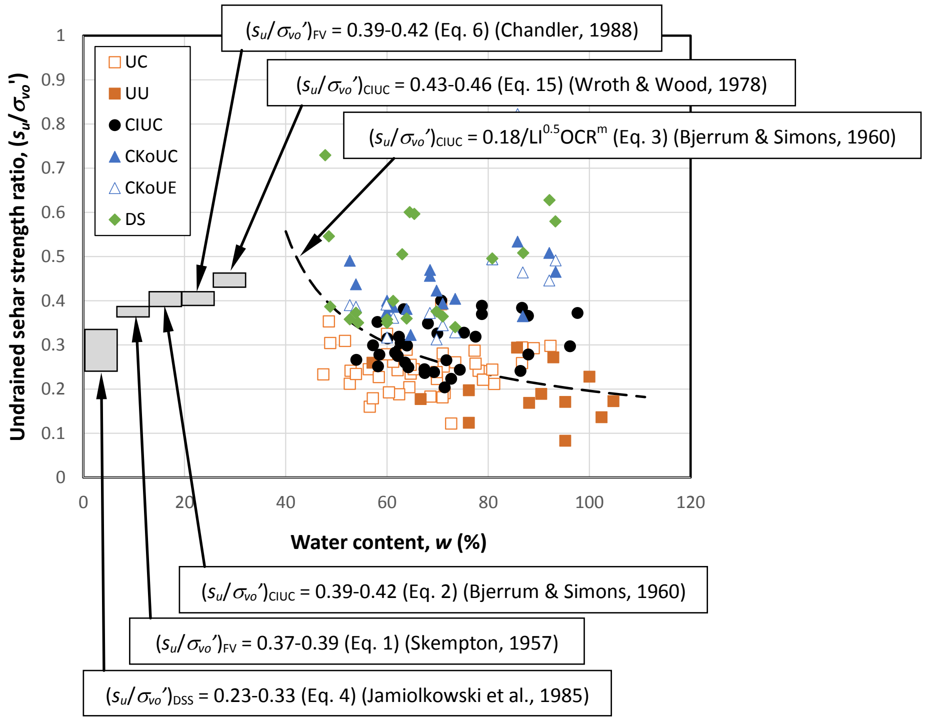



Figure 16 Relationship between $s_u/\sigma_{vo}'$ and water content

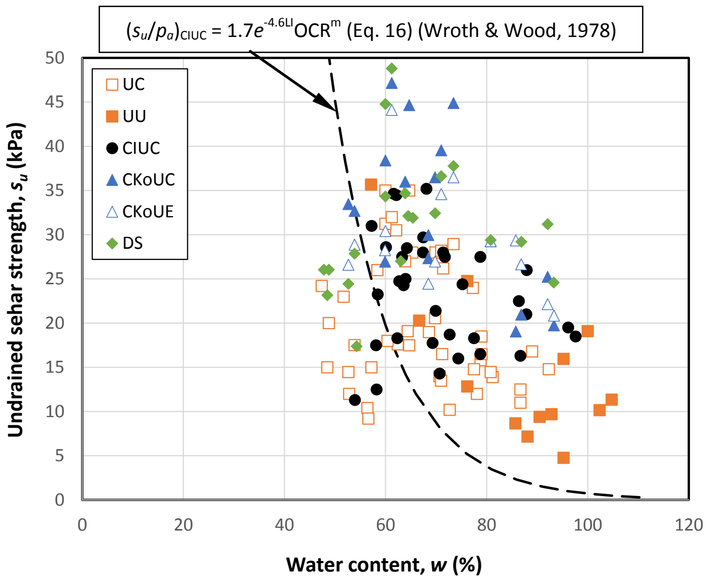



Figure 17 Relationship between $s_u$ and water content

## 6. DISCUSSIONS

The index propertied of Bangkok Clay have been shown in Table 3 as LL = 80-90%, PL = 30-33%, and PI = 52-57%. The water contents of very soft to soft clay and medium stiff clay are 70-80% (LI = 0.75-0.83) and 65% (LI = 0.66), respectively. The Bangkok Clay is of lightly overconsolidated with OCR = 1.3. The CIUC tests from this study yield the friction angle $\phi_{TC}'$ of 27° ($M$ = 1.07) which is relatively consistent with other reported data (e.g. Surarak et al., 2012; Yimsiri et al., 2013). The isotopic consolidation during CIUC test in this study give $\lambda$ = 0.25 and $\kappa$ = 0.025. The Skempton's pore pressure parameter $(A_f)_{CIUC}$ = 0.76 (this study) and $(A_f)_{CKoUC}$ = 0.50 (Seah & Lai, 2003). The $K_o$ value is approximately 0.76 (Shibuya et al., 2001; and Seah & Lai, 2003).

Table 6 shows that the values of $(s_u/\sigma_{vo}')_{NC}$ obtained from this study are mostly agree with the data of Boston Blue Clay summarized by Mayne et al. (2009), except that the data of CK$_o$UE are much larger while the data of UU, CIUC, and CIUE are much smaller. Figure 16 shows the relationship between $(s_u/\sigma_{vo}')$ from various laboratory tests and water content ($w$ = 40-110%). Various relationships between $(s_u/\sigma_{vo}')$ and index properties both empirical (Eqs. 1, 2, 3, 4, and 6) and theoretical (Eq. 15) are calculated using the properties of Bangkok Clay (considering effects of OCR according to Eq. 7) and superimposed. It can be seen that the values of $(s_u/\sigma_{vo}')_{CIUC}$ according to Eqs. (2) and (15) are at an upper boundary of the $(s_u)_{CIUC}$ data of Bangkok Clay, whereas that according to Eq. (3) gives better fit. The values of $(s_u/\sigma_{vo}')_{CIUC}$ can also be estimated according to additional