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Prediction and Determination of Undrained Shear Strength of Soft Clay at Bukit Raja

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ABSTRAK

Kertas kerja ini membentangkan keputusan kajian untuk meramal dan menentukan kekuatan ricih tak tersalir, s_u, satu parameter yang amat penting dalam amalam rekabentuk, bagi lempung Klang, Malaysia. s_u ditentukan menggunakan ujian ricihan bilah di makmal dan di lapangan, dan kaedah mampatan semula menggunakan radas ricih mudah terus. Ramalan s_u diperolehi menggunakan prosedur SHANSEP dan model keadaan genting. Satu lubang korek ujian di Bukit Raja Klang, digunakan dalam kajian ini. Perbandingan nilai s_u menggunakan kaedah-kaedah ini telah dilakukan dan didapati bahawa semua kaedah menunjukkan arah tuju perubahan s_u dengan kedalaman yang sama. Kaedah bilah ricih memberikan penganggaran s_u yang tertinggi, diikuti dengan kaedah mampatan semula, SHANSEP dan kaedah keadaan genting.

ABSTRACT

This paper presents the results of a study to predict and determine undrained shear strength, s_u, a very important parameter in design practice, for Klang clay, Malaysia. s_u is determined using field and laboratory vane shear and recompression method utilizing the direct simple shear (DSS) apparatus. Prediction of s_u was accomplished using the SHANSEP procedure and the critical state model. A test borehole at Bukit Raja, Klangs, was used for this study. Comparisons of s_u values obtained by these methods are made. It is found that all the methods employed show the same trend of s_u with depth. The vane shear test gives the highest estimation of s_u, followed by the recompression method, SHANSEP method and critical state method.

Keywords: undrained shear strength, OCR, SHANSEP, critical state, direct simple shear, vane test, recompression

INTRODUCTIOIN

In geotechnical design practice, two important considerations which need careful examination are whether construction will cause deformation of the soil and/or instability due to shear failure. Therefore, an engineer has to ensure that the structure is safe against shear failure in the soil that supports it and does not undergo excessive settlement. Knowledge about the stress-strain behaviour, deformation and shear strength of the soil is essential. These considerations are more complicated and challenging when dealing with soft clay that is known to be highly deformable and have low shear strength.

Shear strength is a very important parameter for the design of the foundation of a structure. It can be determined either in the field or in the laboratory, or both. The tests employed in the laboratory may include unconfined compression test, triaxial test, laboratory vane, direct shear box and direct simple shear (DSS) test. *In situ* tests are normally conducted to test the validity of the laboratory tests and also for design purposes. The *in situ* tests available include field vane, standard penetration test, cone penetration test, piezocone and pressuremeter.

This paper forms part of a study to investigate and characterize soft marine clays in Malaysia. A location at Bukit Raja, Klang was chosen for soil sampling and testing. The research was funded by the Intensified Research in Priority Areas (IRPA) program under RM5. It is also supported by Universiti Kebangsaan Malaysia (UKM) and the Geotechnical Research Unit of the Institute Kerja Raya Malaysia, IKRAM.

Methods to predict and determine undrained shear strength, s_u, of soft clay are presented. The techniques are SHANSEP, critical state concept, recompression and *in situ* and laboratory vane shear strength. The first three methods utilize the DSS apparatus. It is noted that all the methods employed show the same trend with slight differences in value. It was also found that the average field and laboratory vane test gives the highest estimation of s_u,followed by recompression method, SHANSEP and critical state predictions, respectively.

MATERIALS AND METHODS

Location of the Study Area

The study was conducted at Bukit Raja, Klang, Selangor in the west coast of Peninsular Malaysia. *Fig. 1* shows the location; of the study area. The profile of the borehole is shown in *Fig. 2* it demonstrated that the depth of the soft clay at Bukit Raja goes down to about 10m before fine sand is encountered.

