CHAPTER 4

RESULTS AND DISCUSSIONS

4.1 Basic Properties of Soil

4.1.1 At Memorial Bridge

The basic properties of the soft clay at Memorial Bridge are summarized in Table 4.1. Below the depth of 3.50 m., i.e. the crust, the water content generally decreased with depth, it ranged from 62.6 to 33.20 %. The liquid limit also showed the same trend, and the values were closed to the natural water content. This illustrats that the clay is practically in the normally or slight overconsolidated stage. Plasticity index ranged from 36.50 % at the upper part and decreased to 12.60 % at the lower part of soft clay. It was classified according to Unified Soil Classification as clay with medium to high plasticity (CH). The bulk unit weight generally varies from 1.50 to 1.87 t/m^3, increasing with depth

4.1.2 At Teves

The basic properties of the soft clay at Teves are summarized in Table 4.2. The water content ranged from 51.29 to 78.92 % from the depths of 3.50 m to 12.50 m. The value generally decreased with depth excepting at 9.50 m, where the clay water content sharply increases. The liquid limit is closed to the natural water content, generally 1 to 5 % higher. The plasticity index vary from 28.80 to 47.90 %, showing no definite trend with depth. They are also classified according to Unified Soil classification as clay with high plasticity

Table 4.1 Basic Properties and Unit Weight.

At Memorial Bridge

Depth (m)	Description of Sample	Properties of Samples					$\bar{\sigma}_{vc}$	$\frac{C_c}{1+e_o}$	$\bar{\sigma}_{vm}$
		W %	LL %	PL %	PI %	γ_T (t/m^3)	(t/m^2)		(t/m^2)
3.50	Clay, Some silt trace of organic matter, grey, very soft (CH)	62.60	69.60	26.90	32.70	1.50	3.70	0.17	4.30
5.00		57.54	57.80	26.20	31.60	1.66	4.90	0.17	5.40
6.50		51.35	57.30	26.20	31.10	1.75	6.20	0.24	6.20
8.00		42.66	52.80	27.80	25.00	1.67	7.77	0.21	7.77
9.50	Clayey silt, trace of mica, li-brown&gray, medium (CH)	44.51	65.10	28.60	36.50	1.69	9.70	-	-
11.00		41.80	44.50	24.90	19.60	1.70	11.73	-	-
12.50	Silty very fine sand with clay layer alternating greenish gray, medium dense (ML)	33.20	38.20	25.40	12.60	1.87	13.90	-	-

Table 4.2 Basic Properties and Unit Weight at Teves

Depth (m)	Description of Sample	Properties of Samples					$\bar{\sigma}_{vc}$	$\frac{C_c}{1+e_o}$	$\bar{\sigma}_{vm}$
		W %	LL %	PL %	PI %	γ_T (t/m^3)	(t/m^2)		(t/m^2)
3.50	Clay trace to some fine sand & Organic matter, shell, grayish brown, very soft (CH)	66.65	67.20	27.80	39.40	1.70	2.75	0.09	10.55
5.00		67.25	68.40	32.20	36.20	1.60	3.75	-	-
6.50	Clay trace some shell & mica, gray soft (CH)	60.91	61.00	29.50	31.50	1.64	5.53	0.12	11.30
8.00		58.60	61.00	30.30	30.70	1.65	7.32	-	-
9.50		78.92	79.20	31.30	47.90	1.54	9.09	0.11	9.25
11.00	Clay trace to fine sand, gray, soft (CH)	51.40	63.80	28.00	35.80	1.64	12.33	-	-
12.50		51.29	57.10	28.30	28.80	1.70	16.44	-	-

(CH). The bulk unit weight generally varies from 1.54 to 1.70 t/m^3, increasing with depth.

The value of $\frac{C_c}{1+e_o}$ for Taves certainly indicates that the soft clay samples are somewhat disturbe, as $\frac{C_c}{1+e_o}$ is rather low for his clay PI.

4.1.3 Comparison of Basic Properties at Memorial Bridge and Teves including in situ Pore Pressure Distribution.

The soft clay at Teves tend to have higher water content and the plasticity index than those at Memorial Bridge site, and the bulk unit weight is lower at Teves sites. The distribution of pore pressure at both sites are previously shown in Fig. 3.3 and Fig. 3.4.

