COMPRESSIBILITY AND SHEAR STRENGTH OF REMOULDED DHAKA CLAY

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A Thesis
by
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ABSTRACT

This study is aimed at evaluation of the intrinsic properties of Dhaka clay, their interactions and to develop a model for predicting undrained shear strength. In this study, the undrained shear strength (S_u) behaviour was investigated for reconstituted Dhaka clay subjected to K_o, isotropic consolidation followed by a number of loading and unloading stress history. The intrinsic parameters of the clay thus obtained were used to predict the strength of natural Dhaka clay for various stress history and stress conditions.

For the laboratory investigation, samples of Dhaka clay were mixed with water (1.5 times the liquid limit) to form a slurry. From slurry, normally consolidated and overconsolidated samples of various overconsolidation ratios (OCR) were prepared under different stress conditions. The slurry was consolidated in a Rowe Cell. A 210 mm diameter Rowe consolidation Cell and test set up was developed for this purpose. Four categories of samples were prepared. For each category, cylindrical samples were prepared by trimming into size of 38.1 mm diameter and 76.2 mm height from the soil cake extruded from the Rowe Cell after consolidation of slurry. The first category of samples (K_oNC) were K_o consolidated and sheared in triaxial compression in undrained state. The second category of samples (K_oOC) were K_o consolidated and unloaded to a specified OCR value and then sheared in undrained triaxial compression. The third category of samples (K_oINC) were K_o consolidated, extruded from Rowe cell and reconsolidated isotropically to a higher pressure and then tested to undrained compression failure. The fourth category of samples (K_oIOC) were subjected to same conditions as that for third category but was unloaded to a preferred OCR before undrained compression failure in triaxial apparatus. Stress-strain behaviour for each category of samples were recorded.

From the test results, normalizing parameters for strength and modulus of deformation were established for Dhaka clay in normally loaded and overconsolidated states. The undrained shear strength ratio, $S_u^{(nc)}/\bar{\sigma}_v$ for normally loaded Dhaka clay was found as 0.186 and 0.295 for K_o and isotropic stress conditions respectively. The equation for the strength ratio, $S_u^{(nc)}/\bar{\sigma}_v$ derived from critical-state Cam-clay theory was found applicable and hence recommended for reconstituted Dhaka clay. Mayne's (1980) K_o model and Mitachi and Kitago's (1976) isotropic model were found to be suitable for predicting intrinsic undrained shear strength (S_u^*) for overconsolidated Dhaka clay. The following equations were derived for predicting intrinsic undrained shear strength for Dhaka clay,

$$S_u^{(oc)} = 0.197(OCR)^{0.821} \times \bar{\sigma}_{vo}$$

for K_o stress condition

and $$S_u^{(oc)} = 0.295 \times (\bar{p})^{0.734} \times (\bar{p}_o)^{0.266}$$

for isotropic stress condition

where, $\bar{\sigma}_{vo}$ and $\bar{p}_o$ are overburden pressures and $\bar{p}$ is the maximum past consolidation pressure.

Other models for predicting shear strength for Dhaka clay are also obtained. This includes relations derived from α i.e $[S_u^{(oc)}/\bar{\sigma}_{vo}]/[S_u^{(nc)}/\bar{\sigma}_v]$ versus OCR and μ $[=S_u^{(oc-\bar{\sigma}_{vo}\ or\ \bar{p}_o)}/S_u^{(nc-\bar{\sigma}_v\ or\ \bar{p})}]$ versus OCR. From the available results, Intrinsic compression Line (ICL) is drawn of Dhaka clay which coincided with the ICL for most clays furnished by Burland (1990). Also intrinsic swelling line (ISL) is drawn for Dhaka clay. These are compared with a few available test results on natural Dhaka clay. All the foregoing demonstrate that strength and compressibility of Dhaka clay is predictable from simple measurements of in-situ water content or void ratio, field overburden pressure and OCR.

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NOTATIONS

Note: Suffix f indicates a failure condition

prefix Δ indicate a change

A bar over a stress indicates an effective stress

A bar over a shear test symbol indicates pore pressure measurement

A	- Skempton's pore pressure parameter = $(\Delta u - \Delta\sigma_3)/(\Delta\sigma_1 - \Delta\sigma_3)$
B	- Skempton's pore pressure parameter = $\Delta u/\Delta\sigma_3$
C_e	- True Cohesion
C_c	- Compression index from consolidation test
$C_c(K_o)$	- Compression index from K_o consolidation test
$C_c(iso)$	- Compression index from isotropic consolidation test
C_s	- Swelling index from consolidation test
$C_s(K_o)$	- Swelling index from K_o consolidation test
$C_s(iso)$	- Swelling index from isotropic consolidation test
e	- Void ratio
e_o	- Initial void ratio
e_L	- Void ratio at liquid limit
E_s	- Young's modulus
E_u	- Undrained Young's modulus
E_i	- Initial tangent modulus
$E_u(50)$	- Undrained secant modulus at 50% of ultimate shear strength

f	- initial shear stress ratio
I_v	- Void index
K_o	- Co-efficient of earth pressure at rest
K	- Ratio of horizontal stress to vertical stress
K_o line	- Line through P and q for soil subjected to one-dimensional consolidation
K_f	- K at failure
K_f line	- Line through p_f versus q_f
I_L	- Liquidity index$=(w_N-w_p)/(w_L-w_p)=(w_N-w_p)/PI$
N.C. or n.c.	- Normally consolidated
O.C. or o.c.	- Overconsolidated
OCR	- Overconsolidation ratio
p	- $(\sigma_1+\sigma_3)/2$ or $(\sigma_v+\sigma_h)/2$
p_f	- p at failure = $(\sigma_1+\sigma_3)_f/2$
p'	- Mean principal effective stress $=(\bar{\sigma}_1+\bar{\sigma}_2+\bar{\sigma}_3)/3$
p'_e	- Equivalent mean effective pressure (Hvorslev)
PI	- Plasticity index = w_L-w_p
$\bar{p}_o$	- In-situ overburden pressure or reduced all round swelling pressure under istropic stress system for overconsolidated soil.
p'_c	- Predicted value of preconsolidation pressure
q	- $(\sigma_1-\sigma_3)/2$ or $(\sigma_v-\sigma_h)/2$
q_f	- q at failure
S	- undrained shear strength ratio for normally consolidated clay ($S_u/\bar{\sigma}_v$ or $S_u/\bar{p}$)

S_d	- Drained shear strength
S_u	- Undrained shear strength
S_u'	- Predicted value of undrained shear strength
$S_{u(nc)}$	- S_u in normally loaded state
$S_{u(oc)}$	- S_u in overconsolidated state
$S_{u(nc-\bar{\sigma}_{vo})}$	- $S_{u(nc)}$ with overburden pressure $\bar{\sigma}_{vo}$
$S_{u(oc-\bar{\sigma}_{vo})}$	- $S_{u(oc)}$ with overburden pressure $\bar{\sigma}_{vo}$
$S_{u(nc-\bar{p}_o)}$	- $S_{u(nc)}$ with overburden pressure $\bar{p}_o$
$S_{u(oc-\bar{p}_o)}$	- $S_{u(oc)}$ with overburden pressure $\bar{p}_o$
W	- Water content
W_N	- Natural water content
W_f	- Water content at failure
W_L	- Liquid limit
W_p	- Plastic limit
α	- Ratio of $[S_{u(oc)}/\bar{\sigma}_{vo}]/[S_{u(nc)}/\bar{\sigma}_v]$
β	- Slope of the line of $\log[S_{u(oc)}/\bar{\sigma}_{vo}]$ vs. log OCR Plot found from Mitachi and Kitago's experimental results.
β'	- Slope of the plot of log C_e vs. of W_f
ϵ	- Axial strain
λ	- Ratio of swelling index to compression index=C_s/C_c
$\Lambda_{o(ko)}$	- Critical-state pore pressure parameter for K_o stress condition.
$\Lambda_{o(iso)}$	- Critical state pore pressure parameter for isotropic stress condition
μ	- The ratio of $S_{u(oc-\bar{\sigma}_{vo}\ or\ \bar{p}_o)}/S_{u(nc-\bar{\sigma}_v\ or\ \bar{p})}$
σ	- Normal stress

σ_1	- Major principal stress
σ_3	- Minor principal stress
$\bar{\sigma}_1$	- Major principal effective stress
$\bar{\sigma}_3$	- Minor principal effective stress
$\bar{\sigma}_{1f}$	- Major principal effective stress at failure
$\bar{\sigma}_{3f}$	- Minor principal effective stress at failure
$\bar{\sigma}_h$	- Horizontal effective stress
$\bar{\sigma}_v$	- Vertical effective stress
$(\sigma_1-\sigma_3)$	- Deviatoric stress
$(\sigma_1-\sigma_3)_f$, $\Delta\sigma_f$	- Deviatoric stress at failure
σ_e'	- Equivalent vertical effective pressure (Hvorslev)
$\bar{\sigma}_{vc}$	- Vertical effective consolidation pressure for normally loaded soil
$\bar{\sigma}_{vm}$, $\bar{\sigma}_{vmax}$	- Maximum past vertical effective consolidation pressure for overconsolidated soil
$\bar{\sigma}_{vo}$	- In-situ overburden pressure or reduced vertical effective pressure for overconsolidated soil.
$\bar{\sigma}_{(IKo)}$	- Initial K_o consolidation pressure prior to isotropic consolidation
$\bar{\sigma}_c$, $\bar{P}$	- All round consolidation pressure in isotropic stress system for normally loaded soil
$\bar{\sigma}_{cm}$	- Maximum past all round consolidation pressure in isotropic stress sytem for overconsolidated soil.
$\bar{\sigma}_p$, $\bar{P}_c$	- Preconsolidation pressure
τ	- Shear stress
τ_{ff}	- Shear stress on failure plane at failure
$\bar{\phi}$	- Effective angle of internal friction

$\bar{\phi}_o$ - True angle of internal friction

Ψ - Normalized undrained secant Modulus

$= E_u(50)/S_u$

Shear Test

CD - Consolidated-drained test

CU - Consolidated-undrained test

UU - Unconsolidated-undrained test

CK_oU - CU with K_o consolidation

CIU,CID - CU, CD with isotropic consolidation

CIUC, CK_oUC - CU triaxial compression test

CIUE, CK_oUE - CU triaxial extension test

DSS - Direct simple shear test

TC - Triaxial compression test

TE - Triaxial extension test

CK_oUTC - CK_oU triaxial compression test

CK_oUTE - CK_oU triaxial extension test

CK_oUDSS - CK_oU direct simple shear test

Samples

K_oNC - K_o consolidated sample in normally loaded state

K_oOC - K_o consolidated sample in overconsolidated state

K_oINC - Isotropically consolidated sample in normally loaded state

K_oIOC - Isotropically consolidated sample in overconsolidated state

বাংলাদেশ প্রকৌশল বিশ্ববিদ্যালয়
নং.....
তাং.....
লাইব্রেরী, ঢাকা

CHAPTER 1
INTRODUCTION

1.1 General

The foundation engineers are required to have a knowledge of two most important properties of a soil namely compressibility and shear strength in order to design, analyse and construct various geotechnical works. The compressibility of a soil determines the susceptibility of a soil to volume change under load. On the other hand, the shear strength of a soil mass governs the criterion of load carrying capacity. Soil strength is the resistance developed from a combination of particle rolling, sliding and crushing and is reduced by any excess pore pressure which develops during particle movement. This resistance to deformation reveals the shear strength of the soil as opposed to the compressive or tensile strength of other materials. By definition, the shear strength of soil is generally regarded as the resistance to deformation by continuous shear displacement of soil particle along surfaces of rupture. Shear strength of the soil is not regarded solely in terms of its ability to resist peak stresses, but it must be viewed in the context of deformation which may govern its performance. In that light, shear failure is necessarily viewed as the state of deformation when the functional performance of the soil mass is impaired.

It is very much essential to understand the compression susceptibility and stress-strength behaviour of the soil of the fast growing capital city, Dhaka. It is required to evaluate shear strength and compressibility parameter of the clay and to investigate whether the clay shows normalized behaviour or not. A strength model is of paramount importance with a view to understand, interpret and predict the strength of the clay. A study is required to know whether the available strength models

can be applicable to Dhaka clay. The evaluated model for the clay should be such that it's in-situ soil strength can be predicted simply by using some easily available in-situ parameters of the clay such as water content, void ratio, existing overburden pressure etc. The model will help to interpret, countercheck, curtail volume of lengthy investigation program or even eliminate the sub-soil investigation works with undisturbed sampling since it is not always possible and also not desirable to accomplish the detail soil investigation for all individual structure as the concern is laborious, time-consuming, costly and sometimes render difficulties for interpretation.

1.2 Geological Aspects of Dhaka Clay

The city of Dhaka situated on the southern part of Madhupurgar which is formed by older Pleistocene terrace sediments. Dhaka soil belongs mainly to the catagory of Pleistocene terraces and some smaller part to Recent Alluvium. Pleistocene and Recent samples do not differ in maximum grain size, but an excess of very fine-grained material is found in the Pleistocene samples (Morgan et.al., 1959). Pleistocene sediments are well oxidized and typically are reddish, brown or tan and are mottled.

Mostly of Dhaka soil belongs to Dihing formation of Pleistocene age. This formation having thickness ranging between 8 to 30 metres consists of clay, fine sand and pebbles which are mostly mottled and 'Red clay' (Interim Report, WASA, 1990).

It is also covered by highland and lowland alluvium in some places. Dupi Tila formation underlies the Modhupur clay Residuum which is locally called Dhaka clay. Dupi Tila formation consists of clay, fine sands, medium sands, clayee lenses,sub-ordinate shale and poorly consolidated sand stone. It is massive thick bedded, yellow to yellow brownish in colour.

A portion of east and west side of Dhaka consists of recent alluvium. The alluvium deposit consisting of stream deposits, natural levee and backslope deposits, swamp deposits and interstream deposits. Though some part of Dhaka is covered by recent alluvium, most of Dhaka soil is of older Pleistocene deposits.

Most of the natural Dhaka soil deposits are at slightly to heavily overconsolidated state. The city of Dhaka is at the elevation of 8 to 10 metres from mean sea level. The top layer of Dhaka soil generally consists of a mixture of clay and silt. The depth of this top layer varies from 8 metres to 30 metres. It is this clay layer that is under present research. Layer of courser materials such as sand and gravel exists at the lower levels and finer particles dominates at the top surface.

1.3 Research Area

No general constitutive relationship and model suitable for use is yet available for determination of shear strength characteristics of in-situ Dhaka clay in order to use in the design and analysis of geotechnical problems. Around the world, works have been done on this aspect on different types of clays specifically on the overconsolidated clays. Different types of relationships and models formed the basis of analysis and design of geotechnical works on those clays. But regarding Dhaka clay, little is known. Consequently, due to non-availability of the appropriate characterization of Dhaka clay for different loading principles and stress history, the prediction of shear strength of the overconsolidated clay may be grossly underestimated and the resulting design may be unnecessarily conservative and expensive.

Finding the above deficiencies, Dhaka clay has been undertaken for research investigation with a view to determining its undrained shear strength behaviour at different loading principles such as K_o and isotropic consolidation and at different stress history i.e at normally loaded and preloaded states. As shear strength behaviour is different for different stress history and loading conditions, the task is to study the behaviour at various states and conditions separately. With reconstituted soil, it is only possible to reproduce such states and conditions. The characterization of the soil at particular state and condition is not possible by using natural soil, because natural soil, generally is non-uniform and sometimes appear in complex form which produces anomaly and ambiguity in results. Moreover, the stress-path dependency of the preloaded behaviour of natural non-uniform soil becomes uncertain in characterization. So reconstituted soil has been used in the research work instead of natural soil.

The area of the research includes the determination of the various essential intrinsic properties of Dhaka clay and their various interactions at different stress history and stress conditions. These intrinsic properties found from reconstituted Dhaka clay can form the fundamental aspects of reference for interpreting the corresponding characteristics of natural Dhaka clay. The research program incorporates the evaluation of a strength model for the clay using its intrinsic properties.

CHAPTER 2
THEORETICAL BACKGROUND AND REVIEW OF LITERATURE

2.1 Introduction

One of the most important aspect of the study of the soil mechanics is the prediction of failure stress, that is, stress under which excessive deformation is produced. Any load will produce stresses and result in strain and deformation or settlement within the stressed zone. The stress and strain developed within a soil mass due to applied load such as fillings, building foundation or unloading such as excavation etc. is of great interest for engineers. Since soil is a particulate material, failure is primarily by a rolling and slippling of grains and not by simple tension or compression. Because of this failure mode, the stresses of interest are shear stresses and the soil resistance or soil strength of interest is the shear strength. For rapid construction involving saturated cohesive soils, the appropriate strength to use in a total stress stability analysis is the in-situ undrained shear strength, S_u. In other case, the undrained shear strength is that corresponding to soil which has been consolidated (or partially consolidated) under the stresses existing just prior to superimposing the additional load.

This review covers the theoretical background of the shear strength behaviour of saturated clays, mainly under undrained conditions. The stress-strain-strength characteristics of clays have also been incorporated.

2.2 Undrained Stress-Strain Behaviour of Clay

The immediate or undrained settlement is referred to as elastic settlement. The immediate or undrained settlement occurs without sufficient dissipation of excess pore water pressure due to sudden application of load. Undrained settlement is closely related to the undrained stability of foundation and is very important in foundation design. Linear displacement theory is widely used for predicting initial settlement. In this theory, the soil is characterized as an uniform elastic layer. The calculation of undrained settlement is based on the theory of elasticity which requires a value of the Young's modulus, E_s for the clay layer involved for undrained loading condition. This can be done by evaluating the undrained Young modulus of the clay.

The modulus of soil for undrained loading is not a unique property but varies widely with stress level, stress history, time, type of loading and soil disturbance. Bjerrum (1964), Ladd (1964), Desai (1971) and Yudhbir (1975) has dealt with Young's modulus of clays.

Fig. 2.1 shows stress-strain curves for five normally consolidated clays (Ladd, 1964). The general similarities are evident. At larger strain level, the clays experience slow and gradual increase of stress with increasing strain. It can be observed that the clays do not have any peak values where recession occurs. This is the general trend which all the normally consolidated clays experience.

2.2.1 Determination of Young's Modulus in Undrained Condition

Tangent and secant modulus are two commonly used methods for computing the Young's modulus from non-linear stress-strain curves found either from conventional undrained test or unconfined compression test (Davis and Poulos, 1968).

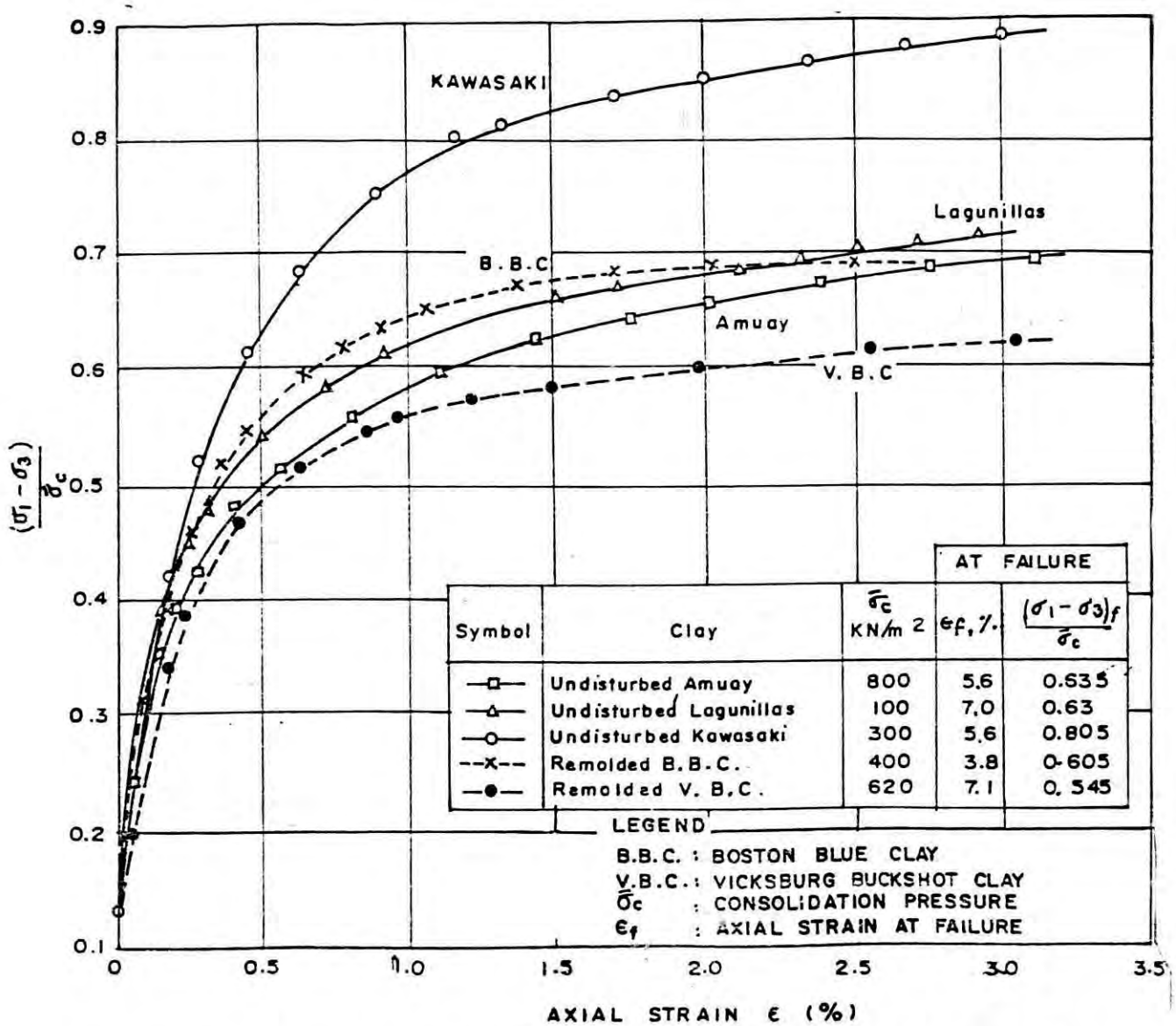


FIG. 2.1 STRESS – STRAIN RELATIONS FOR NORMALLY CONSOLIDATED CLAYS IN UNDRAINED CONDITIONS (FROM LADD, 1964)

Unconsolidated undrained tests and consolidated undrained triaxial tests are usual methods to determine Young's modulus of clays.

Due to difficulties in obtaining undisturbed soil samples, a number of investigator have attempted to correlate undrained modulus (E_u) to undrained shear strength (S_u). In saturated clay soils, the undrained Young's modulus can be obtained by the relation as reported by Cooling & Skempton (1942) and Skempton & Henkel (1957):

$$E_u = \Psi S_u \qquad (2.1)$$

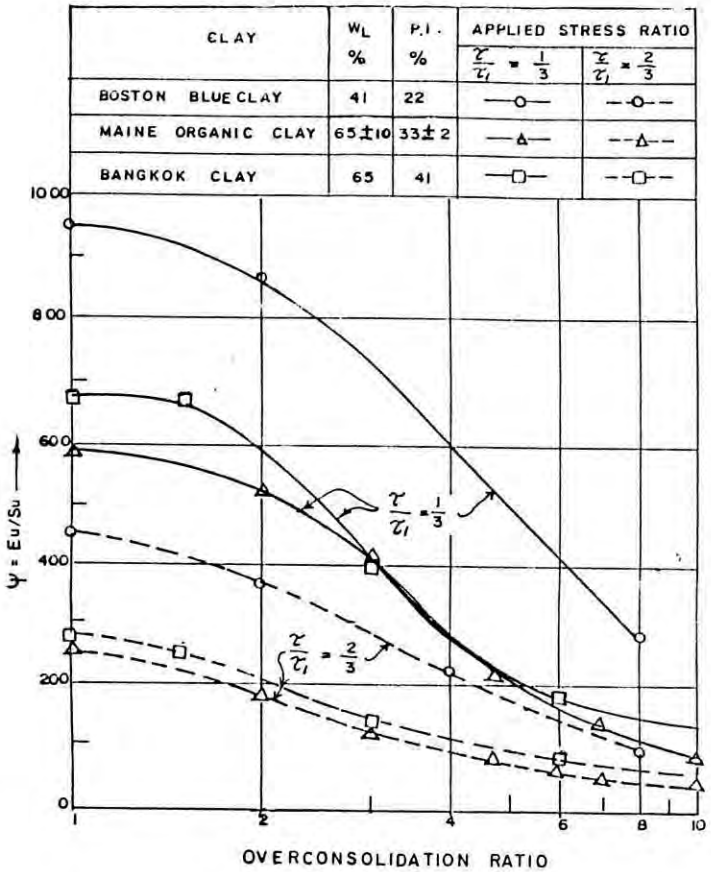
Where Ψ varies about 250 to 500 (Bjerrum, 1964), but other investigators such as Cooling & Skempton (1942) and Skempton & Henkel (1957) suggested Ψ value equal to 140. Based on present knowledge, the following comments can be made in the value of Ψ and E_u:

1) The value of Ψ decreases with increase of overconsolidation ratio of the clay. This is shown for three clays in Fig. 2.2. (D'Appoloria, Poulos and Ladd, 1971).

2) The value of Ψ generally decreases with the increase of the plasticity index of the clay (Duncan and Buchigani, 1976).

3) The value of E_u determined from unconfined compression tests and unconsolidated undrained triaxial tests are generally low (Liepens, 1957; Crawford and Burn, 1962; D'Appolonia et al., 1971).

4) The value of Ψ decreases with the organic content in the soil (Liepens et al., 1957).



DATA FROM $\overline{CK_0U}$ TYPE DIRECT-SIMPLE SHEAR TESTS

$E_U = \frac{3\tau}{\gamma}$ τ = APPLIED HORIZONTAL SHEAR STRESS

$E_U = 3G$ τ_l = MAXIMUM VALUE OF τ

$S_U = \tau_l$ γ = SHEAR STRAIN

FIG. 2.2 RELATIONSHIP BETWEEN E_u/S_u AND OCR FROM CU TESTS ON THREE CLAYS (AFTER D.J.D. APPOLONIA, POULOS AND LADD, 1971)

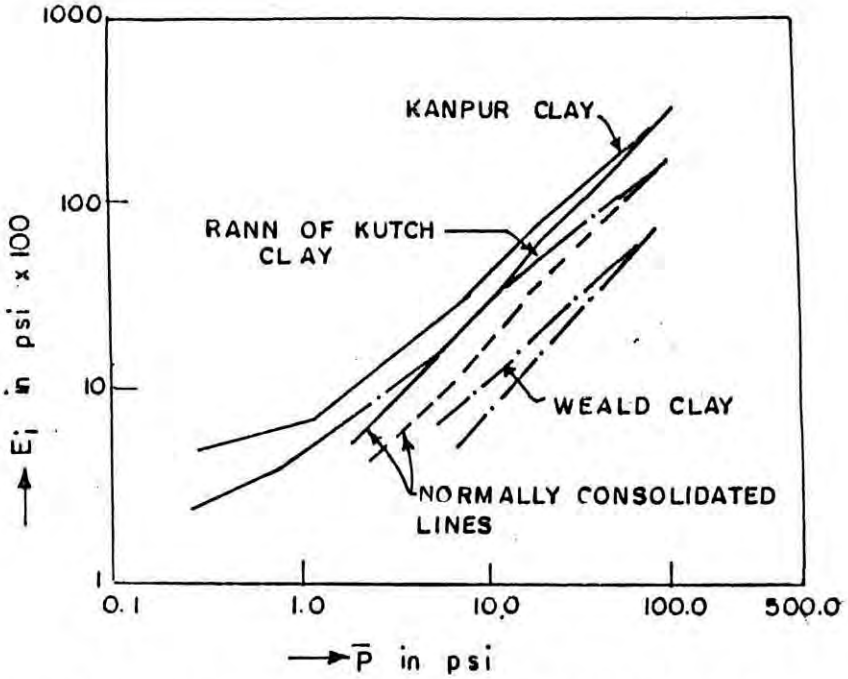


FIG. 2.3 INITIAL TANGENT MODULUS (E_i) VERSUS CONSOLIDATION PRESSURE ($\bar{P}$) FOR THREE CLAYS. (AFTER YUDHBIR ET AL, 1975)

5) For most cases, CIU or CK_oU type of tests on undisturbed samples yield values of E_u that are more representative of field behaviour (D'Appolonia et al., 1971).

6) For highly plastic clays, CU tests yield E_u values generally indicative of field behaviour (Liepens et al., 1957).

During undrained test, Ladd (1964) investigated the effect of the magnitude of the confining pressure. He concluded that the amount of confinement has no effect on the stress-strain relationship of a saturated cohesive soil.

2.2.2 Factors Affecting Undrained Young's Modulus of Clay

Any factor which will modify the slope of the stress-strain curve will affect the Young's modulus. These factors include consolidation pressure, stress level, OCR, type of initial consolidation pressure, soil unit weight, thixotropy, aging, strain-rate, soil fabric, sample size, type of tests such as U or CU, sample disturbance, factor of safety, load cycle and anisotropy. Some of the salient factors are discussed below briefly:

Consolidation Pressure

Yudhbir (1975) found that initial tangent modulus (E_i) increases with consolidation pressure. This has been shown in Fig. 2.3 (Yudhbir, 1975). The variation, as a crude approximation, is linear. The same has also been observed by Janbu (1963), Ladd (1964) and Varadarajan (1973). In Fig. 2.4, Lambe (1964) presents the results of undrained triaxial tests for three clays in the form of stress paths through which strain contours have been drawn. This is particularly informative type of plot. If the effective stress paths are geometrically similar and the strain

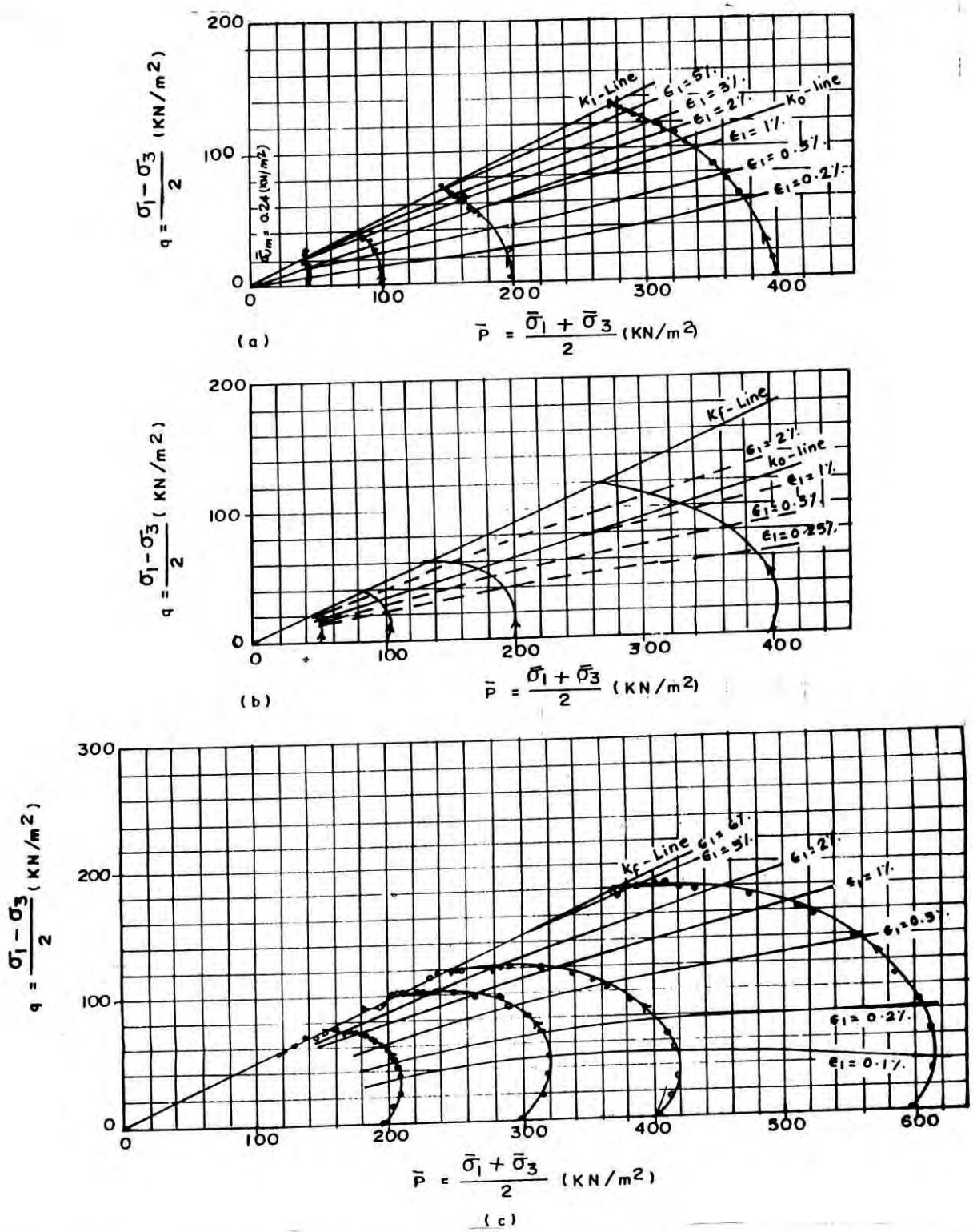


FIG. 2.4 CONTOURS OF EQUAL STRAIN FOR THREE NORMALLY CONSOLIDATED CLAYS (a) Amuay clay. (c) Remolded Boston blue clay. (b) Lagunillas clay. (from Lambe, 1964)

contours are straight radial lines for a group of tests, then a plot of $q/\bar{\sigma}_c$ (where, $q = (\sigma_1 - \sigma_3)/2$ and $\bar{\sigma}_c$ = consolidation pressure) versus strain would be unique. If the plot is unique, the modulus is then proportional to consolidation pressure.

Stress Level

Ladd (1964), Berre and Bjerrum (1973) found that at a particular consolidation condition, modulus decreases with the increase of stress level. Fig. 2.5 (after Ladd and Varallyay, 1965) presents a plot of normalized secant modulus, $E_u/\bar{\sigma}_{vc}$ (ratio of secant modulus and vertical consolidation pressure) versus applied shear stress ratio, $\Delta q/\Delta q_f$ i.e. $(\sigma_1 - \sigma_3)/(\sigma_1 - \sigma_3)_f$ for tests on normally consolidated Boston blue clay. The data suggest that the stress level is more important than the type of CKoU test.

Overconsolidation

Yudhbir (1975) showed variation of the ratio of initial tangent modulus to reduced swelling pressure ($E_i/\bar{P}_o$) with OCR for three clays in Fig. 2.6. The trend is pronounced at high values of OCR. Ladd (1964) and Varadarajan (1973) found similar trend.

Factor of Safety

Fig. 2.7 (Ladd, 1964) shows the ratio of secant modulus to consolidation pressure ($\bar{\sigma}_c$) for clays of Fig. 2.1 plotted against the factor of safety. The plot shows that $E_u/\bar{\sigma}_c$ increases with the increase of factor of safety.

2.3 Basic Strength Principles

Ladd (1971) referred that there are two basic strength principles that are commonly assumed in soil engineering.

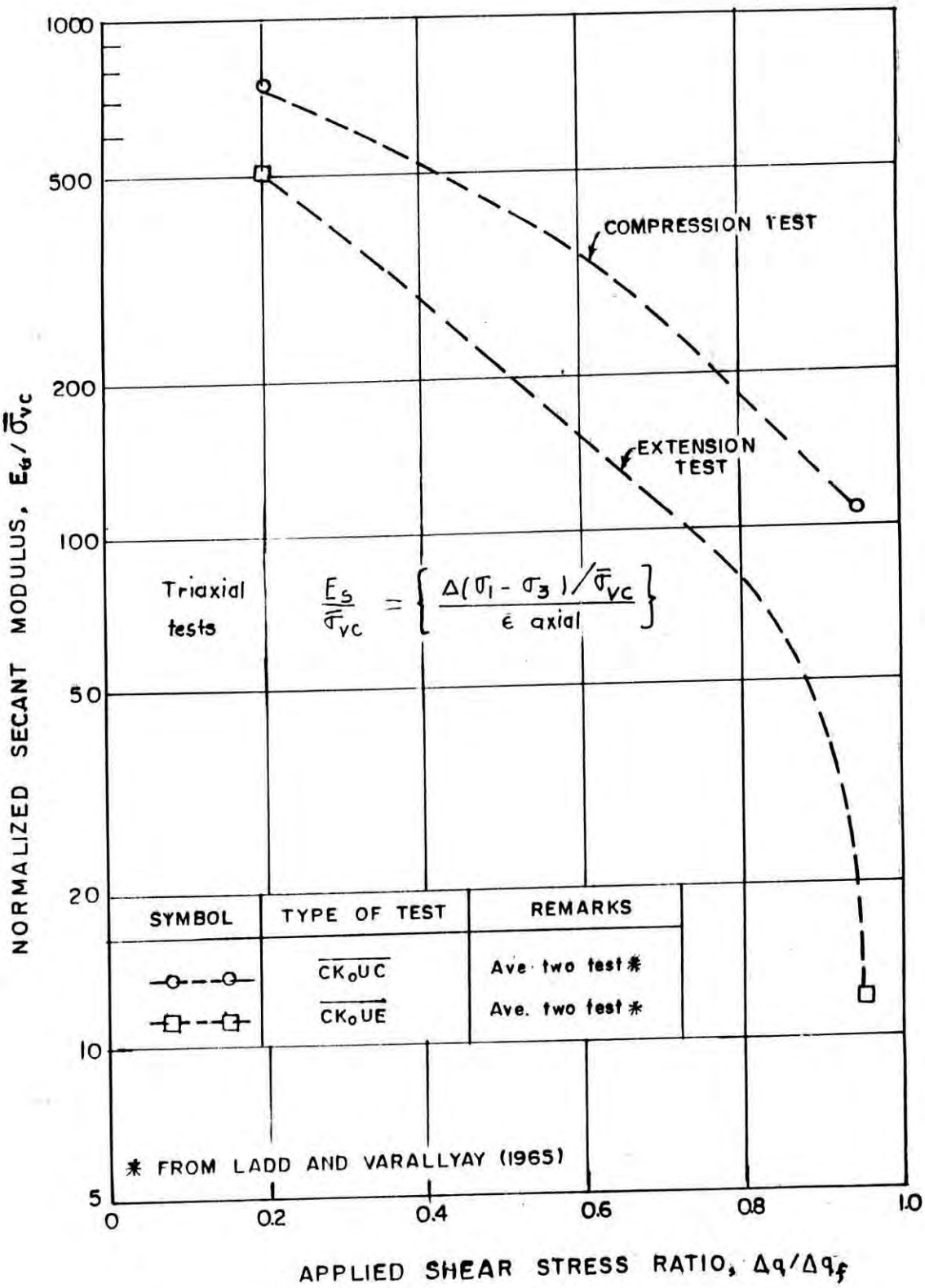


FIG. 2.5 NORMALIZED SECANT MODULUS FROM $\overline{CK_oU}$ TRIAXIAL TESTS ON NORMALLY CONSOLIDATED BOSTON BLUE CLAY. [AFTER LADD AND VARALLYAY (1965)]

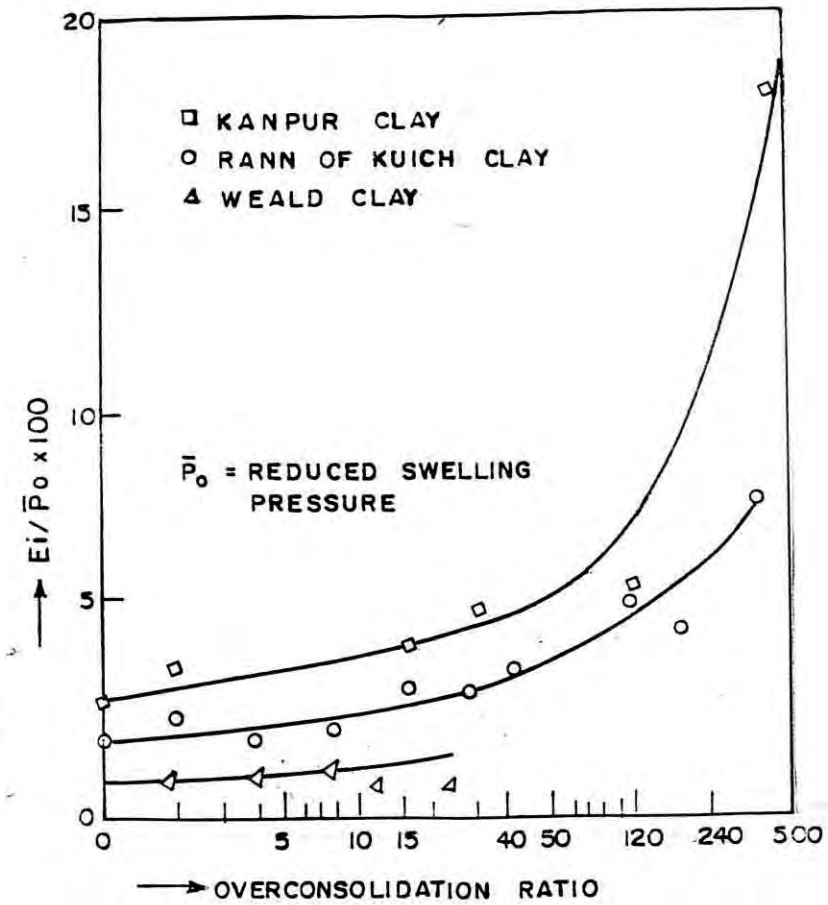


FIG.2.6 NORMALIZED INITIAL TANGENT MODULUS ($E_i/\bar{P}_o$) AS A FUNCTION OF OVERCONSOLIDATION RATIO (AFTER YUDHBIR ET AL, 1975)

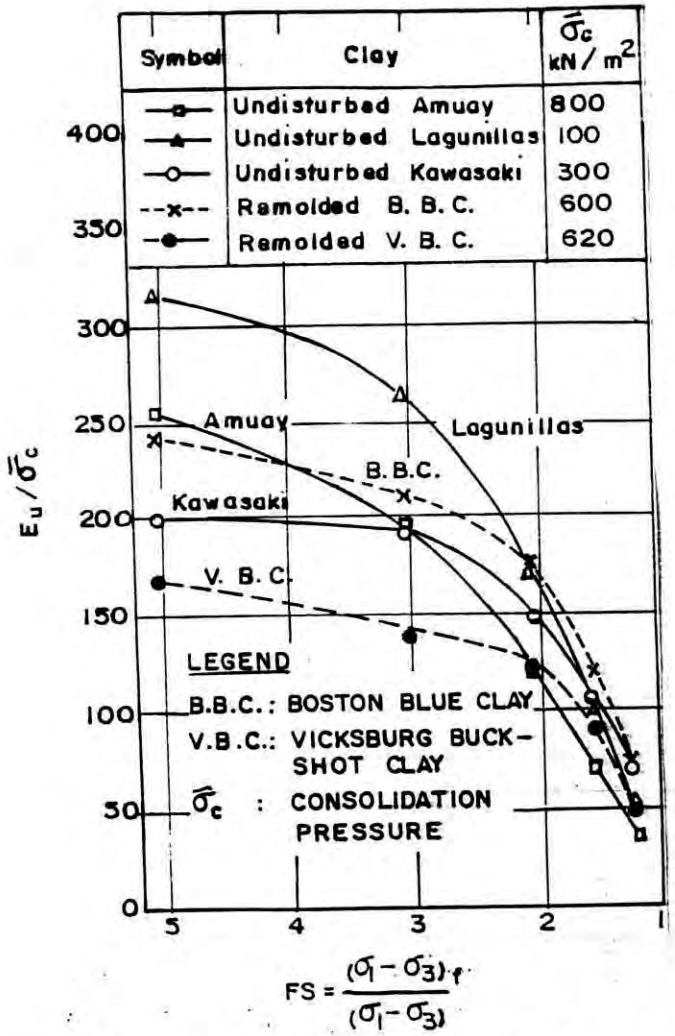


FIG. 2.7 UNDRAINED MODULUS FOR FIVE NORMALLY CONSOLIDATED CLAYS. (AFTER LADD, 1964)

1st Principle

For normally consolidated samples, or for overconsolidated samples with the same past stress hisotry, there is an unique relationship between strength and effective stress at failure.

2nd Principle

For normally consolidated samples or for overconsolidated samples with the same maximum past stress history, there is an unique relationship between water content, shear stress and effective stress.

The above principles are assumed to hold independent of

1) The drainage condition [i.e. whether an CU, CD or UU test]

2) The total stress path during shear [i.e. whether the soil is loaded or unloaded].

3) The K ratio [i.e. σ_h/σ_v or σ_r/σ_a] at consolidation.

2.4 K_o Consolidation and Isotropic Consolidation

The process of K_o consolidation and isotropic consolidation is illustrated in Fig. 2.8. The figure shows a comparison between the classical representation of one dimensional consolidation of Terzaghi which is K_o consolidation and the isotropic consolidation used in the critical state of soil mechanics (CSSM) approach (Wroth, 1968). The K_o consolidation has void ratio, e plotted against $\log_{10}\sigma_v$ [where, $\bar{\sigma}_v$ = effective vertical consolidation pressure] and the isotropic consolidation has specific volume, V plotted against $\log_e p'$ [where, p' = mean principal effective stress]. The specific volume, V is the total

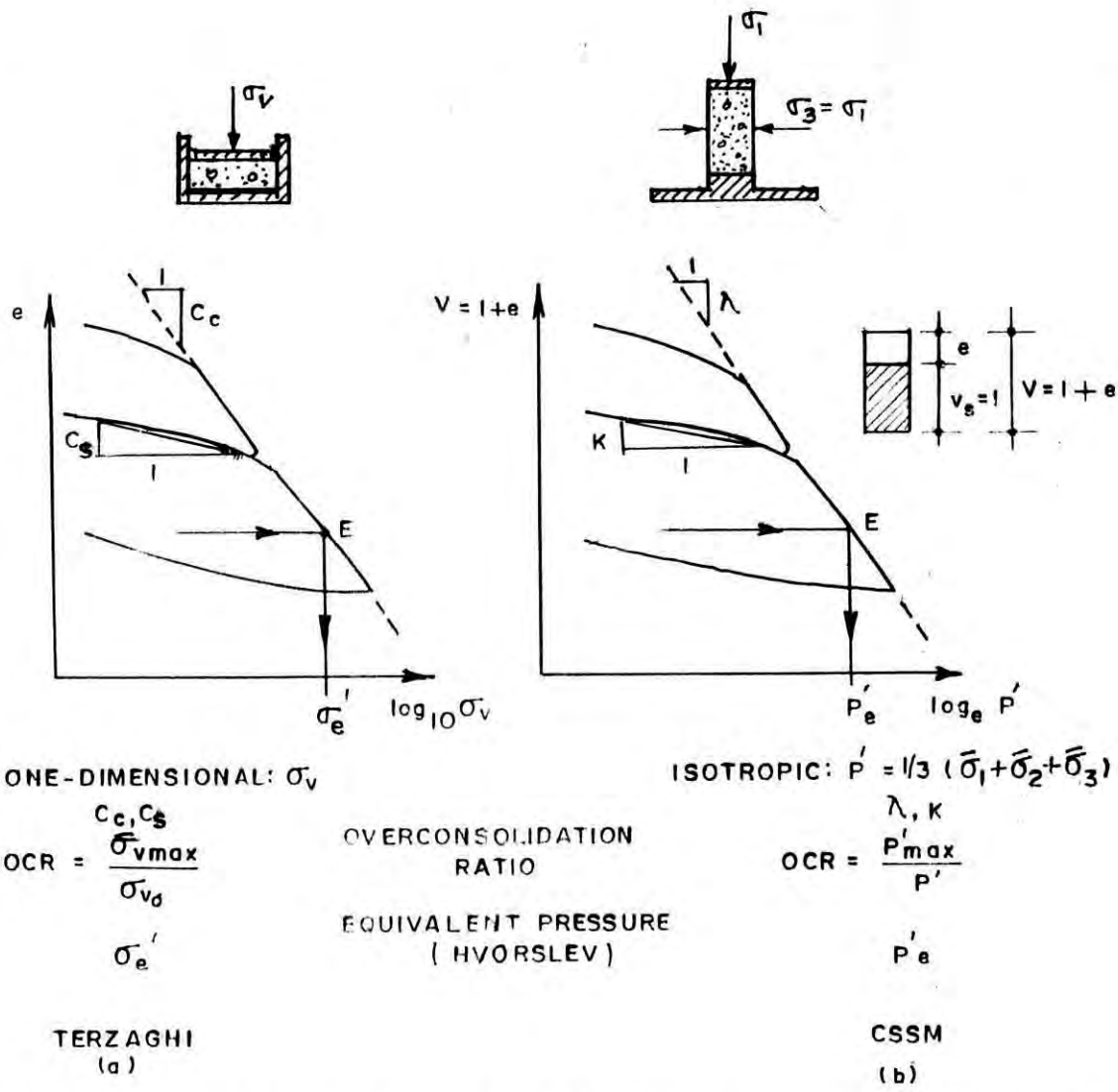


FIG. 2.8 DEFINITION OF CONSOLIDATION IN ONE DIMENSIONAL (Ko) AND ISOTROPIC CONDITIONS. [AFTER WROTH (1968)]

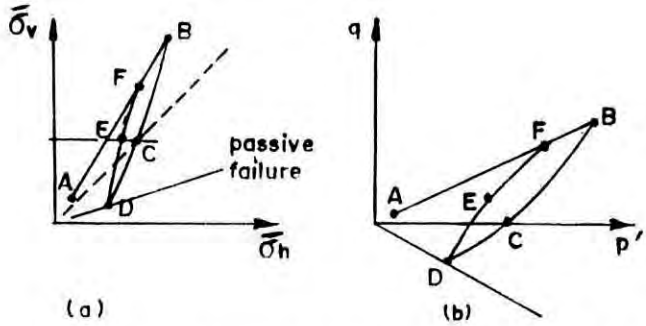


FIG. 2.9 EFFECTIVE STRESS PATHS IN ONE DIMENSIONAL (Ko) CONSOLIDATION AND SWELLING. [WROTH (1968)]

volume occupied by the unit volume of soil particles i.e.V = 1+e. For normally consolidated state, principal stresses increase proportionally and the two normal consolidation lines of the two plots of Fig. 2.8 have same gradient. The equivalent pressure of a specimen, introduced by Hvorslev (1937) is defined as the pressure on the normal consolidation line, such as at point E in Fig. 2.8. This proves to be an useful and convenient way of converting from the void ratio (or water content) of a clay specimen into a pressure variable.