Geology of the Area

Geologically, coastal deposits in Peninsular Malaysia have been classified as quaternary deposits from the Cenozoic era. The geology of the study area consists mainly of Holocene deposits of the quaternary period termed Gula formation; this consists primarily of clay, silt and sand with minor amounts of

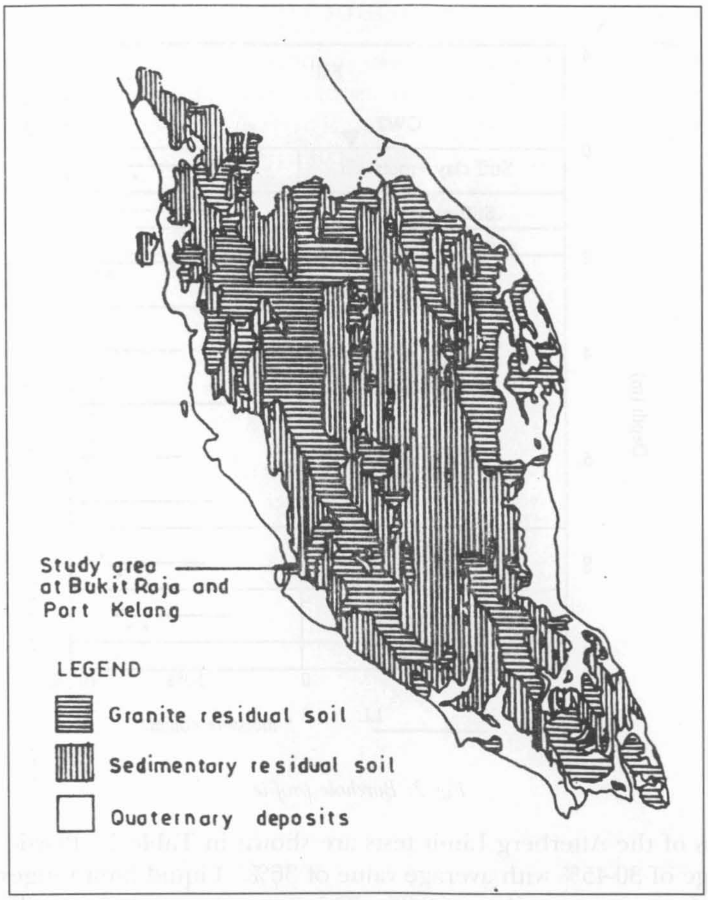

Fig 1: Location of the study area

gravel, shells and corals. Generally the soft deposits are thicker near the coastline and major river mouth, and become thinner away from the coastline. The thickness may range from 5-30 m (Abdullah *et al.* 1987). Shells and decayed wood are often found in the deposits. The predominant clay minerals recorded are koalinite, illite and a mixed layer of montmorillonite while the important foraminifera belong to the rotalia and nonion species (Mahilah Bibi 1971). The tidal range along the Selangor coast is the highest in Peninsular Malaysia. The recorded difference between the spring and the mean high water spring is 4.1 m in Port Klang (Bosch 1988).

Basic Properties

Generally, the colour of the clay in Bukit Raja is dark grey. Organic materials such as decayed wood and roots are found in the clay to a depth of 10 m [see Mohd Raihan Taha *et al.* (1991) for more details].

