CHAPTER 2

LITERATURE REVIEW

2.1 Definition of Undrained Shear Strength

2.1.1 General Failure Criteria

In 1776 COULOMB proposed his failure criterion for the shear strength as

$$\tau_{ff} = C + \sigma_{ff} \tan \phi$$

where:

C is the cohesion

σ_{ff} is the normal stress at the failure surface

ϕ is the angle of internal friction at failure

In 1900 MOHR pastulated that on the failure plane, the shear stress was a function of the normal stress, then

$$\tau_{ff} = f(\sigma_{ff})$$

and as shown in Fig. 2.1

However, for most calculations regarding the stability of a soil mass, we generally uses the failure relationship which is a straight line. Thus, it is expedient to represent the Mohr failure envelope by the Coulomb equation, called the Mohr-Coulomb Envelope as shown in Fig. 2.2.

A failure criterion of greater generallapplication is obtained by expressing the shear strength as a function of the effective stress, $\bar{\sigma}_{ff}$. In accordance with Terzaghi's fundmental concept that the strength and deformation characteristics of soils are governed by the effective stresses rather than the total stresses, then

$$\bar{\sigma}_{ff} = \sigma_{ff} - u_f$$

and

$$\tau_{ff} = \bar{C} + \bar{\sigma}_{ff} \tan \bar{\phi}$$

where:

u_f is the pore water pressure at failure

$\bar{C}$ is the effective cohesion intercept

$\bar{\phi}$ is the effective angle of internal friction at failure

2.1.2 Undrained Shear Strength and the $\phi = 0$ concept, Undrained shear occurs in practical problem whenever external loads changes at a rate much faster than the rate at which the induced pore pressure can dissipate. This condition is thus of great practical importance in the case of clays where the clays remain at constant water content during stress changes. For such conditions it is assumed that the clay will behave as a cohesive material. To obtain the cohesive strength of a clay both in situ and laboratory test can be conducted as long as these tests provide undrained condition, though such tests generally yield different results.

The assumption is generally made that the undrained strength of clay is dependent upon the water content and the preconsolidation pressure. The failure criteria is also generally based on $\phi = 0$ concept.

For rapid construction involving saturated cohesive soils, because of their low coefficient of permeability upon the application of a stress, the water has no time to drain out of the voids of the clay. Thus, the external stress induced a exess pore water pressure. This pore water will partly or fully carrys the applied stress during the initial period of time, dependent upon the loading condition. The inducedpore pressure, Δu, is dependent upon $\Delta\sigma_1$, $\Delta\sigma_2$, $\Delta\sigma_3$ and including shear stress

The shear strength of the soil, expressed as the apparent cohesion, when considering $\phi = 0$, is used in a stability analysis carried out in term of total stress, which know as $\phi = 0$ analysis(SKEMPTION, 1948). This concept is based on the condition that the shear strength depends only on the initial effective stress in the soil, providing that the shearing process is indentical.

2.2 Measurements of Undrained Shear Strengths

Measurements of shear strength under conditions corresponding closely to the actual field condition are necessary for engineers to investigate any stability problems. However, there are many difficulties in doing this. In the laboratory, we can not simulate in the specimens exactly the in situ stress conditions, and samples can not be taken without disturbance. The ideal undistrubed sample, or "Perfect sample"

(LADD & LAMBE, 1963), inevitable has to undergo stress release on removal from the ground. Field testing also has problems with interpretations. Therefore, both in the field and in laboratory, the strength of a soil obtained is limited by the mode of failure using different types of equipment which may not correspond to the mode of actual soil failure in the field.

Table 2.1 lists some of the more common methods for measuring undrained shear strength.

Table 2.1 Common methods for measuring undrained strength (LAMBE & WHITMAN, 1969)

Method	Comments
Insitu measurements	
1. Vane test	Usually considered to give best result,. but is limited as to strength of soil with which it can be used and difficult to interpret
2. Penetration test	Give crude correlation to strength
Measurements on undisturbed samples	
1. Unconfined Compression	Best general purpose test, under estimate strength because disturbance decreases effective stress.
2. UU. test in-situ Confining pressure	Most representation of loboratory tests, because of usually compensating of errors.
3. CU. Triaxial test at in-situ confining pressure	Over estimate strength because disturbance lead to smaller water content upon reconsolidation, especially in very sensitive clay.