Typical results of one-dimensional consolidation tests in which the lateral stress has been measured are illustrated qualitatively in Fig. 2.9. The effective stress paths have been presented in two separate plots. The first plot, Fig. 2.9(a), is in terms of the principal stresses, $\bar{\sigma}_1 = \bar{\sigma}_v$ and $\bar{\sigma}_2 = \bar{\sigma}_3 = \bar{\sigma}_h$, whereas the second plot of Fig. 2.9(b) is in terms of parameter $\bar{p}$ and q [where $\bar{p} = (\bar{\sigma}_1 + \bar{\sigma}_3)/2$ and $q = (\bar{\sigma}_1 - \bar{\sigma}_3)/2$]. Both serve useful purposes. The plot of Fig. 2.9(b) has the virtue of being based on the preferred stress variables for describing axially symmetric conditions. During one-dimensional normally consolidated condition along path AB in Fig. 2.9, the effective stresses are in constant proportion and the value of K_o is constant. The unloading phase is BCD which ends with the state of passive failure being reached at D. During the unloading phase K_o is not constant but varies with OCR. If the soil is reloaded one-dimensionally from an overconsolidated state such as D, the effective stress path DE rises very steeply in the plot of Fig. 2.9(a) until the initial normally consolidated condition is approached near point F. The relatively large hysteresis loop observed for heavily overconsolidated clays as illustrated in Fig. 2.9 serves an important warning. A clear distinction must be made between primary unloading BCD and reloading DEF. States E and C have the same current value of $\bar{\sigma}_v$, the same preconsolidation pressure (given by state B) and, hence, the same OCR, but they do not have same value of K_o.

2.5 Undrained Shear Strength

In most practical problems, undrained shear occurs wherever the load imposes at a much faster rate than the rate at which the induced pore pressures can dissipate. The excess pore pressures dissipate relatively slowly from a clay than from a cohesionless soil. The undrained shear strength is thus of great practical importance in the case of clays. Under this heading, different aspects of undrained shear strength are discussed below.

2.5.1 Applicability of Undrained Shear Strength Parameters

In conducting the analysis of stability of various foundations, the applicability of undrained shear strength parameters arises. The stability of those foundations is generally analyzed by assuming the condition as undrained. In UU analysis (also called end of construction or short term or immediate or undrained analysis), the shear strength controlling stability is the in-situ undrained shear strength, S_u that existed prior to construction. It is generally assumed that the change in total stresses during construction does not affect the in-situ undrained shear strength. There is no drainage and hence no change in shear strength. The analysis is called "$\phi = 0$" analysis since there is no change in strength with the change in total stress (Skempton, 1948; Bishop and Bjerrum, 1960).

Tests of this type can be performed quickly since drainage is not allowed. For a saturated cohesive soil, the deviator stress at failure, $\Delta\sigma_f$, is practically the same irrespective of the confining pressure, σ_3 as shown in Fig. 2.10. So the total stress failure envelope can be assumed to be a horizontal line, and $\phi = 0$. The undrained shear strength can be expressed as $S_u = \Delta\sigma_f/2$. This is generally referred to as shear strength based on "$\phi = 0$" concept. The fact that the strength of saturated clays

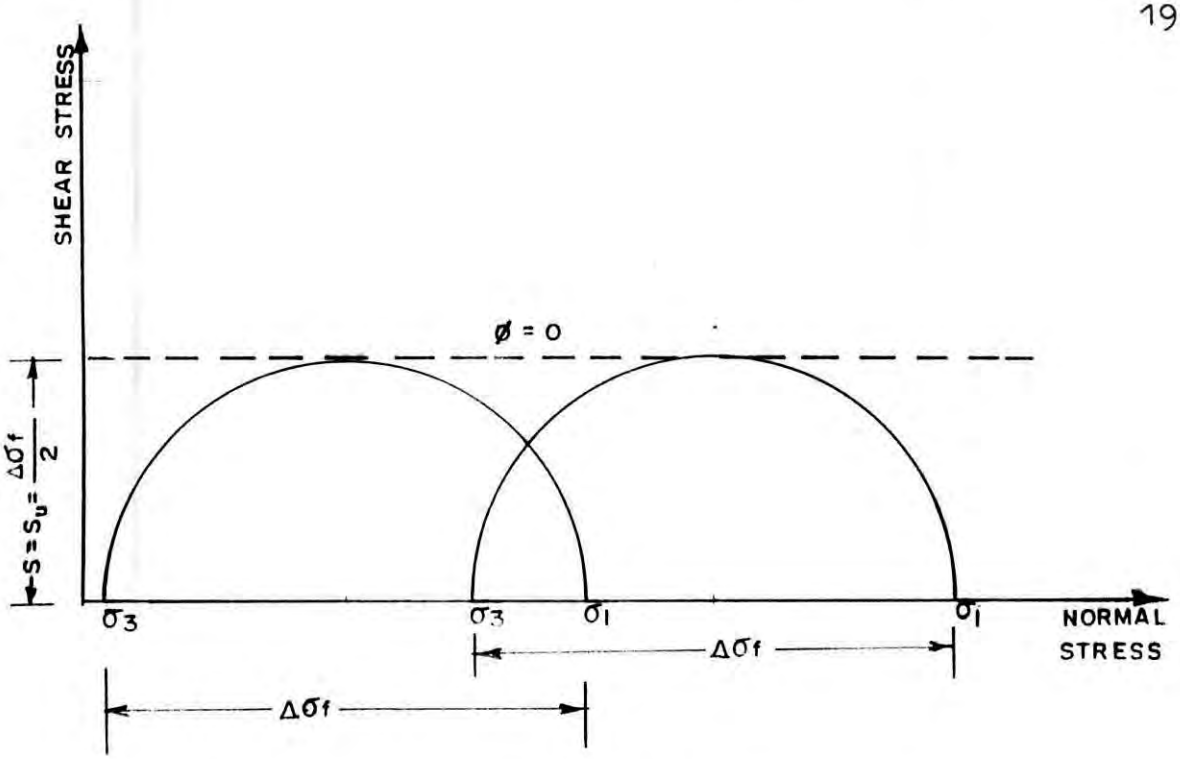


FIG. 2.10 UNCONSOLIDATED UNDRAINED TRIAXIAL TEST.

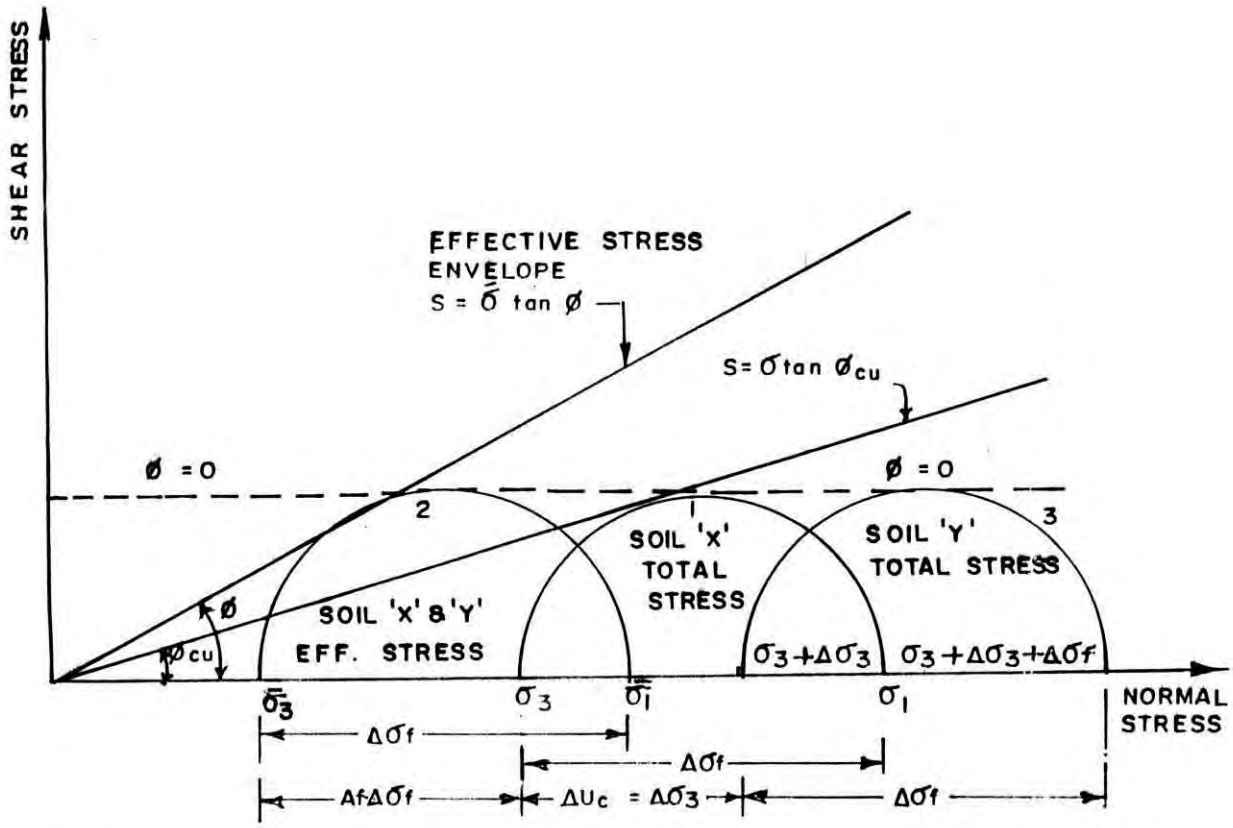


FIG. 2.11 UU TEST-'ø-0' CONCEPT.

in unconsolidated undrained loading conditions is the same irrespective of the confining pressure σ_3 can be explained with the help of Fig. 2.11. The effective-stress Mohr's circle corresponding to circle no. 1 of a specimen 'x' is circle no.2, which touches the effective stress failure envelope. If a similar soil specimen 'Y' is subjected to an additional confining pressure $\Delta\sigma_3$ without allowing drainage, the pore water pressure will increase by $\Delta\sigma_c$ and we know $\Delta\sigma_c = B\,\Delta\sigma_3$. For saturated soils, $B = 1$. So, $\Delta\sigma_c = \Delta\sigma_3$. Since the effective confining pressure of specimen 'Y' is the same as specimen 'X', it will fail with the same deviator stress, $\Delta\sigma_f$. It is shown in Fig. 2.11 that the effective stress Mohr's circle at failure for specimen 'Y' will be same as that for specimen 'x' i.e. circle no. 1. The value of $\Delta\sigma_3$ could be of any magnitude in specimen 'Y', in all cases, $\Delta\sigma_f$ would be the same.

2.5.2 Undrained Shear Strength Correlations and the $S_u/\bar{\sigma}_v$ Ratio

Shear strength correlations or parameters are widely used for both preliminary and final design studies. Many clay soils have been found to exibit normalized behaviour between the undrained shear strength, S_u and the in-situ overburden pressure, σ_v and some index property, I_i as

$$S_u/\bar{\sigma}_v = f(I_i)^k \qquad (2.2)$$

In 1957, Skempton established the following empirical correlation for normally consolidated clays based on field vane test results.

$$S_u/\bar{\sigma}_v = 0.11+0.0037(PI) \qquad (2.3)$$

Where, $\bar{\sigma}_v$ = effective overburden pressure
PI = plasticity index in percent

This statistical relationship has been found to be broadly valid over a wide range of sedimented clays (Peck et al., 1974). Use is usually made of this relationship to estimate the undrained shear strength of samples from Atterberg limit tests, if the deposit is known to be normally consolidated. The $S_u/\bar{\sigma}_v$ ratio can be utilized to predict the preconsolidation pressure for clay deposits once the Atterberg limits and undrained shear strength have been determined.

Another use of the S_u/σ_v ratio is the prediction of the undrained shear strength of a clay deposit. The predicted value of the undrained shear strength (S_u) is

$$S_u' = p.\ (S_u/\bar{\sigma}_v) \qquad (2.4)$$

Where P represent $\bar{\sigma}_v$ or measured most probable preconsolidation pressure (Pc) and $S_u/\bar{\sigma}_v$ is determined by Skempton's equation (2.3). Wu et al. (1978) reported measurement of the shear strength of a slightly overconsolidated clay deposit near Toledo, Ohio.

Similarly, the $S_u/\bar{\sigma}_v$ ratio can be utilized to predict the increase in shear strength from a change in effective stress for staged construction of embankments over soft ground.

Several other correlations of the general form (eq. 2.2) have been formulated for normally consolidated clays. Bjerrum and Simons (1960) established,

$$S_u/\bar{\sigma}_v = 0.45(PI)^{\frac{1}{2}} \qquad (2.5)$$

where, plasticity index, PI>5%

and $S_u/\bar{\sigma}_v = 0.18\ (I_L)^{-\frac{1}{2}}$ (2.6)

where, liqudity index, $I_L > 0.5$

Karlsson and Viberg (1967) formulated

$$S_u/\bar{\sigma}_v = 0.5\ w_L \qquad (2.7)$$

where, Liquid limit, $w_L > 20\%$

In above equations, PI, I_L and w_L to be used as decimal values.

In Fig. 2.12, Ladd and Foott (1974) presents experimental results for the variation of $S_u/\bar{\sigma}_{vo}$ [where, $\bar{\sigma}_{vo}$ = reduced swelling pressure] with OCR for six clays. Using the results of Fig. 2.12, Ladd and Foot (1974) presented a non-dimensional plot of α versus OCR in Fig. 2.13.

$$\text{where, } \alpha = \frac{(S_u/\bar{\sigma}_{vo})\ \text{overconsolidated}}{(S_u/\bar{\sigma}_{vo})\ \text{normally consolidated}}$$

The scatter of the plot has been bounded by upper and lower limits. The upper and lower limits are not too for apart. The average curve may be used for estimation of undrained shear strength of an overconsolidated clay having certain overburden pressure ($\bar{\sigma}_{vo}$) if OCR and the value of undrained shear strength of the clay at normally consolidated state [$S_u^{(nc)}$] is known.

The variation in normalized drained and undrained strength [$S_d/\bar{\sigma}_c$ and $S_u/\bar{\sigma}_c$] with OCR for simple clay is shown in Fig. 2.14 (Ladd, 1971). For loading tests, the undrained strength is less than the drained strength until the OCR exceeds 3.6. It is at this point that the pore pressure parameter A_f becomes negative. For unloading, S_u always equals or exceeds S_d.

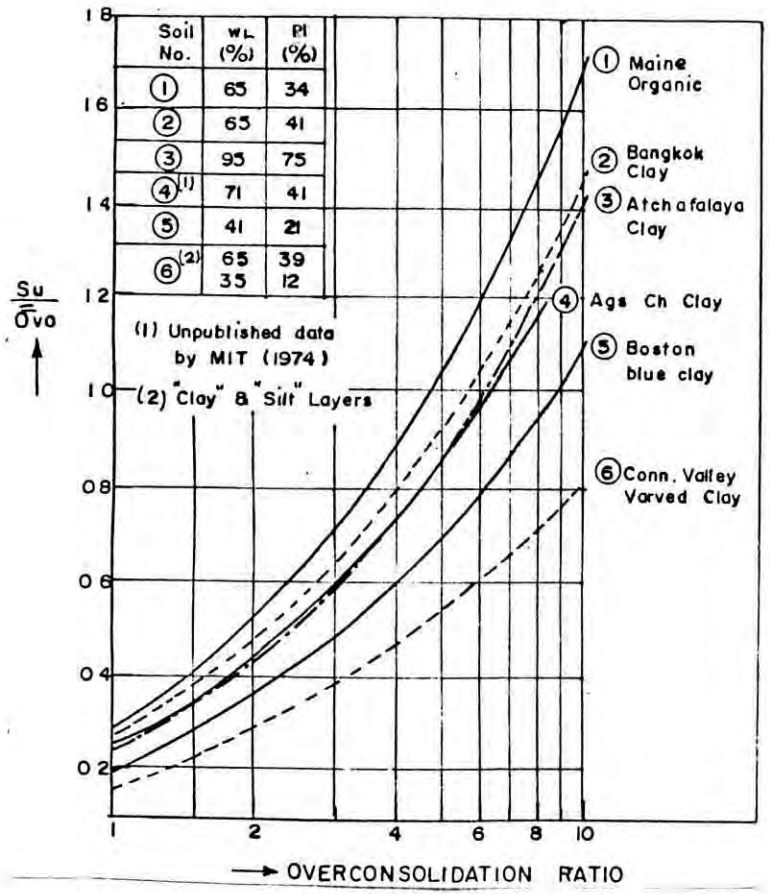


FIG. 2.12 UNDRAINED STRENGTH RATIO VERSUS OCR FROM CK_0 UDSS TESTS ON SIX CLAYS. (AFTER LADD AND FOOT, 1974)

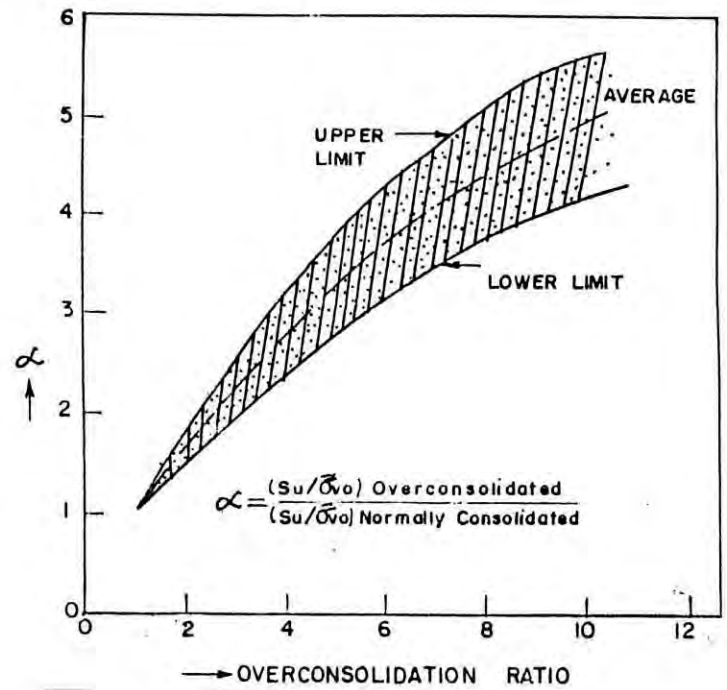


FIG. 2.13 PLOT OF $(S_u/\bar{\sigma}_{vo})$ OVERCONSOLIDATED / $(S_u/\bar{\sigma}_{vo})$ NORMALLY CONSOLIDATED AGAINST OVERCONSOLIDATION RATIO BASED ON RESULTS OF FIG. 2.12 [AFTER LADD AND FOOT, 1974]

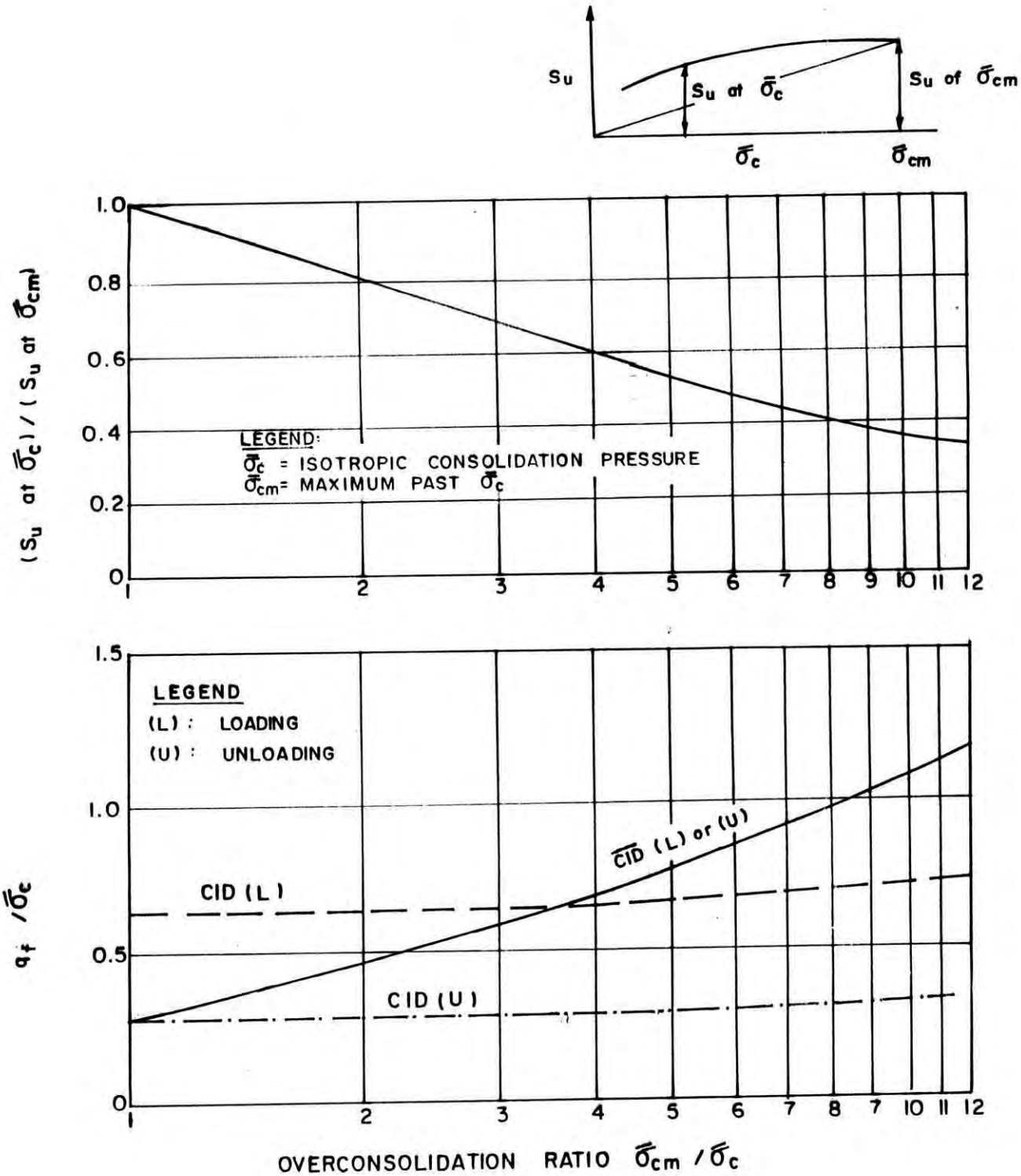


FIG. 2.14 EFFECT OF OCR ON DRAINED AND UNDRAINED STRENGTH OF SIMPLE CLAY. [LADD (1971)]

2.5.3 Relationships Between Water Content, Void Ratio, Undrained Shear Strength and Effective Consolidation Pressure

A unique relationship exists between the undrained shear strength and water content at failure for normally consolidated clay samples (Henkel, 1960). As shown in Fig. 2.15, the curve for undrained shear strength (S_u) is approximately parallel to the curve for effective consolidation pressure. The latter curve is the same as the pressure-void ratio curve from one-dimensional consolidation test.

As a clay deposit consolidates under the maximum effective applied pressure, a reduction in water content and a corresponding increase in shear strength occurs. The soil deposit can be considered to be normally consolidated under the maximum effective applied pressure, and there is definite relation between preconsolidation pressure and undrained shear strength.

The strength of a soil at failure [$(\sigma_1-\sigma_3)_{failure}$] is dependent on the moisutre content at failure. The Fig. 2.16 shows the unique relationship between the moisture content w at failure and the strength of a clay (Henkel, 1960).

For normally consolidated clays, the variation of moisture content, w versus log $(\sigma_1-\sigma_3)_{failure}$ is approximately linear. For overconsolidated clays, this relationship is not linear but lies slightly below the relationship of normally consolidated specimens as shown in Fig. 2.16. The curves merge when the strength approaches the overconsolidated pressure. Here it may be mentioned that slightly different relationships for moisture content, w versus log $(\sigma_1-\sigma_3)_{failure}$ are obtained for axial compression and axial extension tests.

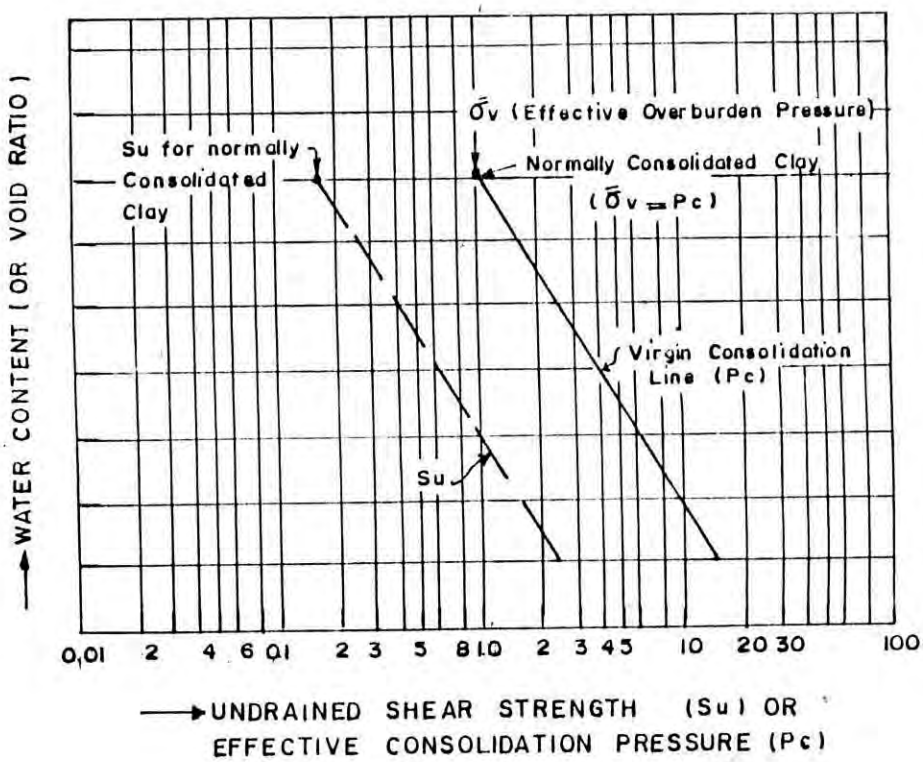


FIG. 2.15 RELATIONSHIP BETWEEN UNDRAINED SHEAR STRENGTH OR EFFECTIVE CONSOLIDATION PRESSURE AND WATER CONTENT OR VOID RATIO. [ANDERSON AND LUKAS (1981)]

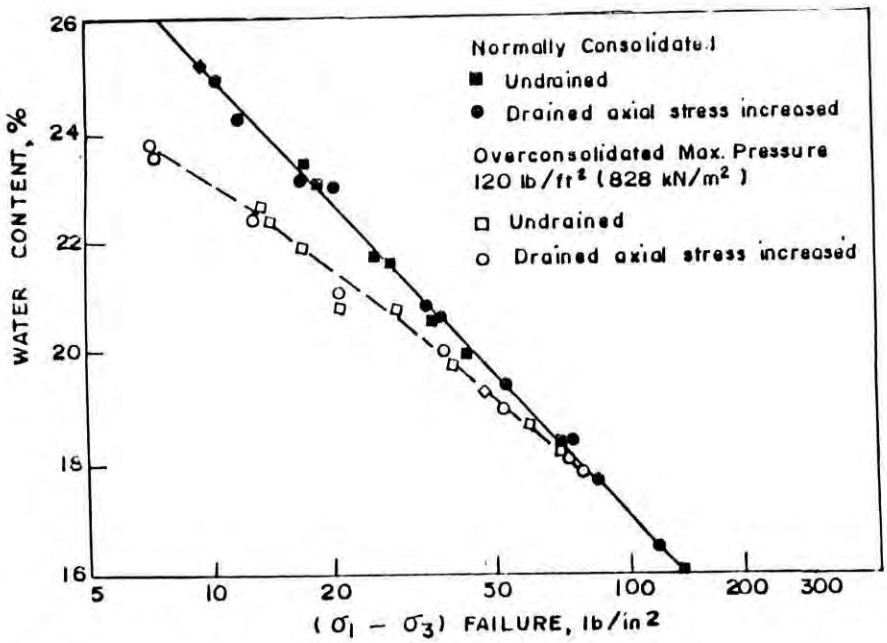


FIG. 2.16 RELATIONSHIP BETWEEN WATER CONTENT AND STRENGTH FOR N.C. AND O.C. WEALD CLAY COMPRESSION TESTS (AFTER HENKEL, 1960)

In Fig. 2.17, the virgin compression line for isotropic consolidation and values of q_f [i.e. $q_f = (\sigma_1 - \sigma_3)/2$ at failure] and $\bar{P}_f$ [i.e. $\bar{P}_f = (\sigma_1 - \sigma_3)/2$ at failure in terms of effective stress] are plotted to a log scale against water content. For normally consolidated clay, the lines of log q_f and log $\bar{P}_f$ versus w_f [water content at failure] are parallel to log σ_c [where $\bar{\sigma}_c$ = consolidation pressure].

The water content versus log stress relationships are given in Fig. 2.18. For the same water content at failure, a normally consolidated sample has a higher strength q_f than an overconsolidated sample. However, for the same $\bar{P}_f$, the overconsolidated sample is slightly stronger.

2.5.4 Use of Normalized Parameters of Undrained Shear Strength

It is a convenience in many cases to normalize the parameters of interest to obtain it in dimensionless form. Ladd et al. (1977) gave an extensive discussion on soils to obtain normalized soil parameters (NSP) for design use. Ladd et al. (1977) summarized the effect of OCR on K_o and undrained stress-strain-strength parameters for a variety of clays. Most of those data came from SHANSEP (Stress History and Normalized Soil Engineering Properties) test programs, wherein undisturbed samples were K_o consolidated in the laboratory into the virgin compression range and then rebounded to varying OCR. Hence in-situ preconsolidation pressure, σ_p' for all SHANSEP test specimens is caused by mechanical overconsolidation.

Fig. 2.19 plots log OCR versus log $S_u/\bar{\sigma}_{vo}$ [where $\bar{\sigma}_{vo}$ = in-situ consolidation pressure] from CK_oUTC, TE and DSS tests (Bjerrum and Landva, 1966). The log-log plot gives essentially straight lines and hence is closely approximated by the relationship:

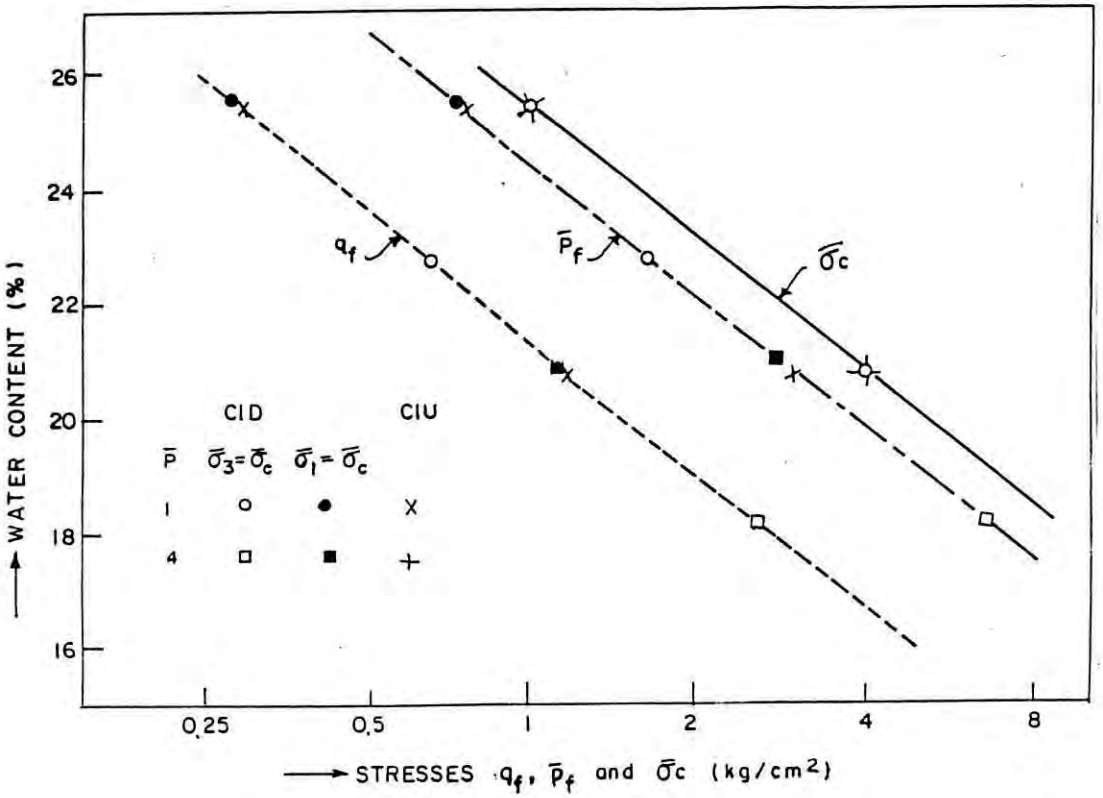


FIG. 2.17 WATER CONTENT VERSUS LOG STRESS FOR TRIAXIAL TESTS ON N.C. SIMPLE CLAY. [LADD (1971)]

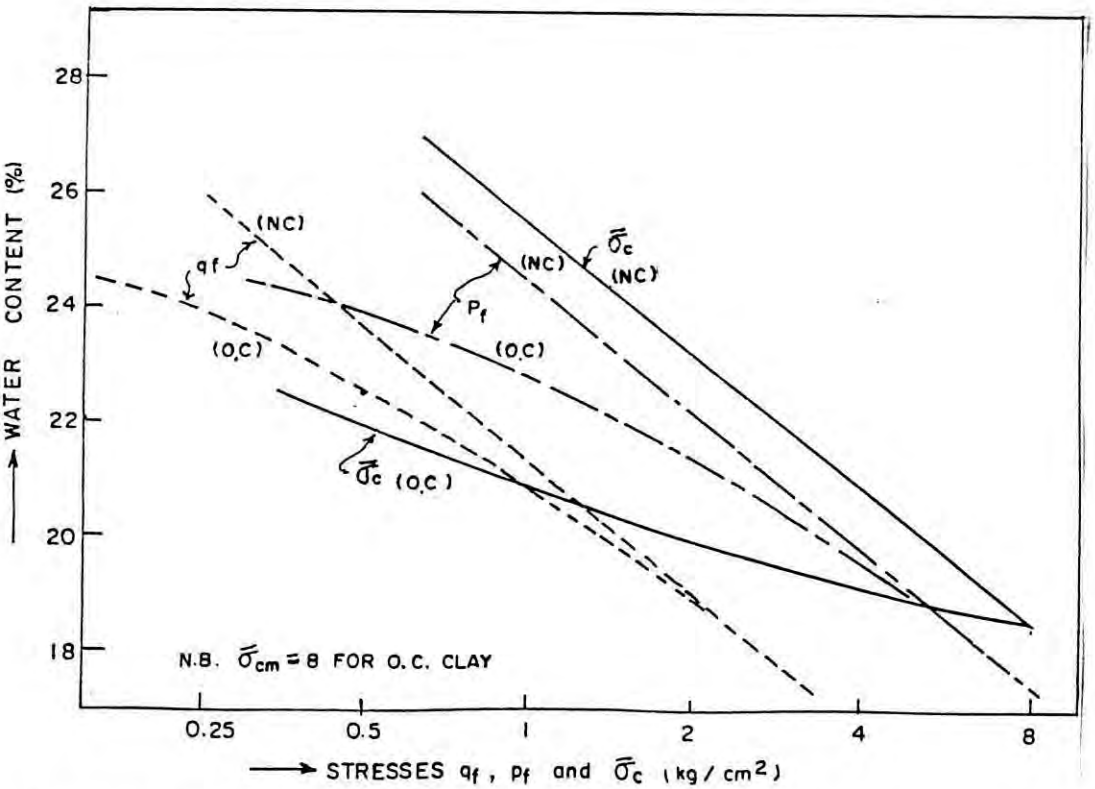


FIG. 2.18 WATER CONTENT VERSUS LOG STRESS FOR CIU AND CID ON N.C. AND O.C. SIMPLE CLAY. [LADD (1971)]

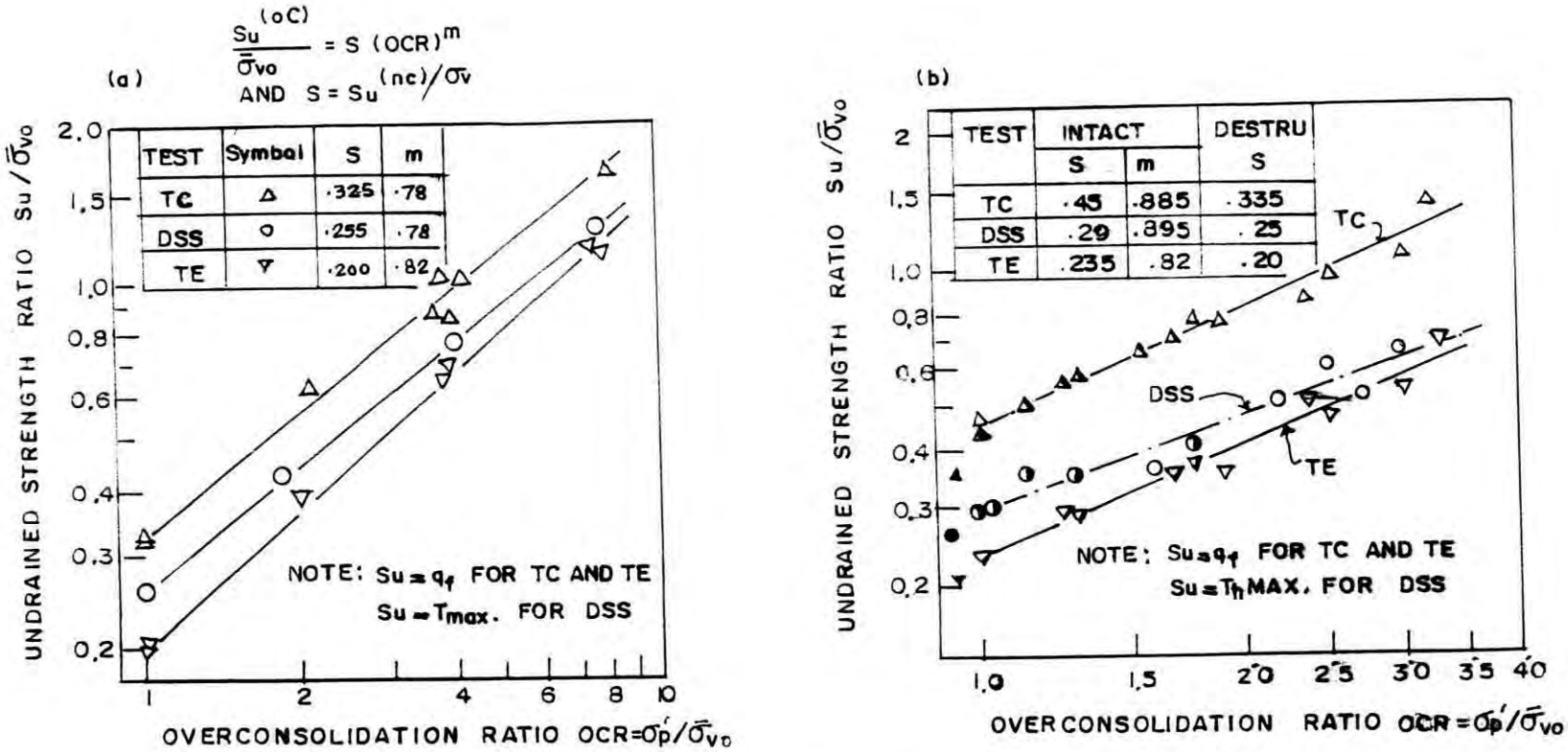


FIG.2.19 UNDRAINED STRENGTH RATIO VERSUS OCR

(a) FROM 'SHANSHEP' CKoU TESTS ON AGS CH MARINE CLAY (KOUTSOFTAS AND LADD (1984)

(b) FROM CKoU TESTS ON INTACT AND DESTRUCTURED MARINE CLAY [LEFEBVRE (1983)]

$$S_u/\bar{\sigma}_{vo} = S(OCR)^m \qquad (2.8)$$

Where, S is the undrained shear strength ratio ($S_u/\bar{\sigma}_{vo}$) for normally consolidated (OCR=1) clay.

Fig. 2.20 presents the relative increase in undrained shear strength ratios with increasing overconsolidation ratio. In this figure, the normalized shear strength ratio at a particular overconsolidation ratio, $[S_u/\bar{\sigma}_{vc}]^{oc}$, is divided by the corresponding normalized shear strength ratio for the normally consolidated soil, $[S_u/\bar{\sigma}_{vc}]^{nc}$. The data of the figure are from a comprehensive series of undrained shear tests performed on a lean sensitive marine clay in both the normally consolidated and overconsolidated state. The type of tests performed included triaxial compression and extension and direct simple shear tests on K_o consolidated specimens. The data from all three types of tests fall within a narrow range. The relationship between the ratio of normalized strengths and overconsolidation ratio may be described as suggested by Ladd et al. (1977) by an equation of the form

$$\frac{[S_u/\bar{\sigma}_{vc}]^{oc}}{[S_u/\bar{\sigma}_{vc}]^{nc}} = OCR^m \qquad (2.9)$$

Where m is a dimensionless co-efficient. For the data shown in Fig. 2.20, the value of m varies from 0.80 to 0.85. It is interesting to note that Ladd et al. (1977) reported m-values ranging from 0.75 to 0.85 from direct shear tests on five other clays.

Ladd et al. (1977) gave a means of estimating the undrained shear strength of preloaded soil as illustrated in Fig. 2.21 based on DSS (direct simple shear) test. These clays were tested in CK_oDSS

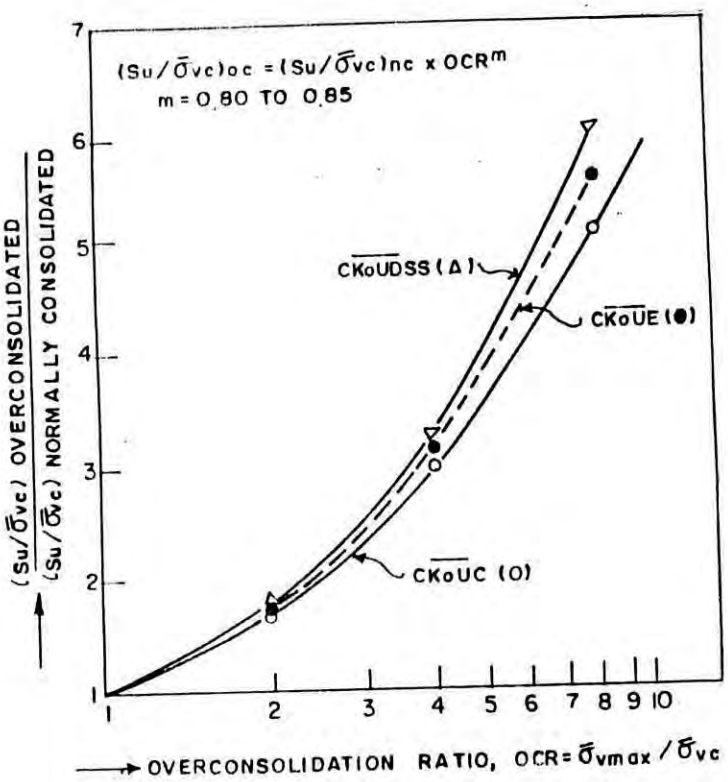
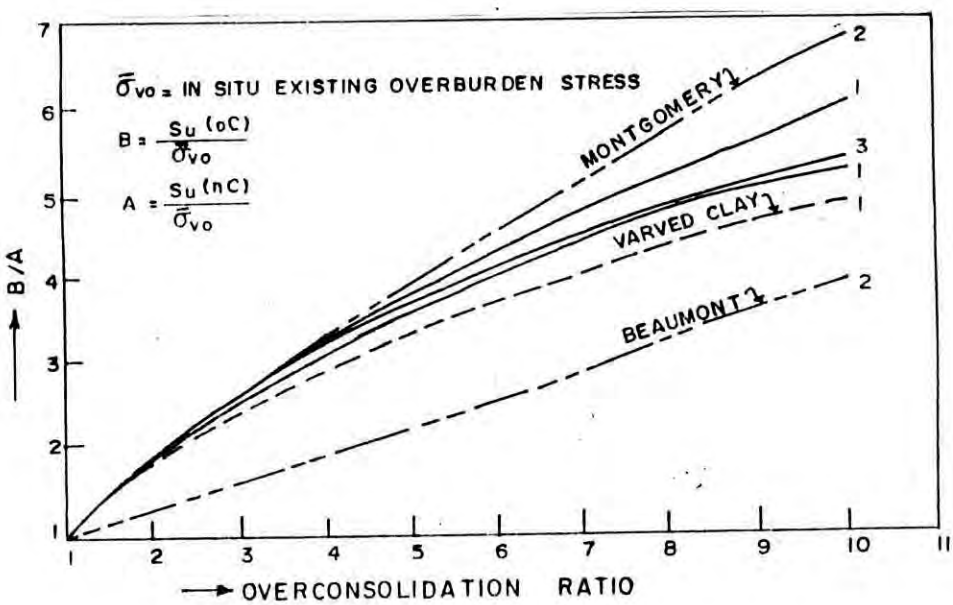


FIG. 2.20 RELATIVE INCREASE IN UNDRAINED STRENGTH RATIO WITH INCREASING OCR. [AFTER KOUTSOFTAS (1981)]



Code Source

1 CKo UDSS [Ladd et al (1977)]

2 CIUC [Mahar and O'Neill (1983)]

3 CIUC (Simons, 1960)

FIG. 2.21 RATIO (B/A) OF OVERCONSOLIDATED TO NORMALLY CONSOLIDATED CLAYS. [LADD ET AL (1977)]

(K_o consolidated undrained DSS test) at OCR from 1 to large values with the results normalized as follows:

$A = (S_u)^{oc}/\bar{\sigma}_{vo}$ and $B = S_u^{(nc)}/\bar{\sigma}_{vo}$

It is evident that at OCR = 1, B/A is equal to 1. Also when σ_{vo} is the in-situ overburden pressure, the ratio, $B/A = S_u^{(oc)}/S_u^{(nc)}$. The initial curve had only a modest scatter and would appear useful for almost any clay. Other tests data from Simons (1960) and Mahar and O'Niell (1983) have been plotted on to this curve (Codes 2 and 3 of Fig. 2.21). Clays range from inorganic to organic and highly dessicated (Code 2). Code 1 Covers five clays, Code 2 is at same locale but for two separate stratum, Code 3 is from oslo, Norway. The general curve trend are present and it can be suggested that these curves might be used for similar soils and the same local test method.

2.6 The Critical-State Pore Pressure Parameter from CU Shear Tests

The critical-state theory can be used to represent the undrained shear strength of overconsolidated soils. Atkinson and Bransby (1968) has derived an expression for the undrained shear strength of an overconsolidated soil in terms of its normally consolidated strength times OCR raised to a power function. This equation can be used in a normalized fashion for an overconsolidated soil which has experienced simple rebound.

$$S_u^{(oc)}/\bar{\sigma}_{vo} = [S_u^{(nc)}/\bar{\sigma}_v] \times (OCR)^{\Lambda_o} \qquad (2.10)$$

where Λ_o = Critical state pore pressure parameter

$$= 1 - [C_s^{(iso)}/C_c^{(iso)}]$$

$S_u^{(oc)}/\bar{\sigma}_{vo}$ = Ratio of undrained shear strength to overburden pressure of o.c soil.

$S_u^{(nc)}/\bar{\sigma}_v$ = Ratio of undrained shear strength to overburden pressure of n.c. soil.

$C_c^{(iso)}$ & $C_s^{(iso)}$ = Isotropic virgin compression and swelling/recompression indices respectively i.e. slope of elogP curve for loading and unloading respectively for isotropically consolidated soil.

The results of isotropic and anisotropic consolidated undrained shear tests (CIU, CK_oU) can be used to determine the critical-state pore pressure parameter. The effects of OCR and initial stress state (K_o) on Skempton's pore pressure parameter (A) can significantly alter effective stress predictions of undrained strength. The critical-state parameter is independent of OCR, K_o and level of shear to failure, thus requiring only two basic soil constants in order to predict undrained shear strength, namely, the effective stress friction angle ($\bar{\phi}$)and the critical-state pore pressure parameter (Λ_o).

Different scientists have shown that $C_s^{(iso)}$ is in fact a variable with OCR and is different during loading than for reloading. To avoid this anomaly, the critical-state pore pressure parameter (Λ_o) has been redefined in terms of overconsolidated strength.

$$\Lambda_o = \frac{\text{Log}[S_u^{(oc)}/\bar{\sigma}_{vo}]-\log[S_u^{(nc)}/\bar{\sigma}_c]}{\log[OCR]} \qquad (2.11)$$

Ladd and Foot (1974) demonstrated that normalized strengths ($S_u/\bar{\sigma}_{vo}$) are a function of OCR. The "critical-state failure" described by Schofield and wroth (1968) is defined as the stress-level at which continuous deformation occurs at no change in volume. The basic concepts proposed by the theory would be valid if applied to undrained shear strength data. Therefore, for simplicity, 'failure' has been defined as the undrained strength at maximum deviator stress.

Most soils encountered in nature are overconsolidated to some degree. The OCR is not often known during testing unless supplementary consolidation testing is conducted or a SHANSEP approach is employed. The parameter Λ_o may be determined from the results of CU shear tests conducted at confining pressures less than the preconsolidation pressure without knowledge of OCR. The critical-state pore pressure parameter is then defined by the absolute value of the slope of a linear relationship between log $[S_u(^{oc})/\bar{\sigma}_{vo}]$ and log $\bar{\sigma}_{vo}$ as shown in Fig. 2.22.

It is important to note that critical-state pore pressure parameter (Λ_o) has been determined from a total stress approach and it is a soil constant. The parameter Λ_o can be used to predict undrained shear strengths for various stress histories and initial stress conditions. The application of the pore pressure parameter is best presented in its relationship with the critical-state theory. The normally loaded undrained strength of a clay soil can be determined from the relationship between log $[S_u(^{oc})/\bar{\sigma}_{vo}]$ and log [OCR] as the intercept at OCR=1 as shown in the Fig. 2.22. Alternatively, an effective stress approach by schofield and Wroth (1968) proposes that the undrained stress path to failure for any isotropically normally loaded soil is given by

$$q/p' = M\log_e(p'_o/p')/\Lambda_o \qquad (2.12)$$

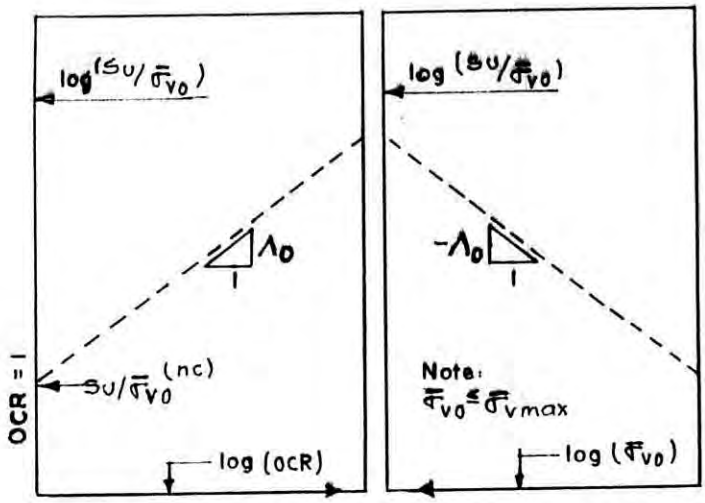


FIG. 2.22 DEFINITIONS OF CRITICAL STATE PORE PRESSURE PARAMETER FROM UNDRAINED STRENGTH DATA.