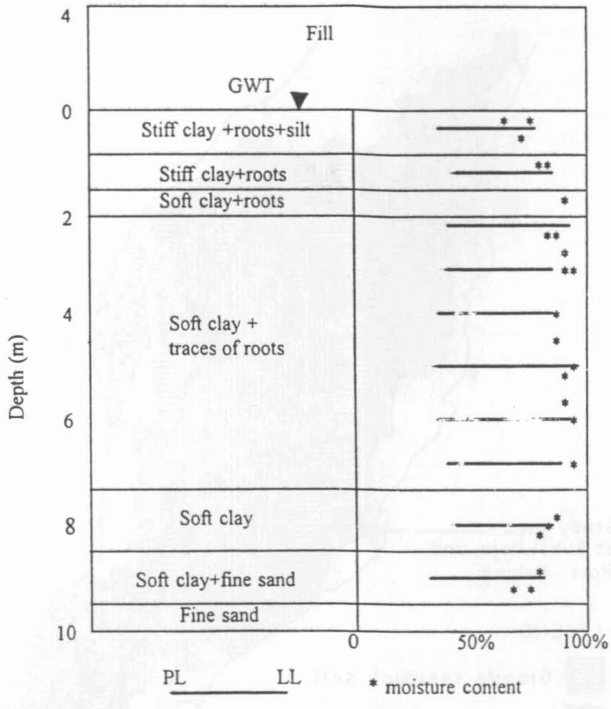

Fig 2: Borehole profile

Results of the Atterberg Limit tests are shown in Table 1. Plastic limit was in the range of 30-45% with average value of 36%. Liquid limit ranges between 75-95% with an average value of 86%. The water content was mostly found to

TABLE 1
Results of some basic soil tests

Depth (m)	Moisture Contents (%)	Liquid Limit (%)	Plastic Limit (%)	Specific Gravity of Solid Solids
4.20	77	75	30	2.51
5.26	78	84	41	2.50
6.20	88	94	36	2.50
7.05	90	84	35	2.57
8.07	84	82	33	2.54
9.07	92	95	35	2.62
10.05	92	93	36	2.62
11.05	97	90	39	2.62
12.07	84	86	45	2.70
13.07	74	80	30	2.55

TABLE 2
Comparison of basic properties of different clays

Clay	w (%)	LL (%)	PL (%)	PI (%)	Unit weight kN/m^3
Klang	66-107	75-95	30-45	41-60	13.8-15.1
Singapore	50-83	50-90	18-22	30-50	12.7-18.6
Bangkok	68	65	24	41	12.7-14.7
Boston	38.42	45-55	23-24	19-31	-
London	32	95	30	65	-
Weald	-	55-85	20-49	365-46	
Norway	27-40	25-36	17-20	18-24	15.7
Leda	28-50	20-45	18-24	5-20	16.7-19.6
James Bay	22-38	26-38	14-18	5-18	17.7-19.6

PL - Plastic limit w = Water Content
LL - Liquid limit PI = Plastic Index

be almost equal to its liquid limit. This leads to a liquidity index of unity and therefore indicates the possibility that the soil is normally consolidated.

The coordinates of the plasticity index and liquid limit are mainly concentrates at the upper end along the A-line of the Cassagrande Chart.

Therefore, the clay may be classified as a highly plastic clay, CH or high plastic organic clay, OH. Using the AASHTO system, Klang clay can be grouped under A-7-5.

The specific gravity values of the soil are not consistent due to the existence of line sand, roots and old wood pieces in some parts. These values are also shown in Table 1. The average unit weight of the soil is about 14.35 kN/m^3. Comparisons of the index properties with other clays such as London clay, Norwegian clay and Boston Blue clay are given in Table 2. It can be observed that Klang clay values are higher than those of the other clays.

DSS Apparatus

Direct simple shear apparatus available in UKM is of type NGI (Norwegian Geotechnical Institute), Model H-12. In this apparatus, the soil sample is sheared by introducing lateral stress to the top part of the sample while the bottom remains unmoved. Constant volume tests are performed using an automatic sample height. The primary feature of this apparatus is the use of a rubber membrane reinforced by wires. This membrane restrains the changes in its perimeter in order to ensure uniform distribution of stress and strain in the sample.

The simple shear test gives a better uniform stress-strain relationship compared to the direct shear test. In addition, the direct shear test creates problems in interpretation of test results due to the rotation of principal stresses. Further details of this apparatus may be found in Geonor (1968) and Azmi (1991).

As various kinds of construction lead to the application of different loading conditions to the soil, several laboratory tests were developed to reproduce or simulate the loading conditions in the laboratory. The strength measured using DSS apparatus represents the average mobilized strength for embankment stability of soft clay (Trak *et al.* 1980), soft ground beneath spread footings (Kinner and Ladd 1973), and shaft resistance along pile foundation (Randolf and Wroth 1981). Based on the s_u of 50 different soft clays, Mayne (1985) observed that the DSS test gave values between that of triaxial compression and triaxial extension tests.