The drawdown of the hydrostatic pressure at Memorial Bridge is greater than at Teves, and the soft clay layer at Memorial Bridge tends to be more compressed by the higher insitu effective stress than that at Teves. This results from the fact that there is a near by pumping station near the Memorial Bridge site. At Memorial Bridge site the OCR value varies from 1.20 to 1.0 from 3.50 to 9.50 m. depth clearly illustrated partly the influence of deep well pumping that causing the declined in the OCR value in Bangkok soft and medium clay.

At Teves, the OCR decreases linearly with depth from the 3.50 m. depth down to 6.50 m depth. The value ranges from 3.83 to 2.04. At Teves, from 9.50 m depth down to 12.50 m depth, the OCR value of the soft and medium clay is equal to one, resulting from the influence of deep well pumping.

The OCR values versus depth at Memorial Bridge and Teves are shown in Fig. 4.1 and Fig. 4.2 respectively. The consolidation test results at both sites are shown in Appendix A. The value of compression ratio $(\frac{C_c}{1+e_o})$ and the maximum past pressure are also shown in Table 4.1 and Tabld 4.2 for Memorial Bridge and Teves, respectively.

4.2 Shear Strength of the Clay Samples

4.2.1 Field vane shear test

The field vane shear strength was measured using Geonor vane shear device at both sites. The dimension of vane is 55 mm and 110 mm for diameter and height, respectively. The measurement was performed using the rotation rate of 0.1 degree per second. Geonor vane tests were performed at every 0.50 m interval. The values presented with depth are shown in Fig. 4.3 and Fig. 4.4 for locations at Memorial Bridge and Teves, respectively.

4.2.1.1 Comparison of Field vane shear test at Memorial Bridge and Teves.

At Memorial Bridge site, the field vane shear strength of the clay layer from 5.0 m to 12.50 m increases rapidly with depth from 1.86 t/m^2 to 6.48 t/m^2. However from 3.50 to 4.50 m the field vane shear strength is practically constant with depth, and equal to 1.89 t/m^2.

The field vane shear strength at Teves site also shows the same trend. However, due to having higher OCR and the plasticity index, the field vane shear strength at upper 5.0 m depth is higher i.e. 2.75 t/m^2 and the changes in shear strength with depth are gradually vary from 2.75 t/m^2 at 5.0 m depth to 4.80 t/m^2 at 12.50 m depth.

The field vane value was corrected using corrected coefficient (μ) recommended by Bjerrum (1972), depending upon plasticity index of clay which is also varied with depth. The corrected Geornor vane strength values showed the same trend at those which are not corrected. Generally, the value was lower than that uncorrected, as shown in Fig. 4.3 and Fig. 4.4. The normalized field vane and corrected field vane shear strength versus OCR were plotted in Fig. 4.5

The value of μ versus depth at both sites are listed in table

Table 4.3 The value of μ versus depth for each site

Depth (m)	Value of μ at Memorial Bridge	Value of μ at Teves
3.50	0.90	0.81
5.00	0.91	0.88
6.50	0.91	0.90
8.00	0.96	0.92
9.50	0.88	0.82
11.00	1.0	0.89
12.50	1.03	0.95

4.2.2 The Dutch Cone Test

The Dutch cone test was performed from ground surface to the 14.00 m depth using 60° cone tip. The projected area of tip is 10 cm^2 with the friction sleeve. Both the cone resistance and local friction were continuous registered and they were plotted against depth in Fig. 4.6 and Fig. 4.7 for Memorial Bridge and Teves, respectively.

4.2.2.1 Comparisons of Results from Geonor Vane and Dutch Cone Tests.

From the cone resistance and local friction, the values were correlated with the field vane shear strength and corrected vane shear strength values. The cone resistance q_c values are correlated with Geonor field vane strength and corrected Geonor field vane strengths for both sites, and relations were plotted in Fig. 4.8 and Fig. 4.9 respectively. The local friction from Dutch cone tests were also correlated with field vane, and corrected field vane strengths, and relations were shown in Fig. 4.10 and Fig. 4.11, respectively. As seen in Fig. 4.8 and Fig. 4.9, for these two sites, the field vane shear strengths when plotted with q_c from Dutch cone tests yield good correlations. Relations can be fitted in the linear relationship.