2.2.1 The Direct Shear Test

The direct shear test is the simplest of all the methods of determing shear strength and was first used by COULOMB (1776). The essential elements of the direct shear apparatus are shown in Fig. 2.3

The soil is held in a box that is split across it middle. The confining force is applied, and then a shear force is applied so as to cause relative displacement between the two part of box. The magnitude of the shear force is recorded as a function of the shear displacement, and usually the change in thickness of the soil specimen is also recorded. For a given depth of a particular soil, the strength envelope of the cohesive soil is drawn as shown in Fig. 2.4

The direct shear test, although simple and relatively rapid, however, has the following inherent disadvantages.

1. The failure plane is not the weakest plane of the soil sample but it is pre-determined by the nature of the test. This disadvantage show up more with drained rather than with undrained tests.

2. Most of the distortion occurs in a thin zone on unknown thickness. The strain in this zone, which determines the shear resistance, is thus quite different from the displacement between the two parts of the shear box divided by the thickness of the specimen. Therefore, it is very difficult to get other than qualitative stress-strain data from the usual direct shear test.

3. Drainage conditions cannot be easily controlled.

4. The stress conditions across the soil sample in the shear box are very complex because of the change in the shear area in the split shear box with the increses in shear displacement as the test progress, causing unequal distribution of shear stresses and normal stresses over the potential surface of sliding. The total normal load and the total shear force should, therefore, be divided by the area of the sliding surface at failure, and not by the initial area of the specimen. In other words, the corrected area should be used in determining the value of σ_n and τ_f at failure.

5. The water contents of saturated samples of many types of soil change rapidly as a result of the changes in stress.

6. There exists also the question of the effect of laterral restraint by the side wall of the shear box, which is unknown.

BURMISTER (1952) has demonstrated that the stress conditioning and strain restraints imposed by preconsolidation are greater under a hydrostatic state of stress in the triaxial compression test than in the direct shear test under a natural state of consolidation. As a consequent of the anosotropy nature of the soil, the strength obtained by direct shear test is less than that obtained by the triaxial test.

SANGARIYA VANICH. S. (1973) Compared the undrained shear strength profile of Bangkok clay obtained from the direct shear test and the field vane test. The strength from the quick direct shear test is lower in 1.0-3.0 metre which is in the weathered zone than obtained from the vane shear test from the weathered zone. It was proposed that it would be due to the fissure of this zone. Below 3.0 metre, quick direct shear strength was higher than that of vane results which contrast to the result done by HANSEN & GIBSON (1949) who pointed out that the quick shear box tests should given values of the shear strength theoritically lower than the vane test.

The author believes that the shear strength determined from quick direct shear test should be lower than from vane strength due to the disturbance of soil during sampling

2.2.2 The Vane Shear Test

The vane shear test originated with OLSSON (1928) in Sweden and was developed in to its modified form by CADLING and ODENSTAD (1950). The test can be used to obtain a direct and accurate amount in-situ undrained shear strength with economically under relative little soil structure distrubance compared to sampling and testing methods.

To obtain in-situ undrain shear strength, the vane (usually consist of 4 blades vane) covered by the casing, was pushed in to the soil. When the desired depth reached, the vane was pushed out to the soil and the vane is rotated with the torque to bring the soil at the outer edge of the blades to failure at the rate of 0.1 degree/sec (6 degree/min). Fig. 2.5 shows the vane blades, protection shoe, casing and "Geonor" Torque measuring device.