Prediction of s_u using SHANSEP Procedure

The SHANSEP method was first introduced by Ladd and Foott (1974). It is a procedure that can be used for design purposes and for examining stability of soft clay that shows a normalized behaviour. A soil parameter is normalized by reducing it to a dimensionless number. Here, a normalized parameter is obtained by dividing s_u to the *in situ* effective stress σ'_{vc} or to effective confining pressure σ'_{vc} of the test. In this method, stress history, which is represented by the overconsolidation ratio, OCR, is believed to have a great influence on the strength of soil. This technique requires a plot of normalized s_u versus OCR. OCR at any depth can be predicted using a high quality oedometer test. The past maximum consolidation pressure, σ'_{vmax} can be computed from an oedometer test, which gives OCR values at any depth. This can be used to predict s_u for that particular depth using a plot of normalized s_u versus OCR. The SHANSEP method can be obtained using different equipment such as triaxial apparatus, shear box apparatus and DSS apparatus

A study was conducted to obtain a plot of normalized s_u versus OCR for soft marine clay in Klang in order to facilitate design and construction. *Fig. 3* shows the normalized s_u versus OCR plot for soft clay for Klang clay using the DSS apparatus. The values for other commonly reported clays are also plotted in the figure. It can be seen that the Klang clay plot follows the same pattern as the other plots. The equation for this curve can be written as follows.

$$s_u / \sigma'_{vc} \text{ (CK}_0\text{UDSS)} = 0.259\text{xOCR}^{0.78} \quad (1)$$

The plot is used to estimate s_u at any particular depth if OCR from the oedometer test at the same depth is available. For this calculation, the OCR was obtained from the work of Jamilah Jaadil (1991). s_u at different depths is shown in Table 3. A sample calculation is demonstrated below.

Sample Calculation

At the depth of 4.94 m, σ'_{vo} and σ'_{vc} are 22.4 kN/m² and 61 kN/m², respectively. So OCR is 2.7. Therefore,

$$s_u / \sigma'_{vo} = 0.259 \times 2.7^{0.78} = 0.56$$

$$s_u = 0.56 \times 22.4\text{kN} / \text{m}^2 = 12.6 \text{ kN} / \text{m}^2$$

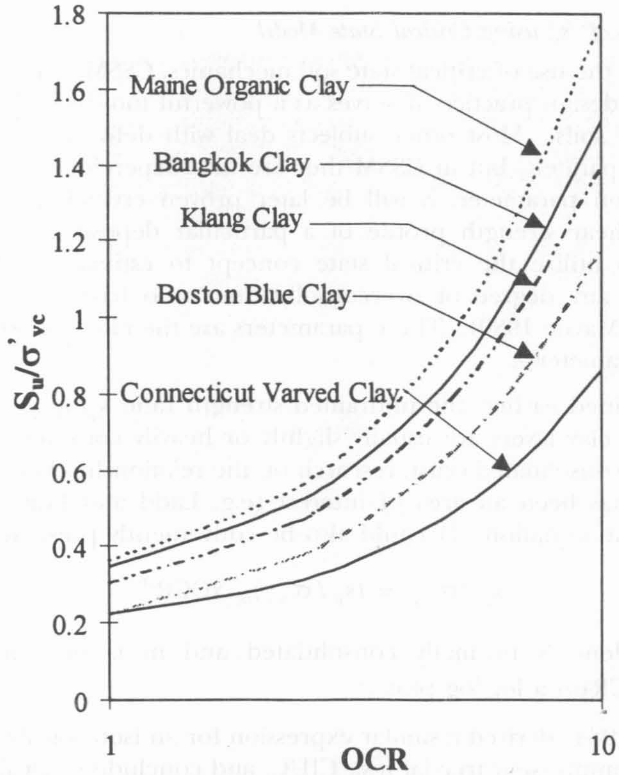

Fig 3: Normalized S_u versus OCR

TABLE 3
s_U from SHANSEP method

Depth (m)	σ'_{vo} (kN/m^2)	σ'_{vmax} (kN/m^2)	OCR	s_U/σ'_{VO}	s_U (kPa)
1.1	5.20	132	25.5	3.19	16.5
2.9	13.3	64	4.8	0.87	11.6
3.9	17.8	41	2.3	0.49	8.8
4.9	22.4	61	2.7	0.56	12.6
5.9	27.0	32	1.2	0.3	8.0
6.9	31.5	27	0.9	0.23	7.2
7.9	36.0	40	1.1	0.28	10.1
8.9	40.6	54	1.3	0.32	13.1