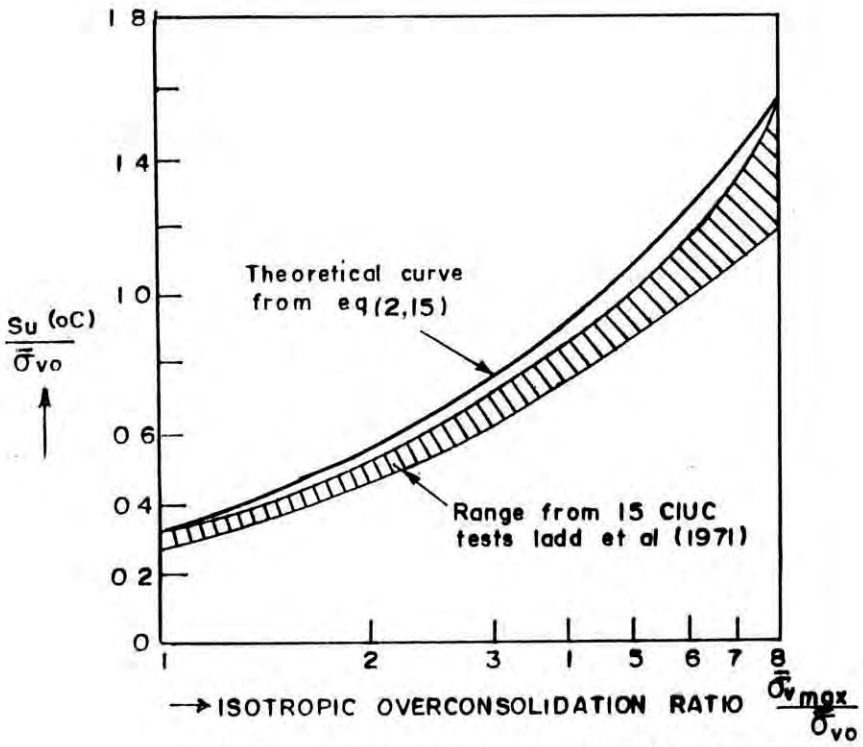


FIG. 2.23 VARIATION OF UNDRAINED STRENGTH RATIO WITH OVERCONSOLIDATION RATIO FROM TRIAXIAL COMPRESSION TESTS. (data from Ladd et al 1971)

where $M = 6 \sin \bar{\phi}/(3-\sin \bar{\phi})$ (2.13)

M is the slope of the failure line in q-p space [where $q = \sigma_1 - \sigma_3$ and $p = 1/3\ (\bar{\sigma}_1 + 2\bar{\sigma}_3)$]. P'_o and p' are initial and current values of mean normal effective stress, respectively. Further derivation gives the cam-clay prediction of undrained shear strength ratio for normally consolidated soils as

$$S_u(nc)/\bar{\sigma}_v = 0.5\ M \exp(-\Lambda_o) \qquad (2.14)$$

Equation (2.10) and (2.14) can be combined to represent the undrained strength for a clay soil over a range of OCR:

$$S_u(oc)/\bar{\sigma}_{vo} = 0.5M(e^{-1}\ OCR)^{\Lambda_o} \qquad (2.15)$$

For a given soil, M and Λ_o are constants, so that in theory, the undrained shear strength ratio is proportional to the OCR raised to a power Λ_o. A comparison between theory from eq. (2.15) and experimental data is presented in Fig. 2.23 for Boston blue clay.

Equation (2.13) and (2.14) can be combined to form the equation for undrained strength ratio for normally consolidated clay.

$$S_u(nc)/\bar{\sigma}_v = \frac{3 \sin \bar{\phi} \exp(-\Lambda_o)}{(3-\sin\bar{\phi})} \qquad (2.16)$$

2.7 Mitachi and Kitago's Derivation and Mayne's Model to Predict Undrained Shear Strength

Mitachi and Kitago (1976) investigated the undrained shear strength behaviour under K_o and isotropic stress conditions for

three types of remoulded clays. They reported that the log-log plot of normalized undrained shear strength ($S_u/\bar{\sigma}_{vo}$) versus overconsolidation ratio is a straight line. From experimental observations, they found that the semilog plots of water content versus consolidation pressure are straight lines for both loading and unloading portions and their slopes (i.e. compression index, C_c and swelling index, C_s) are constants for a given clay irrespective of stress-systems during consolidation and secondly, the lines of w log $\bar{\sigma}_v$ and w log $\bar{p}_f$ [where, $\bar{p}_f = (\bar{\sigma}_1 + \bar{\sigma}_3)/2$ at failure] are parallel and the rebound part of wlog p_f curve is approximated as straight line (Fig. 2.24). Based on these assumptions, they developed a relationship to predict the undrained strength of any clay,

$$\frac{S_u(oc)}{\bar{\sigma}_{vo}} = \frac{S_u(nc)}{\bar{\sigma}_v} OCR^{(1-\lambda)} \qquad (2.17)$$

where, $\lambda = C_s/C_c$

The experimental results of Mitachi and Kitago (1976) yielded straight line having slope of β (Fig. 2.25) and showed excellent compliance with their relation of eq. (2.17). The equation hold good for any clay satisfying the assumptions as stated above. The slope of line from eq. (2.17) is $1-\lambda$.

Mayne (1980) investigated undrained shear strength behaviour of clay and silt soils that are normally consolidated to lightly and heavily overconsolidated. He compared results of 96 different soils from data provided from different publications. Using such data bank, Mayne (1980) developed a model to predict undrained shear strength of overconsolidated soil. The model is simple and valid for any clay. The model is able to find the undrained shear strength of any clay for any degree of overconsolidation and any

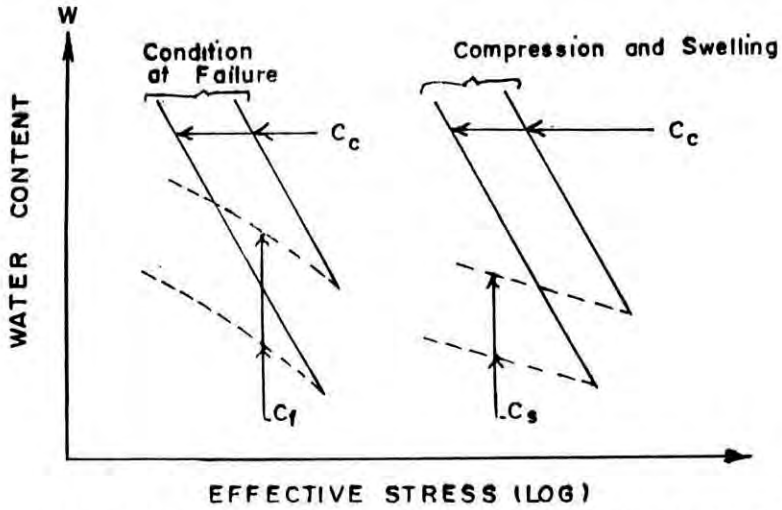
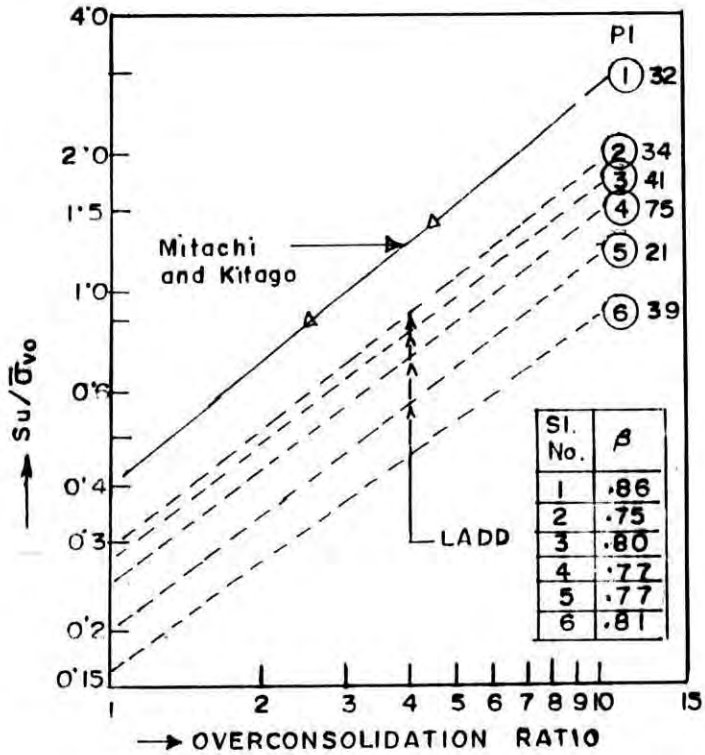


FIG. 2.24 ASSUMPTION MADE BY MITACHI AND KITAGO (1976) ABOUT WATER CONTENT VERSUS EFFECTIVE STRESS.



Comparison of the value $(1-\lambda)$ and β
Source : Mitachi and Kitago

Sample	β	$1-\lambda$
1	0.81	0.80
2	0.85	0.84
3	0.86	0.89

FIG. 2.25 NORMALIZED UNDRAINED SHEAR STRENGTH AS A FUNCTION OF OVERCONSOLIDATION RATIO (SOURCE: EXPERIMENTAL RESULTS OF MITACHI AND KITAGO, 1976)

in-situ overburden pressure if critical-state pore pressure (Λ_o) and effective stress friction angle ($\bar{\phi}$) of the soil are known.

As stated earlier, the critical-state pore pressure parameter (Λ_o) is basic soil constant which is independent of initial confining pressure, OCR, K_o and shear to failure. Λ_o can be determined from CIU or CK_oU tests without direct measurement of pore pressure. The parameter Λ_o is equal to $1-[C_s(iso)/C_c(iso)]$.

By the extensive research effort, Mayne (1980) concluded that following relationship can be used to predict the undrained shear strength of overconsolidated soils.

$$\frac{S_u(oc)}{\bar{\sigma}_{vo}} = \frac{S_u(nc)}{\bar{\sigma}_v} \times (OCR)^{\Lambda_o} \qquad (2.10)$$

Mitachi and Kitago's (1976) have used $(1-\lambda)$ in their equation (2.17) as power to OCR, whereas Mayne (1980) has used A_o instead of $1-\lambda$. The parameter A_o can be found from experiments and can be formulated as follows:

$$A_o = \frac{\text{Log}[S_u(oc)/\bar{\sigma}_{vo}]-\log[S_u(nc)/\bar{\sigma}_v]}{\log[OCR]} \qquad (2.11)$$

So it shows that Λ_o is the slope of the straight line found from the log-log plot of $S_u(oc)/\bar{\sigma}_{vo}$ versus OCR. Mayne (1980) determined the values of Λ_o for each soil from linear regression analysis. Sample correlation co-efficient (r) from the linear regression studies are generally high ($r>0.98$) indicating an excellent fit (Here to note that $r=0$ for no correlation and $r=1$ indicates a perfect fit).

The critical state theory predicts by conservation of energy that a limiting condition exists [Λ_o(max) = 1] whereby the work dissipated in the soil must either be equal or less than energy input. Mayne's (1980) study reveals that these soils which exibit high values of Λ_o (greater than 0.9) are sensitive clays. As we have seen cam-clay theory give the following equation in terms of effective stress.

$$\frac{S_{u(nc)}}{\bar{\sigma}_v} = \frac{3 \sin \bar{\phi}}{3 - \sin \bar{\phi}} \times \frac{1}{(\exp \Lambda_o)} \qquad (2.16)$$

Replacing the equation (2.16) in equation (2.10), it can be obtained the equation for predicting the undrained shear strength of o.c. soils.

$$\frac{S_{u(oc)}}{\bar{\sigma}_{vo}} = \frac{3 \sin \bar{\phi}}{3-\sin\bar{\phi}} \times \frac{OCR\ \Lambda_o}{\exp \Lambda_o} \qquad (2.18)$$

The equation requires two basic soil parameters Λ_o and $\bar{\phi}$ which are constants for a particular soil and may have different values for different soils.

Mahar and O'Neill (1983) reported that higher values of liquid limit of soil will produce lower value of β. They plotted $S_u/\bar{\sigma}_{vo}$ versus OCR for two different types of soils and found two different curves whereas they found that the plot of $S_u w_L/\bar{\sigma}_v$ versus OCR for the two types of soils produced a common curve.

A summary of available equations for modelling clay soil behaviour in normally consolidated and overconsolidated conditions are now shown in Table 2.1.

TABLE 2.1: SOME MODEL EQUATIONS TO REPRESENT UNDRAINED SHEAR STRENGTH

Sl. No.	Model equations	Stress	Source of Development
1.	$\frac{S_u^{(oc)}}{\bar{\sigma}_{vo}} = \frac{S_u^{(nc)}}{\bar{\sigma}_v}.(OCR)^{\Lambda o}$	Overconsolidated	Derived by Atkinson and Bransby (1968)
2.	$\frac{S_u^{(oc)}}{\bar{\sigma}_{vo}} = \frac{S_u^{(nc)}}{\bar{\sigma}_v}.(OCR)^{1-\lambda}$	"	Mitachi and Kitago (1976)
3.	$\frac{S_u^{(oc)}}{\bar{\sigma}_{vo}} = \frac{3\sin\bar{\phi}}{3-\sin\bar{\phi}} \times \frac{(OCR)^{\Lambda o}}{\exp(\Lambda_o)}$	"	Equation derived from critical-state Cam-clay theory. Mayne (1980) used it as model.
4.	$\frac{[S_u/\bar{\sigma}_{vc}]^{(oc)}}{[S_u/\bar{\sigma}_{vc}]^{(nc)}} = (OCR)^m$	"	Ladd et al. (1977)
5.	$S_u/\bar{\sigma}_{vo} = S(OCR\ )^m$	"	Bjerrum and Landva (1966)
6.	$\frac{S_u}{\bar{\sigma}_v} = \frac{3\sin\bar{\phi}}{3-\sin\bar{\phi}} . \frac{1}{\exp(\Lambda_o)}$	Normally consolidated	Critical-state Cam-Clay theory developed by Schofield and Wroth (1967)
7.	$S_u/\bar{\sigma}_v = 0.11+0.0037(PI)$	"	Skempton (1957)
8.	$S_u/\bar{\sigma}_v = 0.45(PI)^{\frac{1}{2}}$ where, $PI>5\%$	"	Bjerrum and Simons (1960)
9.	$S_u/\bar{\sigma}_v = 0.18(I_L)^{-\frac{1}{2}}$ where, $I_L>0.5$	"	Bjerrum and Simons (1960)
10.	$S_u/\bar{\sigma}_v=0.5\ w_L$ where $w_L>20\%$	"	Karlsson and Viberg (1967)

2.8 The Variation of Initial Shear stress Ratio with Stress History

The in-situ stress state has a major effect on the subsequent behaviour of a soil element. The relative values of σ_{ho} and σ_{vo} i.e. the value of K_o dictate the magnitude of stress change that the element can sustain before local yield or failure is induced. This can have major influence, for example, on the performance of a retaining wall or a settlement experienced by a surface loading. A convenient way of representing this effect in dimensionless form is by means of the initial shear stress ratio, f introduced by Davis and Poulos (1968) and defined as

$$f = \frac{\frac{1}{2}(\bar{\sigma}_{vo}-\bar{\sigma}_{ho})}{S_u} = \frac{1-K_o}{2S_u/\bar{\sigma}_{vo}} \qquad (2.19)$$

This ratio will vary markedly with K_o and hence with overconsolidation ratio. For normal consolidation, as shown in Fig. 2.9, the effective stresses are in constant proportion and the value of K_o is constant. For normally consolidated soils, Jacky (1944) derived a theoretical relationship:

$$K_{o(nc)} = \frac{[1+(2/3)\ \sin\bar{\phi}](1-\sin\bar{\phi})}{(1+\sin\bar{\phi})} \qquad (2.20)$$

where $\bar{\phi}$ = effective stress friction angle.

Several empirical expressions have been suggested for relating K_o with OCR during unloading phase (Vide Fig. 2.9), since K_o varies with OCR in this phase. These expressions have been reviewed by Ladd et al. (1977) and the one most favoured is that proposed by Schmidt (1966) and Alpan (1967).

$$K_o^{(oc)}/K^{(nc)} = (OCR)^n \qquad (2.21)$$

where n is a constant for a particular soil. Experience gained with thirty italian sedimentary clays (Schmidt, 1966) tested in undisturbed state indicate that

$$n = 0.46 \pm 0.06$$

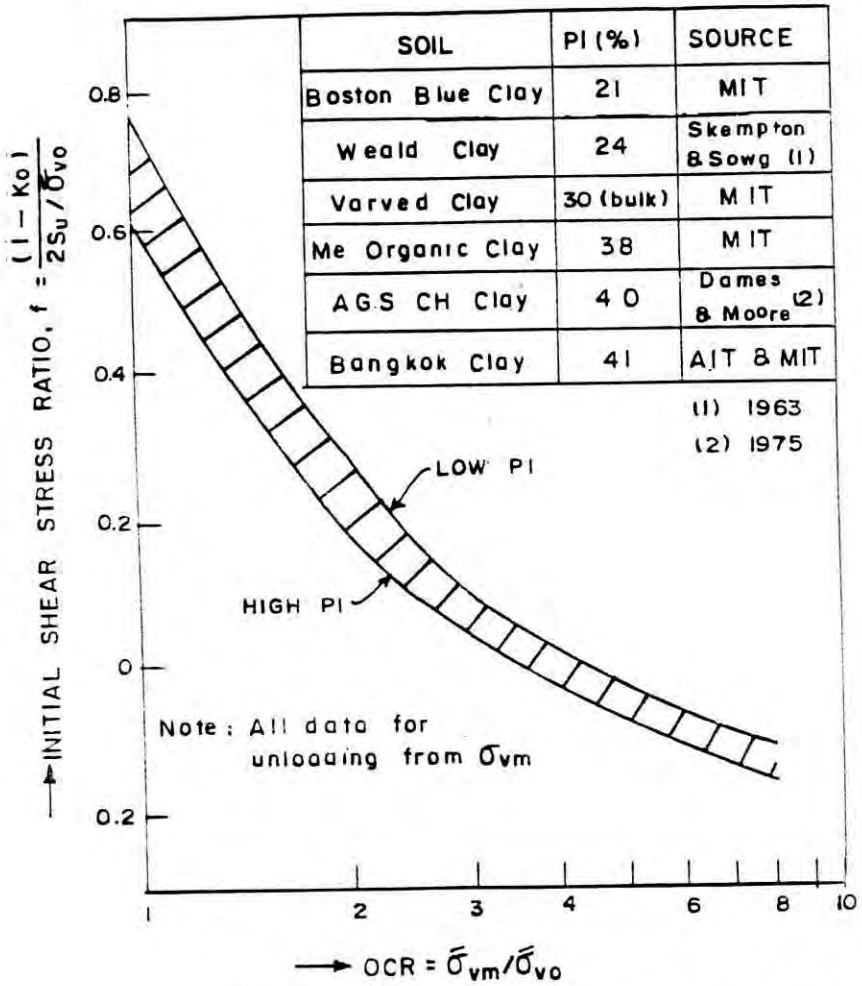
By using eq. (2.21) to express K_o as a function of OCR, eq. (2.10) to express $S_u^{(oc)}/\bar{\sigma}_{vo}$ as a function as a function of OCR and eq. (2.19), it follows that

$$f = \frac{1-[K_o^{(nc)} x OCR^n]}{2[S_u^{(nc)}/\bar{\sigma}_v] x (OCR)^{\wedge o}} \qquad (2.22)$$

The computed variation of f with OCR can be compared with experimental evidence in Fig. 2.26. The data are directly reproduced from the general report by Ladd et al. (1977) and refer to six different clays, some tested in triaxial compression and some in plane strain conditions.

2.9 Void Index and Intrinsic Compression Line Concept for Clay Materials

Burland (1990), in his 30th Rankine Lecture termed the properties of reconstituted clays as 'intrinsic' properties since they are inherent to the soil and independent of natural state. A reconstituted soil has been defined as one that has been thoroughly mixed at a moisture content equal to or greater than the liquid limit (w_L). The term 'intrinsic' has been used to describe the properties of clays which have been reconstituted at a water content of between w_L and 1.5 w_L and then consolidated



SOIL	PI (%)	SOURCE
Boston Blue Clay	21	MIT
Weald Clay	24	Skempton & Sowg (1)
Varved Clay	30 (bulk)	MIT
Me Organic Clay	38	MIT
AGS CH Clay	40	Dames & Moore (2)
Bangkok Clay	41	AIT & MIT

(1) 1963
(2) 1975

Note: $S_u = 0.5(\bar{\sigma}_1 - \bar{\sigma}_3)_f$ from CKoU triaxial or plane strain tests

FIG.2.26 VARIATION OF INITIAL SHEAR STRESS RATIO WITH OVERCONSOLIDATION RATIO (AFTER LADD ET AL, 1977)

$$I_V = \frac{e - e^*_{100}}{e^*_{100} - e^*_{1000}} = \frac{e - e^*_{100}}{C^*_c}$$

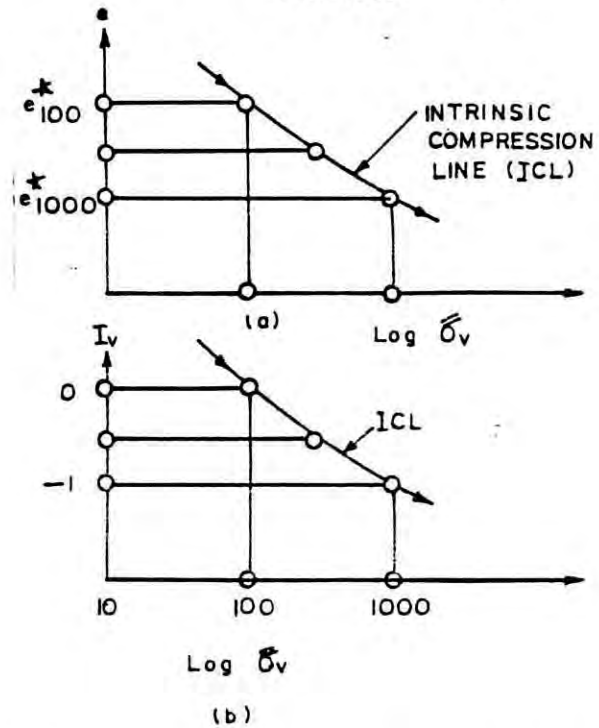


FIG. 2.27 THE USE OF VOID INDEX, I_V TO NORMALIZE INTRINSIC COMPRESSION CURVE (BURLAND, 1990)

under one-dimensional condition. The term 'intrinsic' was chosen since it refers to the basic or inherent properties of a given soil prepared in a specified manner and which are independent of natural state. An asterik is used to denote an intrinsic property.

The compressibility and strength characteristics of reconstituted clays are used as a basic frame of reference for interpreting the corresponding characteristics of natural sedimentary clays. The intrinsic properties provide a frame of reference for assessing the in-situ state of a natural clay and the influence of structure on its in-situ properties. For overconsolidated natural clays, the intrinsic compression line provides a useful means of assessing the degree of overconsolidation. Burland (1990) introduced a new normalizing parameter called the void index to aid in correlating the compression characteristics of various clays. One-dimensional compression curve for reconstituted natural clay is normalized by assigning fixed values to e^*_{100} and e^*_{1000}. The quantities e^*_{100} and e^*_{1000} are the intrinsic void ratio corresponding to consolidation pressure, $\bar{\sigma}_v$=100 kPa and $\bar{\sigma}_v$=1000 kPa. The normalizing parameter chosen has been defined as the void index, I_v such that,

$$I_v = \frac{e - e^*_{100}}{e^*_{100} - e^*_{1000}} = \frac{e - e^*_{100}}{C_c^*} \qquad (2.23)$$

The intrinsic compression index, C_c^* is defined as $e^*_{100} - e^*_{1000}$. Following Terjaghi (1925), the parameters e^*_{100} and C_c^* are called as the constant of intrinsic compressibility.

The compression curve shown in Fig. 2.27(a) may be transformed to the normalized curve in Fig. 2.27(b) where the void index, I_v defined by above equation (2.23) is the ordinate. When $e=e^*_{100}$, then I_v=0 and when $e=e^*_{1000}$, then I_v=-1. The void index may be

thought of as a measure of the intrinsic compactness of a sediment. When I_v is less than zero, the sediment is compact and when I_v is greater than zero, the sediment is loose.

It has been seen that a reasonably unique line is achieved when void index, I_v versus $\log\bar{\sigma}_v$ is plotted for clays covering a wide range of liquid limits and of pressures. This line has been termed as intrinsic compression line (ICL). Burland (1990) has given a cubic equation as follows which may represent the co-ordinates of the ICL with sufficient accuracy

$$I_v = 2.45-1.285x+0.015x^3 \qquad (2.24)$$

where $x = \log\bar{\sigma}_v$ in kPa

The intrinsic compression line may either be measured directly for a clay or if the values of e^*_{1000} and C_c^* are known for the clay, the ICL may be constructed using the above equation (2.24). If it is required to plot the ICL in terms of e versus $\log \bar{\sigma}_v$, then the values of e corresponding to various values of $\log \bar{\sigma}_v$ may be obtained from equation (2.23)

$$e = I_v C^*_c + e^*_{100} \qquad (2.25)$$

Where, again, the values of I_v may be obtained from equation (2.24).

Here to refer that a more fundamental parameter than liquid limit (w_L) is the void ratio corresponding to the liquid limit (e_L) for a clay. Two clays may have same liquid limit but may posses two different e_L due to their different specific gravity values. At any given value of $\bar{\sigma}_v$, the void ratio is related to e_L increasing as e_L increases.

2.10 The Properties of Dhaka Clay

Serajuddin and Ahmed (1967) investigated and furnished data on physical and engineering properties of soils of Bangladesh. His study was for soils upto depth of 20 feet. From those data, it reveals that Dhaka soil consists of mainly clay or a mixture of clay and silt. Soil classification by unified system was CL and CH mainly. Most of their samples of Dhaka soil fall above A-line of Cassagrande plasticity chart. Similar sort of information on Dhaka clay can be found from the soil profile data reported by Eusufzai (1967).

Bore charts of Dhaka city show that the top clay layer which exists above sand layer posses an approximate thickness of 25 to 30 feet. Index and other properties of the clay are summarized in Table 2.2 which were found from different soil reports such as sub-soil investigation of proposed Bishal Bhaban, Maghbazar, Dhaka, soil testing reports of a number of proposed residential buildings, Gulshan, Dhaka, sub-soil investigation reports of housing and building research institute for proposed buildings at Mirpur, Dhaka.

The reports furnished by Safiullah (1977), Islam (1980) and Ameen (1985) indicate that Dhaka soil deposits are slightly to heavily Overconsolidated or at least have an overconsolidated crustal stratum. Their data yielded straight lines with the plot of undrained shear strength ratio versus OCR. The soil studied by Ameen (1985) was collected from the campus of Bangladesh Unviersity of Engineering & Technology, Dhaka from a depth of five feet from ground level. He reported the values of different geotechnical parameters of the clay which are presented in Table 2.3. Ameen (1985) found linear relationship between undrained shear strength to reduced consolidation pressure, S_u/σ_{vo} and overconsolidation ratio, n in log-log scale. The plot is shown in

TABLE 2.2: INDEX AND OTHER PROPERTIES OF DHAKA CLAY

Sl.No.	Properties	Generally vary from
1	Liquid limit	39% to 50%
2	Plastic limit	18% to 25%
3	Plasticity index	18 to 29
4	Clay content (less than 2 micron)	15% to 35%
5	Sand content	0% to 11%
6	Silt content	65% to 85%
7	Water content	17% to 37%
8	Co-efficient of consolidation	0.14 to 0.34
9	Soil classification under unified classification system	CL and CH

TABLE 2.3: GEOTECHNICAL PARAMETERS OF DHAKA CLAY (AFTER AMEEN, 1985)

Sl. No.	Parameter	Value
1	Isotropic compression index [$C_c^{(iso)}$]	0.27
2	Isotropic swelling index [$C_s^{(iso)}$]	0.04+0.01
3	K_o compression index [$C_c^{(K_o)}$]	0.27
4	K_o swelling index [$C_s^{(K_o)}$]	0.02+0.005
5	Critical-state pore pressure parameter, $\Lambda_o^{(K_o)}$	0.907
6	Critical-state pore pressure parameter, $\Lambda_o^{(iso)}$	0.87
7.	Λ_o for natural soil	0.958
8	K_o value at normally loaded state	0.46
9	K_o value at preloaded state (from OCR = 1.2 to 12)	0.512 to 3.23
10	Undrained secant modulus [$E_u(50)/S_u$] for K_o consolidated soil at normally loaded state	192
11.	Undrained secant modulus [$E_u(50)/S_u$] for isotropically consolidated soil at normally loaded state	165
12.	Undrained secant modulus [$E_u(50)/S_u$] for K_o consolidated soil at preloaded state (OCR from 2 to 24)	100 to 182
13.	Undrained secant modulus [$E_u(50)/S_u$] for isotropically consolidated soil at preloaded state	109 to 142
14.	S for K_o consolidated soil	0.19
15.	S for isotropically consolidated soil	0.30

Fig. 2.28. In this figure, the undrained shear strength ratio, $S_u/\bar{\sigma}_{vo}$ for overconsolidation ratio 1 is the intercept of the curve and the slope of the line is represented by Λ_o. For isotropically consolidted samples, the intercept is 0.3 and the slope is 0.746. For K_o consolidated sample, the intercept is 0.19 and the slope is 0.83. Ameen (1985) used this plot as model to predict the undrained shear strength of the clay for both K_o and isotropically consolidated samples.

Ameen (1985) replotted the Fig. 2.28 and showed the results of undisturbed undrained shear strength of natural Dhaka clay in Fig. 2.29. The undrained shear strength data had been collected from Safiullah (1977) and Islam (1980). In Fig. 2.29, it can be seen that although there is a scatter, some amount of agreement is observed between prdicted and measured undrained shear strength.

Ameen (1985) presented a plot of $S_{uWL}/\bar{\sigma}_{vo}$ (normalized undrained shear strength) versus overconsolidation ratio in semi-log scale which is shwon in Fig. 2.30. Similar normalized plot was also drawn by Mahar and O'Neill (1983) for desiccated clays. From this figure, it can be observed that the curve for isotropically consolidated Dhaka clay is very close to the normalized curve drawn by them. Data points for natural Dhaka clay are also shown in this figure. These points are close to the curve shown by Mahar and O'Neill (1983). The curve for K_o consolidated samples is different but parallel to the curve drawn by them. This probably answers an important question regarding the mechanism through which Dhaka clay has been overconsolidated. Ameen (1985) commented that the clay has been overconsolidated might be through the process of desiccation.

Ameen (1985) determined the co-efficient of lateral earth pressure at rest (K_o) experimentally from special consolidation test. He found the value of K_o equal to 0.46 for Dhaka clay in

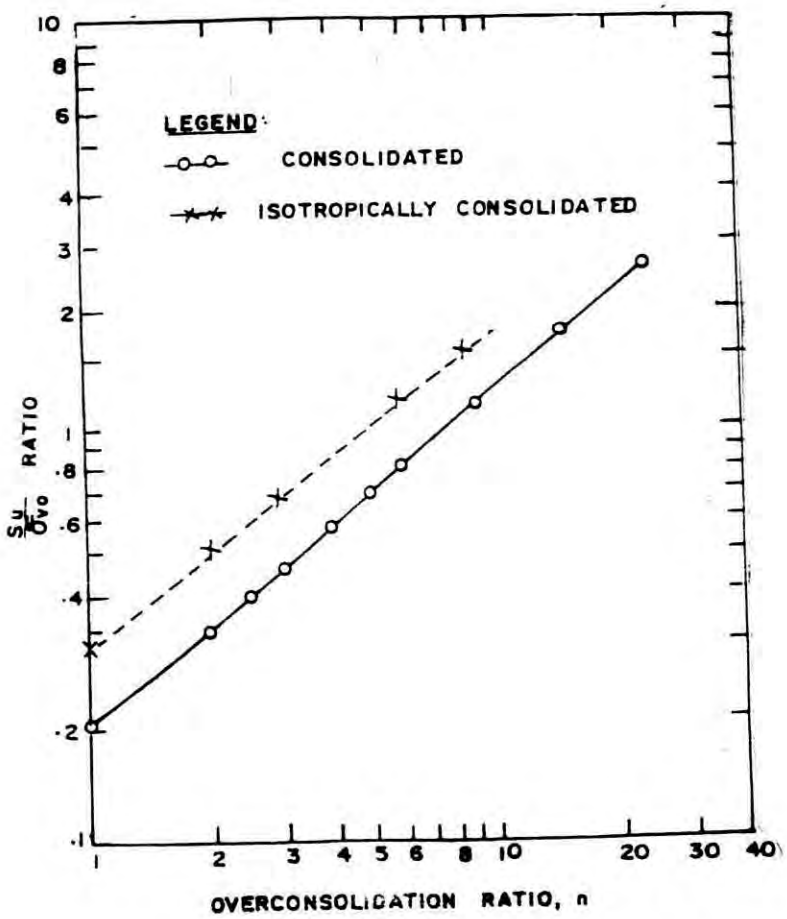


FIG. 2.28 VARIATION OF NORMALIZED SHEAR STRENGTH WITH OVERCONSOLIDATION RATIO FOR DHAKA CLAY (AMEEN, 1985).

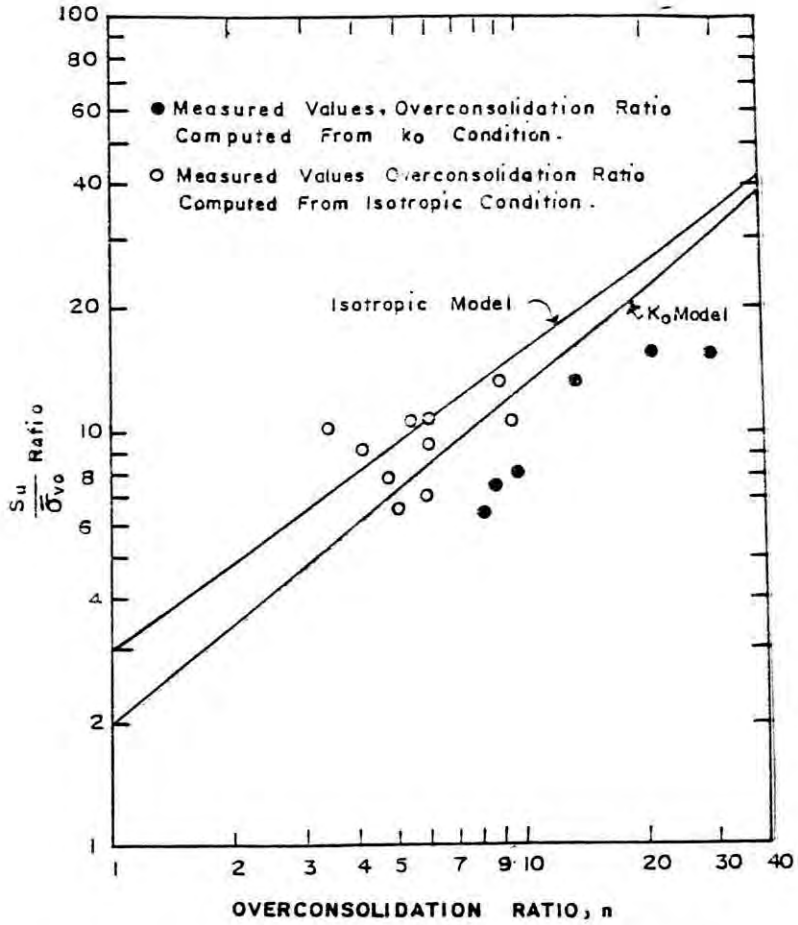


FIG. 2.29 COMPARISON OF MEASURED VALUES OF UNDRAINED SHEAR STRENGTH WITH THOSE PREDICTED FROM THE MODEL (AMEEN, 1985).

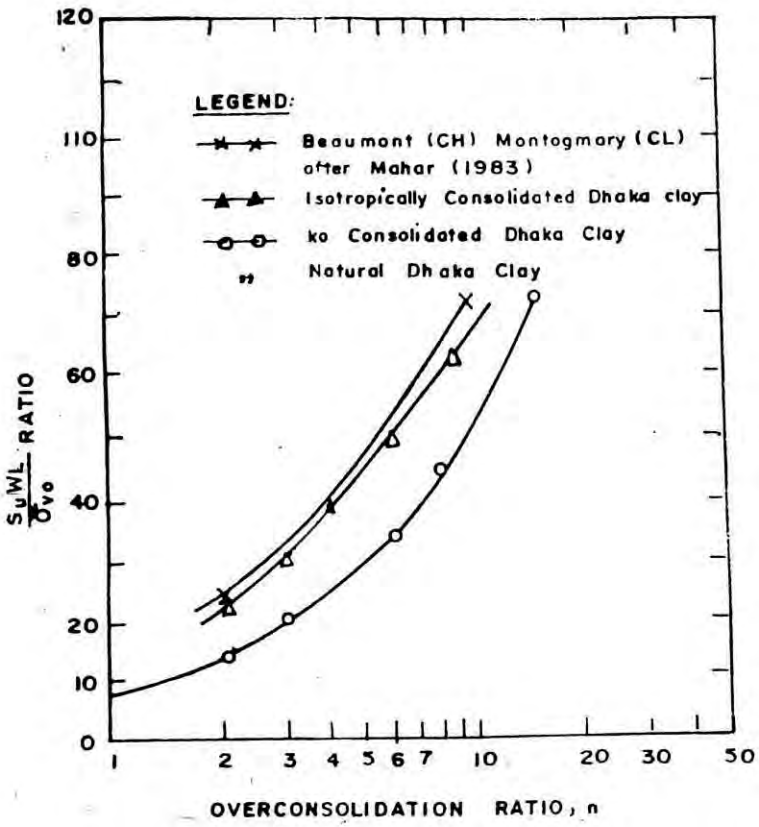


FIG.2.30 GENERAL NORMALIZED UNDRAINED SHEAR STRENGTH VERSUS OVERCONSOLIDATION RATIO (AMEEN, 1985).

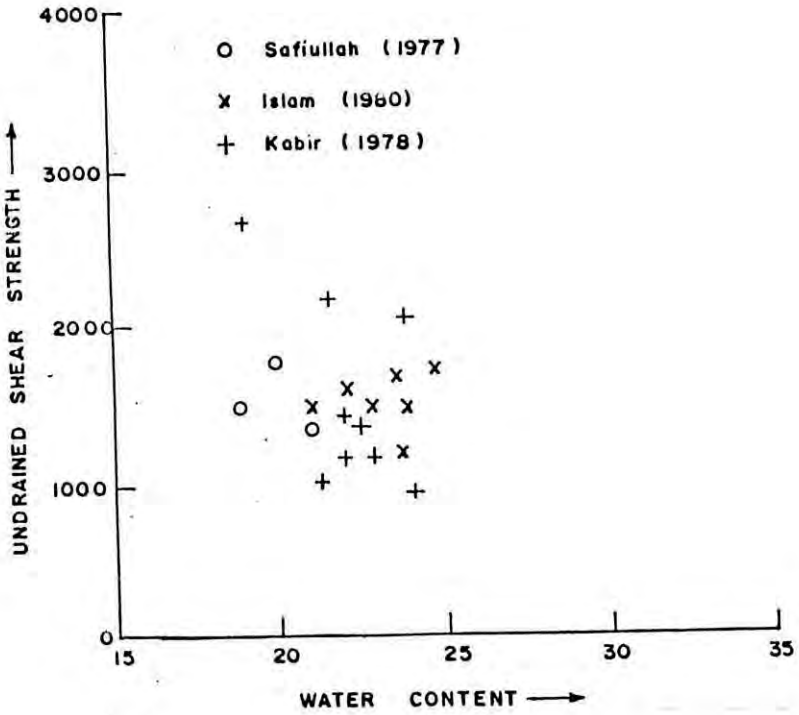


FIG.2.31 RELATIONSHIP BETWEEN UNDRAINED SHEAR STRENGTH AND WATER CONTENT FOR NATURAL OCCURING DHAKA CLAY.

normally loaded state and increases with the increase in overconsolidation ratio to 3.23 at a overconsolidation ratio of 12.

Undrained shear strength versus water content had been plotted from data of naturally occurring Dhaka clay (Fig. 2.31). These data were collected after Safiullah (1977), Kabir (1978) and Islam (1980). The plot showed that no definite relationship existed between the water content and shear strength. It is known that, at the same water content, undrained shear strength may be different at normally loaded state and preloaded state. Thus it shows that Dhaka soil deposits are overconsolidated to various degree.

2.11 Summary

The main points of the above literature review can be summarized as follows:

1) A convenient way to characterize the stress-strain behaviour of a clay is with its undrained modulus. In general, the modulus of a soil increases with an increase in consolidation pressure, OCR, aging and strain rate but decreases with an increase in deviator stress and soil disturbance.

2) Undrained Young's modulus can be given by the equation $E_u = \Psi S_u$, where Ψ decreases with an increase of OCR, plasticity index, organic content of the soil. Various investigators reported that the value of Ψ varies from 500 to 140.

3) For normally consolidated samples or for overconsolidated samples with same past stress history and for same stress system applied during shear, firstly, there is an unique relationship between the strength and effective stress at failure and

secondly, there is another unique relationship among water content, shear stress and effective stress.

4) With same water content, the undrained shear strength may be different for normally consolidated clay and overconsolidated clay (Henkel, 1959).

5) The $S_u/\bar{\sigma}_v$ ratio developed by Skempton (1957) can be utilized to predict the undrained shear strength of a normally consolidated clay deposit.

6) The arithmatic plot of B/A [where $B = S_u^{(oc)}/\bar{\sigma}_{vo}$ and $A = S_u^{(nc)}/\bar{\sigma}_{vo}$] versus OCR can be used to predict undrained shear strength of any clay (Ladd et al., 1977).

7) The critical-state concepts can be used to represent undrained strength characteristics of overconsolidted clays. The critical-state parameter is independent of OCR, K_o and level of shear to failure. So it requires only two basic soil constant ($\bar{\phi}$ and Λ_o) in order to predict undrained shear strength. The results of CIU and CK_oU tests are used to determine the critical-state pore pressure parameter, Λ_o.

$$\Lambda_o = \frac{\mathrm{Log}[S_u^{(oc)}/\bar{\sigma}_{vo}] - \mathrm{Log}[S_u^{(nc)}/\bar{\sigma}_v]}{\mathrm{Log}[OCR]}$$

8) Normalized soil parameters can be used for design purposes. The log-log plot give straight lines and hence is closely approximated by the relationship

$$\frac{S_u^{(oc)}/\bar{\sigma}_{vo}}{S_u^{(nc)}/\bar{\sigma}_v} = (OCR)^m$$

9) Using cam-clay theory, the following relation can be developed for undrained shear strength ratio at normally loaded state.

$$\frac{S_u(nc)}{\bar{\sigma}_v} = \frac{3 \sin\bar{\phi}}{(3-\sin\bar{\phi})(\exp \Lambda_o)}$$

10) Based on experimental results, Mitachi and Kitago (1976) developed the following relation for undrained strength ratio for any clay.

$$\frac{S_u(oc)}{\bar{\sigma}_{vo}} = \frac{S_u(nc)}{\bar{\sigma}_v} (OCR)^{1-\lambda}$$

11) The in-situ stress state has a major effect on the magnitude of subsequent stress change of a soil element that can sustain before local yield or failure is employed. Davis and Poulos (1960) introduced a convenient way of representing this effect in dimensionless form by means of the initial shear stress ratio, f which can be defined as

$$f = \frac{1 - K_o}{2S_u/\bar{\sigma}_{vo}}$$

12) Using the critical-state cam-clay theory, Mayne (1980) has developed a simple and useful model to predict the undrained shear strength ratio of overconsolidated clay. the model equation si as follows:

$$\frac{S_u(oc)}{\bar{\sigma}_v} = \frac{3 \sin\bar{\phi}}{3-\sin\bar{\phi}} \times \frac{OCR^{\Lambda_o}}{\exp \Lambda_o} -$$

13) Burland (1990) furnished the concept of void index, intrinsic compression line (ICL), intrinsic swelling line (ISL), sedimentation compression line (SCL) etc. and referred that compressibility and strength of reconstituted clays can be used as basic frame of reference for interpreting corresponding characteristics of natural clays.

14) Ameen (1985) plotted undrained shear strength ratio (i.e S_u/σ_{vo}) versus log OCR for Dhaka clay which showed linear relations and proposed to use this relation for prediction of undrained shear strength of Dhaka clay.

CHAPTER 3
THE RESEARCH SCHEME

3.1 Introduction

It has been shown in the literature review that for a clayey soil a number of unique relations exists between water content and consolidation pressure, water content and undrained shear strength, void ratio and consolidation pressure, undrained shear strength and consolidation pressure, undrained shear strength and OCR, and undrained secant modulus and OCR. Engineering properties of a clay are affected in a complex way by the composite effects of the above interacting and interrelated factors. This complex behaviour is simplified upto required extent by using certain rational assumptions. Some of the relationships mentioned in the literature review can be used to evaluate important properties of a clay such as undrained shear strength, compressibility, undrained elastic modulus etc. It has already been demonstrated in Chapter 2 that the rate of increase of shear strength due to increase of overconsolidation ratio, the phenomenon of increase in shear strength and decrease in compressibility due to increase in overconsolidation ratio, etc. depend on many factors such as stress history, stress sytem, environmental conditions under which the clay layer is consolidated.

The present research is aimed at evaluating appropriate geotechnical parameters for reconstituted Dhaka clay so that these can be used to estimate and predict essential properties such as undrained shear strength, undrained elastic modulus, compressibility, initial shear stress ratio, etc. for this clay from a measurement of some simple quantities such as water content, void ratio, overburden pressure, etc. Since normal loading and preloading process produce different characteristics of a clay, this study looks into two aspects, one for normally

loaded state, and another for preloaded state. The increase of shear strength due to increase of OCR is also considered as an aspect of study in this research.

3.2 Objective of the Research

The research program is directed at assessing geotechnical parameters of reconstituted Dhaka clay and to find out a model for prediction of undrained shear strength of this clay consolidated under K_o and isotropic stress system at normally loaded and preloaded states. This includes study of applicability of the Mitachi and Kitago's (1976) or Mayne's model (1980) for this clay. In this respect, parameters such as critical state pore pressure parameter (Λ_o), effective stress friction angle ($\bar{\phi}$), undrained shear strength ratio(s), compression and swelling indices (C_c and C_s), undrained elastic modulus (E_u) etc. are to determine for the reconstituted Dhaka clay. The stress-strain-strength characteristics and interacting relationships among undrained elastic modulus, OCR, equilibrium water content, void ratio, consolidation pressure, undrained shear strength parameters of Dhaka clay are also required to be studied. These relationships would yield informations about critical-state pore pressure parameter (Λ_o), undrained shear strength ratio,s (i.e. $S_u/\bar{\sigma}_v$) etc. and would be used in the strength model. So in summary, this research program has two-fold objectives:

1) To find out an applicable model to predict undrained shear strength for Dhaka clay.

2) To obtain strength and compressibility data for reconstituted Dhaka clay under various loading and stress condition (i.e. stress hisotry).

The second objective would involve study of relationships shown in Table 3.1.

TABLE 3.1: GEOTECHNICAL PARAMETERS TO BE STUDIED FOR DHAKA CLAY

Sl. No.	Parameters		Stress condition	Stress history	Type of relationships to be studied
	Notation	Description			
1	$(\bar{\sigma}_1-\bar{\sigma}_3)$ vs. ϵ	Deviatoric stress versus axial strain	K_o and isotropic consolidation	n.c. and o.c states	To determine E_u and S_u and to find out the characteristics of stress-strain curves
2	Log $E_u(50)$ vs. Log $\bar{\sigma}_v$ or $\bar{p}$	Undrained secant modulus versus consol. pressure	"	"	To determine the behaviour of Young's modulus with consolidation pressure
3	$E_u(50)/\bar{\sigma}_{vo}$ or $\bar{p}_o$ vs. log OCR	Normalized undrained secant modulus versus OCR	"	o.c state	To determine the behaviour of normalized modulus with OCR
4	Ψ vs. log OCR	Normalized undrained secant modulus versus OCR	"	"	To find the characteristics of and to compare with other clays
5	e vs. log $\bar{\sigma}_v$ or p	Void ratio versus consolidation pressure	"	n.c and o.c state	To find the compression and swelling indices and value of Λ_o
6	w vs. log $\bar{\sigma}_v$ or $\bar{p}$	Water content versus consolidation pressure	K_o and isotropic consol.	"	To find OCR from water content and overburden pressure

TABLE 3.1 (CONTD....)

Sl. No.	Parameters		Stress condition	Stress history	Type of relationship to be studied
	Notation	Description			
7	w vs. log S_u	Water content versus undrained shear strength (S_u)	K_o and isotropic consoli-dation	n.c. and o.c. states	To predict S_u of n.c clay from water content only
8	e vs. log S_u	Void ratio versus undrained shear strength (S_u)	"	"	To predict S_u of n.c. clay from void ratio only
9	S_u vs. $\bar{\sigma}_v$ or $\bar{p}$ and S_u vs. $\bar{\sigma}_{vo}$ or $\bar{p}_o$	Undrained shear strength versus consolidation pressure	"	"	To determine undrained shear strength ratio for n.c. clay and to determine Hvorslev's parameter
10	S_u vs. OCR	Undrained shear strength vs. OCR	"	o.c. state	To find nature of variation of S_u for individual reduced swelling pressure
11	$S_u/\bar{\sigma}_{vo}$ or $\bar{p}_o$ vs. OCR	Undrained shear strength ratio vs. OCR	"	"	To predict S_u of o.c clay
12	($S_u/\bar{\sigma}_{vo}$ or $\bar{p}_o$) Vs. log OCR	Undrained shear strength ratio vs. log OCR	"	"	To predict S_u of the o.c. clay from curve
13	Log($S_u/\bar{\sigma}_{vo}$ or $\bar{p}_o$) vs. log OCR	log undrained shear strength ratio vs. log OCR	"	"	To find value of A_o and to predict S_u of o.c. clay

TABLE 3.1 (CONTD....)