The shear strength of the soil is evaluated from the torque measured. Equation 2.1 gives the general formula usually used to convert to the maximum torque, T, required to rotate the vane to obtain the undrained shear strength, S_{uv}

$$S_{uv} = \frac{2T}{\pi D^3 (\frac{H}{D} + \frac{a}{2})} \qquad \text{..........(2.1)}$$

where D = diameter of vane

H = height of vane

a = factor that depend on shear distribution assumed at top and bottom of vane generated failured cylinder

= 2/3 if uniform (Usual assumption)

= 3/5 if parabolic

= 1/2 if triangular

Although there are some difference in the sizes of vanes used, depending on soil strength, and minor differences in vane shape from the different equipment manufactures, there seem to be world wide agreement to use a 4-bladed vane and a height/diameter (H/D) ratio of 2. The ASTM. VST. Standard D-2573. Includes these requirements. When H/D = 2 and a = 2/3 Eq. (2.1) becomes:-

$$S_{uv} = (6/7)\ \frac{T}{\pi D^3} \qquad \text{..........(2.2)}$$

When using Eq. (2.2) which includes the assumption that S_{uv} has the same value at both the ends and side of the vane generated failure cylinder about 86 % of T, and therefore S_{uv} results from shear on the side (Vertical) of cylindrical face.

Factors affecting vane shear strengths are listed below

a. <u>Rate of Strain</u>

CADLING & ODENSTAD (1950) concluded in the test results that the shear strength decreases as the rate of loading decreases. He found that the rate 6 degree/min seemed to be practical lower limit of rotation rate at which the smallest shear strength is obtained. With the higher rate at 60 degree/min, the strength will be 20 % higher than that of 6 degree/min. SKEMPTON (1948) give the same conclusion as shown in table 2.2 below

<u>Table 2.2</u> Effect of the rate of strain (after SKEMPTON, 1948)

Time of loading to failure	Shear Strength, lb/ft^2	
	Undisturbed	Remolded
1. min (fast)	880	230
10 min (Standard)	830	215
50 min (Slow)	815	200

WIESEL (1973) has shown that the shear strength is a function of the rotation rate as

$$C = K_1 m^{k_2} \qquad \text{.............(2.3)}$$

where

C = shear strength, KN/m^2

K_1 constant, KN/m^2

m = rotation rate, degree/min

K_2 = exponent

The exponent K_2 was determined by the least square method for the investigated depths.

AAs (1965) carry out a series of vane test at different rate and concluded that the faster the rate of loading, the greater the measured shear strength of the soil.

BJERRUM (1972) reported that in analysis the stability of embankment on soft ground using the field vane data, the vane strength seen to give a higher factor safety than that existing in the field. One of the dominent factor influened is the rate to which the clay is brought to failure by the vane test is greater than that of failure mobilized in the field. However he propose a correction factor for the field vane test using the plasticity index of the soil as shown in Fig. 2.6. This chart also takes care for the effects of strength anisotropy and progressive failure.

WIROJANAGUD, P. (1974) studied the rate of effect on vane shear strength of the Bangkok clay in the weathered zone, with the 1-16 degree/min of rotational rate there were no practical significant different in the shear strength of the soil.

b. Vane dimension.

AAș (1965) has found that large change in height diameter ratio can influence thecalculated strengths, due to variations in the mobilized resistance along vertical and horizontal samples. This can result from true undrained shear strength anisotropy and/or conditions peculiar to the mode of failure imposed by the field vane test.

PHUONG (1973) used several vane size to investigate shear strength of Bangkok clay. He found that the values measured are random and the difference is in tbe range of 1 ton/m^2.

WIESEL (1973) found that the shear strength and angle of rotation at failure both decreased with decreasing the height-diameter ratio of the vane.

LAROCHELLE & et.al (1973) using the same size and volume of vane but varying the thickness of the edge of the blade had a pronouned effect on the shear strength as shown in Fig. 2.7. In very sensitive clay, increasing in the blade thickness leads to more sample disturbance and hence decreases the vane strength.

c. Soil disturbance

This effect is brought about from the distrubance of soil around the vane. The soil disturbance may be caused by boring or pushing the vane casing or inserting the vane in to the soil.

CADLING and ODENSTAD (1950) found that the influence zone in boring was within the five time the diameter of the bore hole.

For vane tests without borehole, but using a protection shoes, CADLING and ODENSTAD (1950) and ANDERSON & BJERRUM (1957) found that no resonable distrubance when the vane test is made 50 cm away from the shoe.