Determination of s_U *using Critical State Model*

Even though the use of critical state soil mechanics, CSSM, is not so popular in engineering design practice, it serves as a powerful tool in understanding the behaviour of soils. Most other subjects deal with deformation and strength properties separately, but in CSSM they are interdependent.
A deformation parameter, λ will be later proven crucial in predicting the undrained shear strength profile of a particular deposit. The aim of this section is to utilize the critical state concept to estimate undrained shear strength for any degree of overconsolidation ratio based only on two soil parameters (Mayne 1980). These parameters are the effective friction angle ϕ' and the parameter λ.

As explained earlier, the undrained strength ratio s_u/σ'_{vo} is a function of OCR. Since clay layers are usually slightly or heavily consolidated or at least have an overconsolidated crust, research on the relationship between these two parameters has been an area of interest (e.g. Ladd and Edgers 1972). The relationship in equation (1) could also be conveniently presented as

$$s_u / \sigma'_{vo} = (s_u / \sigma'_{vo})_{nc} \, x OCR^m$$

where, nc denotes normally consolidated and m is the linear slope of s_u/σ'_{vo} vs OCR on a log-log plot.

Wroth (1984) derived a similar expression for an isotropically consolidated undrained compression triaxial test, CIUC, and concluded that the undrained strength ratio is proportional to the overconsolidation ratio with exponent λ. This parameter was also termed a parameter relating swelling with compression (Schofield and Wroth 1968) and the critical state pore pressure parameter (Mayne 1980). It is usually determined from isotropic consolidation test in the trixial cell and is defined as

$$\Lambda = (\lambda - \kappa) / \lambda = 1 - \kappa/\lambda$$

where

λ = slope of isotropic consolidation line from v vs ln p' plot

and

κ = slope of isotropic swelling line from v vs ln p' plot.

Another method to determine λ and κ is from the oedometer test. It has been concluded that both one-dimensional consolidation and swelling lines have slopes close to λ and κ, respectively (Atkinson and Bransby 1978). Therefore, the above equation can be rewritten as

$$\Lambda = 1 - C_s/C_c \qquad (2)$$

where

C_s = swelling index from oedometer test

C_c = compression index from oedometer test

It should be noted that the current practice is to plot the void ratio against $\log_{10}$ σ'_{vc} to determine C_c and C_s. Therefore a factor of 2.303 should be divided to convert these parameters to λ and κ.

Mayne (1980) compared m from strength tests and Λ determined from one-dimensional consolidation tests and concluded that differences between these two parameters are due to the difficulty in determining C_s. Furthermore, the slope C_s is usually not linear.

Based on the critical state concept, Schofield and Wroth (1968) arrived at a simple expression for the undrained strength ratio for a normally consolidated clay:

$$(s_u/(s'_{vo})_{nc} = 1/2\ M \exp(-\lambda)$$

since

$$M = (6 \sin \phi')/(3-\sin \phi')$$

therefore,

$$(s_u/(\sigma'_{vo})_{nc} = (3 \sin (\phi' \exp(-\lambda)) \ / \ (3 - \sin \phi') \tag{3}$$

The value ϕ' can be obtained from several type of tests such as the triaxial, direct shear box or direct simple shear test. Different tests produce different values of ϕ'. The most suitable value of ϕ' to be used in the CSSM concept is from CIUC test at critical state.

The preceding discussion gives sufficient information to predict the shear strength profile from a series of conventional oedometer tests and the value of ϕ' from any tests without actually studying the relationship between OCR and s_u. This is useful for small projects where extensive testing programmes such as SHANSEP method are not economically feasible. For the purpose of clarification, a sample calculation of s_u for the test borehole is given below. Table 4 gives s_u profile from eight sets of consolidation information.