Sl. No.	Parameters		Stress condition	Stress history	Type of relationship to be studied
	Notation	Description			
14	τ vs. σ	Shear stress versus normal stress	Isotropic consolidation	n.c. state	To determine effective stress friction angle
15	α vs. OCR	$\alpha = [S_u^{(oc)}/\bar{\sigma}_{vo}]/[S_u^{(nc)}/\bar{\sigma}_v]$ vs.OCR	K_o and isotropic consol.	o.c state	To predict S_u of the o.c clay
16	μ vs. OCR	$\mu = S_u^{(oc-\bar{\sigma}_{vo} \text{ or } \bar{p}o)}/S_u^{(nc-\bar{\sigma}_{vo} \text{ or } \bar{p}o)}$ vs. OCR	"	"	To study rate of increase of S_u of o.c. Dhaka clay
17	f vs. log OCR	Initial shear stress ratio, $f=(1-K_o)/[2S_u/\bar{\sigma}_{vo}]$ vs. OCR	K_o consolid.	"	To study variation of f of Dhaka clay with OCR
18	I_v vs. log $\bar{\sigma}_v$	Void index versus consolidation pressure	"	n.c. state	To find ICL and ISL for Dhaka clay

3.3 Soil Sample Used

The soil samples for this investigation were collected from a depth of six meters from an excavation site at Maghbazar, Dhaka. From bore-hole records and from inspection of open cuts nearby it was established that the sub-soil at this location is more or less uniform in the upper layer.

The deposit seems to be well oxidized and reddish brown or tan in colur. Samples were collected by digging from an open cut and some quantity preserved in polythene bags for subsequent laboratory investigations. The consistency can be attributed as medium to stiff. The soil may be generally termed as the 'Older Alluvium' deposit or 'Pleistocene Madhupur deposit' at Dhaka or locally called the 'Red Dhaka Clay'.

The details of the physical and index properties of this soil are given in Chapter 6. A uniform slurry prepared from this soil was subjected to various loading conditions to obtain the earlier mentioned parameters.

3.4 The Experimental Scheme

A remoulded soil in form of slurry was prepared by mixing soil with water at approximately 1.5 times its liquid limit. The remoulded soil was consolidated in a Rowe consolidation cell to make a soil cake. At the end of consolidation, the soil cake was extruded and a total of 93 cylindrical samples were prepared. These were further consolidated to K_o and isotropical conditions and to either normally loaded or overconsolidated states before applying deviator stress. Each sample had a diameter of 38.1 mm (1.5 in) and height of 76.2 mm (3 in.). The samples were sheared in triaxial cell under either drained or undrained conditions. A list of different tests performed on the soil samples prepared as above is shown in Table 3.2.

TABLE 3.2: LIST OF RECONSTITUTED SAMPLES TESTED UNDER DIFFERENT LOADING CONDITIONS

Stress condition after sampling from Rowe cell	Stress history	Total nos. of K_o consolidated samples	Total nos. of isotropically consolidated samples	Total nos.of triaxial tests	Drainage condition during triaxial compression test
K_o consolidated	Normally consolidated	6	-	18	Undrained
	Overconsolidated	17	-	51	"
Isotropically consolidated	Normally consolidated	4	12	12	"
	Overconsolidated	4	9	9	"
	Normally consolidated	3	3	3	Drained

Note: All samples are initially K_o consolidated in Rowe cell.

A flow chart for the experimental program is shown in Fig. 3.1. As mentioned earlier, the soil slurry was made by mixing soil water thoroughly in a rotary mixer. The details of the preparation of soil specimen and testing procedures are given in Chapter 4 and 5.

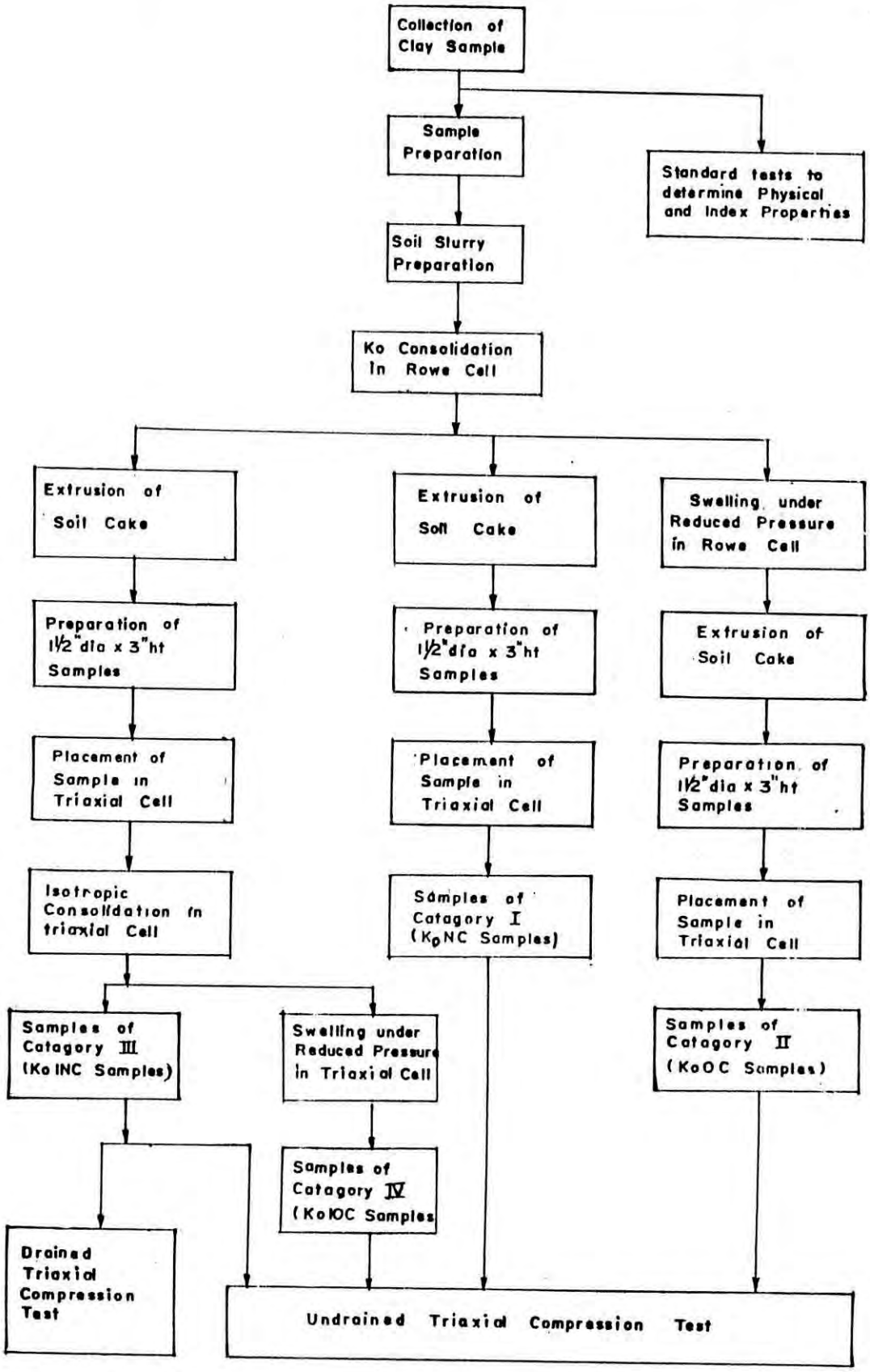


FIG. 3.1 FLOW CHART FOR TEST PROGRAM

CHAPTER 4
EQUIPMENTS USED FOR THE EXPERIMENTAL INVESTIGATION

4.1 General

In order to study the consolidation and shear strength behaviour of Dhaka clay under different stress conditions and at various overconsolidation ratios, an experimental program was developed in line with the research scheme described in Chapter 3. The experiments required design, construction and fabrication of some equipments and accessories that were not available in the Geotechnical Engineering Laboratory of the Bangladesh University fof Engineering & Technology, Dhaka. This chapter describes the details of apparatus used and equipments that were fabricated for this research.

4.2 Equipments Used

For a comparative study of compressibility and shear strength characteristics, a number of identical soil samples having same size and height [such as cylindrical samples of 38.1 mm (1.5 in) dia and 76.2 mm (3 in) height] were required. These were made from a soil cake obtained by consolidating a soil slurry under K_o condition. For soil slurry preparation, a rotary mixter was used. A Rowe cell (Rowe, 1967) was needed for consolidating the soil slurry under K_o condition. In addition to Rowe consolidation cell, air-water cylinder, dash pot and control cylinder system, pressure control valve, air compressor etc. were also needed. Initially a few samples were prepared by using conventional oedometers. Since the laboratory did not have a large diameter Rowe type (Rowe, 1967) cell, this was fabricated locally.

4.2.1 Apparatus Available for K_o Consolidation

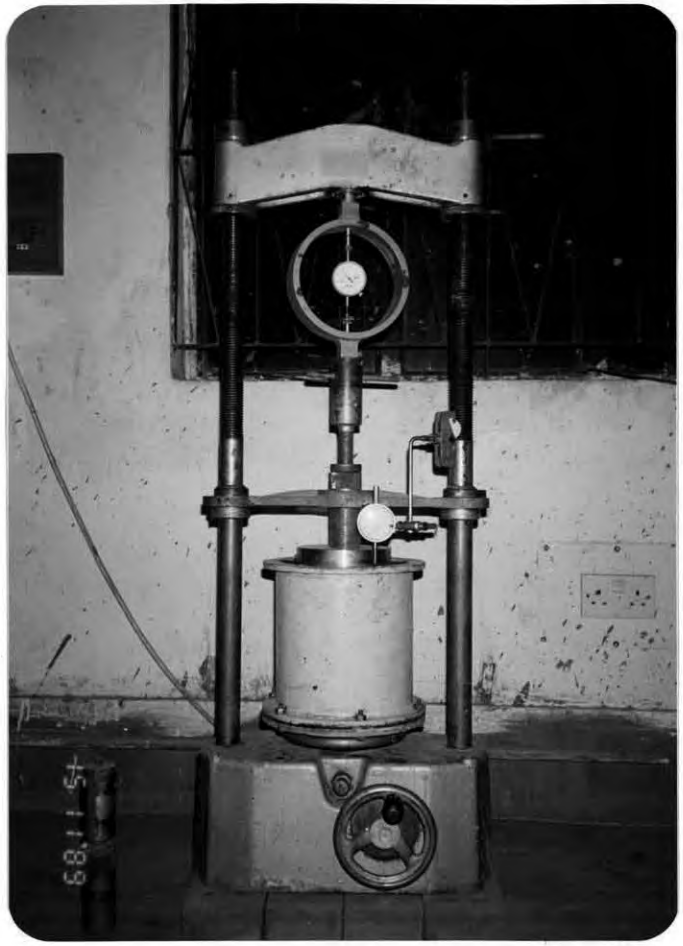
Two types of loading frame (Unit I, Unit II and Unit III) shown in Fig. 4.1 (a & b) and Fig. 4.2 were available for K_o consolidation in the Geotechnical Engineering Laboratory of the Bangladesh University of Engineering & Technology, Dhaka. Thus, in the laboratory, two types of loading arrangements could be made with the existing apparatus for a K_o consolidation test.

1) Loading frame with proving ring and loading ram device [Fig. 4.1 (a & b)].

2) A platform type loading frame with arrangements for measuring weight through a lever arm device (Fig. 4.2).

In the first case, the steel mould containing soil sample is placed on the platform [Fig. 4.1 (a & b)]. The platform is raised manually by rotating a wheel and thus loading the soil sample through a loading ram and proving ring. In this process, continuous raising of platform by manual operation is required to adjust with the deformation i.e. to maintain required pressure on soil sample. The deformation of the proving ring is measured by a dial gauge that gives the load imposed at any time. In the second case, the mould containing soil sample is placed on a platform type balance (Fig. 4.2). Load on soil is measured by the weight lever arm indicator. In this case, also, continuous manual rotation of wheel is required to adjust with deformation i.e. to maintain required pressure on the soil sample.

In both devices, there are a number of disadvantages. Above types of arrangement for loading may produce non-uniform pressure if the plate at the top of soil sample is not sufficiently rigid. The loading arrangement is clumsy or ill-contrived. If the top surface of the soil sample is uneven, then the imposed pressure may not be uniform over the entire area of the soil sample. So



(a) UNIT I

(b) UNIT II

FIG. 4.1 LOADING FRAME ARRANGEMENTS FOR K_o CONSOLIDATION OF SOIL SLURRY.

FIG. 4.2 PLATFORM BALANCE ARRANGEMENT FOR K_0 CONSOLIDATION (UNIT III)

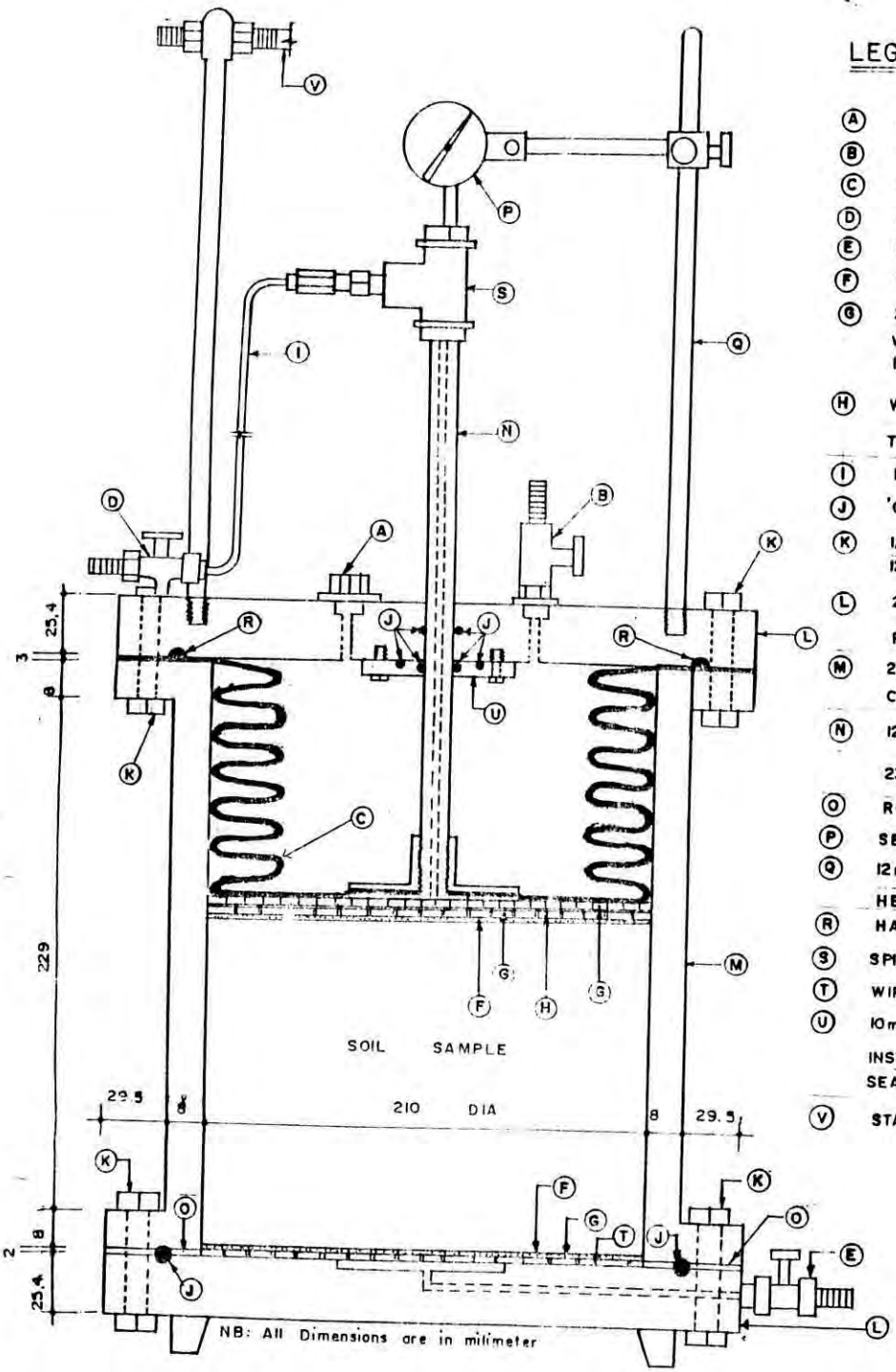
lesser or even zero pressure might be exerted in the depressed zone. Above types of loading arrangemnt require continuous manual adjustment in order to maintain required pressure on the soil sample.

4.2.2 The Rowe Consolidation Cell

To eleminate the disadvantages with the loading frame described above, a Rowe consolidation cell was used for K_o consolidation. In the Rowe cell the top of soil sample is loaded by a convoluted rubber membrane subjected to hydraulic pressure. The use of this convoluted rubber bellow makes it possible to provide uniform pressure over the soil even if the settled surface is uneven. Application of hydrostatic pressure does not require any manual adjustment of the loading arrangement as with continuous squeezing of soil compared to that needed in the case of loading frame described earlier.

A Rowe consolidation cell was, therefore, considered necessary for this research work. As the Geotechnical Engineering Laboratory of BUET, Dhaka did not posses, a Rowe cell of the required diameter, it was made locally. The salient features of this developed experiment is shown in Fig. 4.3, 4.4 and 4.5. Fig. 4.6 shows a photograph of the cell.

The Rowe cell is connected either to a dash pot and control cylinder system or to a air-water cylinder to provide constant hydraulic pressure within convoluted rubber bellows. The Fig. 4.7 shows dash pot and control cylinder arrangement for providing hydraulic pressure to Rowe cell. Alternately, the Rowe cell can be connected to air-water cylinder to provide desired constant hydrostatic pressure to the Rowe cell. The air-water cylinder connected to a air compressor through a pressure regulator. This alternative set-up of air compressor and air-water cylinder to Rowe cell is shown in Fig. 4.8.



LEGEND OF FIGURE-4.3

- (A) AIR VENT PLUG
- (B) CONSTANT PRESSURE SUPPLY LINE
- (C) CONVOLUTED RUBBER BELLOWS
- (D) TOP DRAINAGE CONTROL VALVE
- (E) BOTTOM DRAINAGE CONTROL VALVE
- (F) FILTER PAPER
- (G) 207mm DIA PERFORATED DISC WITH THICKNESS 6mm HAVING 1.5mm DIA HOLES @ 3mm C/C
- (H) WIRE MESH IN BETWEEN THE TWO TOP DISCS
- (I) FLEXIBLE TUBE
- (J) 'O'- RING SEAL
- (K) 1/2 INCH DIA STAINLESS STEEL BOLTS 12 NOS. AT TOP AND 6 NOS. AT BOTTOM
- (L) 285mm DIA AND 25.4mm THICK BRASS PLATE
- (M) 210mm DIA AND 245mm HEIGHT CYLINDRICAL FLANGED STEEL MOULD
- (N) 12mm DIA HOLLOW BRASS SPINDLE 230mm LONG
- (O) RUBBER GUSKET
- (P) SETTLEMENT DIAL GAUGE
- (Q) 12mm DIA BRASS STAND HAVING HEIGHT OF 300mm
- (R) HALF ROUND 'O' RING SEAL
- (S) SPINDLE HEAD
- (T) WIRE MESH AT THE BOTTOM OF MOULD.
- (U) 10mm THICK AND 75mm DIA PLATE INSTALLED TO HOLD EXTRA 'O' RING SEAL
- (V) STAINLESS STEEL LIFTING HANDLE

FIG.4.3 THE ROWE CONSOLIDATION CELL

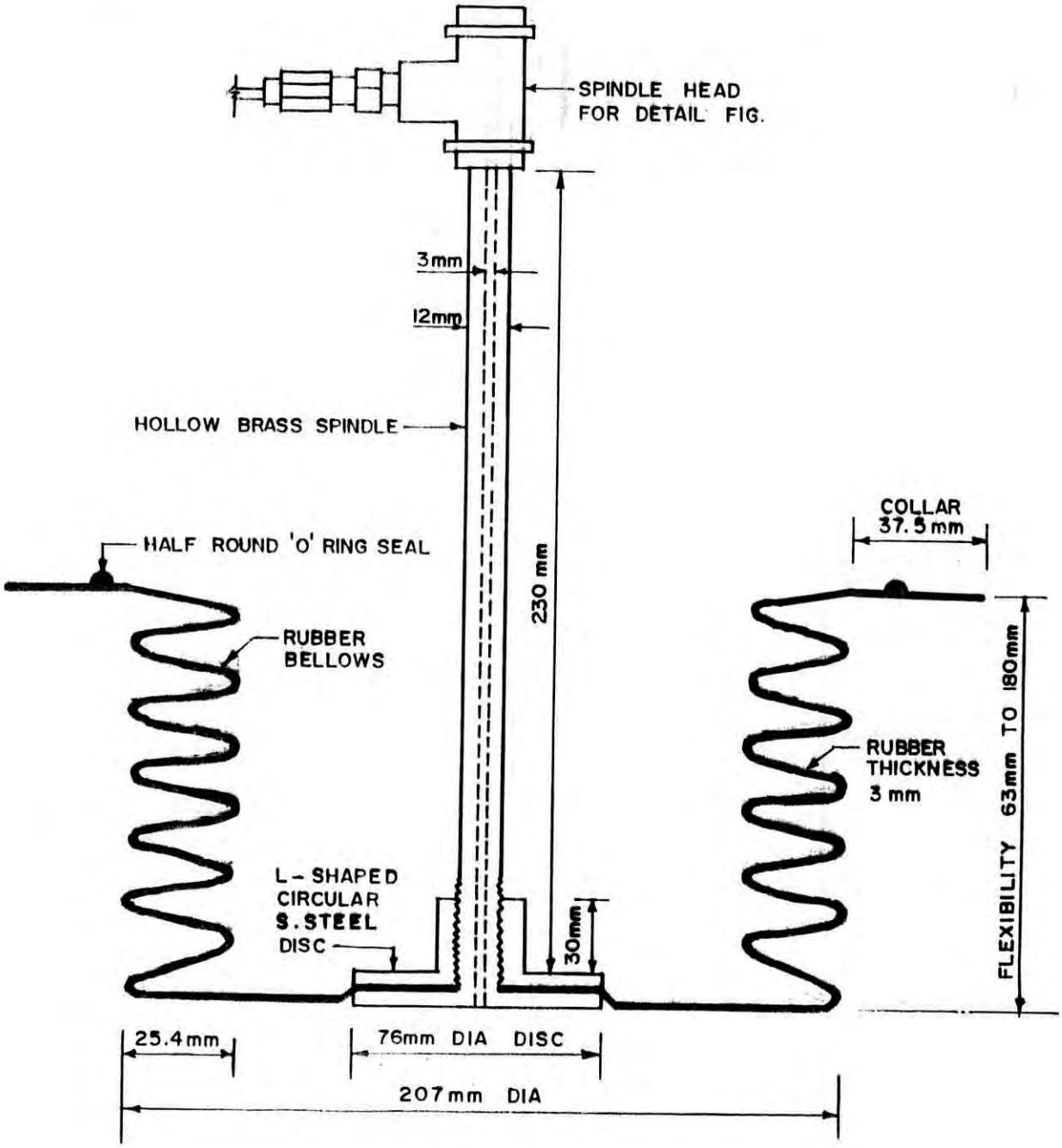


FIG. 4.4 CIRCULAR CONVOLUTED RUBBER BELLOWS AND RELEVANT ACCESSORIES.

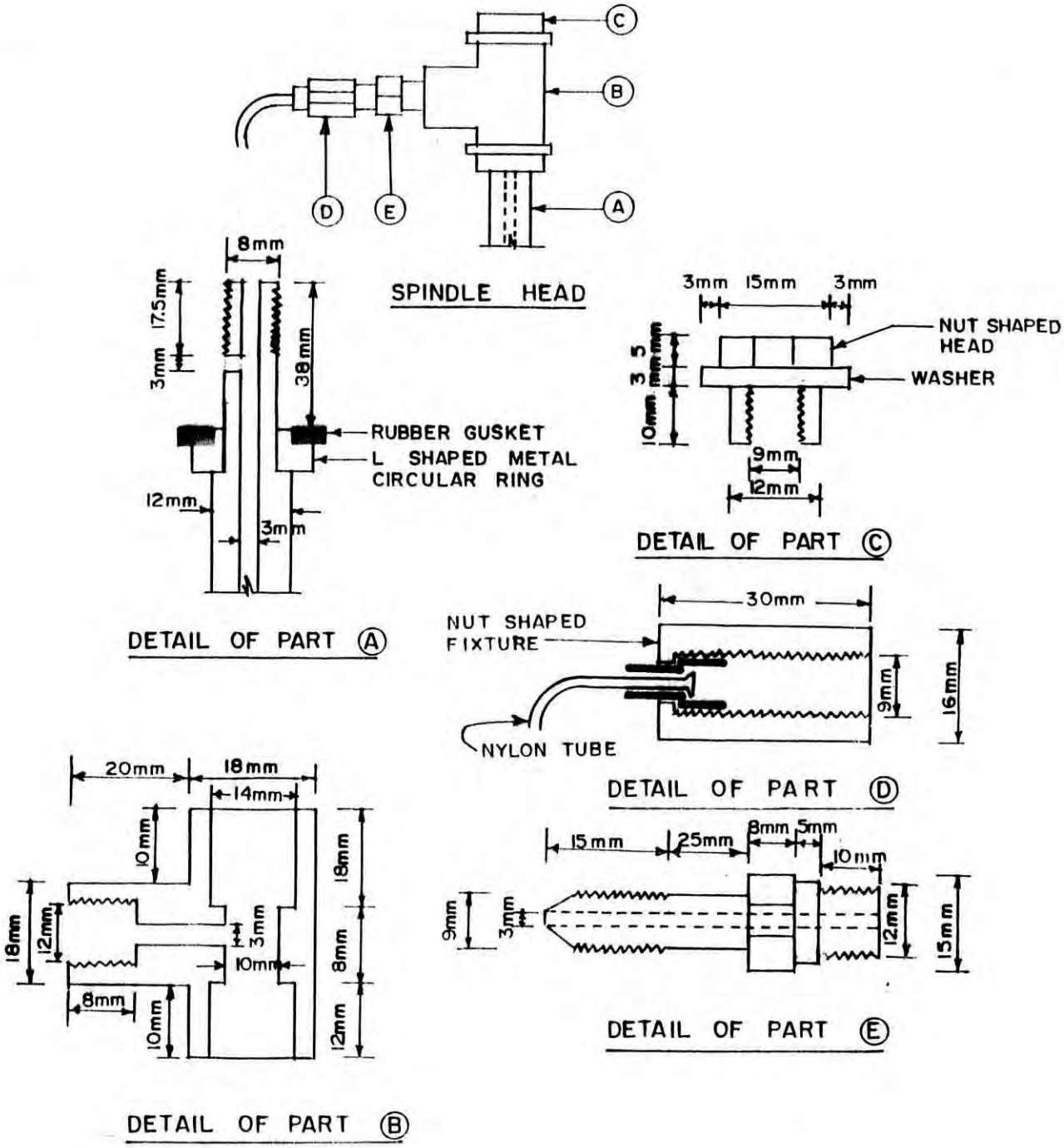


FIG. 4.5 DETAILS OF SPINDLE HEAD OF ROWE CELL.

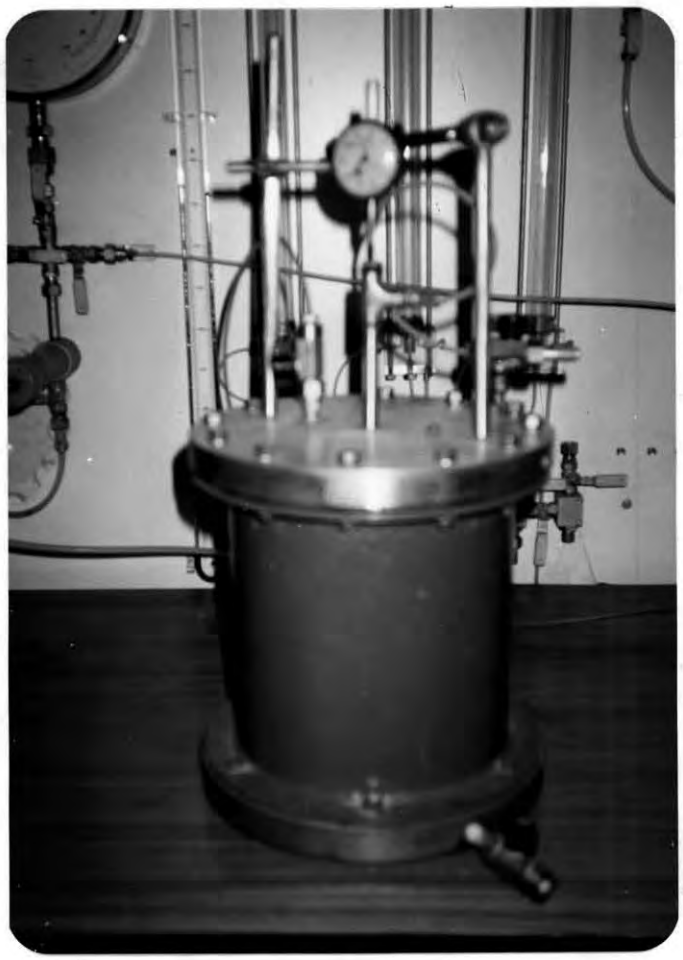
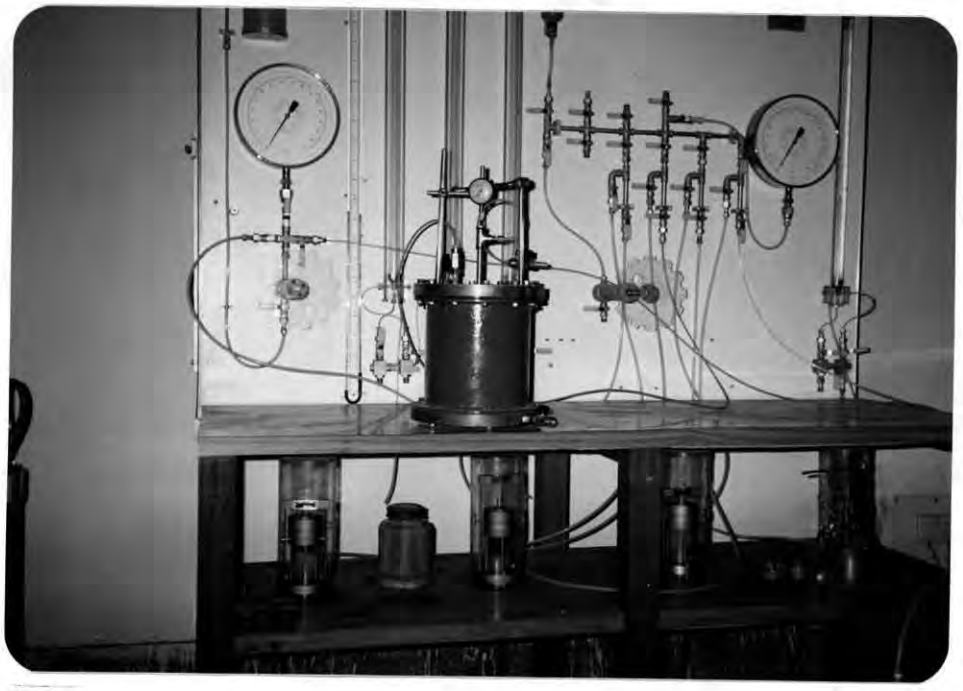


FIG. 4.6 THE ROWE CONSOLIDATION CELL



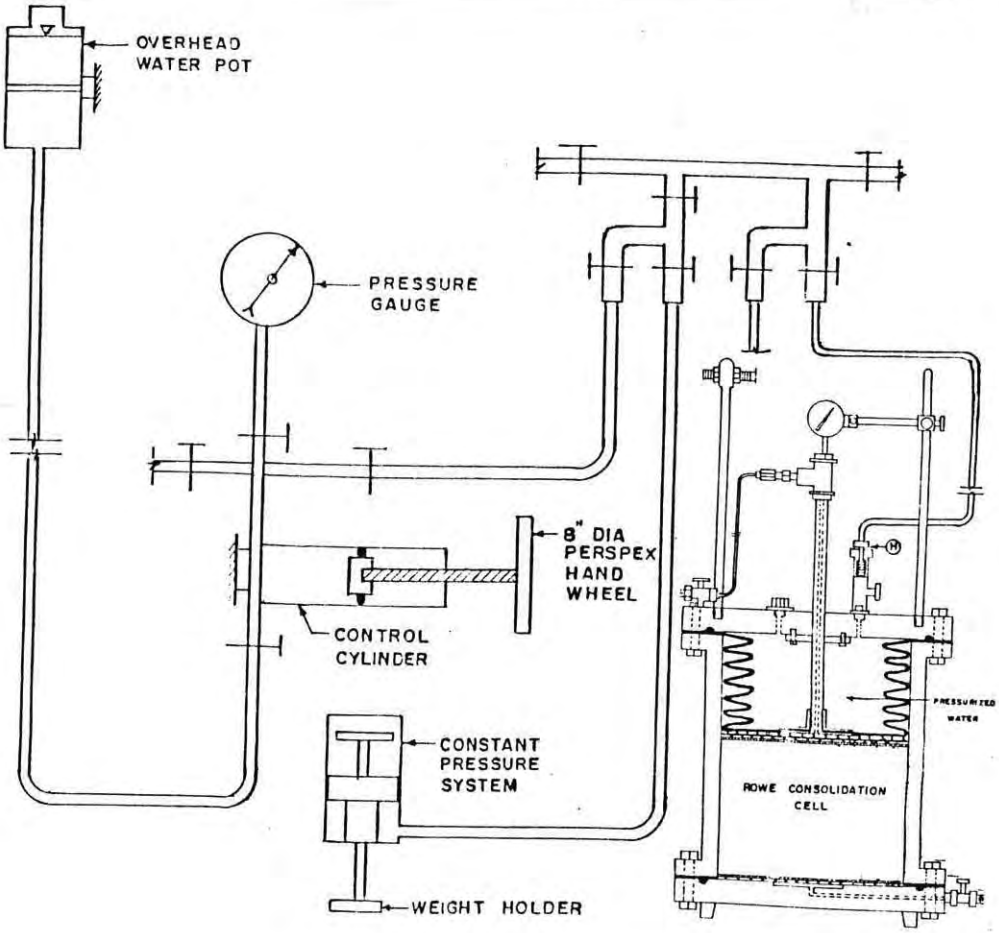
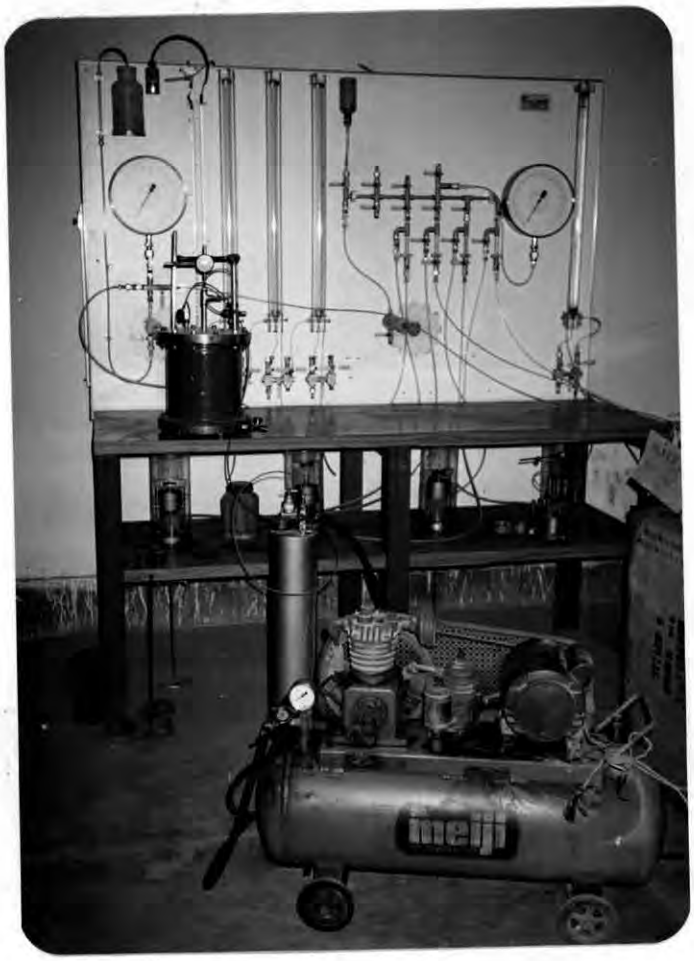


FIG. 4.7 DASH POT AND CONTROL CYLINDER SYSTEM AS A SOURCE OF HYDROSTATIC PRESSURE TO ROWE CELL.



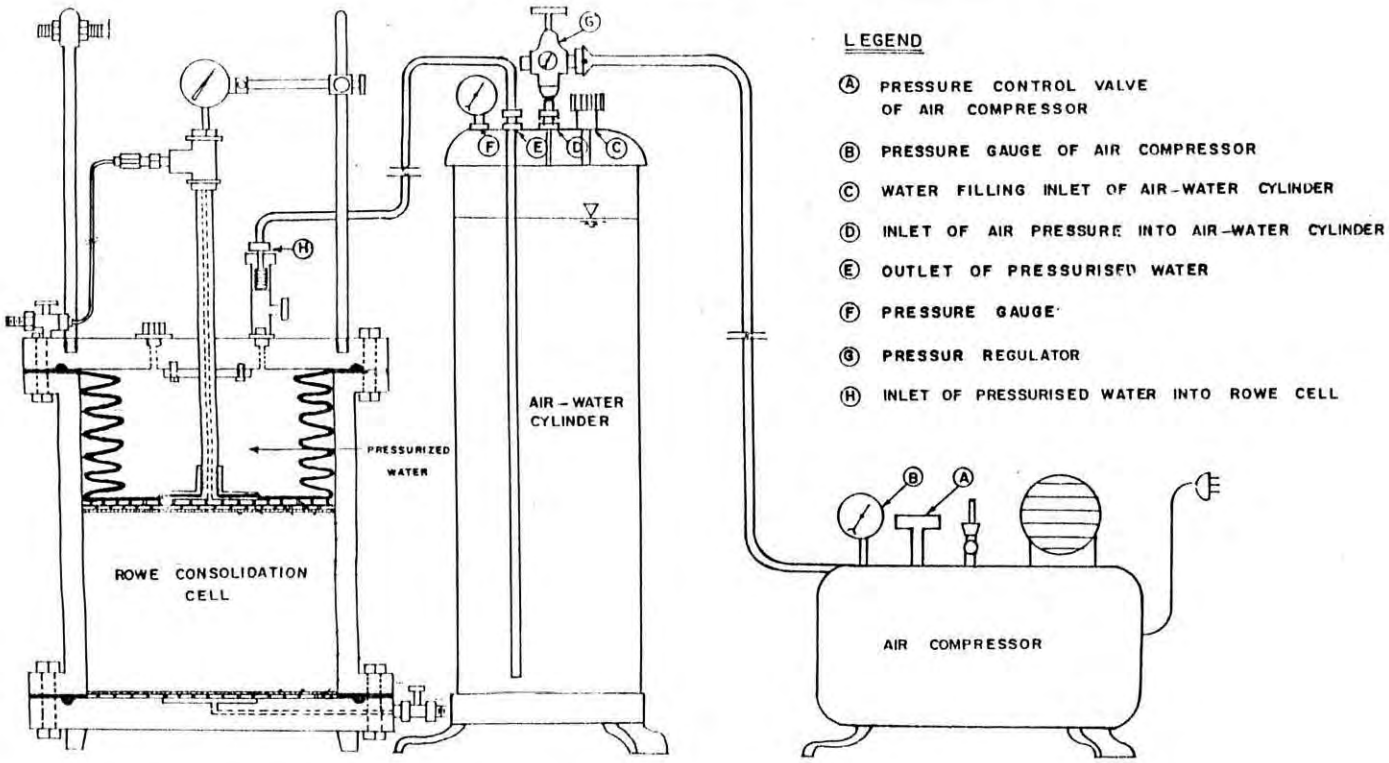


FIG. 4.8 THE EXPERIMENTAL SET UP FOR K_0 CONSOLIDATION

4.2.3 The Rotary Laboratory Mixer

A 'Hobart' mixer machine was used for making soil slurry. The rotary blades of this machine ensured proper mixing of soil particles with water over a short period of time at the required moisture content.

The mixer machine used has a dimension of 738 mm x 406 mm x 489 mm and includes a three-speed gear box driven by a fully enclosed and ventilated motor. The shift handle is mechanically interlocked with the switch, giving definite gear location and making necesary to switch off the motor before changing gear. A shock absorber is provided in the main drive gear and the beater shaft is carried on ball bearings. The bowl locks at the top and bottom of lift travel, which is controlled by convenient hand lever. The speed used in this research work was 113 rpm for 'attachment' and 198 rpm for 'beater'. The mixing time was 30 minutes. A diagram of the rotary mixer, bowl and attachment used for mixing operation is shown in Fig. 4.9.

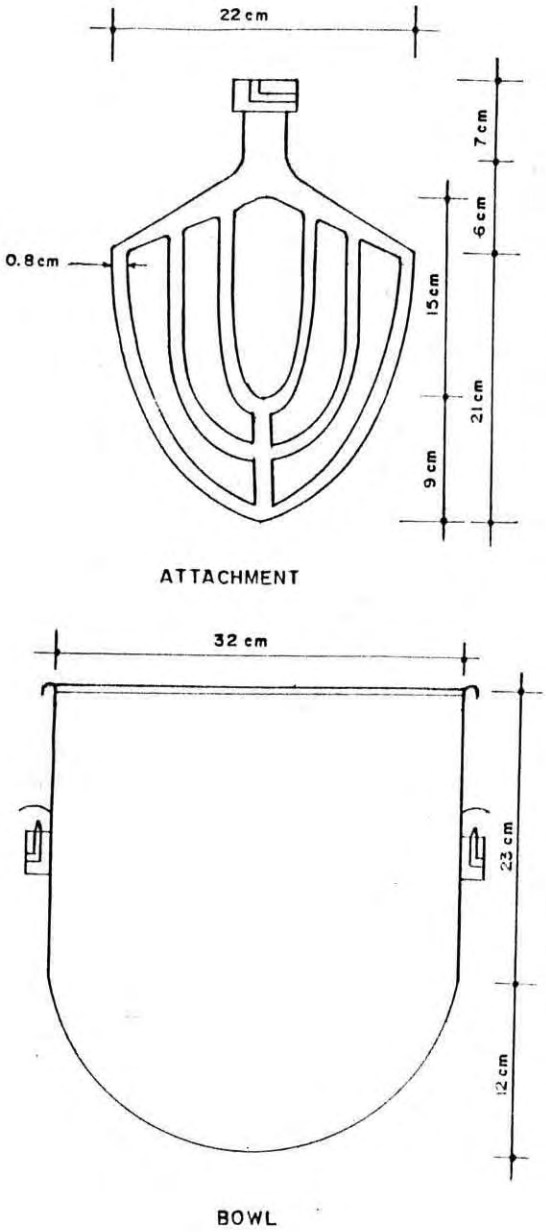


FIG. 4.9 THE ROTARY LABORATORY MIXER

CHAPTER 5
LABORATORY INVESTIGATIONS

5.1 Introduction

This chapter describes, in details, the laboratory investigations made on the soil samples collected from the location described in Chapter 3. Flow chart in Fig. 3.1 was followed for the laboratory investigations.

5.2 Determination of Physical and Index Properties

The soil samples collected were air dried. Then the lumps were broken carefully with a wooden hammer so as to avoid breakage of soil particle. The following tests were performed to determine grain size and index properties of the soil:

1) Specific gravity
2) Atterberg limits
3) Grain size distribution

Although the samples collected from field were disturbed, it's moisture content at the time of collection was recorded. The index and other properties were determined according to ASTM (1979) standards. The soil was classified according to ASTM Standard D 2487-69(1975) based on unified soil classification system. The results of all these tests are presented in Chapter 6.

5.3 Soil Slurry Preparation

Determination of consolidation characteristics and shear strength behaviour required identical soil samples under different stress condition with variation in stress history. This requirement was met from artificially prepared specimens in saturated remoulded

state. Shield and Rowe (1975) suggested that the identical specimens should have uniform density and moisture content. According to them, the remoulded samples should be prepared at a water content close to liquid limit. Berry (1967), however, suggested that this water content to be one and half to two times the liquid limit. For the present study, a water content of one and half times the liquid limit was used. The following procedure was followed in the preparation of an uniform soil slurry.

1) About 7 kg of air dried powdered soil was taken for preparation of the slurry.

2) In order to ensure full saturation, the soil was taken in the mixing bowl of the mixer machine and mixed with water at one and half times the liquid limit. The soil and the water were thoroughly mixed by hand kneading to form a slurry to ensure full saturation.

3) The product was them uniformly mixed by using rotary laboratory mixer such as shown in Fig. 4.9 for about 30 minutes.

4) The slurry was stored in a container. By Stirring with steel rod, effort was made to remove the entrapped air from slurry.

5) The slurry was then poured into the Rowe cell taking care that no air bubble was entrapped within the slurry. After pouring, the top surface of the soil sample was levelled properly.

5.4 Categories of Sample Used in the Investigation

A schematic diagram of the test program is given in Chapter 3 in Fig. 3.1. It can be seen that four categories of samples were prepared for this investigation.

The collected soil samples were air dried, lumps were broken and then it was mixed thoroughly in a mixer machine to form a slurry. Then slurry was consolidated in the Rowe cell in K_o condition. After consolidation, soil cake was extruded from Rowe cell and samples of size 38.1 mm (1.5 in) diameter and 76.2 mm (3 in) height were trimmed. The samples thus prepared are of category I (see Fig. 3.1). The samples of this category will hence forth be described as **"K_o consolidated samples in normally loaded state" or K_oNC samples**. Finally, these samples were sheared in undrained triaxial compression test. Stress path showing sequences followed by this category of samples is shown in Fig. 5.1(a).

Samples of Category II were prepared by consolidating soil slurry in Rowe cell at a desired maximum consolidation pressure and then it was allowed to swell in the Rowe cell under a desired reduced swelling pressure. When the swelling was completed, the soil cake was extruded from the Rowe cell and samples were trimmed from soil cake. Samples thus prepared are of category II as shown in Fig. 3.1. The sample of this category will be called **"K_o consolidated samples in overconsolidated state" or K_oOC samples**. These samples were then sheared in undrained triaxial compression test. Stress path showing the sequences followed by this category of samples is shown in Fig. 5.1(b).

The samples of Category III were prepared by consolidating the soil slurry initially under K_o condition in Rowe cell with a small vertical pressure, then the K_o consolidated soil cake was extruded from Rowe cell and triaxial test samples were trimmed from the soil cake. These samples were consolidated isotropically with desired consolidation pressure in a triaxial cell. Samples thus prepared are of Category III as shown in Fig. 3.1. The samples of this category will be called **"Isotropically consolidated samples in normally loaded state" or K_oINC samples**. These samples were sheared in undrained triaxial compression

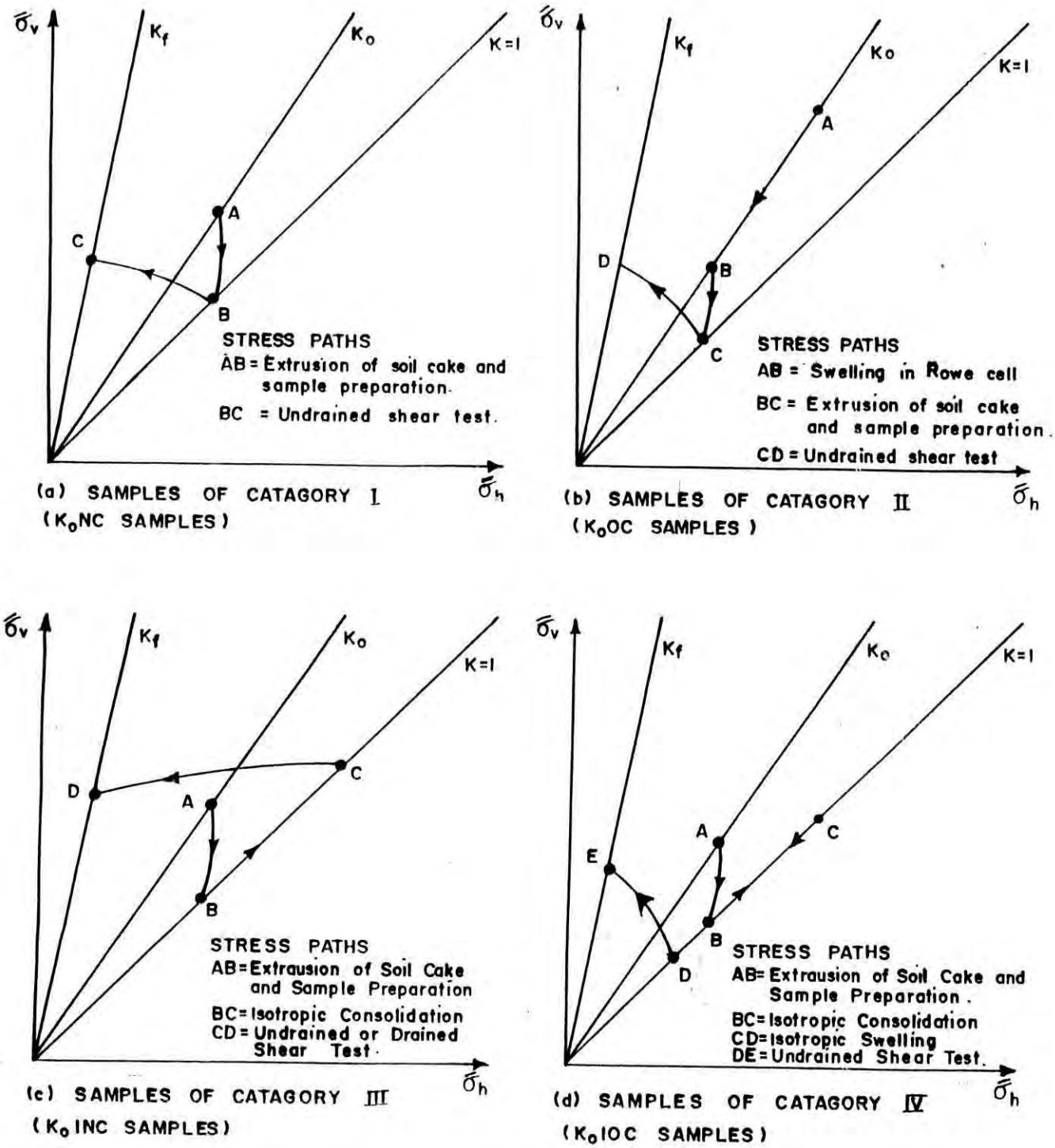


FIG. 5.1 QUALITATIVE EFFECTIVE STRESS PATH FOR LOADING AND UNLOADING SEQUENCE FOLLOWED IN FOUR CATAGORIES OF SAMPLES.

test. A number of samples of this category were sheared in triaxial compression test in drained condition. The stress path showing sequences followed by this category of a sample is shown in Fig. 5.1(c).

The samples of Category IV were prepared by adopting similar procedure as for Category III, but after extrusion of the soil cake and preparation of the triaxial sample, it was consolidated at a desired consolidation pressure, the sample was then allowed to swell at a reduced consolidation pressure. Sample thus prepared are of Category IV as shown in Fig. 3.1. The samples of this category will be called as **"Iostorpically consolidated sample in overconsolidated state" or K_oIOC samples**. Finally, these samples were sheared in undrained triaxial compression test. Stress path showing sequences followed by this category of samples is shown in Fig. 5.1(d).