For the disturbance of only vane rod and vane itself, VEY (1953) showed that without predrilling or protection shoe the clay of one layer may strick to the vane and this increase the area ratio of the vane. Thus the shear strengths measured by the high vane were less than that measured by the lower one.

d. Time effect

When the elapse time between pushing the vane into the soil and start the test increased, the shear strength would also increased, FLAATE (1966). This was due to the consolidation effect of the clay being push in. AAS (1965) compared the result from a series of consolidated undrained (i.e. allowing soil for consolidation) and vane test, he concluded if the vane was left one day after insertion, a substantial increase of shear strength occured. The average ratio between the consolidated undrained shear strength and the vane test varies between 1.28 and 1.52 for the time delay of 24 hours. SCHMERTMAN(1975) in his personal communication to AAS (1975) reported that Nowegian clay the shear strength can vary by + and - 50 % when comparing the ordinary few minutes with 24 hours of pore pressure dissipates allowed after vane insertion.

Interpretation

The vane shear strength is generally considered or strength index tests. However, with the Bjerrum's correction factor μ, this index strength can be interpretted as the average strength of the circular arch failure, used in the analysis of circular arch failure.

2.2.3 The Dutch Cone test

Cone penetration test is a kind of in-situ measurement. The idea arises from a concept that the resistance to penetration of different type of soil will be of different manner. The method is to advance a rod in to the soil vertically, measuring the resistance forces required to produce the cone advancement and interpreting these forces in terms of pile capacity, soil density shear strength, etc. There are many way to classify the variety of the equipment and methods of advancing the rod. Table 2.3 offers a broad classification of cone penetration test types

TABLE 2.3- GENERAL TYPES OF CONE PENETRATION TESTS

Type	Tip Advance		Where Used	Notes
	Method	Rate		
1. Static	During increments of constant load	0	Research	Too slow for general field use
2. Quasi-static	Hydraulic or mechanical jacking	1-2 cm/sec	worldwide	Usually 10 cm^2 60^o cone point
3. Dynamic	Impact of drive weight	variable	worldwide	Great variety sizes, weights, etc.
4. Quasi-static & dynamic	Combines 2. and 3., using dynamic when Q-CPT cannot penetrate further		France Switzerland	Uses special penetrometer tips
5. Screw	Rotation of a weighted, helical cone	variable	Sweden Norway	
6. Inertial	Dropped or propelled into soil/rock surface	variable during measured deacceleration	Offshore, Military	Useful for near-surface soils in inaccessible area

The quasi-static cone penetration test equipments and methods developed in the Netherlands are mostly widely used, these "Dutch" cones having a base area of 10 cm^2, an apex angle of 60° and employing a penetration rate of 1 to 2 cm/sec. The equipment can make separate measurements of the cone resistance and the soil steel resistance along the local friction sleeve. The schematic detial of Dutch cone tip is shown in Fig. 2.8.

Factors Affecting Dutch Cone Test Results

It is obvious that results from the Dutch cone tests are affected by some factors. The important factor, of course, is the physical characteristics of the soils, i.e. the cone resistance is different for different types of soils. Other factors besides the soil properties have been studied by many investigators as their conclusions are presented here in.

a. Physical characteristics of soil - shear strength, sensitivity and compressibility of soil have an influence on the cone resistance. These factors can not be easily shown individually, but their combined effect may be considered by comparing cone resistance with the undrained shear strength of various clays.

THOMAS (1965) has presented the effect of shear strength on the cone resistance for London Clay that for strength greater than 2,000 lb/sq.ft there is a wide scatter of results and average curve tend to rise from the estimated linear curve using N_c equals 18 as shown in Fig. 2.9. For strength lower than 2,000 lb/sq.ft., the linear relationship of results is more pronouned.

b. Shape of cone - After a series of tests in which cones of different shapes and dimensions were used, THOMAS (1965) concluded that the cone penetration resistance is influenced not by the cross-sectional area of the cone but by its shape above the cone and the mode of failure of the soil through which it is penetrating.