TABLE 4
s_u from critical state model

Depth (m)	σ'_{vo} (kPa)	σ'_{vmax} (kPa)	OCR	C_C	C_S	λ	$(s_u/\sigma'_{vo})_{nc}$	s_u/σ'_{vo}	s_u (kPa)
1.14	5.2	132	22.5	0.795	0.099	0.88	0.19	3.25	16.8
2.94	13.3	64	4.8	1.041	0.171	0.84	0.20	0.74	9.8
3.93	17.8	41	2.3	0.900	10.177	0.81	0.20	0.40	7.1
4.94	22.4	61	2.7	1.065	0.237	0.78	0.21	0.46	10.3
5.94	27.0	32	1.2	0.864	0.255	0.70	0.23	0.26	6.9
6.94	31.5	27	0.9	0.912	0.267	0.71	0.23	0.20	6.4
7.94	36.0	40	1.1	0.759	0.213	0.72	0.22	0.24	8.7
8.94	40.6	54	1.3	0.828	0.186	0.78	0.21	0.26	10.7

Sample Calculation

Two direct simple shear tests at 5.77 m and 5.82 m depth gave the average value of $\phi' = 23.4^o$. Consider a depth of 4.94 m which has an effective overburden pressure of 22.4 kPa. Oedometer results for this depth are

$\sigma'_{vmax} = 61$ kPa $\quad C_c = 1.065 \quad C_s = 0.237$

The first step is to calculate (using equation 2)

$\lambda = 1- (0.237\ 1.065) = 0.777$

$(s_u/\sigma'_{vo})_{nc} = (3 \sin 23.4^o \exp(-0.777))/(3 - \sin 23.4^o) = 0.21$

The effect of overconsolidation is then taken into account.

$OCR = 61/22.4 = 2.7$

Since $m = \lambda = 0.777$

$s_u/\sigma'_{vo} = (s_u/\sigma'_{vo})_{nc} x\ OCR = 0.21x2.7^{0.777} = 0.46$

Hence,

$s_u = 0.46x22.4 = 10.3$ kPa

A simpler method to estimate s_u is to use the average values for λ and $(s_u/\sigma'_{vo})_{nc}$. By doing this the undrained strength ratio equation applied throughout the borehole depth is

$$s_u / \sigma'_{vo} = 0.21xOCR^{0.78}$$

This equation is close to the one derived from SHANSEP method and in good agreement with the equation recommended by Jamiolkowski *et al.* (1985) for stability evaluation which is

$$s_u / \sigma'_{vo} = (0.23 \pm 0.04)OCR^{0.8}$$

Analysis made by Mayne (1988) also supports this relationship for the direct simple shear test.

Recompression Method

The recompression method (also known as the reconsolidation method) is usually employed to reduce the effect of disturbance caused by sampling. Bjerrum (1973) noted that samples of soft clay are sometimes disturbed by swelling due to the redistribution of water in the sampling tube. In order to restore the original condition, reconsolidation can be done in the laboratory to bring it back to the former stress in the field before shearing is applied. Two practices are usually chosen to select the reconsolidation pressure (Jamiolkowski 1979). One is to reconsolidate the specimen under K_o conditions, at existing in situ effective stresses σ'_{vo} and $\sigma'_{ho} = K_o\sigma'_{vo}$ (Bjerrum 1973) before shearing starts. The other practice is to reconsolidate the specimen under K_o conditions at pressure well beyond existing *in situ* stresses for the case of effective normally consolidated deposits, and well beyond maximum past vertical consolidation stress, σ'_{vmax} when dealing with overconsolidated deposits (Ladd *et al.* 1977). As with SHANSEP, this method can be done using different apparatus where consolidation is possible, such as triaxial apparatus, shear box apparatus and DSS apparatus. Unlike SHANSEP, s_u is measured directly from the test.

In this study, samples taken at three different depths at Bukit Raja site in Klang are reconsolidated under K_o condition, to σ'_{vo} value, before the application of shear stress. However, it is quite difficult to get an accurate (σ'_{vo} due to problems related to friction in the apparatus (Azmi 1991). Table 5 shows s_u values obtained from this method. It can be seen that the s_u value is high at the top of the borehole, decreasing to certain depth and increasing again.