Samples of all categories have a diameter of 38.1 mm (1.5 in) and height of 76.2 mm (3 in). In all categories, the extrusion of samples from K_o consolidated cake resulted in release of in-situ stresses and the samples developed isotropic effective stresses typical of truly undisturbed samples. This is shown in the stress path diagrams in Fig. 5.1.

5.5 K_o Consolidation and Preparation of Soil Cake

The Rowe consolidation cell, of 210 mm dia and 245 mm height, was used for K_o consolidation of the slurry to form a cake. Prior to pouring of the soil slurry and application of hydrostatic pressure by the rubber bellows of the cell, the following procedure was adopted.

1) Firstly, a brass wire mesh and a 6 mm thick perforated disc were placed at the bottom of the mould of the cell as shown in Fig. 5.2.

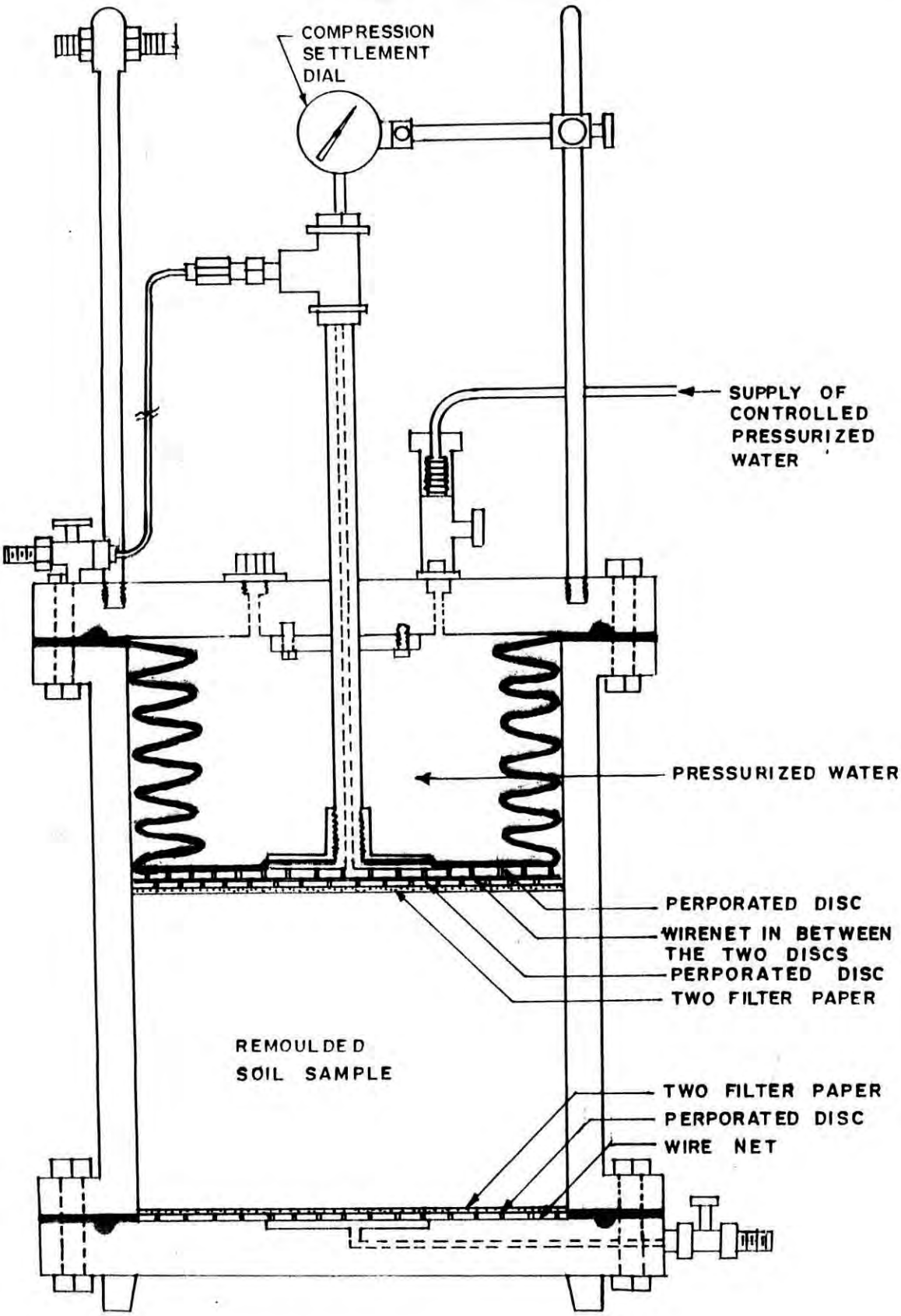


FIG. 5.2 KO CONSOLIDATION IN ROWE CELL.

2) the walls of the cell was coated with a thin layer of silicon grease in order to minimize side friction during consolidation.

3) Two filter papers were placed over the disc at the bottom of the cell.

4) The slurry was then poured in to the cell and the slurry was levelled off taking care that no air is trapped within the soil.

5) Two filter papers followed by two perforated discs were placed at the top of the slurry to permit drainage. A brass wire mesh was used between the two discs for easy flow of water in the horizontal direction. All the perforated discs had a diameter of 207 mm and thickness of 6 mm. These were prepared by drilling holes of 1.5 mm diameter spaced at 3 mm center to center. A clearance of 1.5 mm in between the perforated discs and mould body was provided to eliminate side friction. The arrangement for K_o consolidation is shown in Fig. 4.8 in Chapter 4 and in Fig. 5.2.

6) By using air compressor, required pressure was developed in the air-water cylinder which was nearly full with water (fig. 4.8).

7) The convoluted rubber bellows of the cell was filled up with water through air vent slot and the cell was connected to the air-water cylinder. Pressure was transmitted from air-water cylinder to the rubber bellows of the Rowe cell. A pressure regulator shown in Fig. 4.8 was used to control the pressure of the cell.

The following procedure was followed for loading the soil sample.

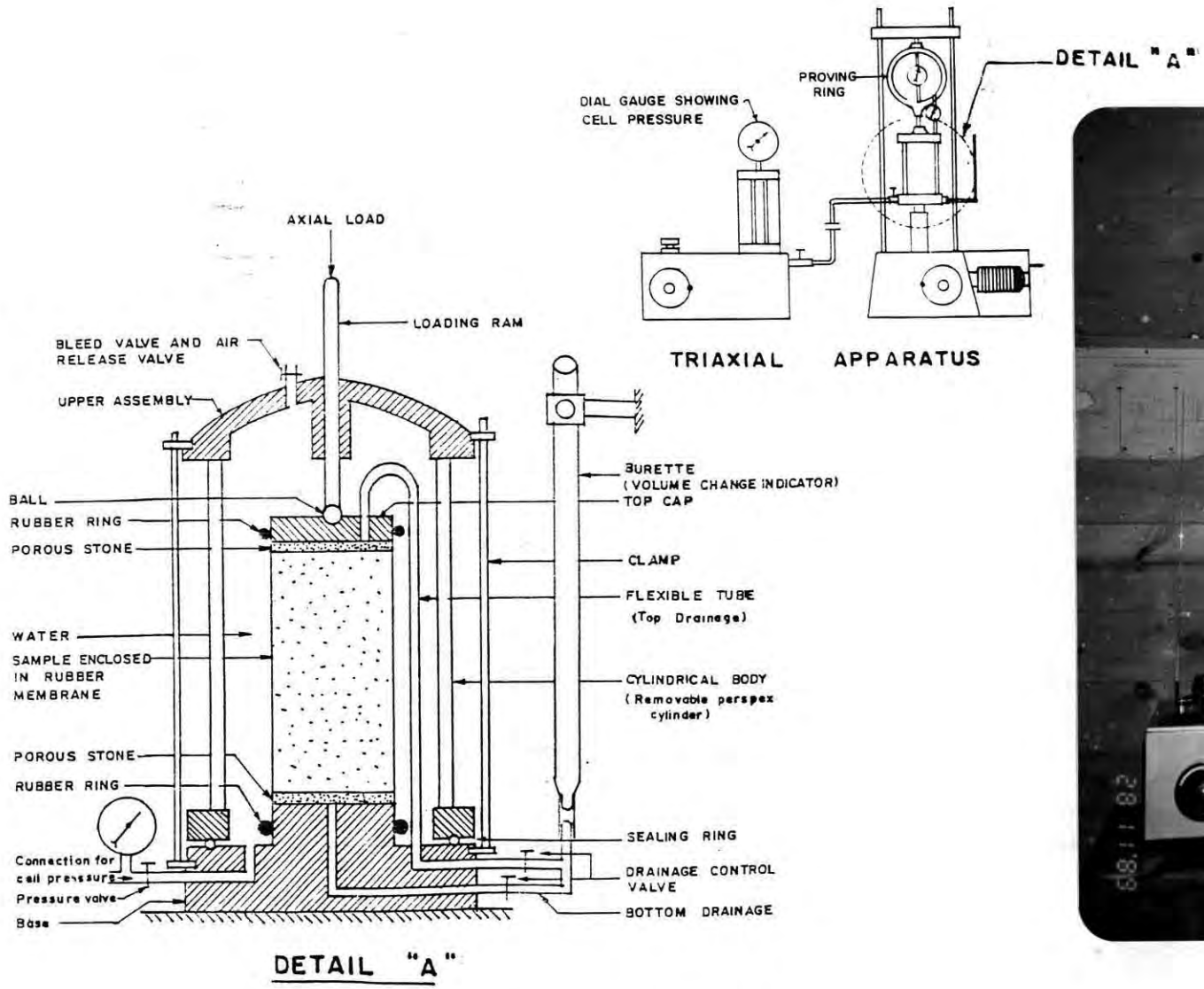
At first, the sample was subjected to an initial consolidation pressure of 10 kPa. Then the pressure was gradually increased to

about 40 KPa over a period of ten hours. This slow increase in load was necessary to avoid extrusion of slurry around the porous discs. During loading, consolidation occurred by vertical drainage through perforated plates at top and bottom of the sample. The settlement or one dimensional compression of the soil sample was indicated by the strain dial gauge (Fig. 5.1). After 24 hours, pressure was gradually increased to 80 kPa. This pressure was maintained until strain dial gauge reading remained constant for atleast 24 hours. This is the equilibrium condition. About four to five days were required to reach this equilibrium condition. Rate of compression was very fast at initial stage and gradually decreased with time. Following above procedure, the load was increased in steps to attain equilibrium. An appropriate consolidated soil cake of about 100 mm thickness was produced by above process. After consolidation, the top and bottom part of the cell was separated and the soil cake was extruded by using an extruder.

The uniformity in density and moisture content within the consolidated soil cake was checked from moisture content and density of specimens taken from five or six different locations within the cake. In all cases, density and moisture content variation was found to be ± 0.25 lb/cft and ± 0.20 percent.

5.6 Undrained Shear Test on K_oNC Samples (i.e. Samples of Category I)

Soil specimens of 38.1 mm (1.5 in) diameter and 76.2 mm (3.in) height were trimmed from the consolidated soil cake using wire cutter and trimming device. The weight of each sample was recorded. Each sample was enclosed in a rubber membrane with help of a membrane stretcher. One filter paper followed by one porous stone was placed at each end of the soil specimen. These specimens prepared from K_o consolidated soil cake were placed in a triaxial cell as shown in Fig. 5.3.



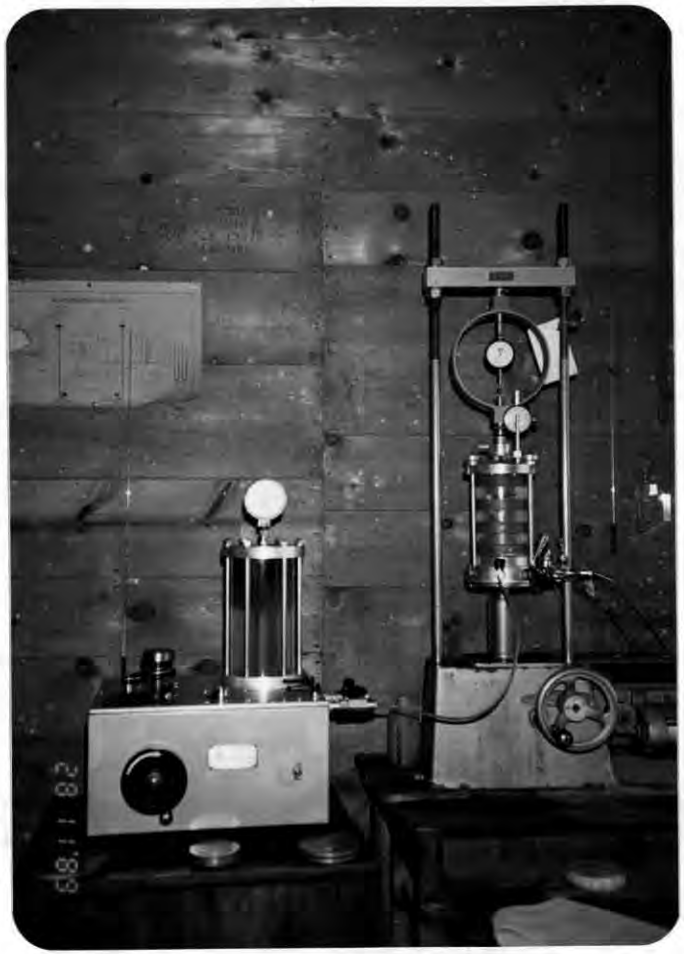


FIG. 5.3 THE TRIAXIAL APPARATUS USED FOR CONSOLIDATING SAMPLES ISOTROPICALLY AND FOR TRIAXIAL COMPRESSION TEST.

Triaxial test procedure details mentioned by Bishop and Henkel (1962) were followed. The rate of strain for undrained test was 1.5 mm per minute (Bishop et al., 1964). The following steps were followed for conduction of triaxial compression test in undrained condition:

1) Prior to placing of the sample and cylindrical body of the apparatus, the top drainage line and bottom drainage line along with volume change indicator were flushed separately with cooled boiled water.

2) To avoid entrapped air in the line, water was allowed to run out slowly. After flushing, the drainage control valves were closed.

3) After placing the sample in the cell, the top cap was placed properly. Then the top and bottom rubber rings were fixed over the rubber membrane as shown in Fig. 5.3.

4) By raising the top nut and upper assembly, the cylindrical body of the triaxial apparatus were placed and the three clamps were fixed properly so that the apex of the loading ram just contacts the centrely placed ball of the top cap.

5) The cell was filled with water by opening the bleed valve which was at the top of the upper assembly. After filling the chamber, when the water was overflowing, the bleed valve was closed.

6) The hydrostatic pressure machine which supplies constant hydrostatic pressure, was switched on and required cell pressure (listed in Table 5.1) attained. This pressure was applied to the cell by opening the pressure value. During the application of cell pressure, drainage from the sample was not allowed by closing top and bottom drainage control valves.

7) The base of the cell was raised slowly by operating the wheel handled manually until the cap was just in contact with the loading ram. This was showed by the movement of the proving ring dial gauge indicator.

8) The proving ring dial gauge indicator and strain dial gauge indicator were set at zero position. Then the motor was switched on. The proving ring dial gauge readings were recorded. The strain was applied at 1.5 mm per minute.

9) The loading continued until the load dial reading remained constant or decreased for a few readings of strain dial gauge.

10) At the end of the test, loading base was lowered and cell pressure was released. The cell was disassembled and sample was carefully removed from the cell. The failure pattern of the sample was recorded, it's weight was measured and water content was determined.

Table 5.1 lists the K_oNC samples (i.e samples of Category I) alongwith consolidation pressure applied in the Rowe Cell, water content of the soil cake, weight of sample after trimming and the confining pressure to the sample prior to application of the deviator stress. It may be mentioned that no drainage was allowed during application of cell pressure. This pressure was maintained during application of deviator stress. No pore pressure was measured during the undrained tests.

5.7 Undrained Shear Test on K_oOC Samples (i.e Samples of Category II)

A slurry soil sample was consolidated under K_o condition by the same process as described in Article 5.5. After achieving the desired maximum consolidation pressure, $\bar{\sigma}_v$ in Rowe cell, the

TABLE 5.1: K_oNC SAMPLES

Sample No.	Vertical consolidation pressure in Rowe cell, $\bar{\sigma}_v$ (kPa)	Water content of K_o consolidated soil cake (percent)	Weight of sample (gm)	Confining pressure during application of deviator stress in triaxial test, $\bar{\sigma}_3$ (kPa)
K_oNC/1	100	31.32	171.17	0
" /2	100	31.38	171.10	100
" /3	100	31.33	171.55	200
" /4	200	27.85	173.95	0
" /5	200	27.87	174.0	200
" /6	200	27.82	173.20	100
" /7	300	25.86	176.95	0
" /8	300	25.84	176.81	300
" /9	300	25.89	177.11	200
" /10	400	24.49	178.62	0
" /11	400	24.52	178.20	400
" /12	400	24.56	178.85	200
" /13	500	23.31	180.05	0
" /14	500	23.28	179.98	500
" /15	500	23.35	180.12	300
" /16	800	20.96	182.79	0
" /17	800	20.90	182.21	800
" /18	800	20.91	182.13	300

sample was allowed to swell under a reduced swelling pressure, $\bar{\sigma}_{vo}$ until the strain dial gauge indicator remained constant. Water was allowed to be drawn into the sample during the unloading state. The swelling process was assumed to be completed when the dial gauge reading remained constant. After completion of swelling, soil cake was extruded, samples were trimmed and prepared for undrained triaxial compression test. The undrained triaxial shear testing procedure was same as described in article 5.6.

Table 5.2 lists the K_oOC samples (sample of Category II) prepared under various maximum consolidation pressure, reduced swelling pressure, OCR alongwith their water content before undrained shearing etc.

5.8 Undrained and Drained Shear Tests of K_oINC Samples (i.e Samples of Category III)

For these samples, soil slurry was consolidated in the Rowe cell following the same procedure as described in article 5.5. The vertical consolidation pressure applied in the Rowe cell was always smaller than the isotropic consolidation pressure to be applied subsequently. After achieving equilibrium condition, the soil cake was extruded from the Rowe cell. Samples were trimmed and prepared as described in article 5.6.

After weighing the sample, placing filter paper and porous stone stone at both ends and enclosing it in a rubber membrane, the sample was placed in triaxial cell for application of an all round pressure to consolidate the sample isotropically. The following steps were adopted for isotropic consolidation in triaxial cell (Fig. 5.3).

1) Prior to application of cell pressure for consolidation, all the operations such as flushing of drainage lines, valves etc.;

TABLE 5.2: K_oOC SAMPLES

Sample No.	Maximum consolidation pressure in Rowe cell, $\bar{\sigma}_v$ (kPa)	Reduced vertical swelling pressure in Rowe cell, $\bar{\sigma}_{vo}$ (kPa)	Water content of K_o consol. soil cake (percent)	Weight of sample (gm)	Confining pressure during application of deviator stress in triaxial test, $\bar{\sigma}_3$ (kPa)	OCR
K_oOC/19	100	100/3	32.45	171.69	0	3
" /20	100	100/3	32.47	171.90	100/3	3
" /21	100	100/3	32.43	172.01	200	3
" /22	200	100/3	28.70	174.08	0	6
" /23	200	100/3	28.76	174.45	100/3	6
" /24	200	100/3	28.72	174.21	200	6
" /25	200	100	28.15	174.76	0	2
" /26	200	100	28.13	174.40	100	2
" /27	200	100	28.18	175.02	200	2
" /28	300	100/3	27.18	175.37	0	9
" /29	300	100/3	27.14	175.02	100/3	9
" /30	300	100/3	27.17	175.42	300	9
" /31	300	100	26.35	175.85	0	3
" /32	300	100	26.39	175.76	100	3
" /33	300	100	26.36	176.02	300	3
" /34	300	150	26.17	176.90	0	2
" /35	300	150	26.15	176.47	150	2
" /36	300	150	26.18	177.02	300	2

TABLE 5.2 :K_oOC SAMPLES (CONTD...)

Sample No.	Maximum consolidation pressure in Rowe cell, $\bar{\sigma}_v$ (kPa)	Reduced vertical swelling pressure in Rowe cell, $\bar{\sigma}_{vo}$ (kPa)	Water content of K_o consol. soil cake (percent)	Weight of sample (gm)	Confining pressure during application of deviator stress stress in triaxial test $\bar{\sigma}_3$ (kPa)	OCR
K_oOC/37	400	100/3	25.66	176.67	0	12
" /38	400	100/3	25.70	177.02	100/3	12
" /39	400	100/3	25.68	176.55	400	12
" /40	400	100	24.98	177.85	0	4
" /41	400	100	25.05	177.81	100	4
" /42	400	100	25.08	177.98	400	4
" /43	400	200	24.66	177.95	0	2
" /44	400	200	24.62	177.76	200	2
" /45	400	200	24.65	177.89	400	2
" /46	500	100/3	24.43	178.82	0	15
" /47	500	100/3	24.46	178.59	100/3	15
" /48	500	100/3	24.51	179.02	500	15
" /49	500	100	24.06	179.60	0	5
" /50	500	100	24.06	179.22	100	5
" /51	500	100	24.02	179.25	500	5
" /52	500	200	23.60	179.84	0	2.5
" /53	500	200	23.59	179.64	200	2.5
" /54	500	200	23.63	179.92	500	2.5

TABLE 5.2: K_oOC SAMPLES (CONTD....)

Sample No.	Maximum consolidation pressure in Rowe cell, $\bar{\sigma}_v$ (kPa)	Reduced vertical swelling pressure in Rowe cell, $\bar{\sigma}_{vo}$ (kPa)	Water content of K_o consol. soil cake (percent)	Weight of sample (gm)	Confining pressure during application of deviator stress stress in triaxial test, $\bar{\sigma}_3$ (kPa)	OCR
K_oOC/55	500	250	23.43	179.62	0	2
" /56	500	250	23.40	179.24	250	2
" /57	500	250	23.38	179.25	500	2
" /58	800	100/3	22.05	180.16	0	24
" /59	800	100/3	22.06	180.42	100/3	24
" /60	800	100/3	22.08	180.67	800	24
" /61	800	100	21.73	180.45	0	8
" /62	800	100	21.70	180.04	100	8
" /63	800	100	21.69	179.91	800	8
" /64	800	200	21.38	182.62	0	4
" /65	800	200	21.40	182.47	200	4
" /66	800	200	21.46	182.92	800	4
" /67	800	400	21.19	183.56	0	2
" /68	800	400	21.16	182.97	400	2
" /69	800	400	21.23	183.21	800	2

placement of sample in the cell, placement of top cap, rubber rings, cylindrical body etc; filling of cell chamber with water, etc. were done following the same procedure as described for triaxial shear test in article 5.6.

2) Then the hydrostatic pressure machine was switched on and required load was placed on the load holder of the machine to develop the desired confining pressure in the cell.

3) The base of the cell was raised slowly by operating the wheel handle manually until the cap was just in contact with the loading ram. This was showed by the movement of the proving ring dial gauge indicator.

4) The drainage valves were opened and consolidation occurred isotropically due to all-round cell pressure. No piston load was applied and loading ram did not move during consolidation. The volume change indicator showed the measurement of water expelled during consolidation. Consolidation was assumed complete when no significant movement of water occurred in volume change indicator. When the consolidation is completed, the sample of this category is produced.

5) At the end of consolidation, the loading ram was brought in contact with the ball of top cap. The distance moved by the loading ram is the change in length of the sample due to consolidation.

The samples thus prepared were finally sheared in undrained triaxial compression test. The triaxial shear testing procedure was the same as described in Article 5.6. Table 5.3 lists the K_oINC samples (Sample of Category III) showing the K_o consolidation pressure initially applied before sampling, corresponding water content, all-round maximum consolidation pressure etc.

TABLE 5.3: K_oINC SAMPLES

Sample No.	Initial K_o consolidation pressure in Rowe cell, $\bar{\sigma}_{(IK_o)}$ (kPa)	Water content of K_o consol. soil cake (percent)	Weight of sample (gm)	Cell pressure for isotropic consol. in triaxial cell, $\bar{p}$ (kPa)	Confining pressure during application of deviator stress in triaxial test, $\bar{\sigma}_3$ (kPa)
K_oINC/70	80	32.30	166.73	100	0
" /71	80	32.38	167.07	100	100
" /72	80	32.33	166.68	100	300
" /73	150	29.40	174.04	200	100
" /74	150	29.32	173.97	200	200
" /75	150	29.37	174.06	200	300
" /76	250	26.79	176.78	300	0
" /77	250	26.83	177.13	300	300
" /78	250	26.85	176.97	300	100
" /79	350	25.13	177.98	400	0
" /80	350	25.16	178.25	400	400
" /81	350	25.09	177.75	400	100

A number of samples of this category were also tested in drained condition in triaxial compression test. The procedure for the drained triaxial test was the same as described for undrained test in Article 5.6 except that the drainage valves were kept open and the rate of strain was 0.0125 mm per minute in the case of drained test. Table 5.4 lists the K_oINC samples (Samples of Category III) which have been sheared in drained condition.

5.9 Undrained Shear Test of K_oIOC Samples (i.e. Samples of Category IV

For these samples, soil slurry was initially subjected to K_o consolidation in Rowe cell with a vertical consolidation pressure [$\sigma_{(IK_o)}$]. The consolidated cake was extruded, then trimmed samples were prepared and placed in triaxial cell adopting procedure as mentioned in Article 5.6. In triaxial cell, all-round consolidation pressure equal to $\bar{p}$ was applied and isotropic consolidation operation was carried out following procedure described in Article 5.8. After completion of consolidation, the cell pressure was reduced to $\bar{p}_o$ and swelling of the sample was allowed. When the swelling was completed, the sample of this category was produced. At the end of swelling, the loading ram was brought in contact with the ball of the cap. The distance moved by the loading ram is the change in length of the sample.

Table 5.5 lists the K_oIOC samples (samples of Category IV) showing the K_o consolidation pressure initially applied before sampling, corresponding water content, all-round maximum consolidation pressure, all round swelling pressure etc.

TABLE 5.4: K_oINC SAMPLES (SHEARED IN DRAINED CONDITION)

Sample No.	Initial K_o consolidation pressure in Rowe cell, $\bar{\sigma}_{(IK_o)}$ (kPa)	Water content of K_o consolidated soil cake (percent)	Weight of sample (gm)	Cell pressure during isotropic consolidation in triaxial cell, $\bar{p}$ (kPa)	Confining pressure during application of deviator stress in triaxial cell, $\bar{\sigma}_3$ (kPa)
K_oINC/82	80	32.28	172.14	100	100
" /83	150	29.35	174.05	200	200
" /84	250	26.87	176.63	300	300

TABLE 5.5: K_o IOC SAMPLES

Sample No.	Initial K_o consolidation pressure in Rowe cell, $\bar{\sigma}_{(IKo)}$ (kPa)	Water content of K_o consol. soil cake (percent)	Weight of sample (gm)	Cell pressure for isotropic consolidation in triaxial cell, $\bar{p}$ (kPa)	Reduced cell pressure in triaxial cell $\bar{p}_o$ (kPa)	Swelling due to reduction of cell pressure (percent)	OCR	Confining pressure during application of deviator stress in triaxial test, $\bar{\sigma}_3$ (kPa)
K_oIOC/85	80	32.35	172.42	100	100/3	1.4	3	100/3
" /86	150	29.39	174.20	200	100/3	2.4	6	100/3
" /87	150	29.35	174.36	200	100	0.9	2	100
" /88	250	26.84	176.59	300	100/2	3.6	9	100/3
" /89	250	26.79	176.73	300	100/3	3.7	9	300
" /90	250	26.81	176.65	300	100	1.8	3	100
" /91	350	25.10	178.15	400	100/3	4.1	12	100/3
" /92	350	25.16	178.51	400	100	2	4	100
" /93	350	25.15	178.90	400	200	0.5	2	200

CHAPTER 6
EXPERIMENTAL RESULTS AND ANALYSIS

6.1 Introduction

The results of the laboratory investigations described in Chapter 5 on Dhaka clay are reported herein. The geotechnical properties of this clay such as undrained elastic modulus, compression and swelling indices, critical state pore pressure prameter (Λ_o) effective stress friction angle ($\bar{\phi}$), undrained shear strength ratio(S) and other important soil parameters and characteristics have been determined for different stress history conditions. Evaluation of some normalized parameters for this clay are also presented. A model for prediction of undrained shear strength for this clay is suggested.

6.2 The Physical and Index Properties of the Soil Used

The index properties such as liquid limit, plastic limit and physical properties of the Dhaka clay investigated are shown in Table 6.1. The soil is CL type. Moisture content at the time of collection of the soil was about 24%. Gradation curve for this soil is presented in Fig. 6.1.

6.3 Undrained Strength Test of K_o NC Samples

Undrained deviator stress and axial strain relations for K_oNC (K_o consolidated normally loaded) samples are presented in Fig. 6.2. In this figure, $\bar{\sigma}_v$ is the pressure upto which these samples were consolidated in the Rowe Cell. It is evident that shear strength increases with consolidation pressure. Also the slope of the initial portion of the curve becomes stiffer with increase in consolidation pressure. There is no defined peak stress for these

TABLE 6.1: THE PHYSICAL AND INDEX PROPERTIES OF THE DHAKA CLAY USED

Physical description of the soil		Specific gravity	Atterberg limits		Grain size distribution		
Colour	Consistency		Liquid limit %	Plastic limit %	Sand %	Silt %	Clay %
Reddish brown	Medium to stiff	2.69	43	20	1	77	22

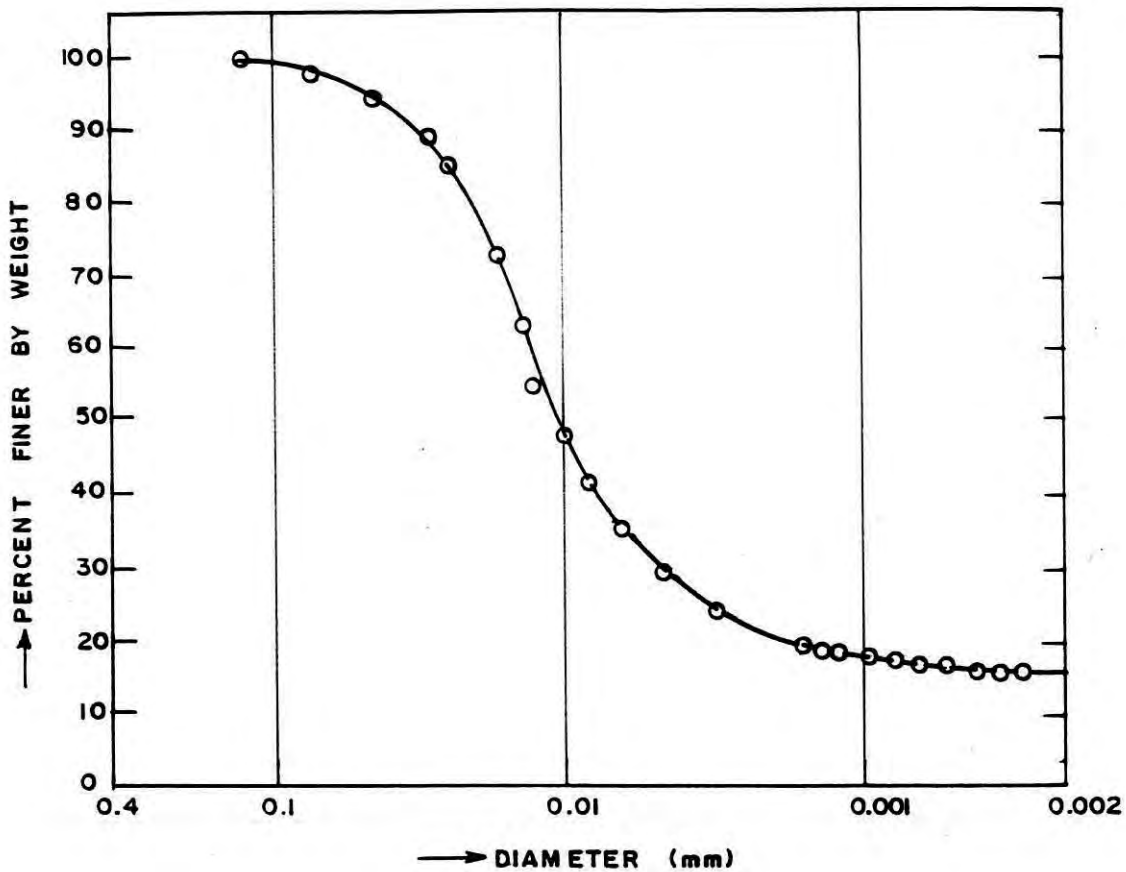


FIG. 6.1 GRADATION CURVE FOR THE SOIL USED.

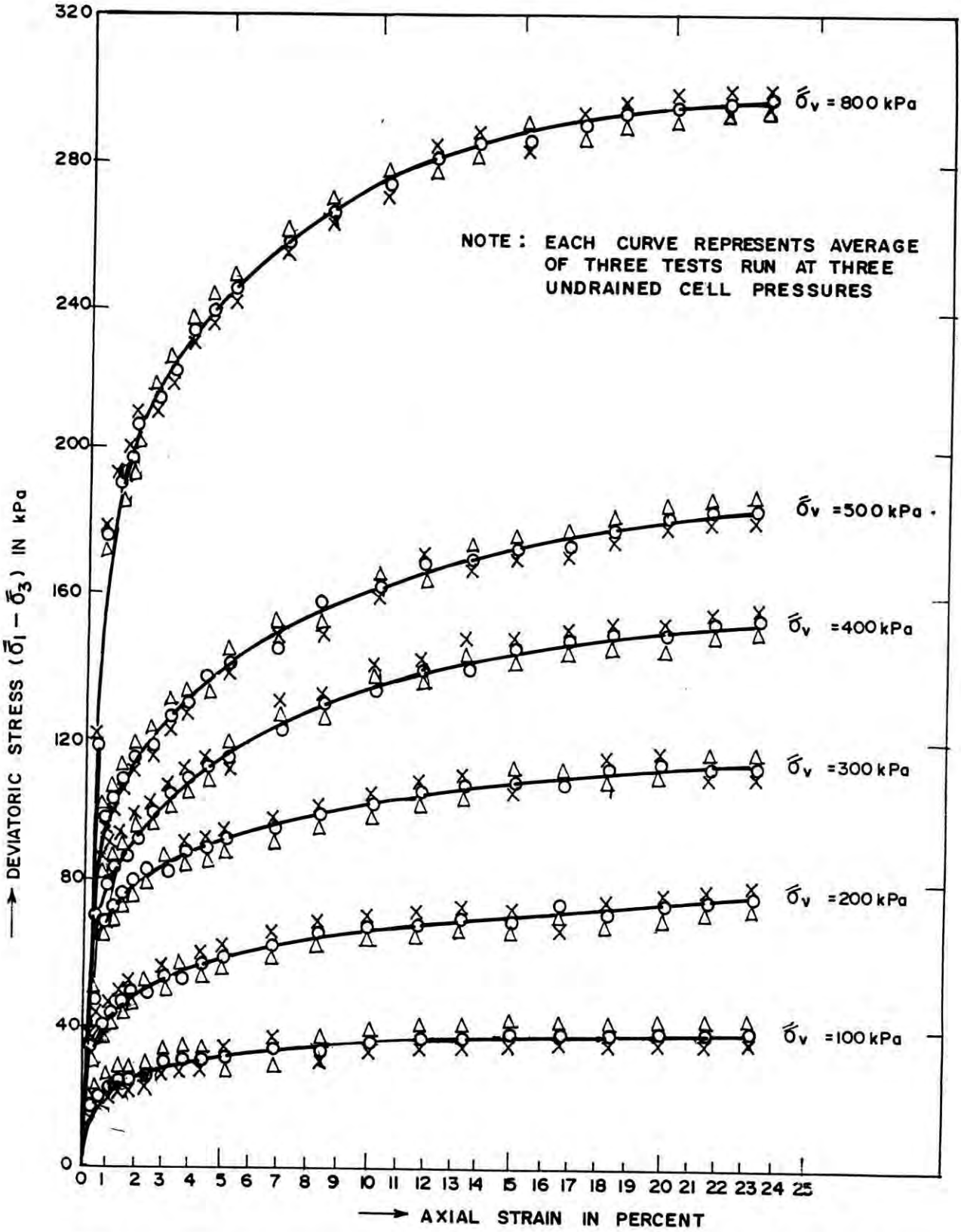


FIG. 6.2 STRESS - STRAIN CURVES OF UNDRAINED TRIAXIAL COMPRESSION TESTS FOR K_0NC SAMPLES.

samples. At higher strain level, the stress increases gradually with increasing strain. Therefore failure stress for these samples have been considered at corresponding strain of 20%. The undrained modulus of elasticity, $E_u(50)$ for all categories of samples have been considered as the slope of line between origin and 50% of the defined failure stress which is at 20% strain for K_oNC and K_oINC samples and peak deviator stress for K_oOC and K_oIOC samples.

The figure shows that cell pressure during undrained shear test have no effect on stress-strain curve whereas the consolidation pressure to which the sample has been consolidated prior to triaxial test greatly influences the stress-strain curve.

Table 6.2 presents for K_oOC samples, some physical quantities, such as $\bar{\sigma}_v$, e, w, S_u, $E_u(50)$, $E_u(50)/\bar{\sigma}_v$, $\Psi = E_u(50)/S_u$ and $S_u/\bar{\sigma}_v$. Here, $\bar{\sigma}_v$ is the maximum vertical pressure applied on the sample during K_o consolidation, S_u is the undrained shear strength of the sample (taken as half of the deviator stress at failure i.e at 20% strain), e and w are equilibrium void ratio and water content respectively. For these samples in normally loaded condition, $S_u/\bar{\sigma}_v$ has an average value of 0.186 and ranges between 0.182 and 0.188. Since the clay has a plasticity index of 23, according to Skempton's (1957) equation, $S_u/\bar{\sigma}_v=0.11+0.0037(PI)$, the value of $S_u/\bar{\sigma}_v$ becomes equal to 0.195, which is close to the average value obtained as above. Ameen (1985) found this value 0.19. From Table 6.2, it is also evident that shear strength increases with an increase of consolidation pressure and decreases with an increase of void ratio and water content. Fig. 6.3 presents plots of void ratio(e) versus consolidation pressure ($\bar{\sigma}_v$), void ratio (e) versus undrained shear strength (S_u), water content (w) versus consolidation pressure ($\bar{\sigma}_v$) and water content (w) vs. undrained shear strength (S_u) for K_oNC samples. The plots are straight lines and show that void ratio or water content

TABLE 6.2: VALUES FOR K_oNC SAMPLES

Maximum vertical consolidation pressure $\bar{\sigma}_v$ kPa (1)	Equilibrium void ratio e (2)	Equilibrium water content w percent (3)	Undrained shear strength S_u kPa (4)	Undrained secant modulus at 50% of ultimate shear strength $E_u(50)$ kPa (5)	$E_u(50)/\bar{\sigma}_v$ (6)		Normalized undrained secant modulus $\Psi=[E_u(50)/S_u]$ (7)		Undrained shear strength ratio $[S_u/\bar{\sigma}_v]$ S (8)	
						Av.		Av.		Av.
100	0.775	31.32	18.60	3676	36.76		197		0.186	
200	0.705	27.83	37.60	7915	39.57		210		0.188	
300	0.658	25.84	55.20	11621	38.73	37.52	210	202	0.184	0.186
400	0.625	24.50	74.80	14247	35.61		190		0.187	
500	0.598	23.30	91.00	17773	35.54		195		0.182	
800	0.540	20.95	148.00	31157	38.94		210		0.185	

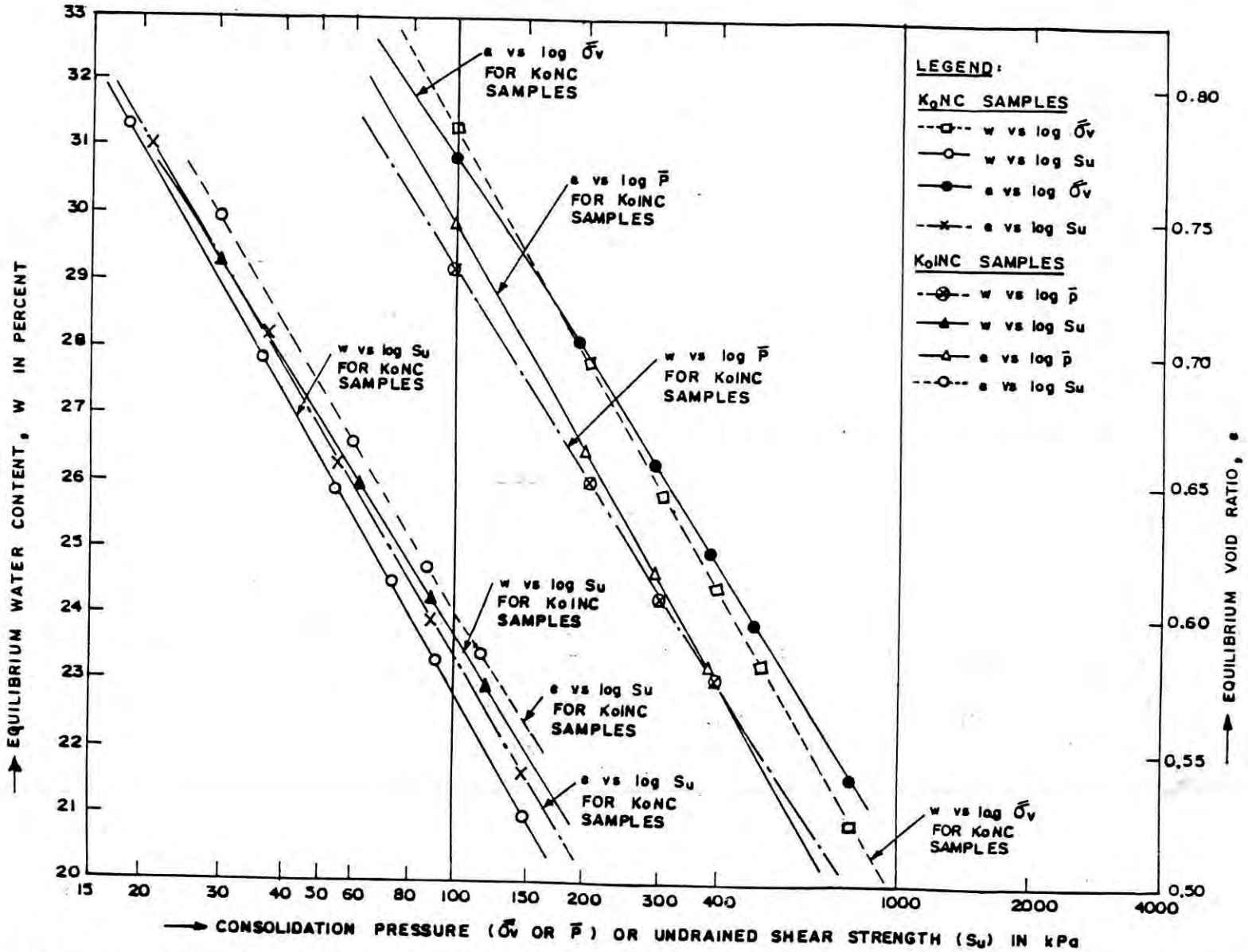


FIG. 6.3 WATER CONTENT OR VOID RATIO VERSUS CONSOLIDATION PRESSURE OR UNDRAINED SHEAR STRENGTH FOR KoNC AND KoINC SAMPLES OF DHAKA CLAY.

decreases linearly with increase of consolidation pressure or undrained shear strength. These relations demonstrate the uniqueness of water content or void ratio relations with consolidation pressure or undrained shear strength for saturated soils earlier demonstrated by Henkel (1959) as shown in Fig. 2.15 and 2.16.

From linear variation of w vs. log S_u plot, the undrained shear strength of K_oNC samples can be obtained as

$$S_{u(nc)} = 10^{(3.99-0.08688w)} \qquad (6.1)$$

The compression index, $C_c^{(K_o)}$ has been found from slope of the relation e vs. log $\bar{\sigma}_v$ for K_oNC sample, which is 0.25.

The log-log plot in Fig. 6.4 shows the variation of undrained secant modulus [$E_u(50)$] with consolidation pressure, σ_v for K_oNC samples by the straight line portion of the plot. The variation is linear and it shows that the undrained secant modulus increases linearly with consolidation pressure.

Shown under column (6), in Table 6.2 is the normalized parameter, $E_u(50)/\bar{\sigma}_v$ for these samples, the average value is 37.52. In the same table, under column (7), the Ψ (= $E_u(50)/S_u$) values have been presented. The average value for these samples is 202. Bjerrum (1964), Skempton & Henkel (1957) and cooling & Skempton (1942) reported that the Ψ values for normally consolidated clays vary from 500 to 140. Ameen (1985) found this value as 192.

6.4 Undrained Strength Test of K_oINC Samples

Undrained deviator stress and axial strain relations for K_oINC samples (isotropically consolidated normally loaded sample with initial K_o consolidation) are shown in Fig. 6.5(a). These samples exhibit behaviour similar to K_oNC samples described in article

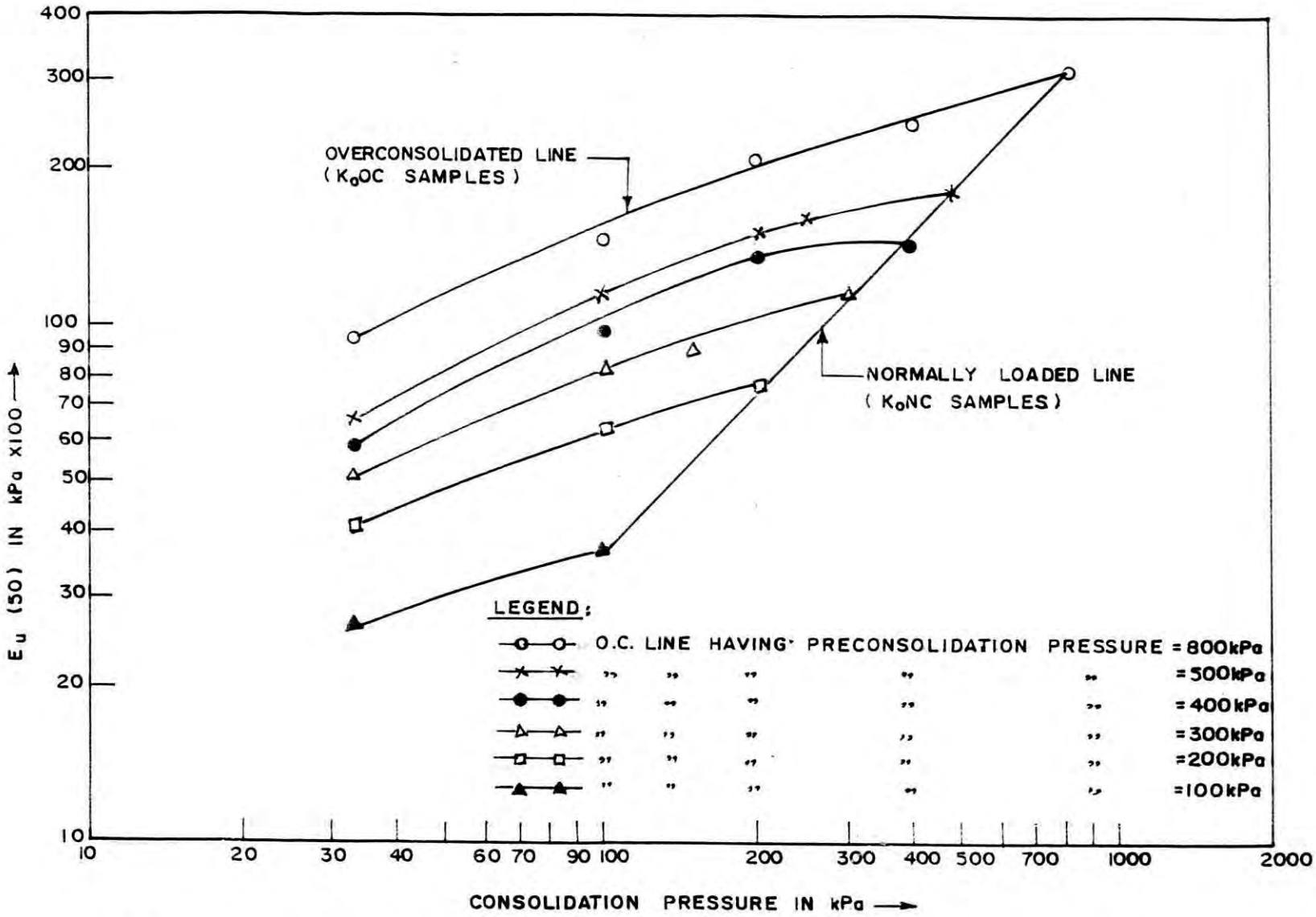


FIG. 6.4 VARIATION OF UNDRAINED SECANT MODULUS WITH CONSOLIDATION PRESSURE FOR K_oNC AND K_oOC SAMPLES OF DHAKA CLAY

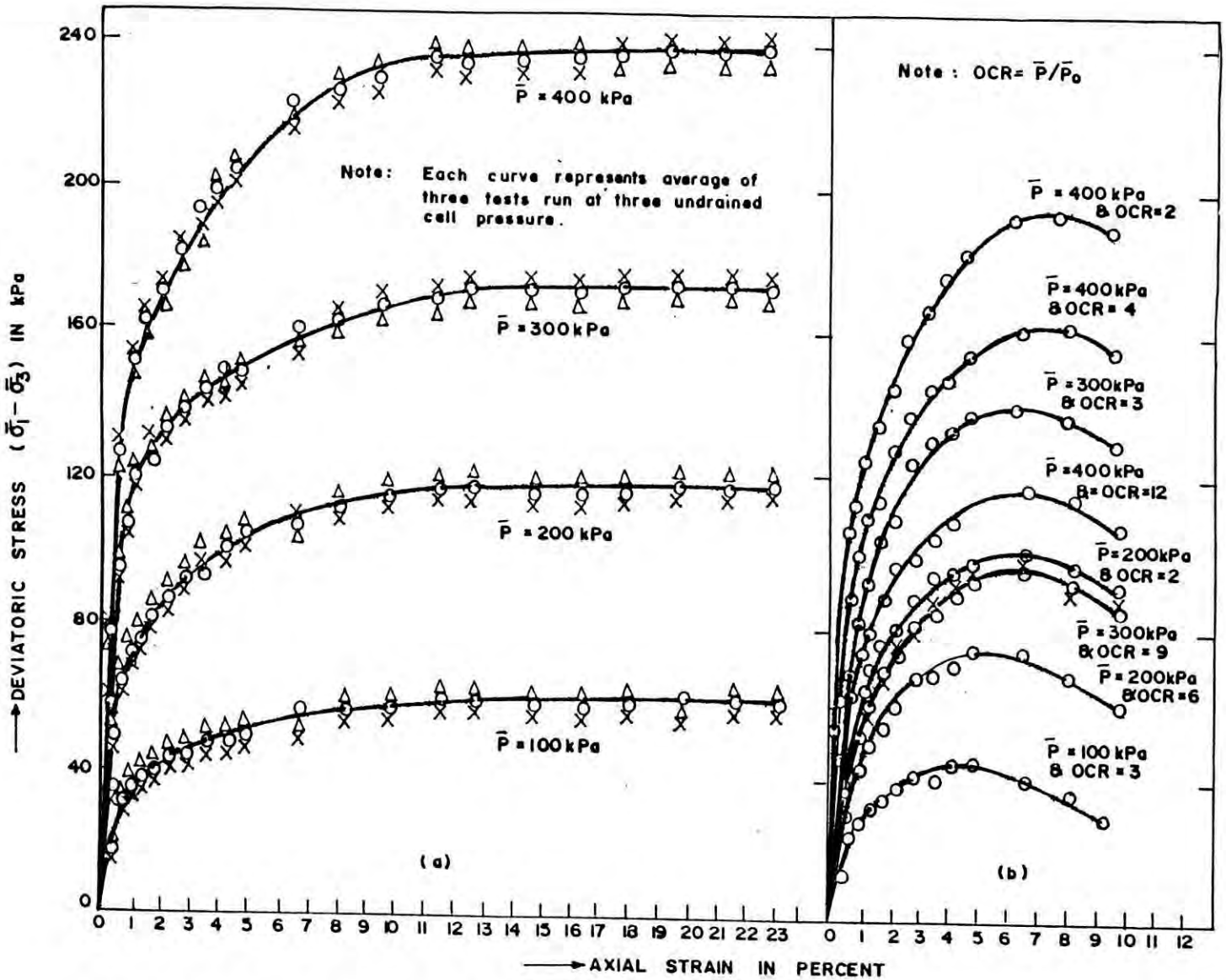


FIG. 6.5 STRESS-STRAIN CURVES FOR UNDRAINED TRIAXIAL COMPRESSION TESTS ON (a) K_oINC SAMPLES AND (b) K_oIOC SAMPLES.