empirical and theoretical equations as follow: 0.40 (Eq. 8), 0.36 (Eq. 18), and 0.36 (Eq. 19) which are also at an upper boundary of $(s_u)_{\mathrm{CIUC}}$ data of Bangkok Clay. The values of $(s_u/\sigma_{vo}')_{\mathrm{CKoUC}}$ can be estimated according to theoretical equations as follow: 0.39 (Eq. 9), 0.39 (Eq. 17), and 0.32 (Eq. 20) which are at average and lower boundary, respectively, of $(s_u)_{\mathrm{CKoUC}}$ data of Bangkok Clay. Moreover, the result of 0.39 from Eqs. (9) and (17) is identical to $(s_u/\sigma_{vo}')_{\mathrm{field}}$ of $(s_u)_{\mathrm{CKoUC}}$ data obtained from this study. The value of $(s_u/\sigma_{vo}')_{\mathrm{DSS}}$ can be estimated according to Eq. (4) as 0.23-0.33 and Eq. (10) as 0.27. It is noted that the value from Eq. (10) is identical to the value of $(s_u/\sigma_{vo}')_{\mathrm{field}}$ of $(s_u)_{\mathrm{DSS}}$ data obtained from this study.

Figure 17 shows the relationship between $s_u$ from various laboratory tests and water content. The value of $(s_u)_{\mathrm{CIUC}}$ can also be predicted by Eq. (16) (considering effects of OCR according to Eq. 7) which gives $s_u$ smaller than the $(s_u)_{\mathrm{CIUC}}$ data of Bangkok Clay. The relationship between $(s_u)_{\mathrm{DSS}}$ and $(s_u)_{\mathrm{CKoUC}}$ in Eq. (11) show relatively similar result with this study which give $(s_u)_{\mathrm{DSS}} = 0.75(s_u)_{\mathrm{CKoUC}}$. The relationship between $(s_u)_{\mathrm{DSS}}$ and $(s_u)_{\mathrm{CKoUC/E}}$ in Eq. (13) can be analyzed by using Eq. (25), which yields $(s_u)_{\mathrm{DSS}} = 1.23(s_u)_{\mathrm{CIUC}}$. This result is slightly larger than the result observed in this study which gives $(s_u)_{\mathrm{DSS}} = 1.10(s_u)_{\mathrm{CIUC}}$. In this study, the ratio of $(s_u)_{\mathrm{UU\ or\ UC}}/(s_u)_{\mathrm{CIUC}}$ = 0.67-0.84 (see Eq. 25) which is larger than the data of Boston Blue Clay (Mayne et al. 2009), which is 0.44-0.59, and also larger than that suggested by Chen & Kulhawy (1993), which is 0.40-0.75.