TABLE 5

s_u from recompression method

Sample Number	Depth (m)	σ'_{vo} (kPa)	σ'_{vmax} (kPa)	σ'_{vc} (kPa)	s_U (kPa)
U14A1	5.6	25.4	40.2	29.4	9.9
U16A1	6.4	29.1	33.3	29.1	9.1
U18A1	7.4	33.6	32.3	34.8	11.3

Field and Laboratory Vane Shear

Field vane test (FVT), is a very popular *in situ* test due to its cost and simplicity. It has been widely used in most parts of the world since its introduction in 1950. As noted by Ladd *et al.* (1977), among the deficiencies of the vane test is its interpretation, which is based on the assumption of full mobilization of the strength along a cylindrical failure surface. However, x-ray pictures taken during a laboratory vane test on a cemented sensitive clay showed no evidence of a shear surface at peak strength. Therefore, it was suggested that FVT should be considered as a "strength index" test requiring empirical correlations to give suitable values for design.

In this study s_u is measured *in situ* using a field vane of Geonor type and in the laboratory using a laboratory vane of ELE type. The strain rate used to perform the tests were 12 mm/min and 10 mm/min for field and laboratory, respectively. These values were then corrected according to the procedure suggested by Bjerrum (1973). It is assumed that the strength is uniform with depth and plasticity index is an interpolated value.

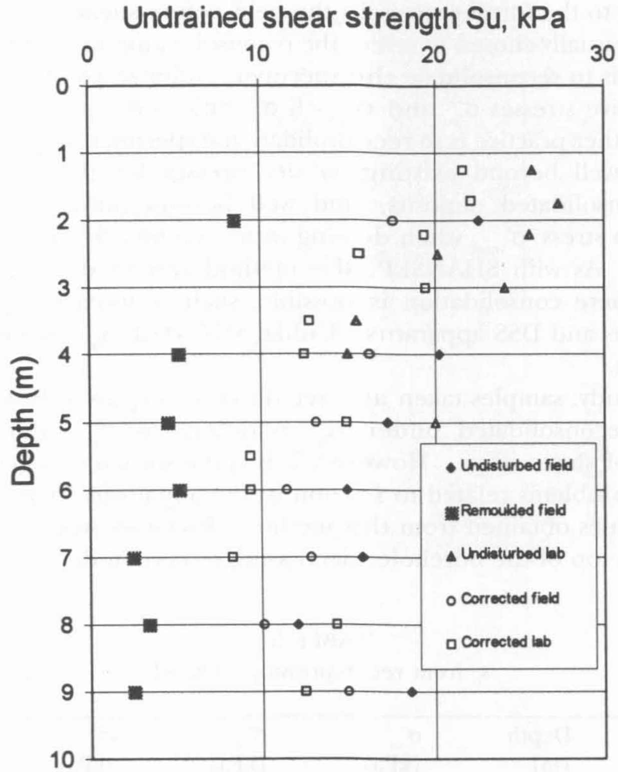

Fig. 4: Actual and corrected values of vane shear test

Fig. 4 gives the actual, corected and remolded s_u from field and laboratory vane with depth. It is noted that the corrected value is about 80% lower than the actual value. The sensitivity was found to be between 2 and 6.

DISCUSSION AND CONCLUSIONS

The SHANSEP method and critical state approach used to estimate the s_u provide comparable results to the s_u from the field, and laboratory vane and the compression method values. In *Fig. 5*, the undrained shear strength from SHANSEP method, critical state concept, recompression method, and field and laboratory vane methods are plotted together. It is noted that both SHANSEP and critical state approach give a close prediction of s_u to one another although the latter seems to show a more conservative prediction. s_u values obtained by the recompression method at three depths are comparable with the s_u values obtained by SHANSEP and critical state methods. For example, at a depth of 6 m, s_u from recompression method, SHANSEP, and critical state is 10, 8.5 and 7 kPa, respectively. The vane test, however, gives the highest s_u. After performing corrections, s_u values seem to come closer to values from the other methods.