For field vane shear strengths

$$q_c = 19.5\ S_{uv} \quad , r^2 = 0.92 \text{ (Fig. 4.8)}$$

$$f_s = 1.2\ S_{uv} \quad , r^2 = 0.87 \text{ (Fig. 4.10)}$$

where r^2 = the coefficient of correlation

and for corrected field vane strength (μS_{uv})

$$q_c = 21\ \mu S_{uv} \quad , r^2 = 0.90 \text{ (Fig. 4.9)}$$

$$f_s = 1.3\ \mu S_{uv} \quad , r^2 = 0.89 \text{ (Fig. 4.11)}$$

Values of f_s from Dutch cone tests could be used for evaluating the ultimate skin friction of the pile. Attempt was made to compute the ultimate pile skin friction first by using data from field vane. Upon knowing the crude estimation of the ultimate pile skin friction in soft and medium clay at a particular depth from field vane strength using adhesion factor from Holmberg (1970), the friction factor (K_c), defined as $K_c = \dfrac{f_s \text{ of the pile}}{f_s \text{ from Dutch Cone test}}$, can be evaluated. These K_c values then can be used for determing the pile ultimate skin friction from f_s values measured from Dutch cone tests.

The pile ultimate skin friction is determned from multiplying the adhesion factor, recommended by Holmberg (1970) for Bangkok clay, with S_u values from field vane tests. The values of α from Holmberg (1970) are plotted against the field vane strength values, shown in Fig. 4.12. The ultimate skin friction of pile for various shear strength values are thus obtained. Dividing the ultimate skin friction of pile by local friction from Dutch cone test at that depth gave the friction factor. These factors were compared with the factors done by Nottingham (1975) see Fig. 4.13. The factors for soft and medium clay derived from this research seem to be less than Nottingham's factor.

4.3 Soft Clay Shear Strengths from Laboratory Tests

4.3.1 Shear Strength of Soil from Direct Shear Tests

The consolidated quick direct shear tests for both sites were tested in laboratory using circular sample ring. The insitu effective vertical stresses are calculated for each depth and used to consolidate sample at that depth. The sample was consolidated at many loading steps until the effective overburden pressure is reached and then sheared with a rate of 0.1 mm/min. At each loading step, the sample was allow to consolidated for one day. The normalized shear strength of consolidated quick direct shear test for Memorial Bridge and Teves site are plotted with depth and they are shown in Fig. 4.14 and 4.15 respectively.

The normalized consolidated quick direct shear strengths at Memorial Bridge site is slightly increase with depth, ranging from 0.40 to 0.53. This is partly due to the increase in sample disturbance with depth. The consolidation of samples having high degree of sample disturbance will yield high strength due to the loss of water. It is taught that there is little change in OCR and PI with depth. The average values are about 0.45 from 3.50 m depth to 9.0 m depth, and about 0.50 from 9.0 to 12.50 m depth. For Teves site the normalized strength generally decreases with depth. They ranged from 0.99 to 0.38 for 3.50 m to 11.0 m depth. The normalized strength seems to decrease corresponding with the OCR values.

The quick direct shear tests are also performed for samples at Teves site only using the circular sample ring. The insitu effective vertical stress at each depth was used as the normal stress for the samples. The soil sample was sheared immediately after putting the normal load with out any consolidation. The normalized shear strength versus depth at Teves site is shown in Fig. 4.15. The normalized quick direct shear strengths at Teves site tends to decrease with depth corresponding to OCR values. They ranged from 0.68 to 0.35 for 5.0 m to 11.0 m depth and from OCR of about 2.9 to 1.0 and 0.37 at 12.50 m depth.