6.3. Only thing is that at larger strain, the stress of K_oINC sample increases very slowly or remains almost constant, but for K_oNC sample the stress increases gradually with increasing strain. For these samples, failure stress has considered at 20% strain. Table 6.3 presents the values of $\bar{\sigma}_{(IKo)}$, $\bar{p}$, e, w, S_u, $E_u(50)$, $E_u(50)/\bar{p}$, $E_u(50)/S_u$ and $S_u/\bar{p}$ for K_oINC samples. $\bar{\sigma}_{(IKo)}$ is the initial consolidation pressure applied in Rowe cell prior to isotropic consolidation in triaxial cell. $\bar{p}$ is the maximum all-round consolidation pressure applied on the sample in triaxial cell.

The normalized parameter, $S_u/\bar{p}$ of these normally consolidated sample has an average value of 0.295 and range between 0.291 and 0.298. This value is high compared to both for K_oNC samples (0.186) and that obtained from Skempton's equation (0.195). Ameen (1985) found this value as 0.30.From Table 6.3, it is evident that shear strength increases with an increase of consolidation pressure and decreases with an increase of void ratio and water content. Fig. 6.3 includes the plots of e vs. log $\bar{p}$, e vs. log S_u, w vs. log $\bar{p}$ and w vs. log S_u for these samples. The plots are straight lines and show that void ratio or water content decreases linearly with increase of consolidation pressure or undrained shear strength. The compression index, $C_{c(iso)}$ has been found from slope of the relation, e vs. log $\bar{p}$ for K_oINC sample, which is 0.278.

From the linear relationship of w vs. log S_u, the undrained shear strength of K_oINC samples can be obtained as

$$S_{u(nc)} = 10^{(4.2815-0.0961w)} \qquad (6.2)$$

Variation of undrained secant modulus [$E_u(50)$] with consolidation pressure, $\bar{p}$ for these samples is shown in Fig. 6.6 in log-log scale. The plot shows that undrained secant modulus increases linearly with consolidation pressure for K_oINC samples. Yudhbir

TABLE 6.3: VALUES FOR K_oINC SAMPLES

Initial K_o consolidation pressure $\bar{\sigma}_{(IK_o)}$ kPa (1)	Maximum all-round consolidation pressure in triaxial cell $\bar{p}$, kPa (2)	Equilibrium void ratio e (3)	Equilibrium water content w percent (4)	Undrained shear strength S_u (5)	Undrained secant modulus at 50% of ultimate shear strength, $E_u(50)$, kPa (6)	$E_u(50)/\bar{p}$ (7)		Normalized undrained secant modulus $\Psi = E_u(50)/S_u$ (8)		Undrained shear strength $(S_u/\bar{p})$ (9)	
							Ave		Ave		Ave
80	100	0.747	29.50	29.50	5306	53.06		179		0.295	
150	200	0.664	26.07	59.60	10365	51.82	50.90	174	172	0.298	0.295
250	300	0.615	24.34	87.30	14846	49.48		170		0.291	
350	400	0.585	23.06	119.40	19702	49.25		165		0.298	

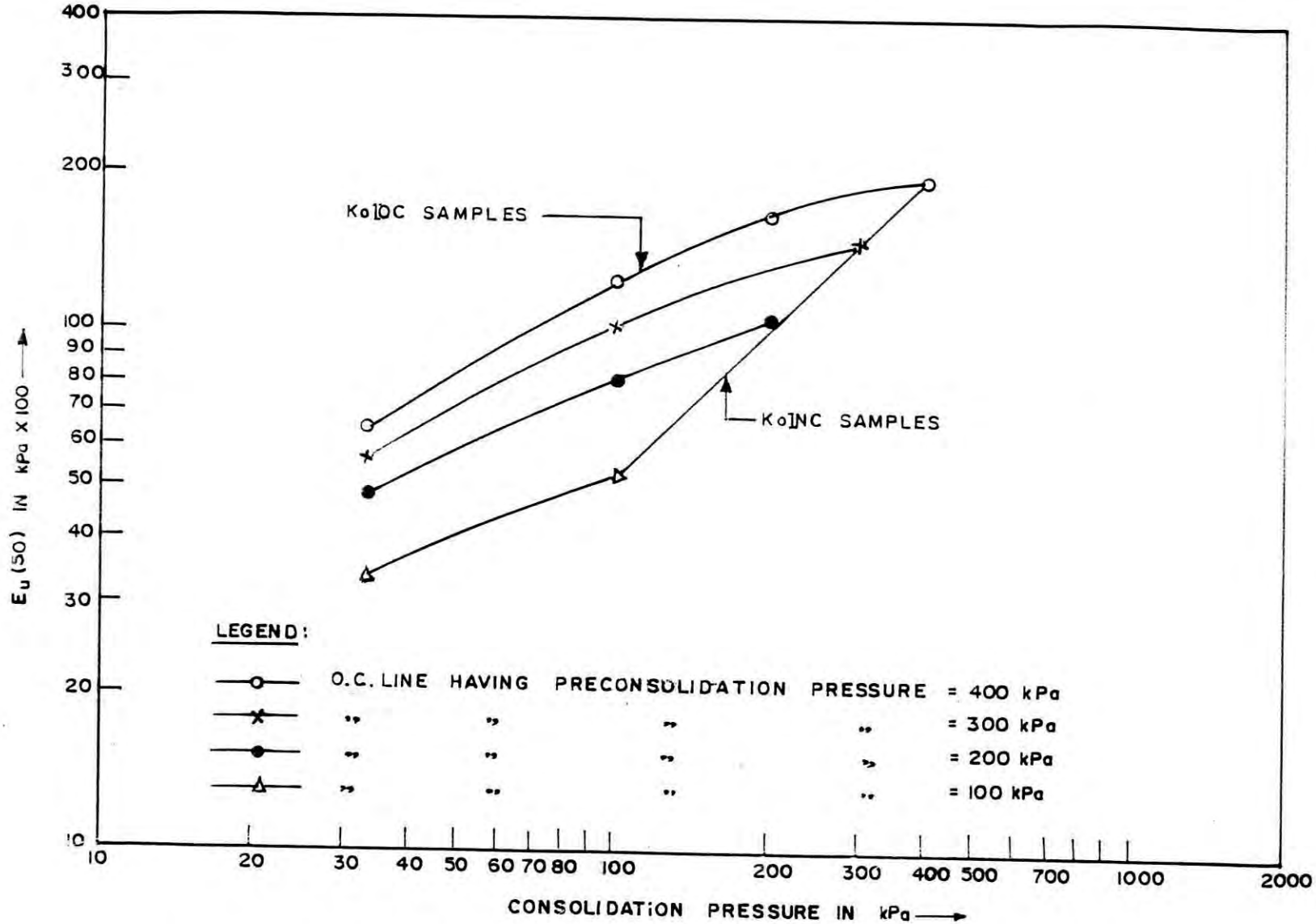


FIG. 6.6 VARIATION OF UNDRAINED SECANT MODULUS WITH CONSOLIDATION PRESSURE FOR KoINC AND KoIOC SAMPLES OF DHAKA CLAY.

et al. (1975) also found a linear relation for Weald clay, Rann of Kutch clay and Kanpur clay.

The normalized parameter, $E_u(50)/\bar{p}$ for these samples is shown under column (7) in Table 6.3. The average value of this parameter is 50.90. In the same table, under column (8), the Ψ values i.e normalized undrained secant modulus, $E_u(50)/S_u$ have been presented. The average value of Ψ for these samples is 170.

6.5 Undrained Strength test of K_oOC Samples

Undrained deviator stress and axial strain relations for K_oOC samples (K_o consolidated sample in overconsolidated state) are shown in Fig. 6.7. It is evident from these curves that undrained shear strength of these samples varies with OCR and consolidation pressure, $\bar{\sigma}_v$. For same $\bar{\sigma}_v$, the strength decreases with increase in OCR. The figure shows that the undrained shear strength is reduced by swelling process. The amount of reduction of shear strength due to swelling depends on the extent of swelling applied on the soil sample. Thus for same maximum consolidation pressure, the undrained shear strength of normally consolidated sample is greater than that of overconsolidated sample since stress recession occurs at larger strain region, all these samples show clear peak strength. Strain to attain peak deviator stress, however, varied within axial strain of 4% to 10%. For these samples, failure stress has been considered at peak deviator stress. The undrained shear strength for these soils have been defined as the half of peak deviator stress.

Table 6.4 presents the values of $\bar{\sigma}_v$, $\bar{\sigma}_{vo}$, e, w, OCR, S_u, $E_u(50)$, $E_u(50)/\bar{\sigma}_{vo}$, $E_u(50)/S_u$ and $S_u/\bar{\sigma}_{vo}$ for K_oOC samples. $\bar{\sigma}_{vo}$ is the reduced consolidation pressure under which the sample was allowed to swell in Rowe cell. Fig. 6.8 shows the variation of undrained shear strength of these samples with OCR in arithmetic scale. Each curve represents a particular overburden pressure as shown

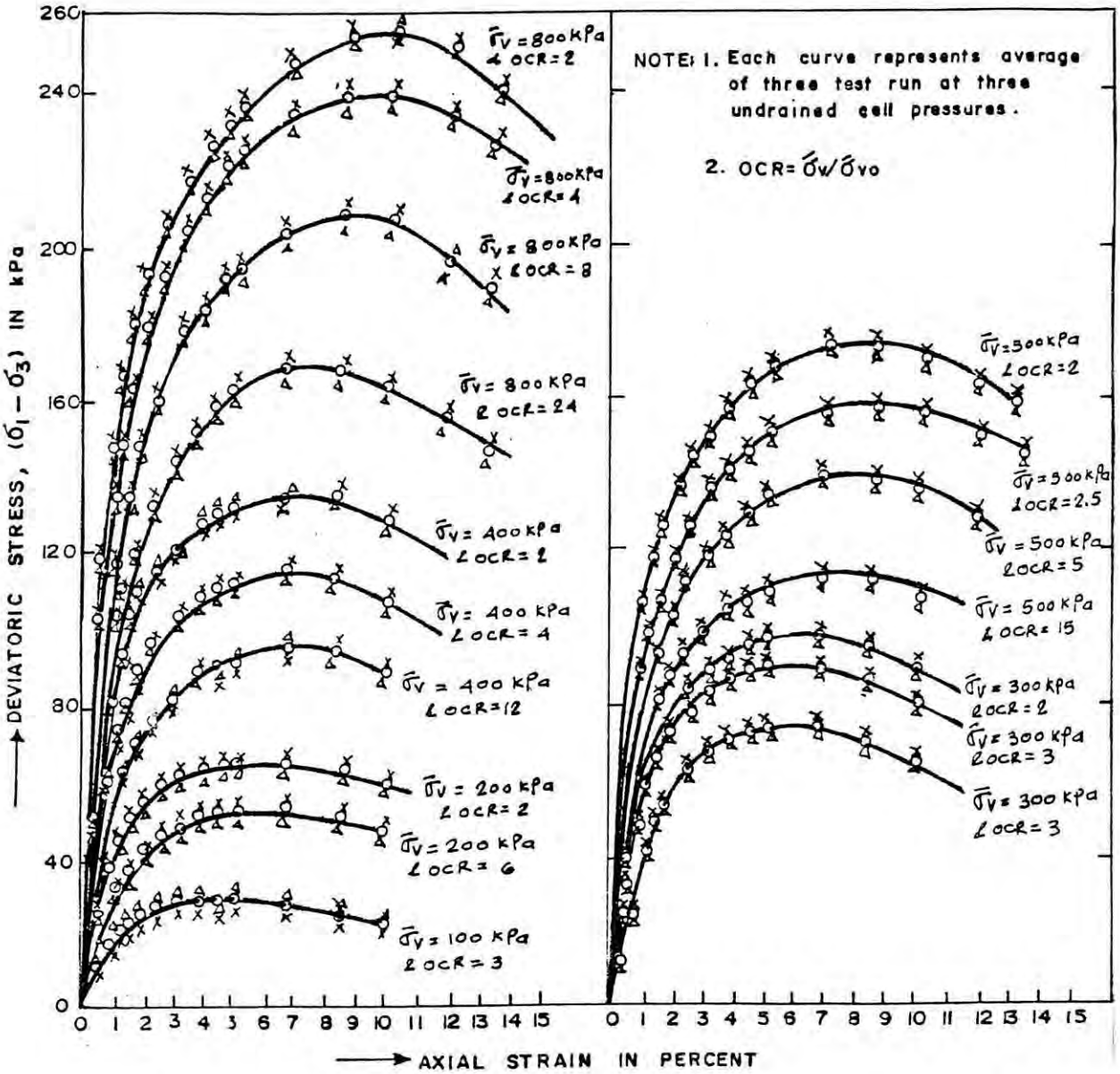


FIG. 6.7 STRESS STRAIN CURVES OF UNDRAINED TRIAXIAL COMPRESSION SHEAR TESTS FOR K_0OC SAMPLES.

TABLE 6.4: VALUES FOR K_oOC SAMPLES

Maximum vertical consol. pressure $\bar{\sigma}_v$ kPa (1)	Reduced consol. pressure $\bar{\sigma}_{vo}$ kPa (2)	Void ratio e (3)	Equilibrium water content w percent (4)	OCR (5)	Undrained shear strength Su kPa (6)	Undrained secant modulus at 50% of ulti. shear strength Eu(50) kPa (7)	$\frac{Eu(50)}{\bar{\sigma}_{vo}}$ (8)	Normalized undrained secant modulus [Eu(50)/Su] Ψ (9)	Undrained shear strength ratio [$Su/\bar{\sigma}_{vo}$] S (10)
100	100/3	0.799	32.46	3	15.00	2608	78.24	174	0.45
200	100/3	0.724	28.70	6	26.67	4103	123.09	154	0.80
200	100	0.710	28.15	2	32.70	6386	63.86	195	0.327
300	100/3	0.691	27.17	9	37.67	5111	153.33	135	1.13
300	100	0.673	26.38	3	45.90	8345	83.45	181	0.459
300	150	0.667	26.15	2	49.35	9071	60.47	184	0.329
400	100/3	0.66	25.68	12	47.00	5875	176.25	125	1.41
400	100	0.64	25.05	4	58.10	9880	98.80	170	0.581
400	200	0.634	24.65	2	68.10	13458	67.29	198	0.34

TABLE 6.4 VALUES FOR K_oOC SAMPLES (CONTD...)

Maximum vertical consol. pressure $\bar{\sigma}_v$ kPa (1)	Reduced consol. pressure $\bar{\sigma}_{vo}$ kPa (2)	Void ratio e (3)	Equilibrium water content w percent (4)	OCR (5)	Undrained shear strength Su kPa (6)	Undrained secant modulus at 50% of ulti. shear strength. Eu(50) kPa (7)	Eu(50) / $\bar{\sigma}_{vo}$ (8)	Normalized undrained secant modulus [Eu(50)/Su] Ψ (9)	Undrained shear strength ratio [Su/$\bar{\sigma}_{vo}$] S (10)
500	100/3	0.626	24.47	15	56.67	6803	204.09	120	1.70
500	100	0.614	24.04	5	70.00	11200	112.00	160	0.70
500	200	0.607	23.60	2.5	80.00	15065	75.32	188	0.40
500	250	0.605	23.40	2	87.75	15954	63.81	182	0.351
800	100/3	0.574	22.07	24	84.00	9459	283.77	113	2.52
800	100	0.559	21.70	8	104.00	14857	148.57	143	1.04
800	200	0.554	21.40	4	120.10	20886	104.43	174	0.60
800	400	0.545	21.20	2	128.12	24404	61.01	90	0.32

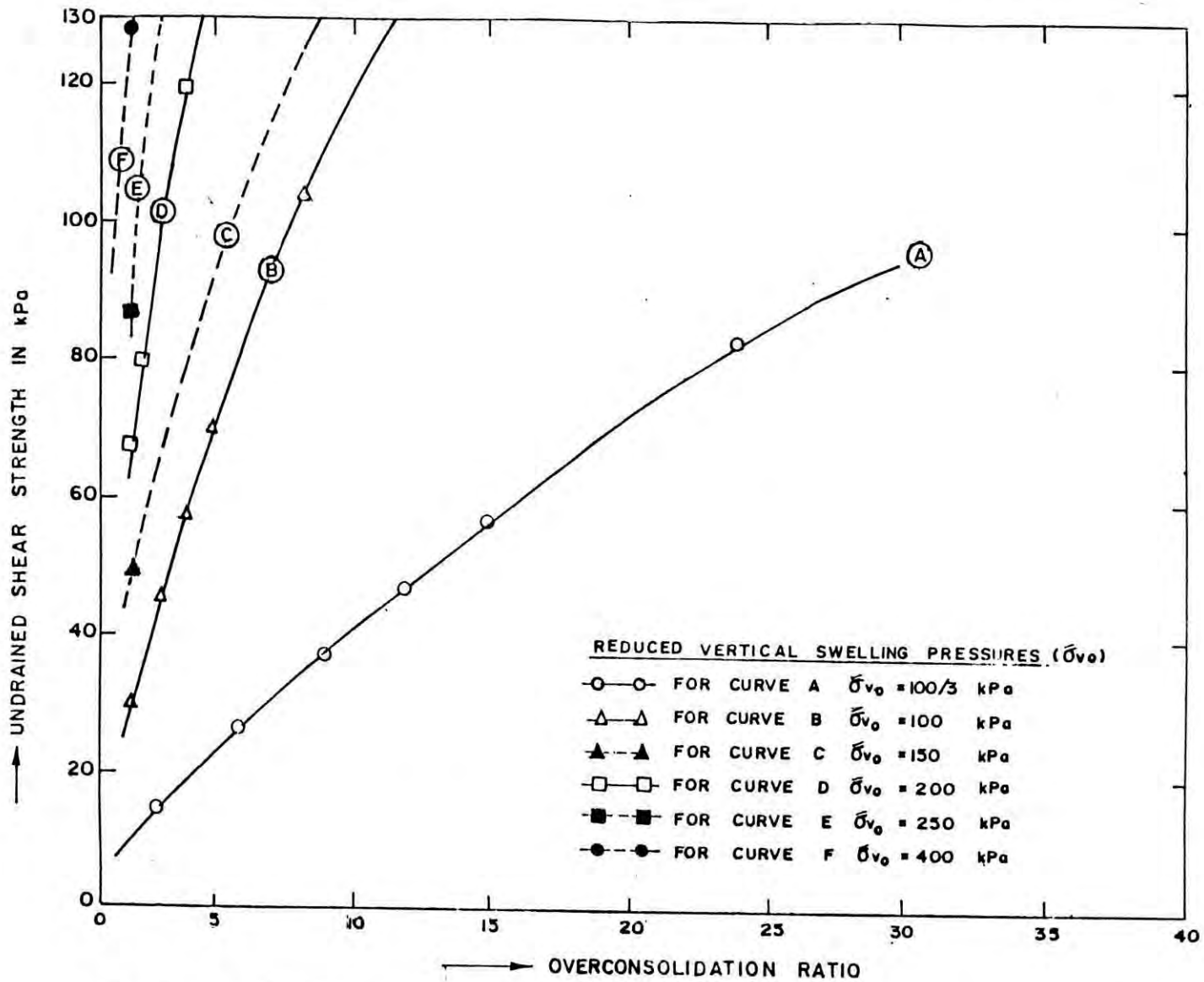


FIG. 6.8 RELATIONSHIP BETWEEN UNDRAINED SHEAR STRENGTH AND OVERCONSOLIDATION RATIO FOR KoOC SAMPLES OF DHAKA CLAY.

in the figure. The general trend of these curves are evident. From the figure, it is clear that as the overburden pressure increases, the influence of OCR on undrained shear strength pronounces highly.

These overconsolidated samples have got the undrained shear strength ratio ($S_u/\bar{\sigma}_{vo}$) which varies from 0.32 to 2.52 (for OCR value from 2 to 24). Dhaka clay clearly shows normalized behaviour and therefore, it is possible to predict its shear strength and modulus. The variation of normalized undrained shear strength, $S_u/\bar{\sigma}_{vo}$ with OCR for these samples is shown in Fig. 6.9 in arithmetic scale. The figure shows that all the data of undrained shear strength ratio ($S_u/\bar{\sigma}_{vo}$) with various reduced swelling pressure fall on a curve. The relation is linear at lower values of OCR (upto OCR value 12).

The variation of the undrained shear strength ratio ($S_u/\bar{\sigma}_{vo}$) with OCR in semi-log scale for K_oOC samples of Dhaka clay is shown in Fig. 6.10 (Curve no.7). In the figure, Dhaka clay has been compared with other six clays (after Ladd & Foot, 1974). The plot produced curve having trend similar to other clays. But when the data of K_oOC samples were plotted in log-log scale, the variation of undrained shear strength ratio ($S_u/\bar{\sigma}_{vo}$) with OCR yielded a straight line as shown in Fig. 4.11. In this figure, data for the six other clays of Fig.6.10 has been plotted in order to have comparison with Dhaka clay.

The straight line of K_oOC sample of Dhaka clay made an intercept at OCR=1. The intercept of the line is the undrained shear strength ratio ($S_u/\bar{\sigma}_v$) at normally loaded state (OCR=1) whose value is 0.186. The slope of the line, $\Lambda_b^{(K_o)}$ which is critical state pore pressure parameter for K_o stress condition, is equal to 0.821.

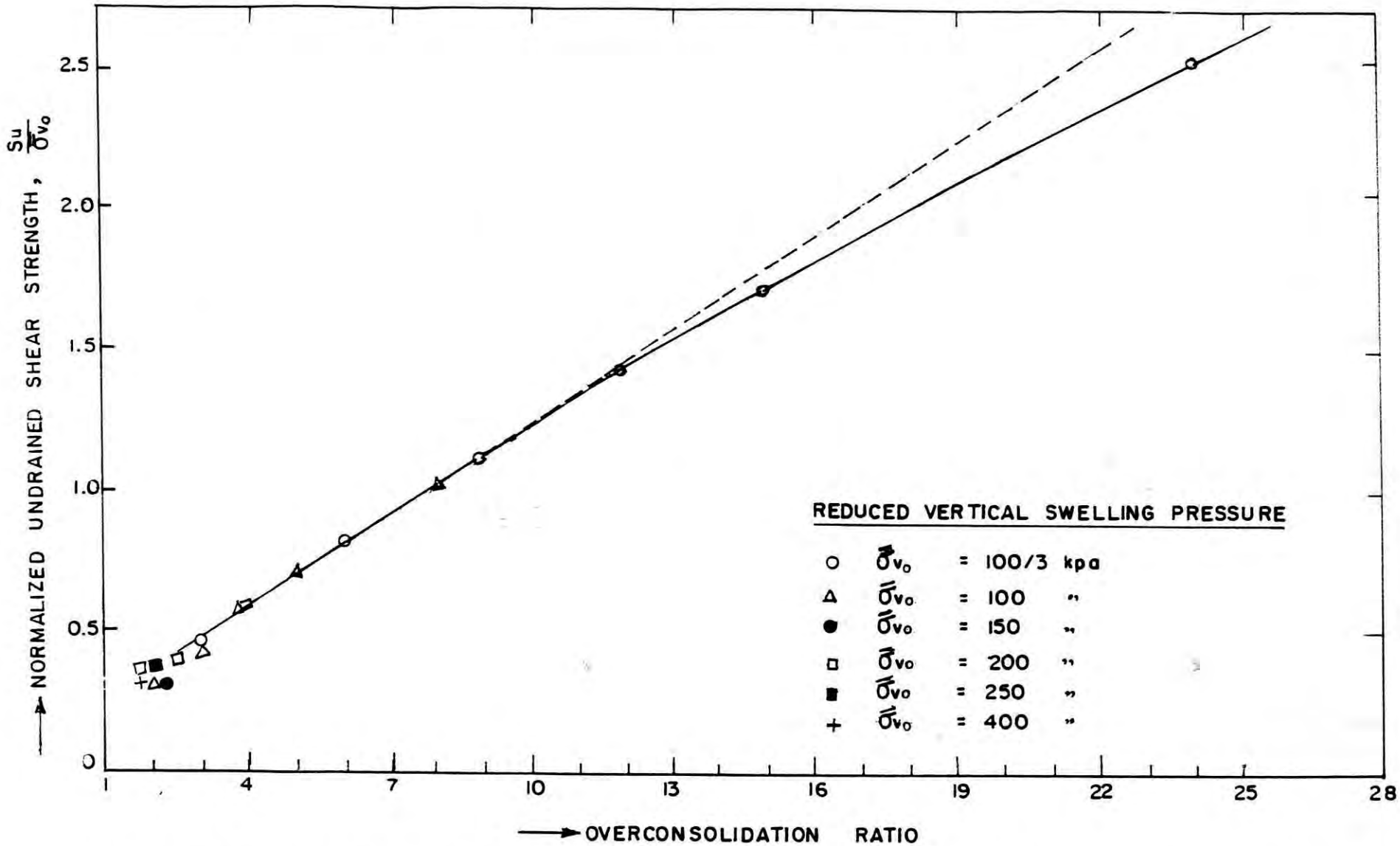


FIG. 6.9 RELATION BETWEEN NORMALIZED UNDRAINED SHEAR STRENGTH AND OVERCONSOLIDATION RATIO FOR K_0OC SAMPLES OF DHAKA CLAY

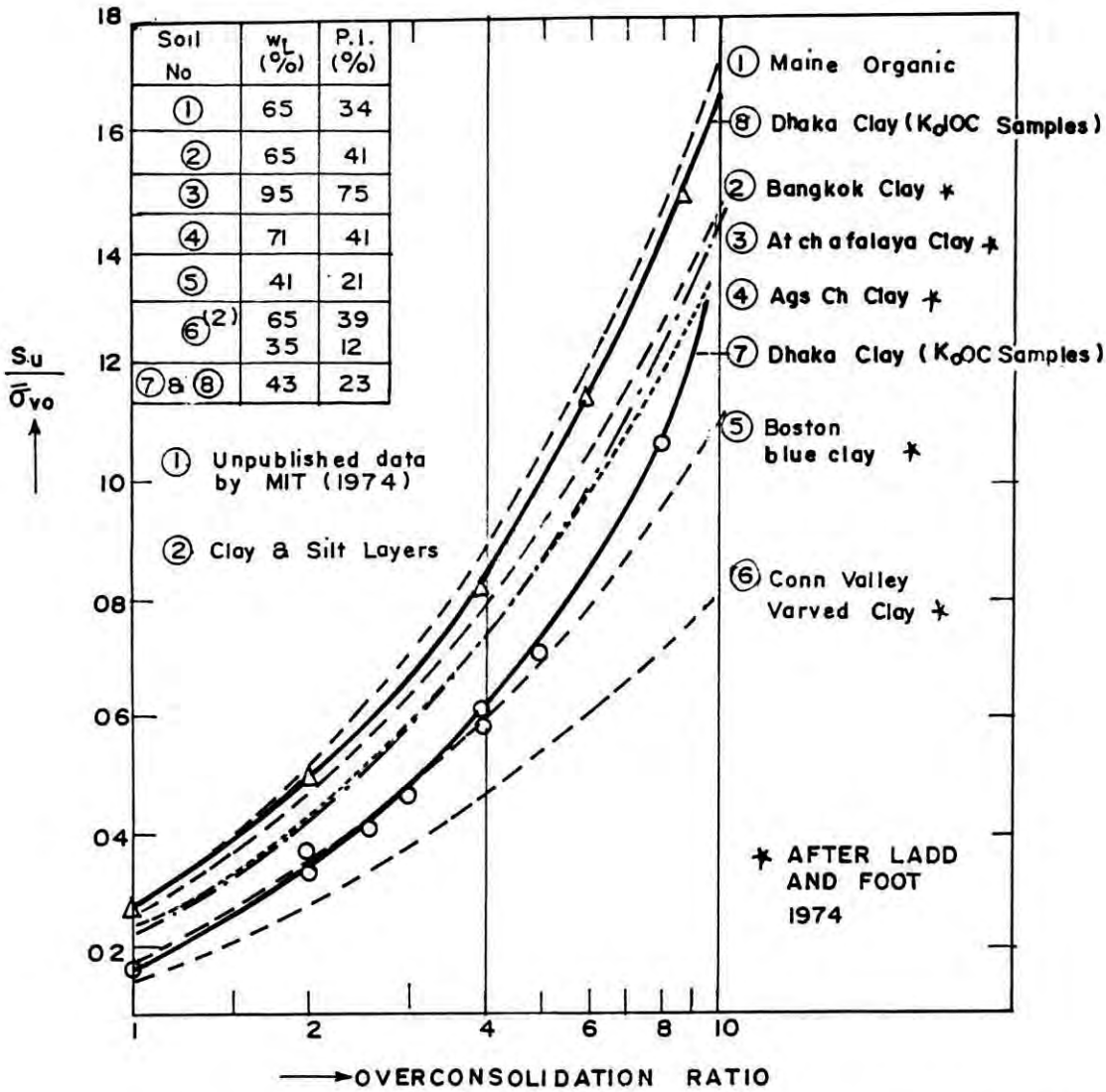


FIG. 6.10 UNDRAINED STRENGTH RATIO VERSUS OCR FOR DHAKA CLAY COMPARED TO OTHER SIX CLAY

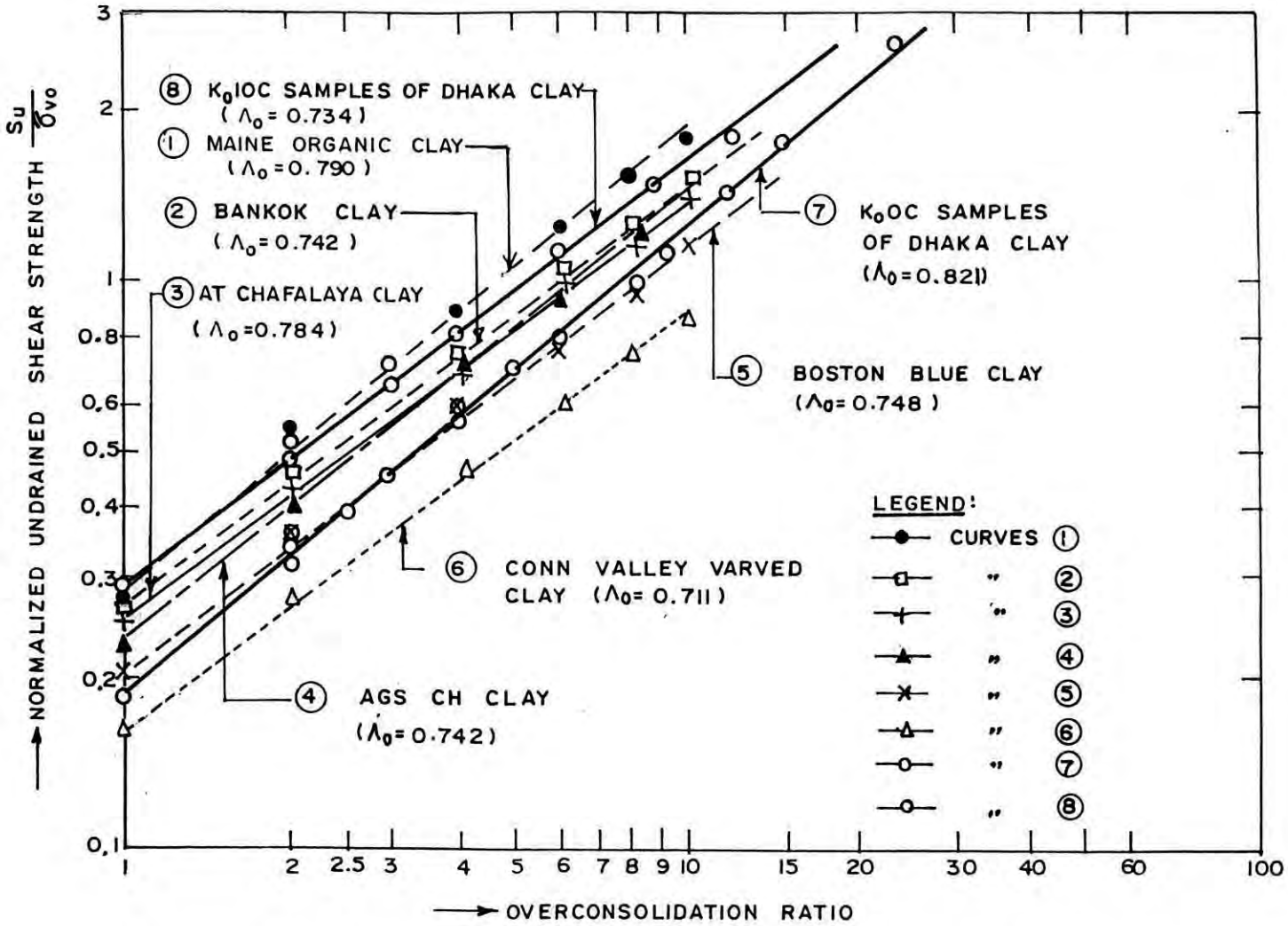


FIG. 6.11 NORMALIZED UNDRAINED SHEAR STRENGTH AS A FUNCTION OF OVERCONSOLIDATION RATIO FOR DHAKA CLAY COMPARED WITH SIX OTHER CLAYS

From Table 6.4, it is evident that shear strength increases with an increase of consolidation pressure and decreases with an increase of void ratio and water content for K_oOC samples also. Fig. 6.12 presents plots of water content versus consolidation pressure and water content versus undrained shear strength for these samples. Fig.6.13 presents plots of void ratio versus consolidation pressure and void ratio versus undrained shear strength. The plots are approximately straight lines and show that water content and void ratio decreases with increase of consolidation pressure or undrained shear strength. The swelling index, $C_s^{(K_o)}$ has been determined from the relation e vs. $\log\sigma_v$ for K_oOC samples, which is 0.025 ± 0.01. The value of $\Lambda_o^{(K_o)}$ found from the relation, $\Lambda_o^{(K_o)}=1-[C_s^{(K_o)}/C_c^{(K_o)})]$, becomes equal to 0.90. Thus it shows that the value of $\Lambda_o^{(K_o)}$ obtained from e vs. $\log\bar{\sigma}_v$ plot found K_o consolidation tests gives higher value than that found from $\mathrm{Log}(S_u/\bar{\sigma}_{vo})$ vs. log (OCR) plot. Mitachi and Kitago (1976) also reported similar observation.

Variation of undrained secant modulus [$E_u(50)$] with consolidation pressure, $\bar{\sigma}_v$ for these overconsolidated samples is shown in Fig. 6.4 in log-log scale. General similarities in trend of these rebound curves are evident. Similar trend has been found by Yudhbir et.al.,(1975) for Weald clay, Rann of Kutch clay and Kanpur clay.

The normalized parameter, $E_u(50)/\bar{\sigma}_{vo}$ for these samples is shown under column (8) in Table 6.4. From the table, it shows that the parameter increases with an increase of OCR. The variation of this normalized parameter as a function of OCR is plotted in Fig. 6.14 in semilog scale. The figure shows that modulus increases with an increase of OCR. At higher OCR values, the same effect is highly pronounced. Yudhbir et al., (1975) has found similar trend for Weald clay, Rann of Kutch clay and Kanpur clay. Ladd (1964) and Varadarajan (1973) reported similar observation for other clays.

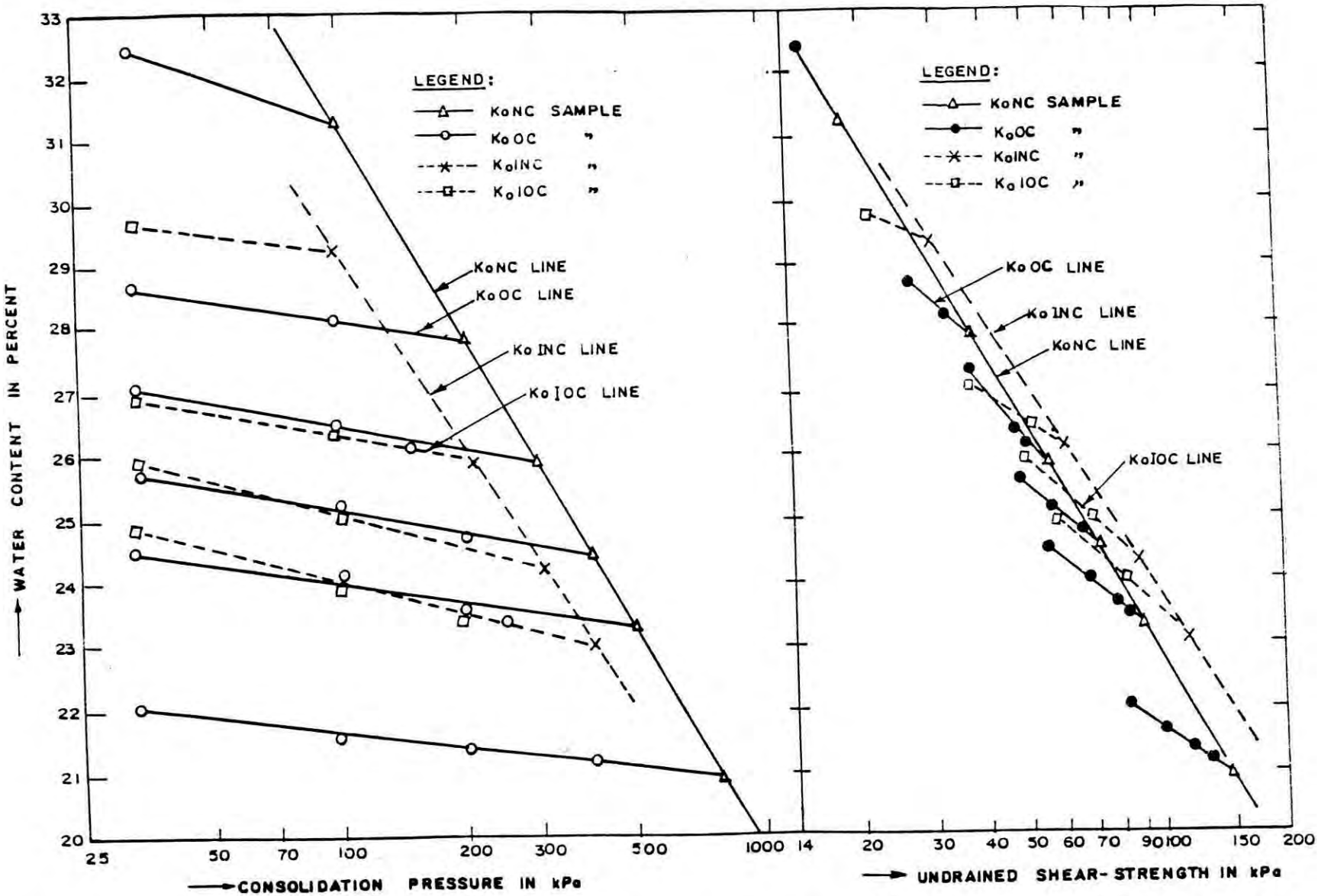


FIG. 6.12 WATER CONTENT VERSUS CONSOLIDATION PRESSURE OR UNDRAINED SHEAR STRENGTH FOR DHAKA CLAY

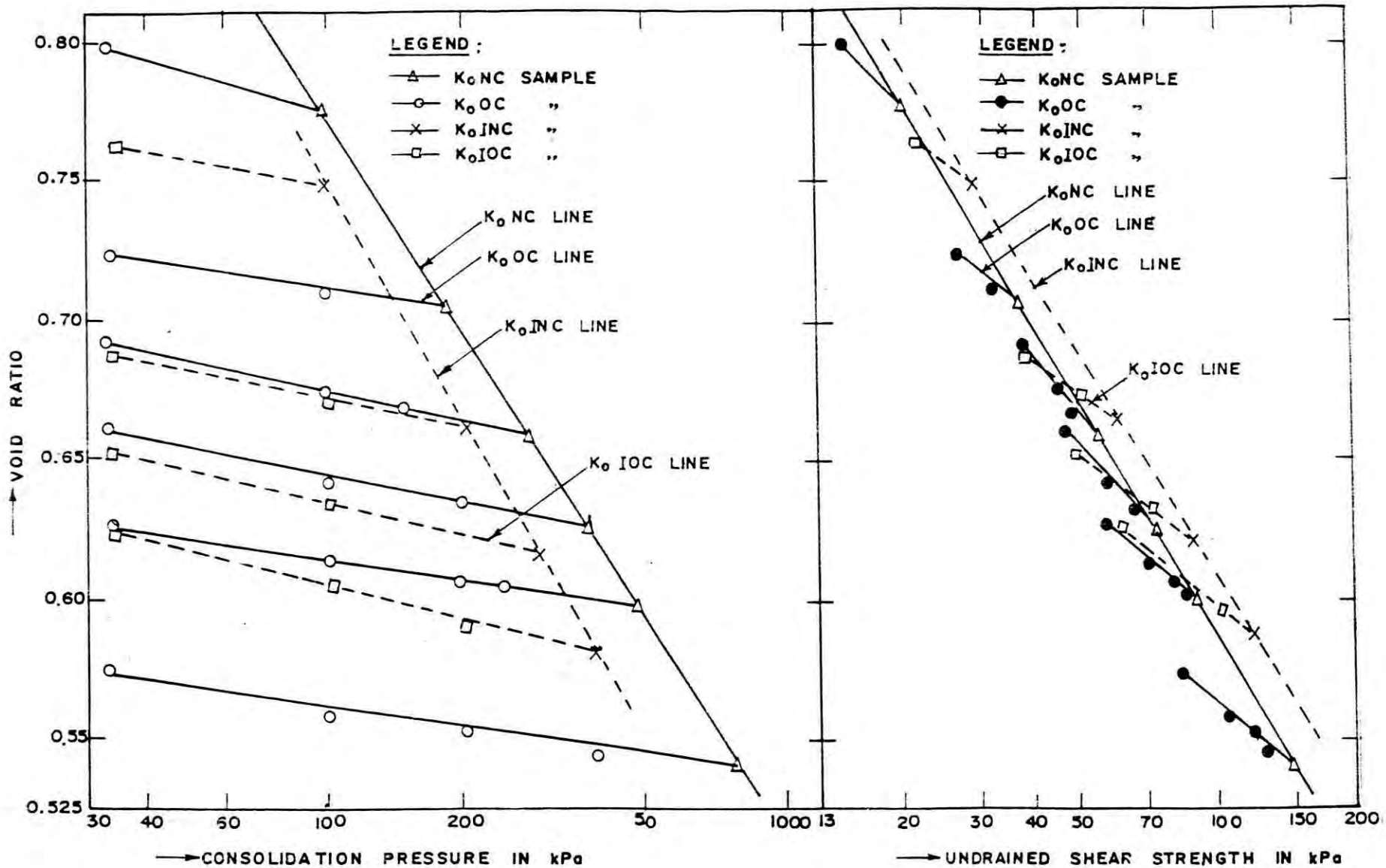


FIG. 6.13 VOID RATIO VERSUS CONSOLIDATION PRESSURE OR UNDRAINED SHEAR STRENGTH FOR DHAKA CLAY

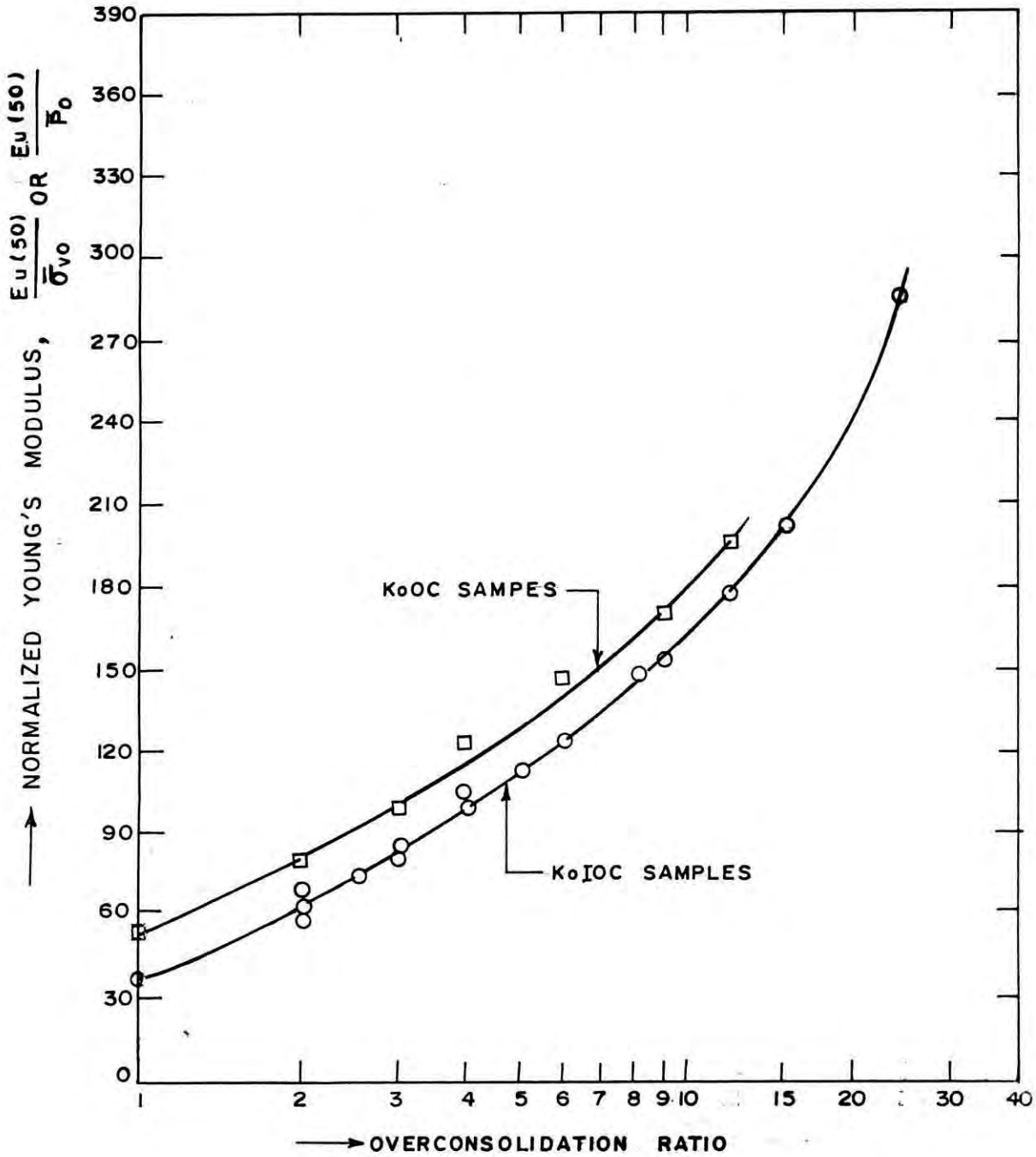


FIG. 6.14 NORMALIZED YOUNG'S MODULUS $E_u^{(50)}/\bar{\sigma}_{vo}$ OR $E_u^{(50)}/\bar{P}_o$ VERSUS OVERCONSOLIDATION RATIO FOR DHAKA CLAY

In Table 6.4, shows under column (9), the $\bar{\Psi}$ values i.e normalized undrained secant modulus, $E_u(50)/S_u$ have been presented. The $\bar{\Psi}$ values for these samples varied from 113 to 198 (for OCR value from 2 to 24). The variation of $\bar{\Psi}$ with OCR for these samples has been shown in Fig. 6.15. The figure shows that $\bar{\Psi}$ gradually decreases with the increase of OCR. In Fig. 6.16, a comparison of $\bar{\Psi}$ value of Dhaka clay has been made with few other clays. The $\bar{\Psi}$ values of these clays has been determined by Appolonia, Poulos and Ladd (1971) from direct simple shear tests, whereas in this study, $\bar{\Psi}$ value has been found from triaxial compression shear tests. However, the general trend of the curve of Dhaka clay is similar to that of other three clays.

It has been seen that for Dhaka clay, the moisture content changes during shearing process and it varies with the effective stress. Because of the rebound characteristics of the clay When subjected to application or removal of effective stress, it is important to carry out study with the objective of determining the variation of shear strength of the clay at a constant moisture content and to determine Hvorslev (1937) parameter. Here to refer that Hvorslev (1937) proposed a shear strength theory to take the above mentioned observations in to account, considering cohesion to be the result of physio-chemical bond forces. The shear strength equation is as follows:

$$S = C_e + \sigma \tan \phi_e \qquad (6.3)$$

for a particular moisture content, where C_e and ϕ_e are true cohesion and true angle of internal friction respectively.