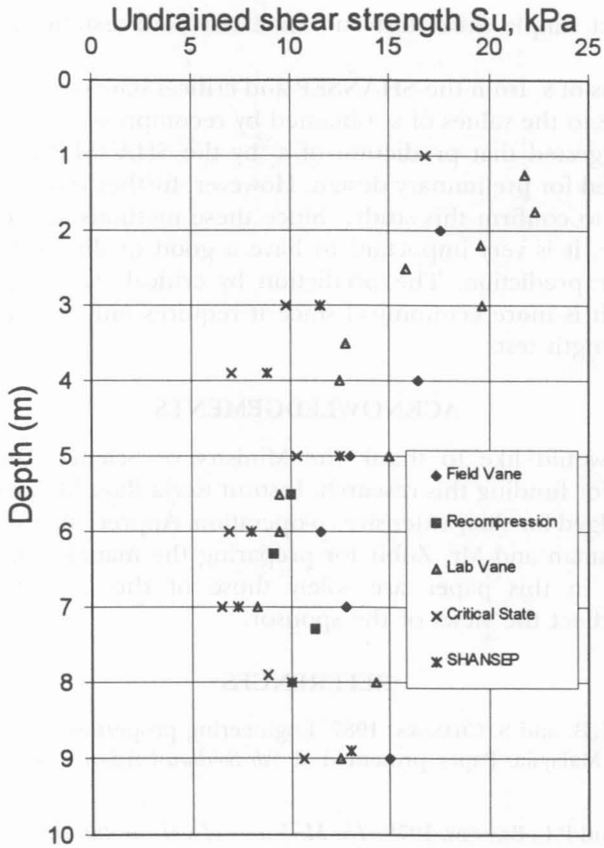

Fig. 5. Comparison of the various methods to determine s_u

Comparing all the s_u from all four methods employed, it is demoustrated that all the methods show the same trend. The s_u values seem to be very high at the top layer and decrease to a depth of about 7.5 m and continue to increase again. For quantitative evaluations of s_u, the vane test was found to give the highest value, followed by recompression method, SHANSEP method and critical state predictions.

The values of s_u determined using all mentioned methods show only slight differences due to several factors. In the recompression method, disturbance might occur during sampling and also, as mentioned before, it is difficult to get σ'_{vc} to be equal to σ'_{vo}. SHANSEP method depends very much on OCR value from the oedometer test and the tests undertaken to get the plot of s_u/σ'_{vc} versus OCR. The critical state method also depends on OCR, C_c and C_s values from oedometer test and ϕ' from direct simple shear tests. The vane test give the highest value of s_u. Therefore, as suggested by Ladd (1977), correction of s_u from vane test according to Bjerrum (1973) is essential in order to prevent an overestimation of the s_u. Another factor that causes the differences is the fact that direct simple shear test and vane test both assume different failure modes.

Predictions of s_u from the SHANSEP and critical state concepts in this study are quite close to the values of s_u obtained by recompression method and vane test. It is suggested that prediction of s_u by the SHANSEP and critical state concept be used for preliminary design. However, further research is needed in this direction to confirm this study. Since these methods are very dependent on OCR value, it is very important to have a good quality oedometer test to ensure a finer prediction. The prediction by critical state is preferred over SHANSEP as it is more economical since it requires only oedometer test and one shear strength test.

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NOTATIONS

s_u	-	undrained shear strength
σ'_{vo}	-	effective overbuden pressure (*in situ*)
σ'_{vc}	-	effective confining pressure (lab)
ϕ'	-	effective friction angle
Λ	-	plastic volumetric strain ratio
λ	-	slope of the isotropic consolidation line in n - ln p' plot
C_s	-	swelling index from oedometer test
C_c	-	compression index from oedometer test
κ	-	slope of the isotropic swelling line in ν - ln p' plot
M	-	slope of the critical state line in q - p' plot
OCR	-	overconsolidation ratio ($s'_{(max)}/s'_{(o)}$)
DSS	-	direct simple shear

SHANSEP	-	stress history and normalized soil engineering parameters
CSSM	-	critical state soil mechanics