The results is well illustrated in Fig. 2.10 for 10 sq.cm cone, the mode of failure is of type (b) both in soft and firm clay. In firm clay, the mode of failure of 20 sq.cm cone is of type (a) because of its immediate reduction in diameter above the cone.

c. Rate of penetration. THOMAS (1965) showed that lower resistances were recorded at the lower rate of penetration. This is in agreement with the result given by LADANYI (1969) who suggested that the effect of the rate of penetration should be taken in to account when comparing results of deep penetration test with results of other types of field tests. He has found that the increase in point resistance is 7.5 % for ten fold increase in penetration rate.

It is clear so far that the penetration resistance of a cone is influenced by a number of factors. In an attempt to correlate cone resistance to other physical properties of soil it is therefore necessary to control the effect of these factors so as to obtain reliable results. To achieve this purpose, type and shape of the penetrometer should be kept unchanged whenever comparisons are made. The rate of penetration must be also controlled and standardized. When using the same equipment and with standard penetration rate, correlations between the cone resistance and other properties of the soil can be made without any effects of these two factors. The cone resistance will be affected only by the properties of the soil

Method for estimating shear strength

Cone penetration test does not measured shear strength directly but measure bearing capacity, q_c, and soil-steel friction along the local friction sleeve, f_s, both of which depend on soil shear strength.

The development of the bearing capacity theories began in 1921 with the work of PRANDTL, who considered the resistance of a metal tool bearing against the surface of a metal half-space. PRANDTL assumed a perfectly smooth, uniformly loaded, infinitely long bearing area underlain by a weightless, perfectly rigid-plastic material which had both cohesion and internal friction. In 1924 REISSNER extended the PRANDTL theory to include cases involving bearing area which were below the level of the adjacent material. To do this, REISSNER replaced the material located above the level of the bearing area with a uniform surcharge and assumed that the surcharge had no shear strength. TERZAGHI (1943) extended the PRANDTL-REISSNER solution so that it could be applied to bearing capacity problem to foundation engineering. He considered bearing materials which passes weight, cohesion, and friction and proposed the following general bearing capacity equation for shallow, uniformly loaded strip footing.

$$q_c = CN_c + \gamma DN_q + 1/2\ \gamma BN_\gamma \qquad \text{.........(2.4)}$$

where N_c, N_q, N_γ are bearing capacity factors which depend on internal friction angle (ϕ) of the soil

γ = the unit weight of the soil

D = the depth of the footing

B = the width of footing

C = the cohesion of the soil

For the case of cone, the width B is relatively small the equation reduce to

$$q_c = CN_c + \sigma_o N_q \qquad \text{..........(2.5)}$$

where σ_o is the total overburden pressure above the cone tip

(a) <u>Cohesionless soils</u>

DE BEER (1948) developed a conservative method for determining ϕ' in sand. The method originates from bearing capacity theory derive from a surface strip load of infinitely length as

$$q_c = \{|e^{\pi\tan\phi'}\tan^2(45^o + \phi'/2) - 1|\frac{\tan\phi}{\tan\phi'} + 1\}\ \sigma_o \quad \text{..(2.6)}$$

where ϕ and ϕ' are effective and De Beer's apparent internal friction angles, q_c is soil resistance. For $\phi = \phi'$ and c = 0 then the equation (2.6) becomes

$$q_c = |e^{2\pi\tan\phi}\tan^2(45^o + \phi/2|\sigma_o \qquad \text{.........(2.7)}$$

The method often produces results too conservative for economical design. A less conservative procedure, with a semi-empirical basis, appears to be in use in the USSR (see Fig. 2.11)

The DEGEBO research in W. Berlin with large-scale model footings on sands, as reported by MUHS and WEISS (1971) noted that

$$q_c\left(\frac{Kgf}{cm^2}\right) = 0.80\ N_\gamma \qquad ....(2.8)$$

N_γ is the ordinary bearing capacity factor from the Terzaghi general shear case

Then ϕ' can be estimated

(b) Cohesive Soil

Estimate of undrained shear strength in clay soil from results usually employ an equation of the following form.

$$S_u = \frac{q_c - \sigma_{vo}}{N_c} \qquad ....(2.9)$$

where σ_{vo} is the in-situ total overburden pressure at depths of q_c

N_c is the cone factor

Many engineers had derived for the N_c values as shown in table 2.4

SCHMERTMAN (1975) reported that the N_c values varies with at least the factors indicated in table 2.5

This is why the N_c values reported from various empirical correlation studies vary from about 5 to 70 (for example in Amar et.al. (1975). Thus, to use a single N_c value for all soil and all penetrometer tips represents a gross oversimplification which can lead to serious error.