The ratio of $s_u$ from triaxial extension to compression can be used to indicate undrained shear strength anisotropy. The results from this study (Eq. 25) show that $(s_u)_{\mathrm{CIUE}}/(s_u)_{\mathrm{CIUC}} = 0.87$ and $(s_u)_{\mathrm{CKoUE}}/(s_u)_{\mathrm{CKoUC}} = 0.89$. These can be compared with the data of Boston Blue Clay (Table 6) which give $(s_u)_{\mathrm{CIUE}}/(s_u)_{\mathrm{CIUC}} = 0.75$ and $(s_u)_{\mathrm{CKoUE}}/(s_u)_{\mathrm{CKoUC}} = 0.48$. It can be seen that the ratios of Bangkok Clay are larger than those of Boston Blue Clay. This is consistent with Tanaka et al. (2001) who showed that the $(s_u)_{\mathrm{CKoUE}}/(s_u)_{\mathrm{CKoUC}}$ of Bangkok Clay is very large and shows more isotropic behavior than those of European and Japanese marine clays.

The undrained shear strengths for design of embankment stability (mobilized undrained shear strength, $((s_u)_{\mathrm{mob}})$, have been proposed by Mesri (1975) as shown in Eq. (5). This yields $(s_u/\sigma_{vo}')_{\mathrm{mob}} = 0.29$ for Bangkok Clay (OCR = 1.3) which is close to the value of $(s_u/\sigma_{vo}')_{\mathrm{field}}$ obtained from CIUC test (Table 6). This suggests that the data from $(s_u)_{\mathrm{CIUC}}$ can be directly used for embankment stability analysis without any correction providing that the strain rate used for the test is close to the reference strain rate indicated in this study. In Japan, $(s_u)_{\mathrm{UC}}$ is traditionally used without correction factor (e.g. Tanaka & Tanaka, 1997). The results from this study indicate that this approach would be too conservative for Bangkok Clay. One may argue that $(s_u)_{\mathrm{UC}}$ can still be used for embankment stability analysis of Bangkok Clay by converting it to $(s_u)_{\mathrm{CIUC}}$ as presented in Eq. (25) before being utilized. The possibility of this approach should be used with caution because of the uncertain nature of $(s_u)_{\mathrm{UC}}$ due to its large scatter as shown in Figure 4. Hanzawa (1992) proposed the $(s_u)_{\mathrm{mob}}$ as $(s_u)_{\mathrm{DS}}$ with correction factor of 0.85 to take into account of strain rate effect. Tanaka et al. (2001) suggested $(s_u)_{\mathrm{mob}}$ as an average of $(s_u)_{\mathrm{CKoUC}}$ and $(s_u)_{\mathrm{CKoUE}}$ together with the correction factor for the rate effect of 0.86. Both of these approaches give $(s_u)_{\mathrm{mob}}$ of 0.32 which is slightly larger than $(s_u)_{\mathrm{mob}}$ proposed by Mesri (1975). This implies that using both of these approaches for embankment stability analysis would be slightly unconservative for Bangkok Clay.

## 7. CONCLUSIONS

This research investigates the values of $s_u$ of very soft to medium stiff Bangkok Clay measured by various laboratory tests, i.e. (i) unconfined compression test (UC), (ii) unconsolidated undrained triaxial test (UU), (iii) isotropically consolidated undrained triaxial compression/extension tests (CIUC/CIUE), (iv) $K_o$-consolidated undrained triaxial compression/extension tests ($\mathrm{CK_oUC/CK_oUE}$), (v) direct shear test (DS), (vi) direct simple shear test (DSS), (vii) laboratory vane test (lab vane), and (viii) triaxial vane test (TX vane). The undrained shear strengths from various laboratory tests are found to be increasing linearly with depth except those from lab vane and TX vane tests which show opposite trend. Therefore, the suitability of these tests to measure undrained shear strength of Bangkok Clay should be further investigated. The straight-line relationships of $s_u$ with depth from various laboratory tests show that UU and UC tests are the most scatter data (without any adjustment) because these tests do not reconsolidate the soil specimens to their in-situ effective stresses and do not ensure fully saturation. Therefore, the $s_u$ from the tests with these conditions should be treated with caution for their scattering nature.