At Teves, data from consolidated quick direct shear test indicate the practically constant $S_u/\bar{\sigma}_{vc}$ of about 0.40 for normally consolidated clay. For about the same plasticity index, $S_u/\bar{\sigma}_{vc}$ is equal to 0.45 for normally consolidated clay at Memorial Bridge.

Except between the depths of 9.0 m to 11.0 m, strengths from unconsolidated quick direct shear test are generally lower than those from consolidated quick direct shear tests.

All test results for consolidated quick direct shear tests, quick direct shear tests are shown in Appendix A.

4.3.2 Triaxial and Unconfined Compression Test.

The anisotropically consolidated undrained triaxial compression tests with pore pressure measurement at both sites were reported by The Asian Institute of Technology. The samples at Memorial Bridge were consolidated with the vertical pressure a little bit lower than the effective overburden pressure and the effective horizontal pressure is 0.60 time the effective vertical pressure. Results of these tests are tabulated in table 4.3. For Teves site, the consolidated vertical pressure are generally higher thant the effective overburden pressure and the effective horizontal pressure ar 0.60 $\bar{\sigma}_{vo}$ for depth 9.50 m, and 0.65 to 0.75 $\bar{\sigma}_{vo}$ for 11.0 to 12.50 m depth, and listed in table 4.4.

The unconfined compression test results were also reported by The Asian Institute of Technology at Memorial Bridge site and listed in table 4.4. For Teves site, the unconfined compression tests were reported by Krungthep Engineering consultants Co., Ltd. (KEC), and listed in table 4.5 Results from nearest bore holes were selected.

The consolidation pressures for anisotropically consolidated undrained triaxial compression tests were different from the effective overburden pressure at the both sites, the normalized shear strength using effective overburden pressure were calculated and they were plotted with depth in Fig. 4.14 and Fig. 4.15 for Memorial Bridge and Teves site respectively.

At Teves, since triaxial samples were consolidated at the vertical effective stress much higher than those at the insitu, and hence having different OCR from that of the insitu. Except data in normally consolidated range, other data can not be compared. The normalized shear strength for Memorial Bridge ranged from 0.42 to 0.45 for 5.0 to 9.50 m depth. This may be about 10 % to 15 % higher than the value which should be, since the effective vertical pressure in the sample is usually lower leading to the samples higher OCR and higher normalized strength compared to the insitu value.

The trend shown from plot of shear strength from unconfined compression test and depth at Memorial Bridge and Teves is different due to sample disturbance effect. At Teves, the shear strength

Depth	Unconfined Compression tests			Triaxial Compression tests ($C\bar{A}U$)								
(m)	γ_T (t/m^3)	W %	S_u (t/m^2)	W_i %	W_f %	γ_T (t/m^3)	$\bar{\sigma}_{hc}$ (t/m^2)	$\bar{\sigma}_{vc}$ (t/m^2)	K_c	$\bar{\sigma}_{vo}$ (t/m^2)	$S_u/\bar{\sigma}_{vc}$	$S_u = \frac{S_u}{\bar{\sigma}_{vc}} \times \bar{\sigma}_{vo}$
3.50	1.63	52.1	1.08	-	-	-	-	-	-	-	-	-
5.00	1.56	59.8	1.36	55.0	66.3	1.57	2.40	4.0	0.6	4.90	0.425	2.08
6.50	1.60	58.1	1.98	-	-	-	-	-	-	-	-	-
8.00	-	-	-	-	-	-	-	-	-	-	-	-
9.50	1.70	53.7	2.21	43.0	42.6	1.69	5.10	8.50	0.6	9.70	0.45	4.37
11.00	1.70	39.6	-	-	-	-	-	-	-	-	-	-
12.50	-	-	-	-	-	-	-	-	-	-	-	-

Table 4.4 Results from Unconfined Compression tests and Triaxial Compression Tests ($C\bar{A}U$) at Memorial Bridge.