The upper part of the Fig. 6.17(a) shows the relationship between water content at failure and consolidation pressure for K_oNC and K_oOC samples whose undrained shear strength is plotted in the lower figure. The consolidation pressure axis is common to both

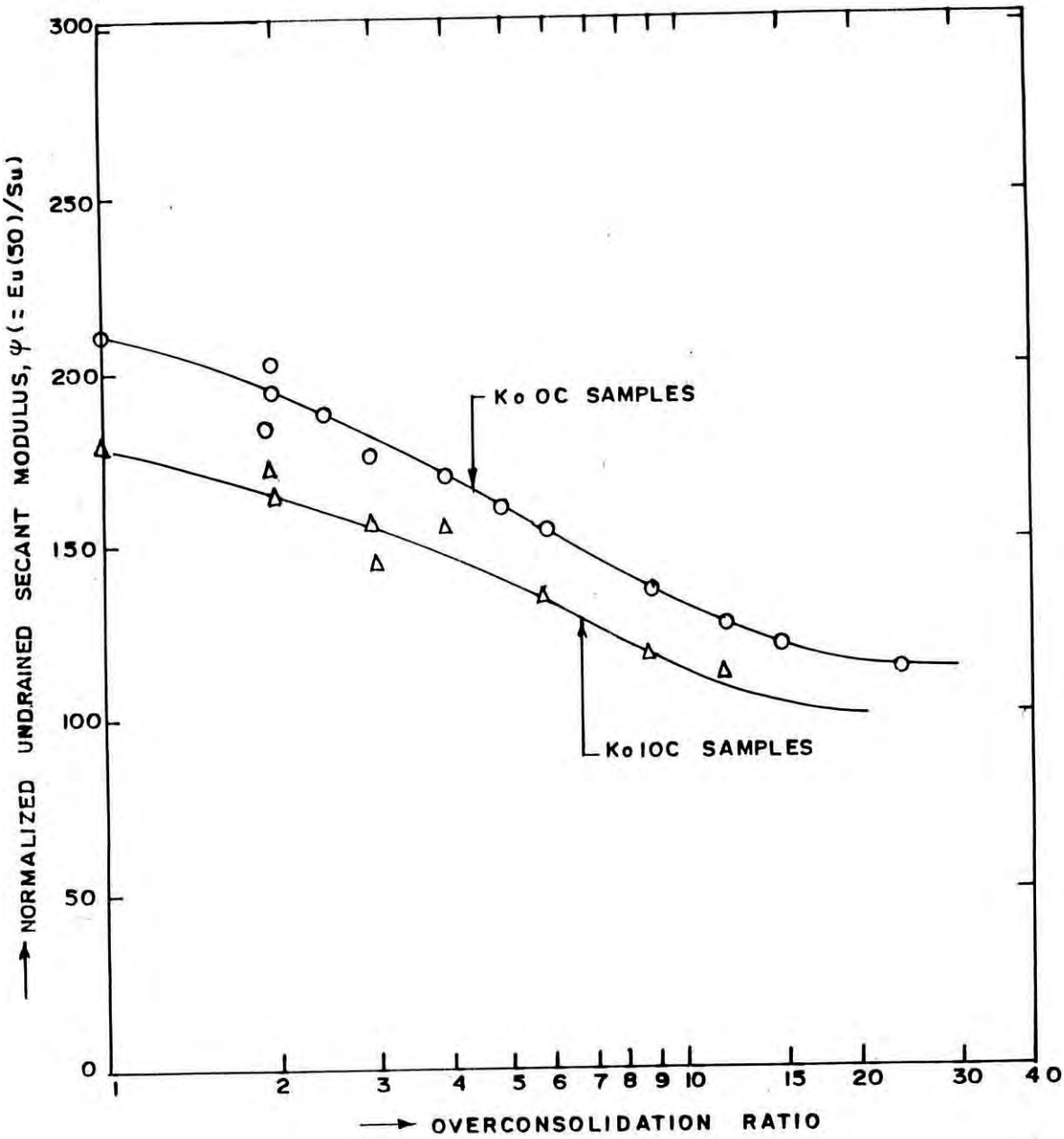


FIG. 6.15 NORMALIZED UNDRAINED SECANT MODOLUS, ψ (= Eu(50)/Su) AS A FUNCTION OF OVERCONSOLIDATION RATIO FOR DHAKA CLAY.

CLAY	WL %	P.I. %	SYM-BOL	SOURCE
BANGKOK CLAY	65	41	--□--	DATA FROM CKoU DSS TESTS (AFTER APPOLONIA POULOS AND LADD, 1971)
MAINE ORGANIC CLAY	65 ± 10	33 ± 2	--△--	
BOSTON BLUE CLAY	41	22	--○--	
DHAKA CLAY (Ko CONSOL.)	41	22	--x--	DATA FROM UNDRAINED TRIAXIAL TEST (AFTER AMEEN, 1985)
" (ISO. CONSOL.)	"	"	--+--	
DHAKA CLAY (KoOC SAMPLES)	43	23	—⊗—	DATA FROM AUTHOR
DHAKA CLAY (KoIOC SAMPLES)	"	"	—⊠—	

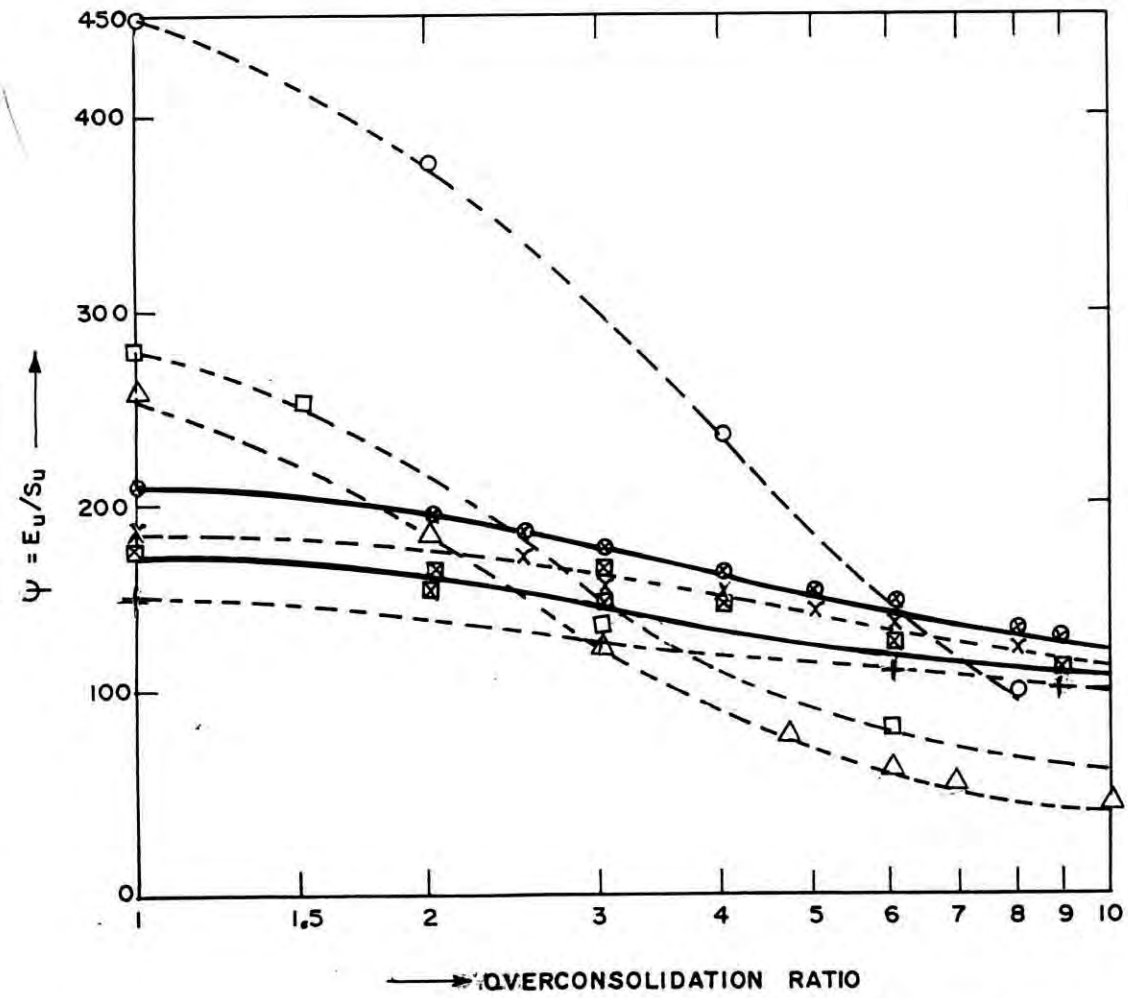
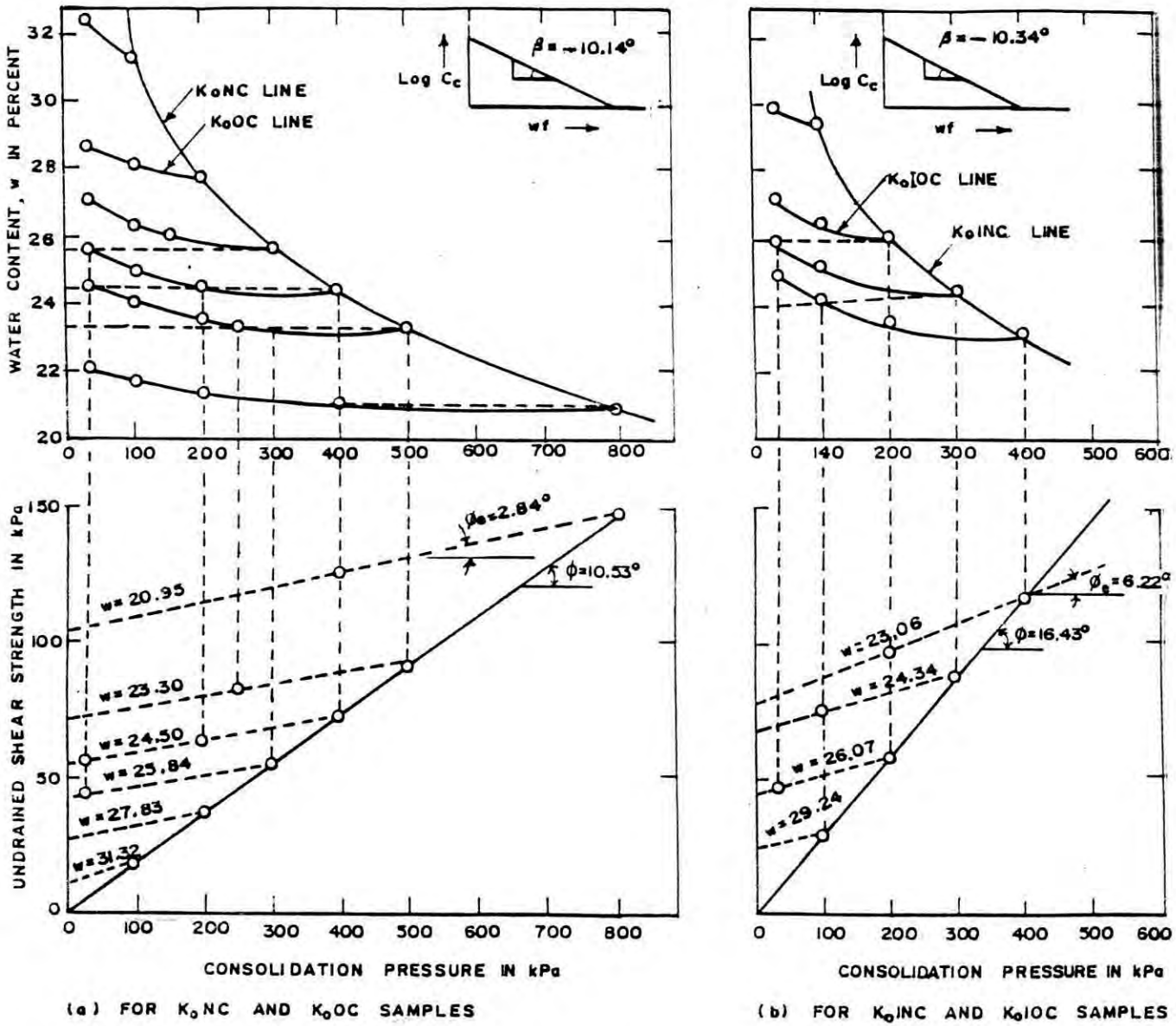


FIG. 6.16 COMPARISON OF NORMALIZED UNDRAINED SECANT MODULUS FOR DHAKA CLAY WITH THAT OF OTHER CLAYS.



(a) FOR K_0NC AND K_0OC SAMPLES

(b) FOR K_0INC AND K_0IOC SAMPLES

FIG. 6.17 DETERMINATION OF COHESION COMPONENTS AT CONSTANT MOISTURE CONTENT.

figures. For same water content (in the upper part of the figure), shear strength points were plotted in the lower part of the figure. Each plot having same water content represents the variation of shearing strength of the clay at one moisture content at failure. The points lie on a straight line of slope, ϕ_e which intercepts the shearing stress axis at a point with ordinate C_e. Each line has got individual C_e but ϕ_e is approximately constant. The equation (6.3) can be written as

$$S = C_o \exp(-\beta' W_f) + \sigma \tan \phi_e \qquad (6.4)$$

where C_o = true cohesion at zero water content

W_f = water content at failure

β' = slope of plot of log C_e vs. water content at failure

Log C_e vs. w_f (i.e. water content at failure) was plotted and its gradient is the value of β'. The values obtained for K_oOC samples are $\beta' = -10.14°$ and $\phi_e = 2.84°$. For K_oNC samples, the value of ϕ obtained from the slope of S_u vs. $\bar{\sigma}_v$ plot found from undrained tests is 10.53°.

6.6 Undrained Strength Test of K_oIOC Samples

Undrained deviator stress and axial strain relations for K_oIOC samples (isotropically consolidated sample in overconsolidated state with initial K_o consolidation) are presented in Fig. 6.5(b). It is evident from these curves that undrained shear strength of these samples varies with consolidation pressure, $\bar{p}$ and OCR. For same $\bar{p}$, shear strength decreases with increase in OCR. All the samples show clear peak strength. Strain to attain peak deviator stress varied within axial strain of 4% to 9%. The undrained shear strength for these samples are considered as half of the peak deviator stress.

Table 6.5 presents the values of $\bar{\sigma}_{(IK_o)}$, $\bar{p}$, $\bar{p}_o$, e, w, OCR, S_u, E_u-

TABLE 6.5: VALUES FOR K_oIOC SAMPLES

Initial K_o consolidation pressure $\bar{\sigma}_{(IK_o)}$ kPa (1)	Maximum all-round consolidation pressure in triaxial cell, $\bar{p}$ kPa (2)	Reduced all-round swelling pressure in triaxial cell, $\bar{p}_o$ kPa (3)	Void ratio e (4)	Equilibrium water content w Percent (5)	OCR (6)	Undrained shear strength S_u kPa (7)	Undrained secant modulus at 50% of ultimate shear strength, $E_u(50)$ kPa (8)	$E_u(50)/\bar{p}_o$ (9)	Normalized undrained secant $(E_u(50)/S_u]$ Ψ (10)	Undrained shear strength ratio $[S_u/\bar{p}_o]$ S (11)
80	100	100/3	0.761	29.73	3	21.67	3365	100.95	155	0.65
150	200	100/3	0.688	27.11	6	37.33	4977	149.31	133	1.12
150	200	100	0.673	26.40	2	49.00	8085	80.85	165	0.49
250	300	100/3	0.651	25.91	9	48.67	5685	170.55	117	1.46
250	300	100	0.633	25.10	3	70.30	10042	100.42	143	0.703
350	400	100/3	0.626	24.91	12	58.74	6483	194.49	110	1.762
350	400	100	0.605	24.02	4	81.10	12476	124.76	153	0.811
350	400	200	0.59	23.50	2	97.60	16626	83.13	170	0.488

(50), $E_u(50)/\bar{p}_o$, $E_u(50)/S_u$ and $S_u/\bar{p}_o$ for these samples. $\bar{P}_o$ is the reduced consolidation pressure under which the sample was allowed to swell in triaxial cell. Fig. 6.18 shows the variation of undrained shear strength with OCR in arithmetic scale. Each curve represents a particular overburden pressure as shown in the figure. From the curves of these samples, it is evident that as the overburden pressure increases, the undrained shear strength increases rapidly with increase of OCR.

Strength ratio ($S_u/\bar{p}_o$) for these samples varies from 0.49 to 1.762 (for OCR value from 2 to 12). The variation of this normalized undrained shear strength, $S_u/\bar{p}_o$ with OCR for these sample is shown in Fig. 6.19 in arithmetic scale. The figure shows that all the data of undrained shear strength ratio ($S_u/\bar{p}_o$) with various reduced swelling pressures fall on a curve. The relation is linear at lower values of OCR (upto OCR value 6).

The variation of undrained shear strength ratio ($S_u/\bar{p}_o$) with OCR in semi-log scale for K_oIOC samples of Dhaka clay is shown in Fig. 6.10 (curve no. 8). The variation is a curve similar to that for K_oOC samples. The figure shows that for same OCR, the undrained shear strength ratio for K_oIOC samples is greater than that of K_oOC samples. In Fig. 6.11, the data for K_oIOC samples have been plotted as $S_u/\bar{p}_o$ vs. OCR in log-log scale with other six clays of Fig. 6.10. In the log-log scale, K_oIOC samples of Dhaka clay produced straight line like K_oOC samples. The intercept of the line which is undrained shear strength ratio ($S_u/\bar{p}$) at normally loaded state (OCR=1) has got value of 0.295. The slope of the line $\Lambda_o^{(iso)}$ which is critical state pore pressure parameter for isotropic stress condition, is equal to 0.734.

From Table 6.5, it is clear that shear strength increases with an increase of consolidation pressure and decreases with an increase of void ratio and water content for K_oIOC sample also. This can be found from Fig. 6.12 and 6.13. The swelling index, $C_s^{(iso)}$ has

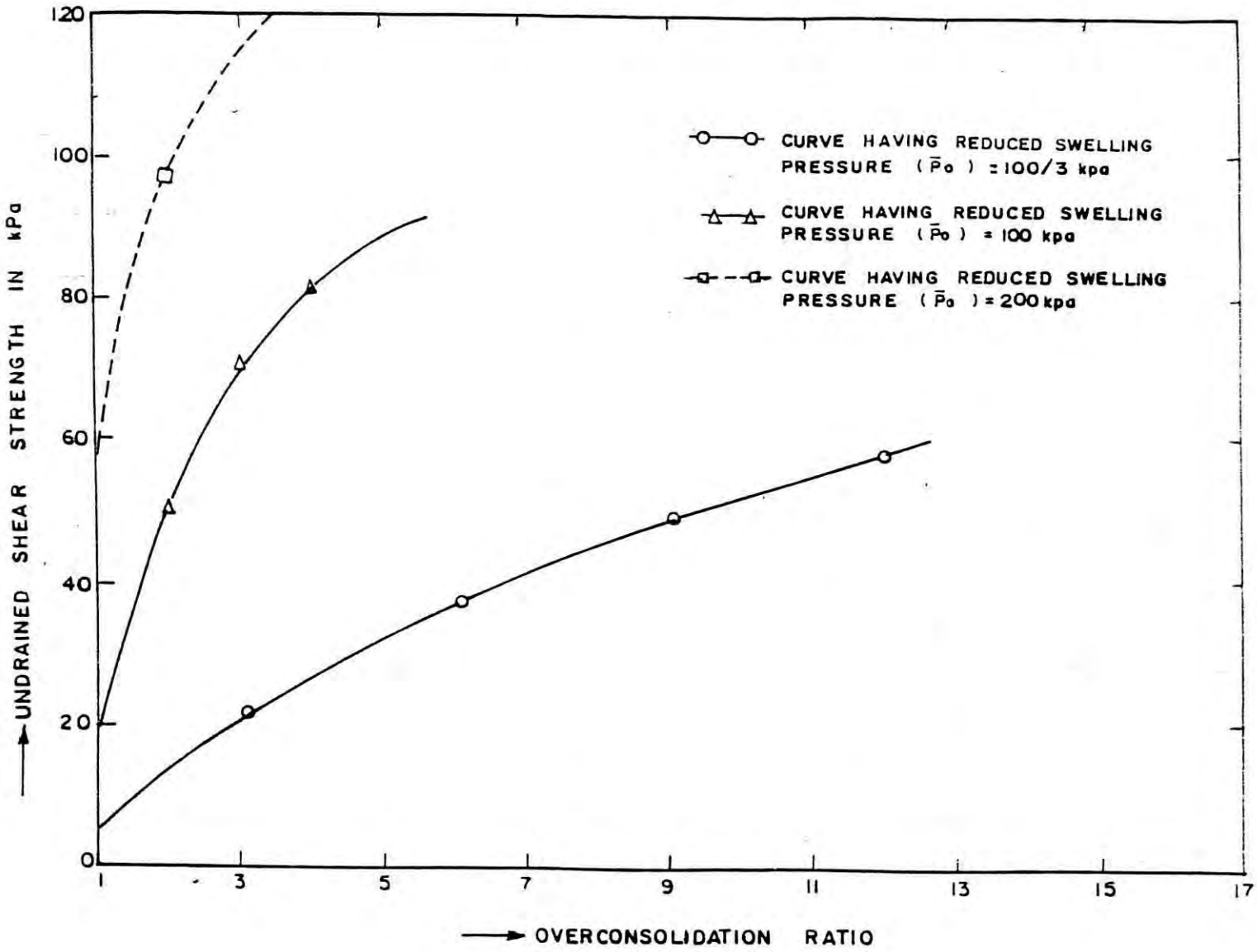


FIG.6.18 RELATION BETWEEN UNDRAINED SHEAR STRENGTH AND OVERCONSOLIDATION RATIO FOR K_0IOC SAMPLES OF DHAKA CLAY

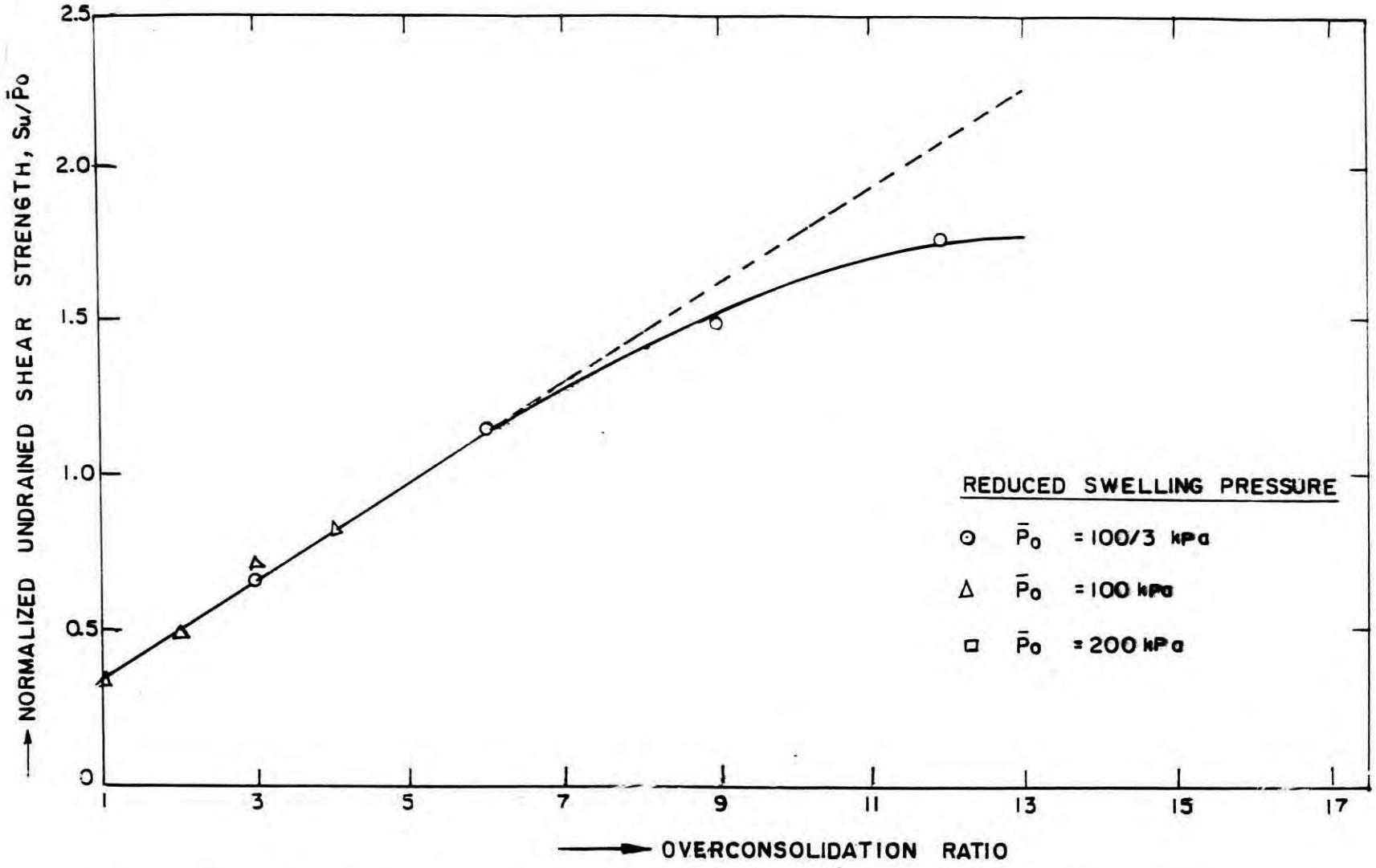


FIG. 6.19 RELATION BETWEEN NORMALIZED UNDRAINED SHEAR STRENGTH AND OVERCONSOLIDATION RATIO FOR Ko10C SAMPLES OF DHAKA CLAY.

been determined from relation e vs. log $\bar{p}$ for K_oIOC samples, which is 0.038±0.01. The value of $\Lambda_o(iso)$, found from relation, $\Lambda_o(iso)=1-[C_s(iso)/C_c(iso)]$, becomes equal to 0.863. Thus it shows that the value of $\bar{\Lambda}_o(iso)$ obtained from e vs. log$\bar{p}$ plot found from isotropic consolidation tests gives higher value than that found from Log(S_u/ $\bar{p}_o$) vs. log OCR plot. Mitachi and Kitago (1976) also reported similar observation.

The variation of undrained secant modulus [E_u(50)] with consolidation pressure, $\bar{p}$ for these overconsolidated samples is shown in Fig. 6.6. General similarities in trend of these rebound curves are evident.

In Table 6.5, the normalized parameter, E_u(50)/ $\bar{p}_o$ for these samples is shown. The variation of this parameter with OCR is shown in Fig. 6.14 in semi log scale which shows that the parameter increases with the increase of OCR. From Table 6.5, it can be seen that the values of Ψ, the normalized undrained secant modulus, E_u(50)/S_u, varies from 110 to 170 for K_oIOC samples (with OCR value from 2 to 12). The variation of Ψ with OCR for these samples is shown in Fig. 6.15. The figure shows that Ψ decreases gradually with increase of OCR. In Fig. 6.16, a comparison of Ψ values for these samples of Dhaka clay has been shown with few other clays. The nature of variation is, however, similar to that of other clay and K_oOC samples of Dhaka clay.

As depicted for K_oOC samples, the variation of shear strength of K_oIOC samples at a constant moisture content has been determined and shown in Fig. 6.17(b). For same water content, shear strength points were plotted which lie on a straight line. Each line represents the variation of shear strength of the samples at one moisture content at failure. The value of β' has been determined from gradient of log C_e vs. W_f plot. The values obtained for K_oIOC samples are β' = - 10.34° and ϕ_e=6.22°. For K_oINC samples,

the value of ø obtained from the slope of S_u vs. p plot found from undrained tests is 16.43°.

6.7 Rate of Increase of Undrained Shear Strength of Dhaka Clay with Increase of OCR

For same overburden pressure, the strength of a clay is increased due to increase of OCR. The rate of increase of shear strength of Dhaka clay can be found from the plot of μ vs. OCR, shown in Fig. 6.20 for K_oNC and K_oOC samples. Here μ is the ratio of undrained shear strength $[S_u(^{oc}-\bar{\sigma}_{vo})]$ at overconsolidated state to the undrained shear strength $[S_u(^{nc}-\bar{\sigma}_{vo})]$ at normally consolidated state, both are at same overburden pressure, $\bar{\sigma}_{vo}$. The dotted line in Fig. 6.20 is the extension of the initial linear relation of μ vs. OCR. The figure shows that the plot is linear at lower values of OCR (upto OCR value 9). Beyond this level of OCR, the rate of increase of strength decreases gradually with increase of OCR value.

Similarly, for K_oINC and K_oIOC samples, μ vs. OCR plot is shown in Fig. 6.21. The figure shows that the plot is linear upto OCR value 6. Beyond this range of OCR, the rate of increase of shear strength decreases gradually with increase of OCR value.

6.8 Model for Prediction of Undrained Shear Strength of Dhaka Clay

6.8.1 Soil in Normally Consolidated State

The cam-clay theory presented by schofield and worth (1968) gives the following relation for undrained shear strength ratio for a normally consolidated clay

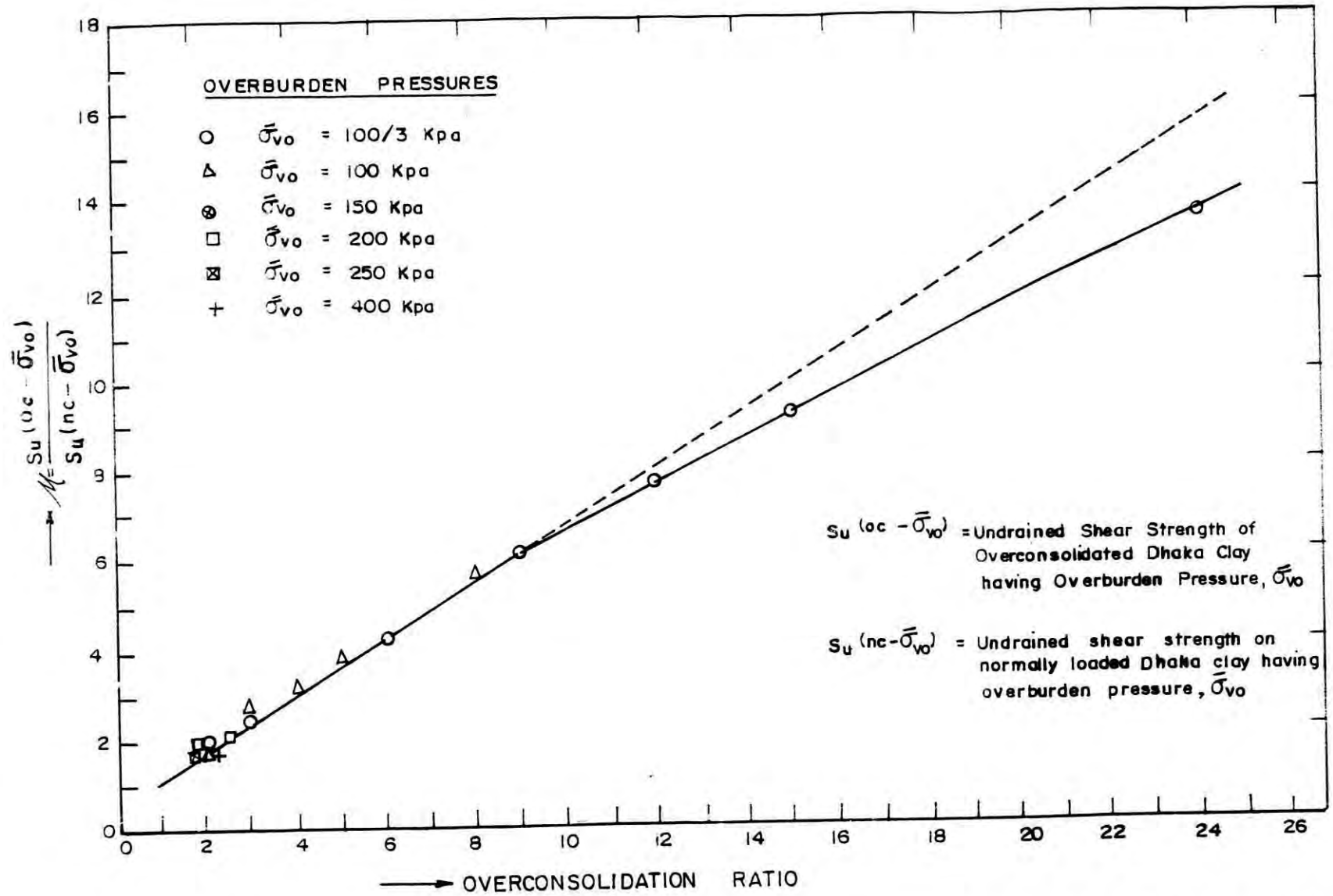


FIG. 6.20 RATE OF INCREMENT OF UNDRAINED SHEAR STRENGTH OF OVERCONSOLIDATED DHAKA CLAY WITH RESPECT TO THAT OF NORMALLY LOADED STATE OF THE CLAY HAVING SAME OVERBURDEN PRESSURE (SUBJECTED TO Ko STRESS CONDITION).

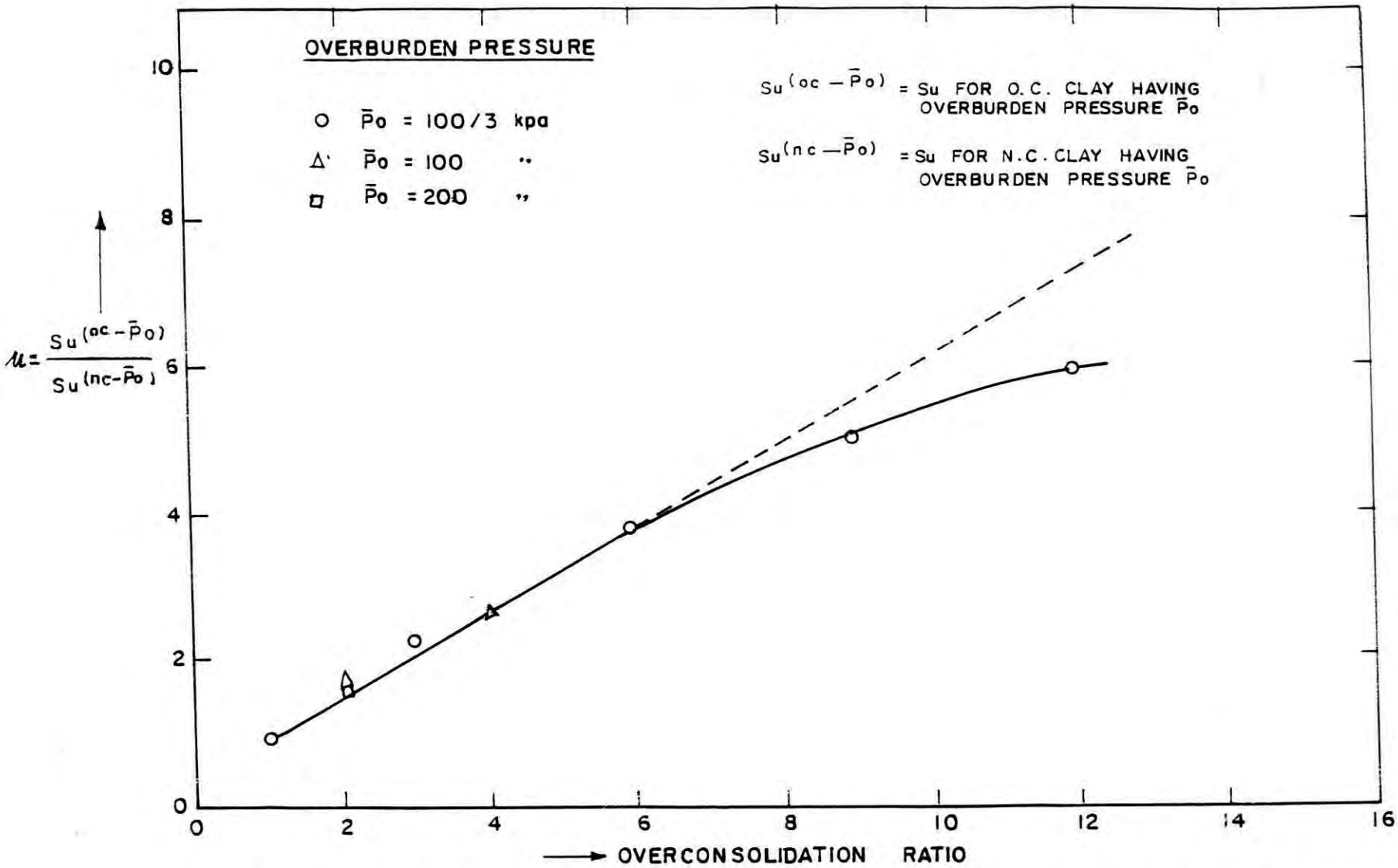


FIG. 6.21 RATE OF INCREMENT OF UNDRAINED SHEAR STRENGTH OF OVERCONSOLIDATED DHAKA CLAY WITH RESPECT TO THAT OF NORMALLY LOADED STATE OF THE CLAY HAVING SAME OVER BURDEN PRESSURE (SUBJECTED TO ISOTROPIC STRESS CONDITION)

$$\frac{S_u^{(nc)}}{\bar{\sigma}_v} = \frac{3 \sin \bar{\phi}}{3-\sin\bar{\phi}} \times \frac{1}{\exp(\Lambda_o)} \qquad (6.5)$$

According to this model, the value of Λ_o can be obtained from the equation, $\Lambda_o = 1 - [C_s^{(iso)}/C_c^{(iso)}]$, in which, as stated earlier, $C_s^{(iso)}$ is the swelling index and $C_c^{(iso)}$ is the compression index in isotropic consolidation. With the values of the indices as depicted in Article 6.6, the value of Λ_o becomes equal to 0.863. The value of $\bar{\phi}$, the effective stress friction angle has been determined from drain tests on K_oINC Samples of remoulded Dhaka clay and shown in Fig. 6.22. The data of these drain tests are shown in Table 6.6. From Fig. 6.22, the value of $\bar{\phi}$ has been determined as 23°. With the value of $\bar{\phi}$ and Λ_o, the equation (6.5) of critical state cam-clay theory, gives the value of undrained shear strength ratio equal to 0.189 which is very close to the experimentally determined value, 0.186.

As stated earlier, according to Skempton's (1957) equation, $S_u/\bar{\sigma}_v = 0.11+0037(PI)$, the value of $S_u/\bar{\sigma}_v$ becomes equal to 0.195, with the plasticity index of the clay as 23. This value is close to the experimentally obtained value, 0.186.

Thus it shows that the equation (6.5) and Skempton's equation, both can be applicable for normally consolidated Dhaka clay. But the result shows that the equation (6.5) derived from Cam-clay theory presented by Schofield and Worth (1968) gives more representative value of undrained shear strength ratio of Dhaka clay which have been consolidated normally.

To predict the undrained shear strength of normally consolidated Dhaka clay, some relationships can be used. As stated earlier, the semilog plot of water content versus consolidation pressure is a straight line. This is true for both K_oNC and K_oINC samples. The equation for the straight lines are,

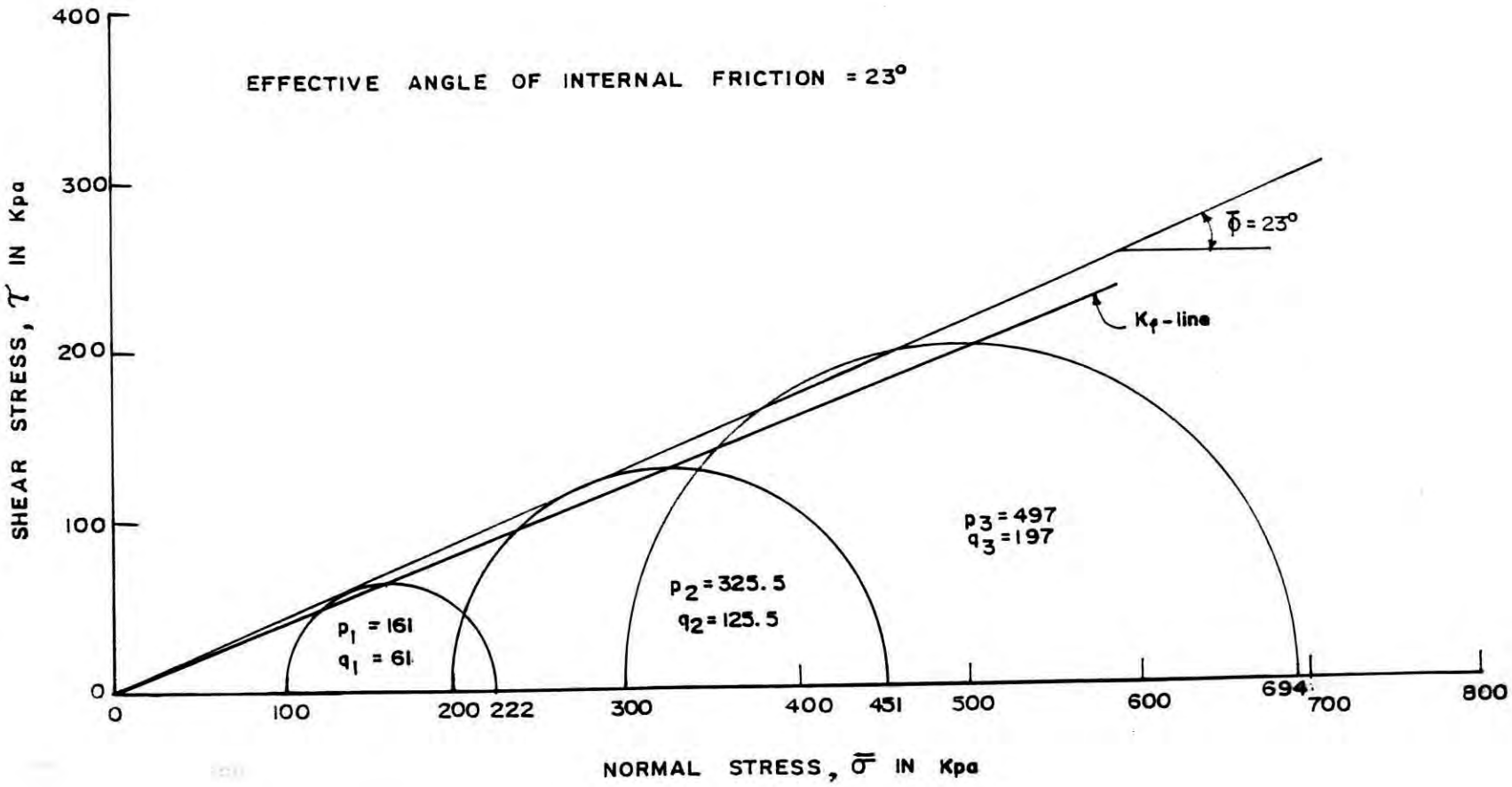


FIG. 6.22 DRAINED TESTS ON Ko INC SAMPLES OF DHAKA CLAY.

TABLE 6.6: DRAINED TESTS ON K_oINC SAMPLES

Initial K_o consolidation $\bar{\sigma}_{(IKo)}$ kPa (1)	Maximum consolidation pressure in triaxial cell, $\bar{p}$ kPa (2)	Confining pressure during triaxial test, $\bar{\sigma}_3$ kPa (3)	Drained shear strength S_d kPa (4)
80	100	100	61
150	200	200	125.5
250	300	300	197

$$S_u(nc) = 10^{(3.99-0.08688w)} \quad (6.1)$$

for K_o stress condition

and $$S_u(nc) = 10^{(4.2815-0.0961w)} \quad (6.2)$$

for isotropic stress condition

where, w = water content in percent.

Thus, by above equations, the undrained shear strength of normally consolidated Dhaka clay can be predicted from it's water content only. At normally loaded state, a particular water content will produce a particular shear strength.

The experimentally determined values of undrained shear strength ratio are 0.186 and 0.295 for K_oNC and K_oINC samples respectively. These values can be used to predict undrained shear strength of Dhaka clay at normally loaded state

$$S_u(nc) = 0.186\ \bar{\sigma}_v \quad (6.6)$$

for K_o stress condition

and $$S_u(nc = 0.295\ \bar{p} \quad (6.7)$$

for isotropic stress condition

where, $\bar{\sigma}_v$ and $\bar{p}$ are overburden pressures.

6.8.2 Soil in Overconsolidated State

Undrained shear strength of Dhaka clay can be predicted by using curves shown in Fig. 6.23, where α vs. OCR has been plotted. Here α is the ratio of undrained shear strength ratio for overconsolidated state $[S_u^{(oc)}/\bar{\sigma}_{vo}]$ and normally consolidated state $[S_u^{(nc)}/\bar{\sigma}_v]$. The values of $S_u^{(nc)}/\bar{\sigma}_v$ for normally consolidated have been found experimentally which are 0.186 and 0.295 for K_oNC and K_oINC samples respectively. So for Fig. 6.23.

$$\alpha = \frac{S_u^{(oc)}}{\bar{\sigma}_{vo}} \Big/ \frac{S_u^{(nc)}}{\bar{\sigma}_v}$$

that is $S_u^{(oc)} = 0.186 \times \alpha \times \bar{\sigma}_{vo}$ (6.8)

for K_o stress condition

and $S_u^{(oc)} = 0.295 \times \alpha \times \bar{p}_o$ (6.9)

for isotropic stress condition

Where, $\bar{\sigma}_{vo}$ and $\bar{p}_o$ are in situ overburden pressures corresponding to K_o and isotropic stress conditions respectively. Simon (1960), Ladd et al., (1977) and Mahar & O'Neill (1983) used such curves to predict $S_u^{(oc)}$ for clays investigated by them.

For a particular value of OCR (where, OCR = $\bar{\sigma}_v/\sigma_{vo}$ or $\bar{P}/\bar{P}_o$), α can be found out from Fig. 6.23 and then from above relations undrained shear strength for overconsolidated soil can be predicted. The OCR of a soil under certain overburden pressure can be known if past maximum pressure of the soil is known. The past maximum pressure ($\bar{\sigma}_v$ or $\bar{p}$) can be predicted from w log $\bar{\sigma}_v$ or $\bar{p}$ curve (Fig. 6.12) if water content and overburden pressure of

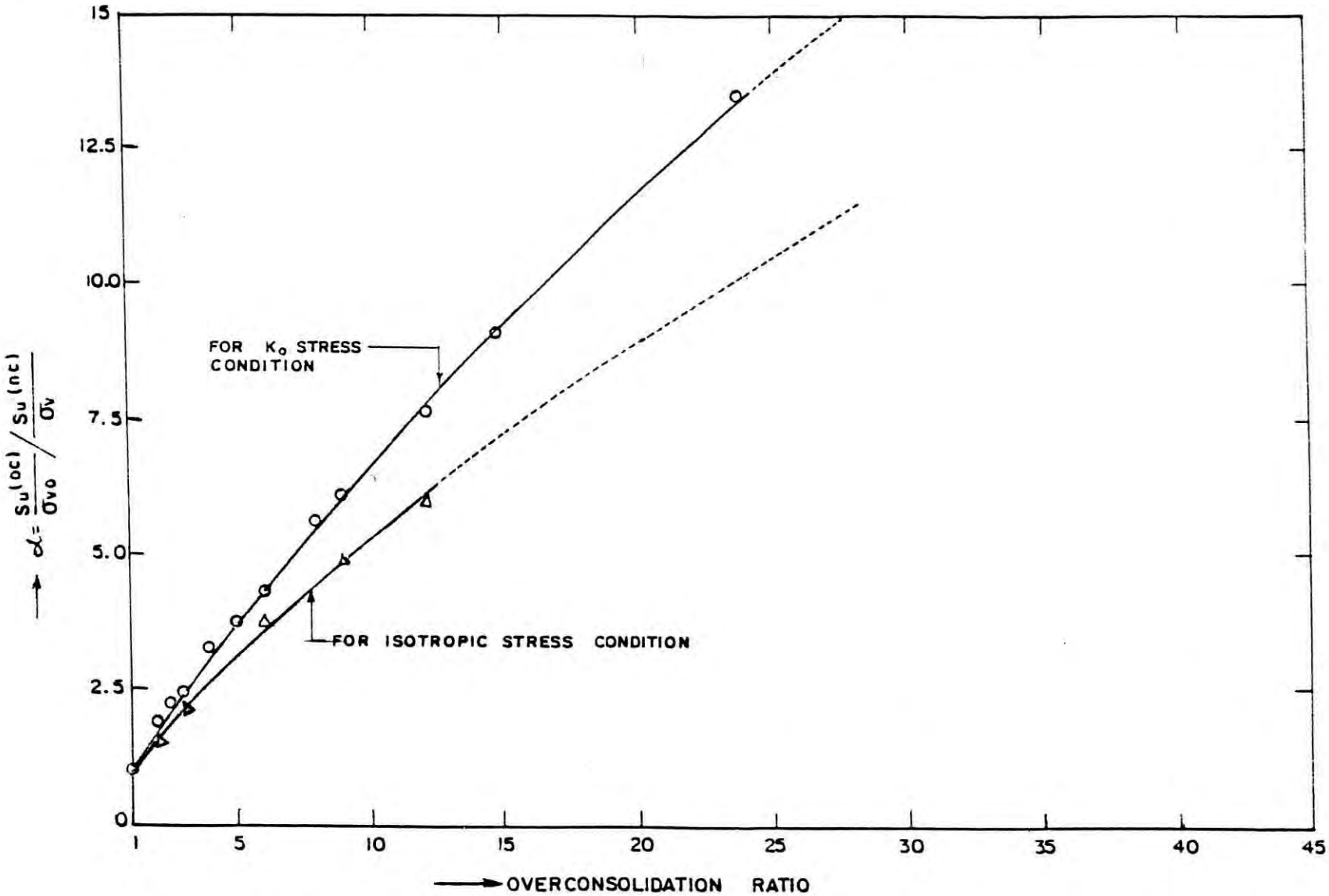


FIG. 6.23 PREDICTION OF UNDRAINED SHEAR STRENGTH OF OVERCONSOLIDATED DHAKA CLAY FROM PLOT OF $\frac{S_u(oc)}{\sigma_{vo}} \Big/ \frac{S_u(nc)}{\sigma_v}$ VERSUS OVERCONSOLIDATION RATIO.

in-situ soil are known. After locating the point of water content and the overburden pressure of in-situ soil, in Fig. 6.12, a line is drawn parallel to the nearest overconsolidated line in the figure. The point of intersection of this drawn line and the normally loaded line in the figure will have its abscissa equal to the predicted value of past maximum pressure ($\bar{\sigma}_v$ or $\bar{p}$) of the soil.

The undrained shear strength of overconsolidated Dhaka clay can also be predicted from the curves of Fig. 6.10. In the figure, $S_u(oc)/\bar{\sigma}_{vo}$ versus log OCR has been plotted for K_oNC and K_oINC samples. For a particular OCR, the undrained shear strength of Dhaka clay for K_o or isotropic stress condition can be found out from these curves. OCR of the soil can be predicted if water content and overburden pressure of the soil is known by using w log $\bar{\sigma}_v$ or $\bar{p}$ plot (Fig.6.12) as stated above.

The plots of μ vs. OCR for K_oOC and K_oIOC samples have been shown in Fig. 6.20 and 6.21. As discussed earlier, the rate of increase of undrained shear strength due to increase of OCR for same overburden pressure has been shown in these figures. In Fig. 6.20 which is for K_o stress condition, the plot is linear upto OCR value 9 and in Fig. 6.21, which is for isotropic stress condition, the plot is linear upto OCR value 6. So upto these level of OCR, undrained shear strength of overconsolidated Dhaka clay can be predicted from following linear equations:

$$S_u(oc) = [0.119(OCR) + 0.065] \times \bar{\sigma}_{vo} \qquad (6.10)$$

for K_o stress condition

and $$S_u(oc) = [0.168(OCR) + 0.14] \times \bar{p}_o \qquad (6.11)$$

for isotropic stress condition

where, $\bar{\sigma}_{vo}$ and $\bar{p}_o$ are overburden pressures.