Author	Correlation	Type of Soil	L.L (%)	P.I. (%)	ω (%)	c (Tons/m^2)
Begemann (1965)*	$c = \frac{q_c}{14}$	Cleyey Soils	-	-	-	-
Thomas (1965)	$c = \frac{q_c}{18}^{+}$	London Clay	-	-	22	5-30 13
Ward & Marsland (1965)	$c = \frac{q_c}{15.6}^{+}$	London Clay	60-71 67	36-43 38	22-26 24	20-55 35
Meigh & Corbett (1969)	$c = \frac{q_c}{16}^{*}$	Arabian Golf Soft Clay	47-61 53	29	30-46 38	0.8-4.0 2
Anagnoslopoulos (1973)	$c = \frac{q_c}{17}^{+}$	Clay of North West Peloponnesus, Greece	35	18	30	1.5-3 2
Sanglerat (1972)	$c = \frac{q_c}{15}$	Annecy Soft Clays	-	-	-	-
Pham (1972)	$c = \frac{q_c}{14}^{*}$	Bangkok Clay	80-96 88	30-40 35	75-90 80	1-3 1.5
Prakob (1974)	$c = \frac{q_c}{19}^{*}$	Soft Nong Ngoo Hao Clay	80-120 100	40-75 60	60-130 95	1.2-4 2
	$c = \frac{q_c}{14.5}^{*}$	Weathered Zone of Soft Nong Ngoo Hao Clay	100-130 115	60-80 70	100-135 130	1.2-2.3 1.8

* undrained shear strengths were obtained from field vane shear tests.

\+ undrained shear strengths were obtained from undrained triaxial tests.

Talbe 2.4 Collection of Cone Factors for Different Clays with Soil Properties.

TABLE 2.5- SOME OF THE VARIABLES THAT INFLUENCE N_c

Variable	Approx. N_c factor potential	Direction	Notes
1. Changing the test method for obtaining reference s_u	2 to 3	Better sampling, thinner vanes, use of s_{uPMT} all decrease N_c	see Eqn 4.
2. Clay stiffness ratio = G/s_u	3	Increases with increasing stiffness	Vesic (1972)
3. Ratio increasing/ decreasing modulus (E^+/E^-) at peak s_u	3	Decreases with decreasing ratio	Ladanyi (1967)
4. effective friction, tan ϕ'	2 to 3	Increases with increasing ϕ'	Janbu (1974)
5. K_o', or OCR	3	Increases with increasing K_o' or OCR	Janbu (1974)
6. Shape of penetrometer tip	2	Clay adhesion on mantle of mechanical tips increases N_c	example in Amar et.al. (1975, Fig. 2)
	1.5	Reduced diameter above cone can decrease N_c in very sensitive clays	Schmertmann (1972b)
7. Rate of penetration	1.2	Increasing rate increases N_c	viscous, no pore pressure effects
8. Method of penetration	1.2	Continuous (electrical tips) penetration decreases N_c compared to incremental (mechanical tips) because of higher pore pressures.	

Baligh (1975) has derived the cone resistance, q_c, of the moving cone using elasto plastic theory in cohesive soil by assuming the rigidity index of clay equal to 60 and the cone tip angle 60 degree. The expression from his analysis is

$$S_u = \frac{q_c - \sigma_h}{16}$$

where σ_h = the horizontal total stress at that depth.

<u>Determination of soil type</u>

Begeman (1965, 1969) has shown that there exists a relationship between unit frictional resistance, f_s, to unit cone resistance, q_c, and the soil type as shown in Fig. 2.12. Schmertman (1967) proposed the following ratios:-

Soil Type	f_s/q_c (%)
Soft rock or shells	0.0-0.5
Sand	0.5-2.0
Silt	2.0-5.0
Clay	>5.0