Depth (m)	Unconfined Compression tests			Triaxial Compression tests ($C\bar{A}U$)								
	γ_T (t/m^3)	W %	S_u (t/m^2)	W_i %	W_f %	γ_T (t/m^3)	$\bar{\sigma}_{hc}$ (t/m^2)	$\bar{\sigma}_{vc}$ (t/m^2)	K_c	$\bar{\sigma}_{vo}$ (t/m^2)	$S_u/\bar{\sigma}_{vc}$	$S_u = \frac{S_u}{\bar{\sigma}_{vc}} \times \bar{\sigma}_{vo}$
3.50	1.73	28.0	3.85	-	-	-	-	-	-	-	-	-
5.00	1.69	59.0	1.63	-	-	-	-	-	-	-	-	-
6.50	1.51	67.0	1.25	-	-	-	-	-	-	-	-	-
8.00	1.59	61.0	0.30	-	-	-	-	-	-	-	-	-
9.50	1.54	78.0	1.28	53.6	47.5	1.58	7.50	12.50	0.60	9.09	0.36	3.27
11.00	-	-	-	52.0	48.9	1.60	9.43	14.50	0.65	12.33	0.29	3.58
12.50	-	-	-	46.4	43.3	1.69	11.63	15.50	0.75	16.44	0.36	5.92

Table 4.5 Results from Unconfined Compression tests and Triaxial Compression Tests ($C\bar{A}U$) at Teves.

decreases from 3.85 to 0.30 t/m^2 betwen 3.50 m to 8.00 m depth, and then increase to 1.28 t/m^2 at 9.50 m depth. The shear strength from unconfined compression tests at Memorial Bridge site increases with depth from 1.08 to 2.21 t/m^2 between 3.50 m to 9.50 m depth.

At both sites the shear strength values from unconfined compression test are generally much lower than these from anisotropically consolidated undrained triaxial compression tests, the deeper the depth is, the greater different in shear strength values as shown in Fig. 4.16, partly due to sample disturbance effect which reduces the strength and reconsolidation in triaxial tests causing the loss in water content leading to too high strength. Compression of consolidated undrained strength from triaxial and direct shear tests indicates higher strength from direct shear test

4.4 Comparison of Shear Strength from Laboratory and Field tests.

4.4.1 Comparison of Shear Strength values using the corrected field vane strengths as a basis.

Shear strength measurement from different laboratory methods are summarized in Fig. 4.17 and 4.18 for locations at Memorial Bridge and Teves respectively. The shear strength determined by Unconfined compression test gives the lowest values for both sites.

At Memorial Bridge site, field vane strength is about the average values between upper bound strength from consolidated quick direct shear strength and corrected field vane strength between the depth of 3.50 m to 11.00 m, and is approximately equal to triaxial strength. The strength from anisotropically consolidated undrained triaxial compression tests is slightly lower than that from consolidated quick direct shear tests at 5.0 m depth but considerably lower at 9.50 m depth.

For Teves site, the following trends were observed.

1. Unconfined compression test gives highest strength at depth smaller than 3.50 m, i.e. in weathered clay partly due to clay at these locations are not homogeneous. Below 3.50 m depth, the unconfined shear strength is lowest, and seems to be very irratic

2. In general, the consolidated quick direct shear tests yield the highest shear strength value except at 3.50 and 9.50 m depth, while the corrected field vane strength is the lowest value except 3.50 and 5.0 m depth when it was compare with other tests except unconfined compression test.

3. The field vane strength is also about the average value of consolidated quick direct shear tests and corrected field vane strength, and is approximately equal to triaxial strength.

4. The quick direct shear strength seems to lower than the field vane strength from 3.50 to 8.0 m and considerable higher than strength from unconfined compression tests.

5. The strength from quick direct shear tests is only a little bit lower than consolidated quick direct shear tests in general.

6. From 5.0 to 8.0 m depth the corrected field vane tests using Bjerrum correction factor, field vane tests, anisotropically consolidated undrained triaxial compression tests, consolidated quick direct shear tests, and quick direct shear tests field smaller variation in strength value compared to that from unconfined compression tests.