Beyond the OCR value 9 and 6 for K_o and isotropic stress condition respectively, the undrained shear strength can be obtained from the Fig. 6.20 and 6.21 directly.

As discussed in literature review, Atkinson and Bransby (1968) has derived an expression of the undrained shear strength of an overconsolidated soil in terms of its normally consolidated strength raised to a power function. Later on, this equation was used by Mitachi and Kitago (1976) as model for undrained shear strength of o.c soils. The equation is as follows:

$$\frac{S_u(oc)}{\bar{\sigma}_{vo}} = \frac{S_u(nc)}{\bar{\sigma}_v} \times (OCR)^{\Lambda_o} \qquad (2.10)$$

It has been shown in Article 6.5 and 6.6 that the experimental data of undrained shear strength ratio ($S_u/\bar{\sigma}_{vo}$) and OCR when plotted in log-log scale produced straight line. This has been shown in Fig. 6.11. In this figure, the equation of the lines are

$$\frac{S_u(oc)}{\bar{\sigma}_{vo}} = \frac{S_u(nc)}{\bar{\sigma}_v} \times (OCR)^{\Lambda_o(K_o)} \qquad (6.12)$$

for K_o stress condition

and $$\frac{S_u(oc)}{\bar{\sigma}_{vo}} = \frac{S_u(nc)}{\bar{\sigma}_v} (OCR)^{\Lambda_o(iso)} \qquad (6.13)$$

for isotropic stress condition

$\Lambda_o(K_o)$ and $\Lambda_o(iso)$ are the slopes of the lines for K_oNC and K_oINC

samples respectively shown in Fig. 6.11. The slope of the lines have been determined from the plot of Fig. 6.11 which are 0.821 and 0.734 for K_o and isotropic stress system respectively. The intercepts of the lines at OCR=1 are undrained shear strength ratio at normally consolidated state which are 0.186 and 0.295 for K_oNC and K_oINC samples. These straight lines can be used as model to determine undrained shear strength of Dhaka clay. By replacing the values of $\Lambda_o(K_o)$ & $S_u(nc)/\bar{\sigma}_v$ and $\Lambda_o(iso)$ & $S_u(nc)/\bar{p}$, the equations (6.12) and (6.13) can be written as

$$S_u(oc) = 0.186(\bar{\sigma}_v)^{0.821} x (\bar{\sigma}_{vo})^{0.179} \qquad (6.14)$$

for K_o stress condition

and $$S_u(oc) = 0.295\ (\bar{p})^{0.734} x (\bar{p}_o)^{0.266} \qquad (6.15)$$

for isotropic stress condition

where, $\bar{\sigma}_v$ & $\bar{p}$ are past maximum pressures and $\bar{\sigma}_{vo}$ & $\bar{p}_o$ are overburden pressures.

As discussed earlier, the past maximum pressure can be predicted from w log $\bar{\sigma}_v$ or $\bar{p}$ curve (Fig. 6.12) if water content and overburden pressure are known.

It has been mentioned in the literature review that, using critical-state Cam-clay theory, Mayne (1980) has developed a simple and useful model to predict the undrained shear strength of overconsolidated clay. The model equation is as follows:

$$\frac{S_u(oc)}{\bar{\sigma}_{vo}} = \frac{3\sin\bar{\phi} \times \exp(-\Lambda_o)}{3-\sin\bar{\phi}} \times (OCR)^{\Lambda_o}$$

The model requires only two soil constants $\bar{\phi}$ and Λ_o to predict undrained shear strength of clay subjected to any degree of overconsolidation. This model equation can be applied for Dhaka clay. Replacing values of $\bar{\phi}$ equal to 23^0 and $\Lambda_o(K_o)$ & $\Lambda_o(iso)$ equal to 0.821 & 0.734 for K_o and isotropic stress condition respectively for Dhaka clay, the equation yield,

$$S_u(oc) = 0.197(OCR)^{0.821} \times \bar{\sigma}_{vo} \qquad (6.16)$$

for K_o stress condition

and $$S_u(oc) = 0.216(OCR)^{0.734} \times \bar{p}_o \qquad (6.17)$$

for isotropic stress condition

where $\bar{\sigma}_{vo}$ and $\bar{p}_o$ are overburden pressures.

Table 6.7 gives a list of model equations, discussed so far, useful for prediction of undrained shear strength of Dhaka clay.

6.9 Comparison of Strength Models for Dhaka clay for Undisturbed Sample

In Table 6.8, a comparison has been made between the values of undrained shear strength predicted by the equations (6.14) and 16.15) [derived from model of Mitachi and Kitago (1976)] and that predicted by the equation (6.16) and (6.17) [derived from model of Mayne (1980)]. In these model, the parameters Λ_o, $\bar{\phi}$ and S are taken from experimental results on reconstituted Dhaka clay as discussed previously. The predicted values of undrained shear strength have been compared with the tested values of undrained shear strength of undisturbed natural Dhaka clays. The data of the natural Dhaka clay have been collected after Islam (1980) and Eastern Engineers, Motijheel, Dhaka (1989). From water content and in-situ overburden pressure, past maximum consolidation

TABLE 6.7 EQUATIONS FOR PREDICTION OF UNDRAINED SHEAR STRENGTH OF DHAKA CLAY

Stress history	Equation derived from	Equation for K_o consolidated clay	Equation for isotropically consolidated clay	Item required to be known for strength prediction
For normally consolidated clay	W vs. log S_u plot (Fig. 6.3/pp.106)	$S_u^{(nc)}=10^{(3.99-0.0888w)}$	$S_u^{(nc)}=10^{(4.2815-0.0961w)}$	Water content (w)
	S_u vs. $\bar{\sigma}_v$ or $\bar{p}$ plot (Fig. 6.17/pp.129)	$S_u^{(nc)}=0.186\ \bar{\sigma}_v$	$S_u^{(nc)}=0.295\ p$	Overburden pressure
	Skempton's (1957) equation (pp.139)	$S_u^{(nc)}=0.195\ \bar{\sigma}_v$	-	"
	Critical-state Cam-clay theory (eq.6.5/pp.139)	$S_u^{(nc)}=0.189\ \bar{\sigma}_v$	-	"
For overconsolidated clay	α vs. OCR plot (Fig.6.23/pp.144)	$S_u^{(oc)}=0.186x\alpha x\bar{\sigma}_{vo}$	$S_u^{(oc)}=0.295x\alpha x\bar{p}_o$	Water content and overburden pressure
	μ vs. OCR plot (Fig.6.20 & 6.21/ pp. 137 & 138)	$S_u^{(oc)}=[0.119(OCR)+0.065]\bar{\sigma}_{vo}$	$S_u^{(oc)}=[0.168(OCR)+0.14]\bar{p}_o$	"
For all stress history	Mitachi and Kitago's (1976) model(pp.147)	$S_u^{(oc)}=0.186(\bar{\sigma}_v)^{0.821}x(\bar{\sigma}_{vo})^{0.179}$	$S_u^{(oc)}=0.295(\bar{p})^{0.734}x(\bar{p}_o)^{0.266}$	"
	Mayne's (1980) model (pp.148)	$S_u^{(oc)}=0.197(OCR)^{0.821}x(\bar{\sigma}_{vo})$	$S_u^{(oc)}=0.216(OCR)^{0.734}x(\bar{p}_o)$	"

TABLE 6.8: COMPARISON OF MITACHI AND KITAGO'S MODEL AND MAYNE'S MODEL IN PREDICTING UNDRAINED SHEAR STRENGTH OF NATURAL DHAKA CLAY

Location	Sl. No.	Plastic limit %	Liquid limit %	Depth of the sample	Water content %	In-situ vertical stress, $\bar{\sigma}_{vo}$ or $\bar{p}_o$ kPa	*Past maximum consolidation pressure from w log $\bar{\sigma}_v$ curve, $\bar{\sigma}_v$ kPa	*Past maximum consolidation pressure from w log $\bar{p}$ curve, $\bar{p}$ kPa	OCR from K_o curve	OCR from isotropic curve	Undrained shear strength from Mayne's model		Undrained shear strength from Mitachi & Kitago model		Undrained shear strength from undisturbed sampling kPa
											K_o kPa	Isotropic kPa	K_o kPa	Isotropic kPa	
Eskaton	1	19	43	6'-6" to 8'-6"	22.00	43.02	805	690	18.71	6.03	93.87	71.23	88.63	97.29	109.36
	2	23	45	13'-9" to 14'-6"	21.00	81.02	940	790	11.60	9.75	119.40	93.10	112.74	127.16	126.11
	3	24	46	31'-6" to 35'-6"	20.00	184.916	1105	840	5.97	4.54	157.92	121.31	149.11	165.69	171.05
Fulbaria	1	19	41	8'-5" to 9'-8"	22.27	51.89	720	540	13.87	10.40	88.57	62.55	83.63	85.43	78.47
	2	19	43	10'-2" to 11'-5"	23.63	61.96	570	420	9.19	6.77	75.47	54.52	71.26	74.47	74.61
	3	22	46	12'-0" to 13'-0"	25.29	71.77	410	310	5.71	4.31	59.12	45.36	55.82	61.96	82.68
	4	22	43	15"-4" to 16'-8"	24.03	91.86	505	410	5.49	4.46	73.32	59.48	69.23	81.24	82.75
	5	19	42	17'-2" to 18'-5"	23.34	102.15	560	420	5.18	4.11	81.35	62.28	76.81	85.06	71.65

* Pressure estimated from water content and overburden pressure of the in-situ soil.

N.B: The undrained shear strength data of natural Dhaka clay for Eskaton has been collected after Eastern Engineers, 71, Motijheel, Dhaka, (1989) and for Fulbaria after Islam (1980).

pressure has been determined from w vs log $\bar{\sigma}_v$ or $\bar{p}$ curves of Fig. 6.12. From Table 6.8 it is seen that Mayne's (1980) K_o model predicts values of undrained shear strength closer to tested samples than that of Mitachi and Kitago's (1976) K_o Model. On the other hand, Mitachi and Kitago's (1976) isotropic model gives more representative values of undrained shear strength than that of Mayne's (1980) model. Fig. 6.24 plots data of Table 6.8.

In light with the above observations, the following equations can be suggested for prediction of undrained shear strength for Dhaka clay.

$$S_{u(oc)} = 0.197\ (OCR)^{0.821} x \bar{\sigma}_{vo} \qquad (6.16)$$

if Dhaka clay is considered to have been consolidated in K_o stress condition

and $$S_{u(oc)} = 0.295(p)^{0.734} x (\bar{p}_o)^{0.266} \qquad (6.15)$$

if Dhaka clay is considered to have been consolidated in isotropic stress condition.

In Fig. 6.25, comparison has been made between the tested values of undrained shear strength of undisturbed samples and that predicted by the above mentioned models.

From Table 6.8 and Fig. 6.24 & 6.25, differences have been observed between the tested values of undrained shear strength of natural Dhaka clay collected by undisturbed sampling and that predicted by the models obtained from parameters of reconstituted samples. The scatter in the value of laboratory test results of undisturbed samples collected from field may be attributed to sampling disturbance, in-situ variation of soil properties, improper estimation of overburden pressure and OCR, variation in soil fabric of individual samples. However, it may be observed

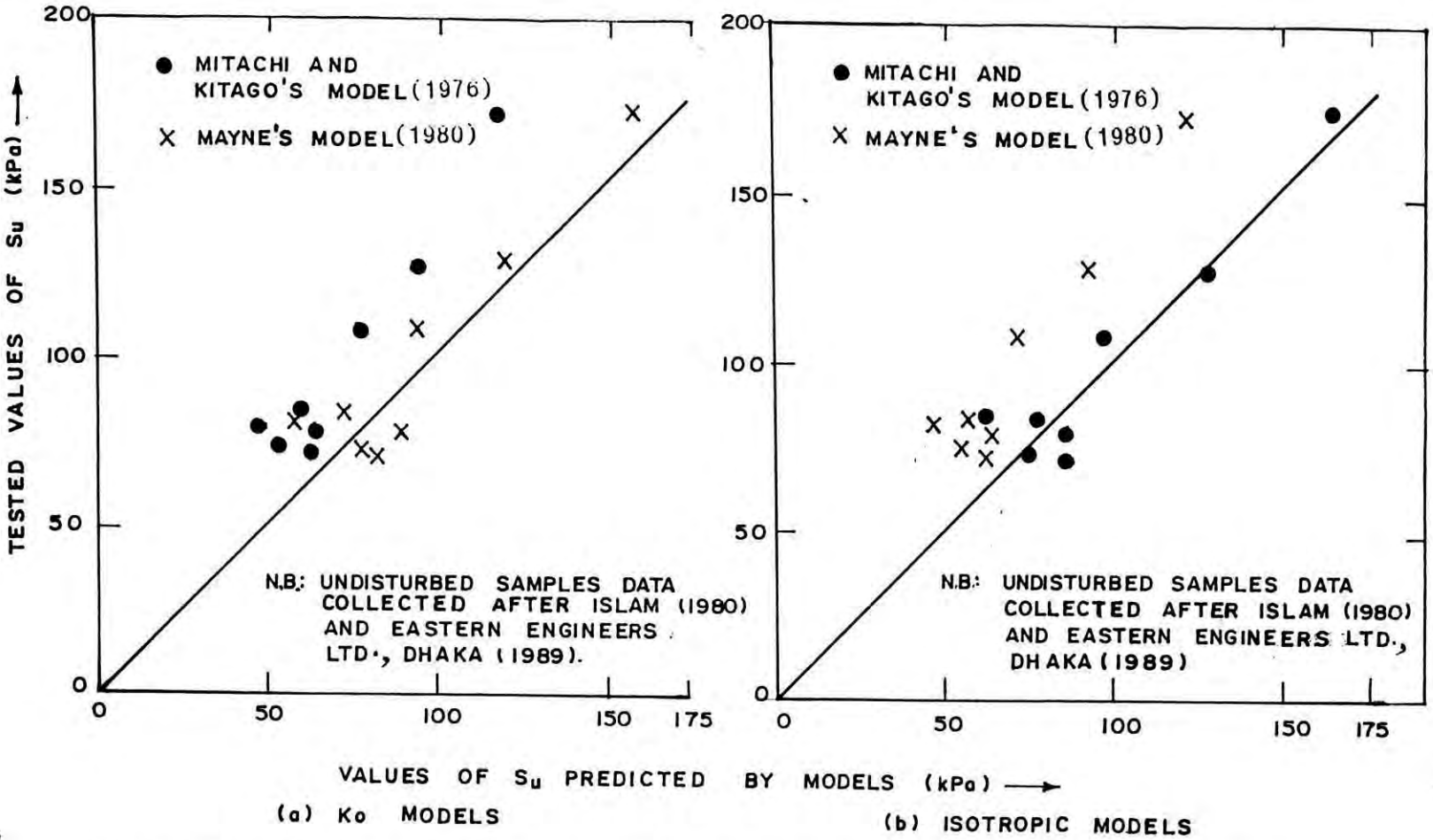


FIG. 6.24 COMPARISON OF TESTED VALUES OF UNDRAINED SHEAR STRENGTH OF NATURAL DHAKA CLAY WITH THAT PREDICTED BY MODELS.

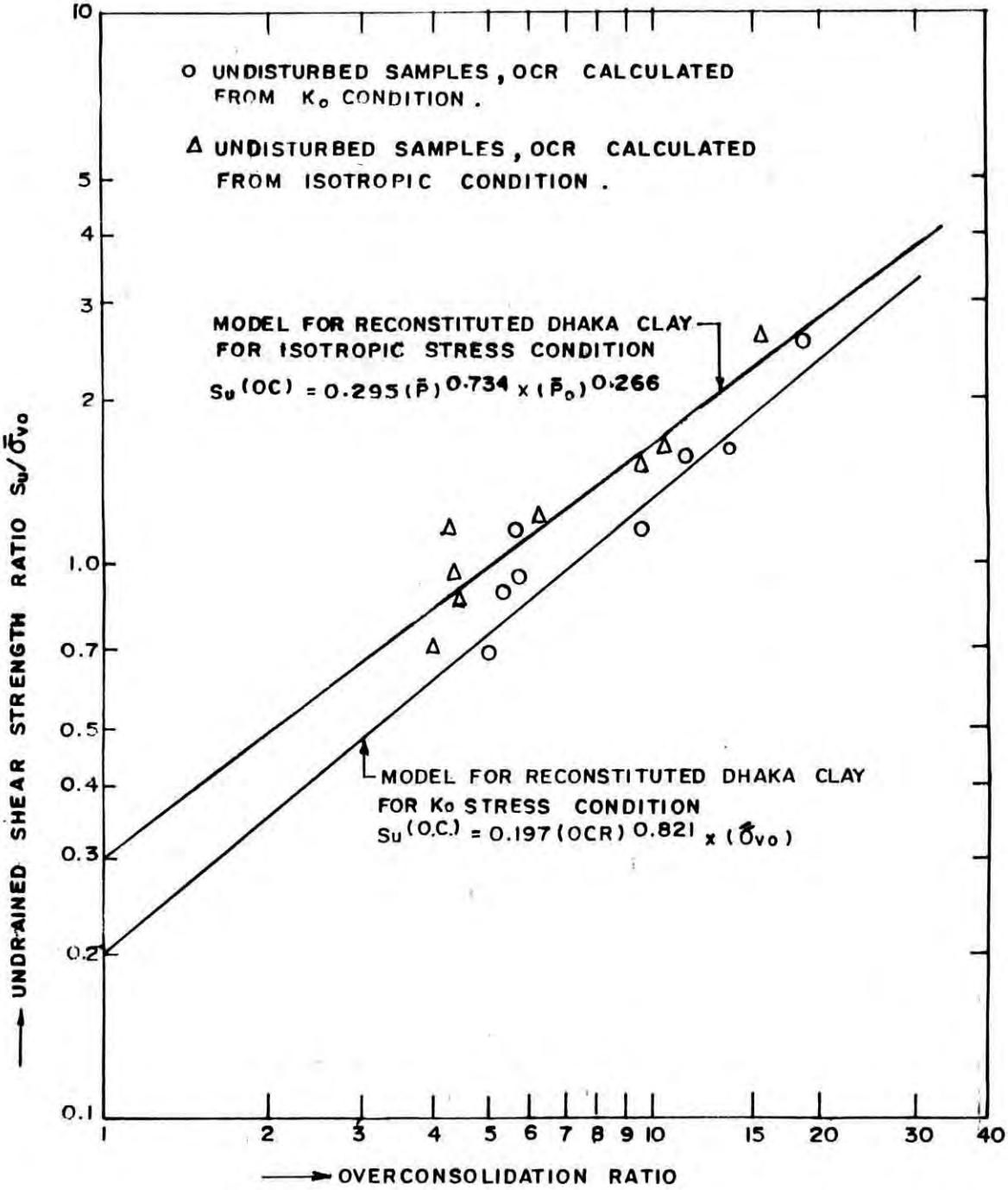


FIG. 6.25 COMPARISON OF MODEL PREDICTED VALUES AND TESTED VALUES OF UNDRAINED SHEAR STRENGTH OF DHAKA CLAY

that variation is reasonable.

6.10 Variation of Initial Shear Stress Ratio with OCR for Dhaka Clay

The in-situ stress state has a major effect on the magnitude of subsequent stress change of a soil element that can sustain before local failure is initiated. As discussed in literature review, this effect can be represented conventiently by initial shear stress ratio, f of a soil (Davis and Poulos, 1968). Fig. 6.26 shows such variation for Dhaka clay with other six clays. The figure shows that the variation curve falls within the zone of Shaded area for other clays and at the side of low plasticity index. The variation of initial shear stress ratio is highly pronounced at lower values of OCR.

6.11 Void Index and Intrinsic Compression Line & Intrinsic Swelling Line for Dhaka Clay

Fig. 6.27 shows the one dimensional compression curve for reconstituted Dhaka clay. The compression curve plotted in Fig. 6.27 represents the intrinsic compression curve for Dhaka clay since the clay was reconstituted at water content of 1.5 times the liquid limit. It is useful to normalize this laboratory compression curve with respect to void ratio. Referring to Burland's concept (1990), the intrinsic compression curve shown qualitatively in Fig. 2.27(a) has been normalized with respect to void ratio as shown in Fig. 2.27(b). Similarly, the intrinsic compression curve for Dhaka clay shown in Fig. 6.27 has been normalized by plotting void index, I_v versus log $\bar{\sigma}_v$ and shown in Fig. 6.28. The intrinsic void ratio, e^*_{100} and e^*_{1000} for Dhaka clay have been found as 0.775 and 0.525 respectively. So the constants of intrinsic compressibility e^*_{100} and C_c^* for Dhaka

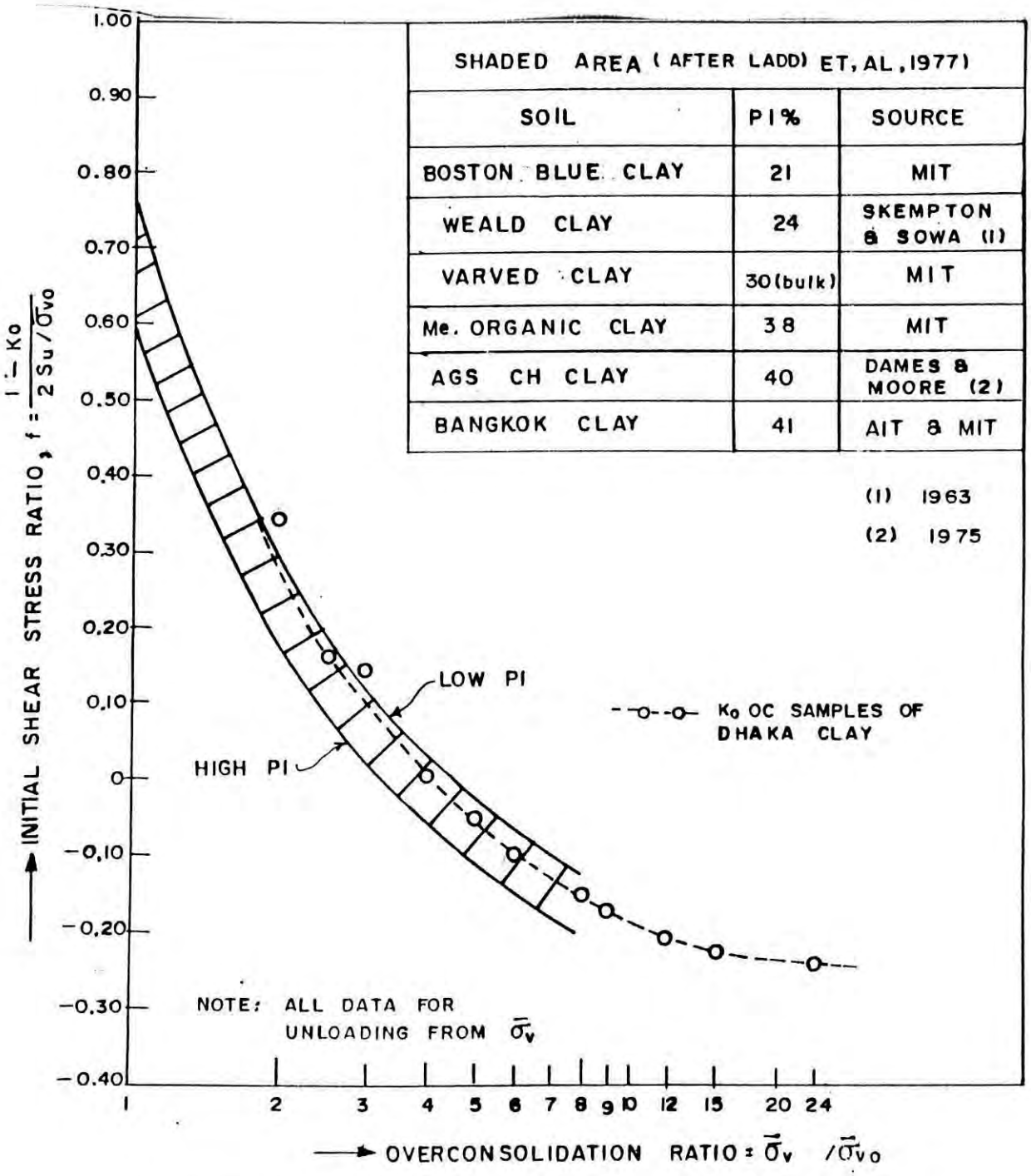


FIG. 6,26 VARIATION OF INITIAL SHEAR STRESS RATIO WITH OVERCONSOLIDATION RATIO FOR DHAKA CLAY COMPARED WITH OTHER CLAYS.

ISL FOR DHAKA CLAY	PRECONSOL PRESSURE	C_s^*	C_s^*/C_c^*
1	100 kPa	0.035	0.14
2	200 "	0.025	0.10
3	300 "	0.035	0.14
4	400 "	0.0324	0.129
5	500 "	0.024	0.096
6	800 "	0.025	0.10

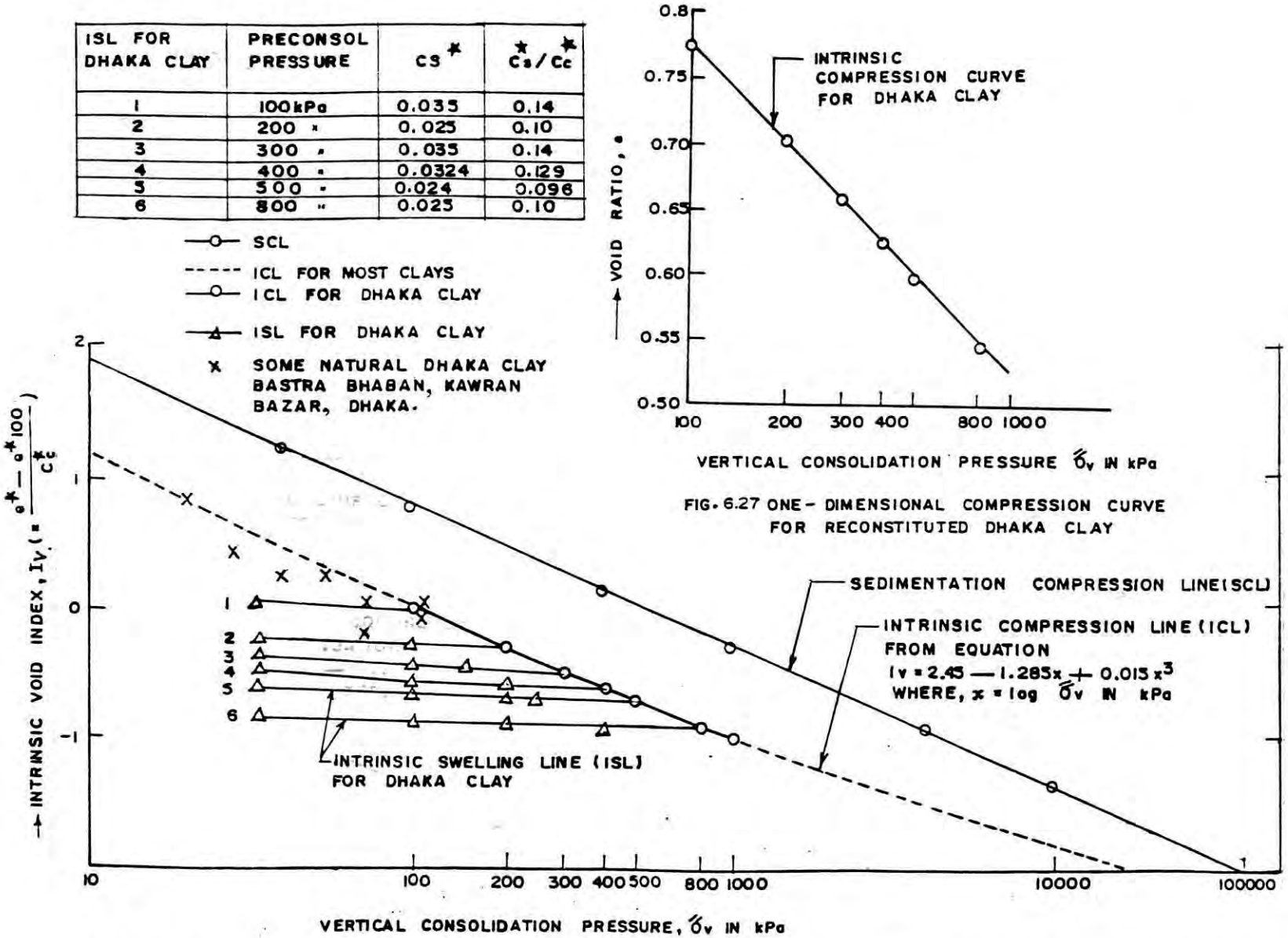


FIG. 6.27 ONE-DIMENSIONAL COMPRESSION CURVE FOR RECONSTITUTED DHAKA CLAY

FIG. 6.28 ICL AND ISL FOR DHAKA CLAY SHOWN WITH ICL AND SCL FOR MOST CLAYS.

clay are 0.775 and 0.25 respectively. Thus the void index for Dhaka clay is

$$I_v = \frac{e-e^*_{100}}{C_c^*} = \frac{e-0.775}{0.25} \qquad (6.18)$$

By using equation (6.18), intrinsic compression line for Dhaka clay was plotted in Fig. 6.28. The curve thus constructed coincides with the intrinsic compression line (ICL) drawn by the following equation (2.24) furnished by Burland (1990) which represents ICL for most clays.

$$I_v = 2.45-1.285x+0.015x^3 \qquad (2.24)$$

where $x = \log \bar{\sigma}_v$ in KPa

Intrinsic swelling line (ISL) from data of reconstituted Dhaka clay has been drawn in Fig. 6.28. The values of C_s^* and C_s^*/C_c^* have been shown in the same figure. The values of C_s^*/C_c^* vary from 0.14 to 0.10. The sedimentation compression line (SCL) which is for most clays have also been shown in the Fig. 6.28.

Some data of natural Dhaka clay (Bastra Bhaban Kawran Bazar, Dhaka) collected from Development Design Consultant Ltd., Eskaton, Dhaka, have been plotted in Fig. 6.28 to show their relative locations with respect to ICL. void ratio corresponding to liquid limit (e_L) is a more fundamental parameter than liquid limit for a clay. The void ratio corresponding to liquid limit (e_L) for Dhaka clay has been found as 1.17.

CHAPTER 7
CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE STUDY

7.1 Conclusions

Undrained shear strength and other geotechnical chaqracteristics of a reconstituted Dhaka clay at different stress history and stress condition have been investigated in the present study. Attempts have been made to evaluate Parameters that are essential for development of an applicable model to predict the undrained shear strength for Dhaka clay. On the basis of experimental results, the following findings and conclusions may now be drawn.

1. Stress-strain curves for normally consolidated reconstituted Dhaka clay tested to undrained compression showed no defined peak. The deviator stress increases with increase in consolidation pressure. The undrained secant modulus, $E_u(50)$ increases linearly with consolidation pressure at normally consolidated state and decreases approximately linearly with reduction of consolidation pressure.

2. The relation, $E_u(50) = \Psi S_u$ can be applied to Dhaka clay to represent its undrained secant modulus, $E_u(50)$. The values of Ψ for Dhaka clay at normally loaded state varies between 190 to 210 under K_o condition and 165 to 179 under isotropic stress condition. For the overconsolidated Dhaka clay, Ψ value varies between 113 to 198 (for OCR 2 to 24) under K_o condition and 111 to 170 (for OCR from 2 to 12) under isotropic stress condtiion. A comparative study showed that Dhaka clay posses a lower value of Ψ than that of some other clays [i.e Bankok clay, Maine organic clay and Boston Blue Clay (Appolonia et al., 1971)].

3. For reconstituted Dhaka clay, under K_o stress condition, the compression index, $C_c^{(K_o)}$ and swelling index, $C_s^{(K_o)}$ determined

from e vs. log $\bar{\sigma}_v$ curve are 0.25 and 0.0.025 ± 0.01 respectively. The isotropic compression index, $C_c^{(iso)}$ and swelling index, $C_s^{(iso)}$ are 0.278 and 0.038 ± 0.01 respectively. These values yield Λ_o for K_o stress condition, $\Lambda_o^{(K_o)}$ and for isotropic stress condition, $\Lambda_o^{(iso)}$ having values of 0.90 and 0.863 respectively.

4. On the basis of Hvorslev's (1937) shear strength theory, cohesion components at constant water content are determined and the relationship between cohesion components and water content at undrained failure condition, is found out. The ϕ_e and β' for K_o stress condition are 2.84° and -10.14° respectively and that for isotropic stress condition are 6.22° and -10.34° respectively.

5. Under undrained shear strength condition, normally loaded Dhaka clay clearly showed $S_u/\bar{\sigma}_v$ as a normalizing parameter. Under this condition, the $S_u/\bar{\sigma}_v$ value is constant and equal to 0.186 under K_o stress condition and 0.295 under isotropic stress condition. The former is closer to the value obtained from Skempton's relation of $S_u/\bar{\sigma}_v$ = 0.11 + 0.0037(PI)=0.195. The above experimentally obtained value of the normalized parameter, $S_u/\bar{\sigma}_v$ is close to that obtained from critical state cam-clay theory proposed by Schofield and Wroth (1978) in the form (eq. 6.5):

$$\frac{S_u}{\sigma_v} = \frac{3\,\mathrm{Sin}\bar{\phi}}{3-\mathrm{Sin}\bar{\phi}} \cdot \frac{1}{\exp(\Lambda_o)}$$

From the available results of $\bar{\phi}$ = 23° and Λ_o = 0.863 for Dhaka clay, the above equation yields $S_u/\bar{\sigma}_v$=0.189 which is closer to the normalized parameter obtained under K_o condition (i.e 0.186). Hence the above cam-clay model is recommended for predicting undrained shear strength of Dhaka clay. Alternately, a $S_u/\bar{\sigma}_v$ = 0.186 value should be used.

6. The $S_u^{(oc)}/\bar{\sigma}_{vo}$ and OCR values for reconstituted Dhaka clay plotted in log-log scale shows linear relation of the form:

$$S_u^{(oc)}/\bar{\sigma}_{vo} = S \times (OCR)^{\Lambda_o}$$

This equation can be used to predict undrained shear strength of overconsolidated Dhaka clay. The experimental data showed that the above equation is valid for Dhaka clay for both K_o and isotropic stress conditions. The values of $\Lambda_o^{(K_o)}$ and $\Lambda_o^{(iso)}$ determined from above relationship are 0.821 and 0.734 respectively. With above experimentally determined values of the parameters, the following model equations for predicting undrained shear strength for Dhaka clay have been found out:

$$S_u^{(oc)} = 0.186\ (\bar{\sigma}_v)^{0.821} \times (\bar{\sigma}_{vo})^{0.179}$$

for K_o stress condition

and $S_u/^{(oc)} = 0.295(\bar{P})^{(0.734} \times (\bar{P}_o)^{0.266}$

for isotropic stress condition.

7. There is a unique relationship between the undrained shear strength (S_u) and water content (w) of a normally consolidated Dhaka clay. On the basis of experimental results, the undrained shear strength of normally consolidated Dhaka clay can be related to water content by the equations:

$$S_u^{(nc)} = 10^{(3.99-0.08688w)}$$

for K_o stress condition

and $S_u^{(nc)} = 10^{(4.2815-0.0961w)}$

for isotropic stress condition

The above equations can be used to predict undrained shear strength of normally consolidated Dhaka clay from water content only.

8. There is unique relationship between the undrained shear strength (S_u) and consolidation pressure ($\bar{\sigma}_v$ or $\bar{p}$) of a normally consolidated Dhaka clay. On the basis of experimental results, the undrained shear strength of normally consolidated Dhaka clay can be related to consolidation pressure by the equations,,

$$S_u^{(nc)} = 0.186\ \bar{\sigma}_v$$

for K_o stress condition

and $S_u^{(nc)} = 0.295\ \bar{p}$

for isotropic stress condition

The above equations can be used to predict undrained shear strength of normally consolidated Dhaka clay from overburden pressure only.

9. α vs. OCR curves were plotted to represent undrained shear strength of overconsolidated Dhaka clay. $S_u^{(oc)}$ can be predicted from relations:

$$S_u^{(oc)} = 0.186 \times \alpha \times \bar{\sigma}_{vo}$$

for K_o stress condition

$$S_u^{(oc)} = 0.295 \times \alpha \times \bar{p}_o$$

for isotropic stress condition

α to be found out from α vs. OCR curves,

10. $S_u^{(oc)}/\bar{\sigma}_{vo}$ vs. OCR curves were plotted in semilog scale. The undrained shear strength of overconsolidated Dhaka clay can also be predicted using those curves for both K_o and isotropic stress conditions.

11. For same overburden pressure, the rate of increase of undrained shear strength with increase of OCR of overconsolidated Dhaka clay can be found from μ vs OCR plot. Following relationships derived from μ vs. OCR plot can be used to predict undrained shear strength of overconsolidated Dhakad clay,

$$S_u^{(oc)}=[0.119(OCR)+0.065]\times\bar{\sigma}_{vo}$$

for K_o stress condition and upto OCR = 9

$$S_u^{(oc)}=[0.168(OCR) + 0.14]\times\bar{P}_o$$

for isotropic stress condition and upto OCR=6.

12. Mayne's (1980) model was investigated to determine its applicability for Dhaka clay. $\bar{\phi}$ was determined from drain tests which is 23°. Replacing values of $\bar{\phi}$ and Λ_o, the following equations were found out to predict undrained shear strength for overconsolidated Dhaka clay,

$$S_u^{(oc)}=0.197(OCR)^{0.821}\times\bar{\sigma}_{vo}$$

for K_o consolidated clay

$$S_u^{(oc)}=0.216(OCR)^{0.734}\times\bar{p}_o$$

for isotropically consolidated clay.

13. A comparison of available strength models for clays shows that Mayne's (1980) K_o model and Mitachi and Kitago's (1976)

isotropic model is most suitable for predicting undrained shear strength of natural Dhaka clay. Therefore, the following equations are recommended to use for predicting the undrained shear strength of overconsolidated Dhaka clay

$$S_u^{(oc)} = 0.197(OCR)^{0.821} \times \bar{\sigma}_{vo}$$

if Dhaka clay is considered to be consolidated under K_o stress system

$$S_u^{(oc)} = 0.295(\bar{p})^{0.734} \times (\bar{p}_o)^{0.266}$$

if Dhaka clay is considered to be consolidated under isotropic stress system.

14. The curve representing variation of initial shear stress f [i.e $f = (1-K_o)/(2S_u/\bar{\sigma}_v)$] with OCR for Dhaka clay falls within the band furnished by Ladd et al. (1971) for a number of clays (see Fig. 6.26).

15. The void index, I_v for reconsolidated Dhaka clay is (e-0.775)/0.25 and the void ratio at liquid limit (e_L) for the clay has been found as 1.17. The intrinsic compression line (ICL) for Dhaka clay found from the intrinsic values coincided with the ICL values for most clays furnished by Burland (1990). The intrinsic swelling line (ISL) for Dhaka clay has also been drawn. ICL and ISL are shown in Fig. 6.28 with sedimentation compression line (SCL).

7.2 Recommendations for Future Study

In this study, the intrinsic properties of Dhaka clay have been determined. The compressibility and shear strength characteristics of the reconstituted clay can be used as basic frame of reference for interpreting the corresponding

characteristics in natural state. Hence, the intrinsic properties of Dhaka clay obtained in this research can provide a frame of reference for assessing the in-situ state of the natural Dhaka clay and the influence of the soil structure on its in-situ properties. As a natural corollary, further study is essential to compare geotechnical properties of natural Dhaka clay with the intrinsic properties obtained from laboratory consolidated reconstituted clay. Therefore, it is proposed that available test results on quality undisturbed samples of Dhaka clay may be collected and compared with intrinsic properties obtained in this study. Comparison methods such as proposed by Burland (1990) in terms of Intrinsic Compression Line (ICL), Sedimentation Compression Line (SCL), Intrinsic Swelling Line (ISL), intrinsic swelling index (C_s*, intrinsic Horslev strength parameter (ϕ_e*) etc. and under critical state framework may be used. These will reveal significant informations as to the nature of the formation of this clay, interparticle bonding state and probable variation in strength and compressibility of this clay in natural state. Since there is likely to be severe shortage of good quality undisturbed sample results for natural clay soils, some field collection fof samples and testing may be necessary.

It may be mentioned here that most of the test results reported here are for undrained condition in which pore pressure was not measured. This study may be repeated with measurement of pore water pressure and local strain (after Burland, 1990) measurement to study the nature of pore water pressure development and dissipation and of post-rupture strength conditions in intrinsic state.

REFERENCES

Abbreviations:

ASCE	: American Society of Civil Engineers
ASTM	: American Society for Testing and Materials
BUET	: Bangladesh University of Engineering & Technology, Dhaka.
ICE	: Institution of Civil Engineers, London.
ICSMFE	: International Conference on Soil Mechanics and Foundation Engineering
JSSMFE	: Japanese Society of Soil Mechanics and Foundation Engineering
JGED	: Journal of Geotechnical Engineering Division
JSMFD	: Journal of the soil Mechanics and Foundation Division
MIT	: Massachusetts Institute of Technology
SEAGC	: Southeast Asian Geotechnical Conference
SEARCSE	: Southeast Asian Regional Conference on Soil Engineering

Ameen, S.F. (1985), "Geotechnical Characteristics of Dhaka Clay", M.Sc.Engg. thesis, BUET, Dhaka.

Anderson, T.C., and Lukas, R.G., (1980), "Preconsolidation Pressure Predicted Using S_u/p Ratio", Laboratory Shear Strength of Soil, ASTM, USA, pp. 502-515.

Azzouz, A.S., Malek, A.M., and Baligh M.M., (1989), "Cyclic Behaviour of Clays in Underained Simple Shear", JGED, ASCE, Vol. 115, No.5, pp. 637-657.

Balasubramanium, A.S., and Choudhury, A.R., (1978), "Deformation and Strength Characteristics of Soft Bankok Clay", JGED, ASCE, Vol.104, GT.9, pp. 1153-1167.

Balasubramanium, A.S. and Brenner, R.P., (1981), "Consolidation and Settlement of Soft Clay", Soft Clay Engineering, 20, Elsevier Scientific Publishing Company, New York, pp. 481-557.

Bishop and Henkel (1964), "The Measurement of Soil Properties in the Triaxial Test", Willium Clowes and Sons Ltd., London.

Bjerrum, L. (1967), "Engineering Geology of Norwegian Normally Consolidated Marine Clay as Related to Settlement of Buildings", 7th Rankine Lecture, Geotechnique, vol.17, pp. 81-118.

Bobby, O.H, (1989), "1-D Stvain in Normally Consolidated Cohesive Soil", JGED, ASCE, Vol.115, No. 5, pp. 689-710.

Bowles, J.E., (1985), "Physical and Geotechnical Properties of Soils", McGraw-Hill Book Company, Singapore.

Bowles, J.E. (1988), "Foundation Analysis and Design", McGraw-Hill Book Company, Singapore.

Burland, J.B. (1990), "On the Compressibility and Shear Strength of Natural Clays", Geotechnique 40, No.3, pp. 1-50.

Cargill, K.W., (1984), "Prediction of Consolidation of Very Soft Soil", JGED, ASCE, Vol. 110, No.6, pp. 775-795.

Chang, C.S., (1985), "Uncertainity of One Dimensional Consolidation Analysis", JGED, ASCE, Vol.111, No. 12, pp. 1411-1423.

Das, B.M., (1985), "Advanced Soil Mechanics", McGraw-Hill Book Company, New York.

Dennehy, W.J., (1979), "The Remoulded Undrained Shear Strength of Cohesive Soils and its Influence on the Stability of Embankment Fill", Proc. of the Conf. held at the ICE, London, pp. 87-94.

Drnevich, V.P., (1980), "Normalized Stress-Strain for Undrained Shear Tests", Laboratory Shear Strength of Soil, ASTM, USA, pp. 387-409.

Eulalio, J.B. and Chen B., (1983), "Consolidation Curves for Clays", JGED, ASCE, Vol.109, No.10, pp. 1303-1312.

Eusufzai, S.H.K., (1967), "Soil Profile Across Dhaka, the Capital City of East Pakistan", the 2nd SEARCSE, Singapore, pp. 73-80.

Fleming, L.F. and Duncan, F. (1990), "Stress-Deformation Characteristics of Alaskan Silt", JGED, Vol. 116, No. 3, pp. 377-393.

Henkel, D.J., (1959), "The Relationships Between the Strengths, Pore Water Pressure and Volume Change Characteristics of Saturated Clays", Geotechnique 9, pp. 119-135.

Islam, A., (1980), "Comparison of Three Dimensional Settlement Analysis with Conventional Methods for Preloaded Clay", M.Sc. Thesis, BUET, Dhaka.

Jamiolkowski, M., Ladd, C.C., Germaine, J.T. and Lancellotta, R., (1985), "New Developments in Field and Laboratory Testing of Soils", XI ICSMFE, Special Lecture.

Kondner, R.L (1963), "Hyperbolic Stress-Strain Response of Cohesive Soil", JSMFD, ASCE, Vol. 89, No. SM1, pp. 115-143.

Koutsoftas, D.C., (1980), "Undrained Shear Behaviour of a Marine Clay", Laboratory Shear Strength of Soil, ASTM, USA, pp. 254-276.

Ladd, C.C., (1971), "Strength Parameters and Stress-strain Behaviour of Saturated Clays", MIT, Cambridge, Research Report R71-23.

Ladd, C.C., (1964), "Stress-strain Response of Clay in Undrained Shear", JSMFD, ASCE, Vol. 90, SM5, 1964, pp. 103-132.

Lambe, T.W., and Whitman, R.V., (1979), "Soil Mechanics", MIT, Wiley Eastern Limited.

Mayne, P.W. and Swason, P.G., (1980), "The Critical State Pore Pressure Parameter from Consolidated Undrained Shear Tests", Laboratory Shear Strength of Soil, ASTM, USA, pp. 410-430.

Mayne, P.W. and Kulhawy F.H., (1982), "K_o-OCR Relationships in Soils", JGED, ASCE, Vol. 108, GT.6, pp. 851-872.

Mayne, P.W. (1984), "K_o-C_u/σ_{vo} Trends for Overconsolidatedn Clays", JGED, Vol.110, No.10, pp. 1511-1516.

Mayne, P.W., (1980), "Cam-Clay Predictions of Undrained Strength", JGED, ASCE, Vol. 106, GT.11, pp. 1216-1240.

Mitachi, T and Kitago, S. (1976), "Change in Undrained Shear Strength Characteristics of Saturated Remoulded Clay due to Swelling" Soils and Found. Vol. 16, No.1, JSSMFE, pp. 45-58.

Mitchell, J.K (1976), "Fundamentals of Soil Behaviour", John Wiley and Sons, New York.

Morgan, J.P and McIntire (1959), "Quarternary Geology of the Bengal Basin, East Pakistan and India", Bulletin of the Geological Society of America, Vol. 70, pp. 319-342.

Moroto, N. and Muramatsu, M. (1987), "K_o Value and Degree of Anisotropy of Overconsolidated Clay", Proc. of 9th SEAGC, Bankok, pp 5-25 to 5-32.

Olson, E. and Ladd, C.C., (1979), "One Dimensional Consolidation Problems", JGED, ASCE, Vol. 105, No. GT. 1, pp. 11-30.

Poskitt, T.J., (1971), "Consolidation of Clay and Peat with Variable Properties", JGED, ASCE, 97 (SM6), pp. 841-880.

Rahman, M.S., (1982), "Time Dependent Deformation Behaviour of Soils", MSC. thesis, BUET, Dhaka.

Rutledge, P.C. (1944), "Relation of Undisturbed Sampling to Laboratory Testing", Transactions, ASCE, 109, pp. 1155-1216.

Safiullah, A.M.M., (1977), "Causes of Failure of Embankments, Dhaka-Narayanganj-Demra Embankments - A Case Study", M.Sc.Engg. thesis, BUET, Dhaka.

Somarasinghe, A.M., Huang, Y.H. and Drnevich, V.P., (1982), "Permeability and Consolidation of Normally Consolidated Soils", JGED, ASCE, Vol. 108, GT 6, pp. 835-850.

Scofield, A. and Wroth, P. (1968), "Critical-State Soil Mechanics", McGrow-Hill Book Company, London.

Sheriff, M.A., and Ishibashi, I., (1981), "Overconsolidation Effect on K_o Values", X ICSMFE, Vol.1, pp. 785-788.

Schmertmann, J.H. (1955), "The Undisturbed Consolidation Behaviour of Clay", Tranactions, ASCE, 120, pp. 1201-1227.

Schmertman, J.H., (1983), "A Simple Question About Consolidation", JGED, ASCE, Vol. 109, No.1, pp. 119-122.

Serajuddin, M. and Ahmed, A. (1982), "A Study of Some Engineering Properties of Soils Occurring in Different Regions of Bangladesh", Proc. of the Seventh SEAGC, Hong Kong, pp. 853-864.

Serajuddin, M. and Ahmed, A. (1967), "Studies on Engineering Properties of East Pakistan Soils", Proc. First SEARCSE, Bangkok, pp. 9-22.

Serajuddin, M., (1987), "Universal Compression Index Equation and Bangladesh Soils", 9th SEAGC, Bangkok, pp. 5-61 to 5-72.
Tavenas, F., (1980), "Some Aspect of Clay Behaviour and Their Consequences on Modelling Techniques", Laboratory Shear Strength of Soil, ASTM, USA, pp. 667-677.

Terzaghi, K., (1943), "Theoretical Soil Mechanics", John Wiley and Sons (Publ.), New York.

Tien H.W.F., Nien Y.C. and Elfatih M.A., (1978), "Consolidation and Strength Properties of a Clay", JGED, ASCE, Vol. 104, GT.7, pp. 889-905.

Topolnicki, M., (1989), "On Shear Strength of Normally Consolidated and Saturated Clay", XII ICSMFE, Vol.1, pp. 127-130.

Wroth, C.P., (1984), "The Interpretation of In-situ Soil Tests", Geotechnique, Vol. 34, No. 4, pp. 449-489.

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WASA, Dhaka, (1990), "Dhaka Region Groundwater and Subsidence Model", Interim Report.

Yong, R.N. and Tabba, M.M., (1980), "On Random Aspect of Shear Strength", Laboratory Shear Strength of soil, ASTM, USA, pp. 485-501.

Yudhbir, Varadarajan, A., Mathur, S.K., (1975), "Evaluation of Stress-Strain Modulus of Saturated Clays", 4th SEARCSE, Kualalumpur, pp. 230-241.

Zhang, C.H., Wang, B.Y., and Wang J.J. (1982), "The Apparent Overconsolidated Behaviour of Shanghai Clay and its Application", Proc. of the Seventh SEAGC Hong Kong, pp. 751-760.