The values of $s_u$ of Bangkok Clay in order from largest to smallest are $\mathrm{CK_oUC}$, DS, $\mathrm{CK_oUE}$, DSS, CIUC, CIUE, UC, and UU, respectively. The interrelationships among various $s_u$ with $(s_u)_{\mathrm{CIUC}}$ are presented in Eq. (25), the $R^2$ of which in decreasing order (increasing scattering) is $\mathrm{CK_oUC}$, UC, DS, $\mathrm{CK_oUE}$, DSS, CIUE, and UU, respectively. The values of $(s_u/\sigma_{vo}')_{\mathrm{NC}}$ from various laboratory tests obtained from this study are generally agree with the data of Boston Blue Clay summarized by Mayne et al. (2009), except that the data of $\mathrm{CK_oUE}$ are much larger while the data of UU, CIUC, and CIUE are much smaller. The ratio of $s_u$ from triaxial extension to compression, i.e. $(s_u)_{\mathrm{CIUE}}/(s_u)_{\mathrm{CIUC}}$ and $(s_u)_{\mathrm{CKoUE}}/(s_u)_{\mathrm{CKoUC}}$, shows that Bangkok Clay has more isotropic behavior (in terms of undrained shear strength) than other reported clays.

The validity of various empirical and theoretical relationships for predicting undrained shear strength ($s_u$) and undrained shear strength ratio ($s_u/\sigma_{vo}'$) of Bangkok Clay are investigated. The $(s_u)_{\mathrm{CIUC}}$ of Bangkok Clay can be conservatively estimated by Eq. (3) (Bjerrum & Simons, 1960), while Eqs. (2), (8), (15), (18), and (19) give values at upper boundary of the data. The $(s_u)_{\mathrm{CKoUC}}$ of Bangkok Clay can be predicted by Eqs. (9) and (17) (Kulhawy & Mayne, 1990), while Eq. (20) gives values at lower boundary of the data. The $(s_u)_{\mathrm{DSS}}$ of Bangkok Clay can be predicted by Eq. (10) (Kulhawy & Mayne, 1990). The relationships of $s_u$ from various mode of shearing obtained from this study show that the obtained ratio of $(s_u)_{\mathrm{DSS}}/(s_u)_{\mathrm{CIUC}}$ is relatively consistent with published data while the obtained ratio of $(s_u)_{\mathrm{UU\ or\ UC}}/(s_u)_{\mathrm{CIUC}}$ is much larger than published data.

According to the mobilized undrained shear strength proposed by Mesri (1975), the results from this study suggest that $(s_u)_{\mathrm{CIUC}}$ can be directly used for embankment stability analysis for Bangkok without any correction. The use of $(s_u)_{\mathrm{UC}}$ without correction factor for embankment stability analysis, as traditionally used in Japan, would be too conservative for Bangkok Clay. On the other hand, the use of $(s_u)_{\mathrm{DS}}$ with a correction factor (Hanzawa, 1992) and the use of an average of $(s_u)_{\mathrm{CKoUC}}$ and $(s_u)_{\mathrm{CKoUE}}$ with a correction factor (Tanaka et al., 2001) would yield slightly unconservative results for Bangkok Clay. An alternative approach of using $(s_u)_{\mathrm{UC}}$ with conversion to $(s_u)_{\mathrm{CIUC}}$ using Eq. (25) before being utilized for embankment stability analysis for Bangkok Clay should be used with caution because of the uncertain nature of $(s_u)_{\mathrm{UC}}$ as presented in this study.

## 8. ACKNOWLEDGEMENTS

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