The shear strength relationships between each test and field vane test are as follows:

$$\mu S_{uv} = 1.6\ S_{u(UC)} \quad , r^2 = 0.96 \text{ (Fig. 4.19)}$$
$$\mu S_{uv} = 0.90\ S_{u(UD)} \quad , r^2 = 0.87 \text{ (Fig. 4.20)}$$
$$\mu S_{uv} = 0.85\ S_{u(CD)} \quad , r^2 = 0.80 \text{ (Fig. 4.21)}$$
$$\mu S_{uv} = 0.90\ S_{u(TXL)} \quad , r^2 = 0.92 \text{ (Fig. 4.22)}$$

where μ = Bjerrum's correction factor value from 0.81to 1.03

S_{uv} = Field vane shear strength

$S_{u(UC)}$ = Unconfined compressive shear strength

$S_{u(UD)}$ = Quick direct shear strength

$S_{u(CD)}$ = Consolidated quick direct shear strength.

$S_{u(TXL)}$ = Anisotropically consolidated undrained triaxial compressive shear strength

Fairly good agreement is found for correlation between strengths from corrected field vane test, and unconfined compression test, and anisotropically consolidated undrained triaxial compression tests. Good agreement was also found in the case of strength from the quick direct shear tests and consolidated quick direct shear tests respectively, upon comparing with the corrected vane strength. From the above relationship, they indicated that shear strength from vane shear strength, quick direct shear tests and anisotropically consolidated undrained triaxial compression tests are about the same while consolidated quick direct shear tests will give the strength higher than the mentioned two laboratory tests. All of these gave higher shear strength than the corrected field vane tests. The unconfined compression test give the lowest strength tests.

The use of corrected field vane strength is based on Bjerrum's opinion. Bjerrum (1972) stated that the discrepancy between the vane shear strength and the value computed from the large scale field loading tests can be attributed to one or more of the following factors.

1. The shear strength depends on the rate of loading

2. The shear strength is anisotropic

3. The shear strength in nature is reduced due to progressive failure

From his conclusion, the coefficient μ should be applied to field vane test to correct these three effects so the strength value can represent the average strength on the circular arc failure surface.

The high values of strengths from consolidated quick direct shear tests compared to those from triaxial tests partly result from the uncontrolable drainage condition in direct shear test. Decrease in water content during tests (3.60 %) leads to the increase in shear strength. Higher strength values of quick direct shear test compared to that from unconfined compression tests also results from uncontrolable drainage condition.

The shear strength normalized with effective overburden pressure for consolidated direct shear tests and corrected field vane tests are plotted with depth in Fig. 4.14 and Fig. 4.15 for Memorial Bridge and Teves, respectively.

At Memorial Bridge site OCR value is varies between 1.20 and 1.0, decreasing with depth, and the normalized consolidated quick direct shear strength and the corrected normalize field vane strengths are approximately constant for the observed depth. The average normalized values is 0.50 for consolidated quick direct shear and range from 0.39 to 0.50 for corrected field vane.

At Teves sites the OCR value decrease with depth and the normalized strength values decrease with depth. The normalized corrected field vane value are lower thán those from consolidated quick direct shear tests for all depths. Normalized strengths range from 0.35 to 0.99 for consolidated qukck direct shear and 0.26 to 0.99 for corrected field vane shear tests. The normalized quick direct shear strength are generally lower than that of consolidated quick direct shear strength except at 9.50 m depth.

The shear strength normalized with effective overburden pressure for each test at Teves site was plotted versus log of OCR value in Figure 4.23. The figure also showed a curve of Ladd & Edgers (1972) for Bangkok clay. It can be concluded that

1. The normalized strengths from consolidated quick direct shear strengths are generally parallel to that of Ladd & Edger relationship, based on direct simple shear test for Bangkok clay but significantly higher.

2. The normalized strength from quick direct shear strength and anisotropically consolidated undrained triaxial compression strength value are very close together. However those values are higher

than those of Ladd & Edger for OCR less than 2.3, but higher for OCR greater than 2.3 for strength from quick direct shear test.

3. The normalized corrected field vane strength are practically equal to that of Ladd & Edger at OCR less than 2.50, and tend to considerably higher from there on. (Note that at high OCR, the clay is in the weathered clay, this may be the reason for such difference)

4. The normalized field vane strength shows the same trend as the values from corrected field vane strength that is they are parallel, but field vane strength is higher.

4.4.2 Correlations of Laboratory Strength Data with Results from Dutch Cone Tests.

The cone resistance, q_c, is also correlate with the strength determined from anisotropically consolidated undrained triaxial compressive strength ($S_{u(TXL)}$) (see tables 4.3 and table 4.4 for summary of results), consolidated quick direct shear strength ($S_{u(CD)}$), quick direct shear strength ($S_{u(UD)}$). The relations were plotted in Fig. 4.24, Fig. 4.25 and Fig. 4.26 respectively. The relationships are the followings.

$$q_c = 19\ S_{u(TXL)} \quad , r^2 = 0.89 \text{ (Fig. 4.24)}$$

$$q_c = 18.5\ S_{u(CD)} \quad , r^2 = 0.80 \text{ (Fig. 4.25)}$$

$$q_c = 19\ S_{u(UD)} \quad , r^2 = 0.84 \text{ (Fig. 4.26)}$$

The relationship of these strength tests show good fit with Dutch cone resistance, as the coefficient of.correlation (r^2) is about 0.84 to 0.92.

Another relationships between local friction and the strengths determined from anisotropically consolidated undrained triaxial compression tests, consolidated quick direct shear tests, and quick direct shear tests also produced good correlations, as the coefficient of correlation (r^2) values vary only from 0.80 to 0.95. These relationships are considered to be a good relation, as the value of r^2 approaching 1.0

$$f_s = 1.15\ S_{u(CD)} \quad , r^2 = 0.87 \text{ (Fig. 4.27)}$$
$$f_s = 1.2\ S_{u(TXL)} \quad , r^2 = 0.95 \text{ (Fig. 4.28)}$$
$$f_s = 1.2\ S_{u(UD)} \quad , r^2 = 0.80 \text{ (Fig. 4.29)}$$

4.5 Interpretation of Strength Tests

In our research the anisotropically consolidated undrained traixaial compression test shows the strength values less than the consolidated quick direct shear test. This may be due to the loss of water content during consolidation and shearing the sample. It would explain why the consolidated quick direct shear strength yield the largest value.

In quick direct shear tests, at least the loss of water due to drainage is an important factor, although disturbance tends to reduce strength. Due to sample disturbance and drainage, the quick direct shear tests would yield the uncertain values. In our case the quick direct shear tests generally lower than consolidated quick direct shear tests mainly due to the sample disturbance effect in quick direct shear tests, i.e. the soil sample has lower preshear effective vertical stress.

Flaate, K. (1966) summarizes various factors known to influence field vane tests results. They are disturbance due to inserting the vane head, rate of rotation, the influence of time delay between vane insertion and testing and influence in the ratio of height to diameter of vane head. These four factors influence the interpretation and it was therefore, consider the device as a "Strength index" test requiring empirical correlations to give suitable strength value for use in design.

Comparison of the field vane strength corrected by Bjerrum coefficient, μ with the anisotropically consolidated undrained triaxial compression tests at both site are shown in Fig. 4.30 and Fig. 4.31.

The corrected field vane strength will be generally slightly lower than anisotropically consolidated undrained triaxial compression tests. The interpret of corrected field vane strength is the estimate average strength value along the cylindrical mode of failure. It is therefore very likly that strengths from anisotropically consolidated undrined triaxial compression tests should be higher.

It can be seen at both sites that at low OCR (OCR <2.5), the vane strength and anisotropically consolidated undrained triaxial compression strength are approximately equal. This is purely accidental. The variation will exist for Bangkok clay at other plasticity index values.

For the consolidated quick direct shear and quick direct shear tests as stated above that some loss of moisture occurs during testing, and reconsolidation of the specimen in case of consolidated quick direct shear tests will occure. Therefore, these tests might yield higher strength than corrected field vane test which is in our case, especially in consolidated quick direct shear test.

Theoretically, the Dutch cone test result can be expressed in the form.

$$q_c = S_u N_c + \sigma_h \quad \text{(Mohsen M. Baligh, 1975)}$$

where q_c = Cone resistance

S_u = Undrained shear strength

N_c = Cone factor

σ_h = the horizontal total stress at that depth

The value of N_c is determine from the curve derive by Mohsen M. Baligh (1975) see Fig. 4.32. To obtain the rigidity index of clay, I_r, the value of undrained shear modulus of the clay, G, is determine by

$$G = \frac{Eu}{2(1+\nu)}$$

where Eu is the undrained Young's modulus of the clay which is measured from the triaxial compression test

ν is the poisson's ratio = 0.5

and $I_r = \frac{G}{S_u}$

where S_u is the undrained snear strength determine from the field vane test.

This theory is based on assuming the soil to be elasto-plastic material and considering the movement of the cone during testing. The predicted strength value should be that for stability analyses.

Base on this equation the cone factor N_c were plotted with depth as shown in Fig. 4.33. The value of σ_h is determined from the equation below which is recommended by Schmidt, B. (1966)

$$K_o(OC)/K_o(NC) = OCR^m$$

where $K_o(OC) = K_o$ for overconsolidated material

$$= \bar{\sigma}_h/\bar{\sigma}_v$$

$K_o(NC) = K_o$ for normally consolidated material

$OCR = \bar{\sigma}_{vm}/\bar{\sigma}_{vo}$

$K_o(NC)$ and value of the exponent m is also suggested by Ladd (1977) which is shown in Fig. 4.34 and Firg.4.35 respectively, and then

$$\sigma_h = \bar{\sigma}_h + u$$

Theoretically, for soft normally consolidated Bangkok clay N_c value vary between 15.25 to 15.95. N_c for corrected field vane strength vary between 11.72 to 19.00 at Teves. At Memorial Bridge N_c for normally consolidate clay is practically constant at value of about 20. This means that the theory will generally predict the strengths for circular failure arc which is too high.

For empirical correlation, the relationships are given below.

For Triaxial test. $q_c = 19.0\ S_{u(TXL)}$

For Direct shear test $q_c = 18.5\ S_{u(CD)}$

$q_c = 19.0\ S_{u(UD)}$

For field vane test $q_c = 19.5\ S_{uv}$

$q_c = 21\ \mu S_{uv}$

For Unconfined test $q_c = 37\ S_{u(UC)}$

4.6 Disadvantages of Triaxial Compression Tests, Direct Shear and Unconfined Compression Tests.

In stability analyses of the embankment on soft ground it should be recognized that the stress system at different point on circular arc failure are varying as shown in Fig. 4.36. The Triaxial compression test gives the good estimate strength on the top portion of circular arc near the embankment, direct simple shear test in the lowest part of circle and the triaxial extension test give the top part near the base ground surface.

The Triaxial compression test give the highest strength, value when the principal stress is in vertical direction, then the triaxial compression would overestimate the strength, thus overestimate the factor of safety.

The direct shear also yield too high strength due to loss of moisture content.

For unconfined compression test, the strength value would be on the conservative due to sample disturbance of the soft clay, and would, therefore under estimate the factor of safety.

4.7 Disadvantages of Dutch Cone and Field Vane Tests

Bjerrum developed the field vane corrections factor, , for stability analyses of embankment on soft ground. The curve derived from only 16 documents leading to seriously question this entire design approach in view of the scatter, probable variations in the determination of plasticity index and uncertainties in the circular arc stability analyses. Variation field vane test procedures can also be added to this list.

For Dutch cone test is less reliable than field vane test because it lacks in the evaluation experience of the field vane test, especially for Bangkok clay.

4.8 Recommendation in using shear strength value from different tests for Bangkok clay

For insitu test, field vane test provides fairly economical, rapid and reliable means for predicting the insitu undrained shear strength of clay for bearing capacity and stability analyses if the data are obtained using established good practice. This, however, has to be used with correction factor, μ, developed from embankment tests in Bangkok clay. This approach is adequate for preliminary design purpose.

For Dutch cone test it is good practice to use the friction resistance and cone resistance to estimate the pile bearing capacity.

This is because the mode of failure in the pile and the Dutch cone penetration is somewhat similar.

Futhure study has to be made on the possible use of the average strength from triaxial compression and extension tests. This approaches can lead to reliable mean of the strengths for stability analyses, provided good quality samples can be obtained.