Stress-Deformation Characteristics of Selected Coastal Soils of Bangladesh and their Sampling Effects

by

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Md. Abul Bashar

Affectionately dedicated
To
My Parents, Wife and Children

for their sacrifice and bearance during this research

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NOTATIONS

A	Skempton's pore pressure parameter
ACU	Anisotropically consolidated undrained triaxial test.
AR	Area ratio = $(D_w^2 - D_i^2) / D_i^2$
B	Skempton's pore pressure parameter
A_p	Skempton's pore pressure parameter A at peak deviator stress
A_u	Skempton's Pore pressure parameter for undrained release of shear stress
C_c	Compression index
$C_c^{(iso)}$	Compression index from isotropic consolidation test
$C_c^{(Ko)}$	Compression index from K_0-consolidation test
C_c^*	Intrinsic compressibility
C_s	Swelling index
C_s^*	Intrinsic swelling index
$C_s^{(iso)}$	Swelling index from isotropic consolidation test
$C_s^{(Ko)}$	Swelling index from K_0-consolidation test
CSL	Critical state line
CIU	Isotropically consolidated undrained test
CK_0U	K_0-consolidated undrained test
D_e	External diameter of sampler tube
D_i	Internal diameter of sampler tube at the cutting shoe
D_s	Internal diameter of sampler tube
D_d	Degree of disturbance = $1 - (\sigma'_i / \sigma'_{ps})$
D_w	External diameter of cutting shoe
Δu	Change in pore pressure
e	Void ratio
e^*	Intrinsic void ratio
e^*_{100}	Intrinsic void ratio at consolidation pressure of 100 kPa
e^*_{1000}	Intrinsic void ratio at consolidation pressure of 1000 kPa
e_L	Void ratio at liquid limit
E_i	Initial tangent modulus
E_{50}	Secant stiffness at half the peak deviator stress
E_u	Secant stiffness
ε_a	Axial strain
ε_p	Axial strain at peak deviator stress
ε_v	Volumetric strain

G_s	Specific gravity of soil grains
H	Slope of Hvorslev surface
I_V	Void index = $(e - e^{*}_{100}) / C_c^{*}$
ICA	Inside cutting edge angle
ISL	Intrinsic swelling line
ICL	Intrinsic compression line
IC-U	Isotropically consolidated undrained triaxial test.
ICR	Inside clearance ratio = $(D_s - D_i) / D_i$
ISL	Intrinsic swelling line
K_0	Coefficient of earth pressure at rest
κ	Average slope of swelling and recompression lines
$\kappa^{(iso)}$	Average slope of swelling and recompression lines for isotropic stress condition
$\kappa^{(Ko)}$	Average slope of swelling and recompression lines for K_0 stress condition
Λ	Critical state pore pressure parameter
$\Lambda^{(iso)}$	Critical state pore pressure parameter for isotropic stress condition
Λ_{o}, $\Lambda^{(Ko)}$	Critical state pore pressure parameter for K_0 stress condition
LL, w_L	Liquid limit
λ	Slope of virgin compression line
$\lambda^{(iso)}$	Slope of virgin compression line for isotropic consolidation
$\lambda^{(Ko)}$	Slope of virgin compression line for K_0-consolidation
M	Slope of critical state line for isotropic stress condition
N	Specific volume from normally (isotropic) consolidation line at $p' = 1$ kPa
NC	Normally consolidated
NCL	Normally consolidation line
N_0	Specific volume from normally (K_0) consolidation line at $p' = 1$ kPa
OC	Overconsolidated
OCA	Outside cutting edge angle
OCr	Outside clearance ratio = $(D_w - D_e) / D_e$
OCR	Overconsolidation ratio
PL, w_P	Plastic limit
PI, I_P	Plasticity index
p'	Mean effective stress = $(\sigma'_a + 2\sigma'_r)/3$
p'_c	Maximum effective consolidation pressure

p'_e	Equivalent pressure
p'^{*}_e	Equivalent pressure on the intrinsic compression line corresponding to the void ratio of the natural soil at yield.
p'_o	Mean effective stress prior to undrained shearing
q'	Deviator stress = $\sigma'_a - \sigma'_r$
R	Correlation coefficient
s'	$(\sigma'_a + \sigma'_r)/2$
S_t	Sensitivity
s_u, c_u	Undrained shear strength
$s_u^{(nc)}$	s_u in normally consolidated state
$s_u^{(oc)}$	s_u in overconsolidated state
s_{up}	Undrained shear strength of "perfect" sample
s_{ut}	Undrained shear strength of "tube" sample
s_u/σ'_{vc}	Normalized undrained shear strength
σ'_a, σ'_1	Axial or vertical effective stress
σ'_c	Isotropic consolidation pressure prior to undrained shearing
σ'_{cm}, σ'_p, σ'_{vc}	Maximum consolidation pressure
σ'_i	Initial or residual effective stress
$\Delta\sigma_h$	Change of horizontal total stress
σ'_{ps}	Isotropic effective stress in a saturated "perfect" sample $= \sigma'_{vc}[K_0 + A_u(1 - K_0)]$
σ'_r, σ'_3	Radial or horizontal effective stress
σ'_v,	In situ vertical effective stress
$\Delta\sigma_v$	Change of vertical total stress
σ'_{vo}	Vertical effective stress prior to undrained shearing
t	Thickness of the sampler tube
t'	$(\sigma'_a - \sigma'_r)/2$
UU	Unconsolidated undrained triaxial test.
USCS	unified soil classification system
v	Specific volume
v_k	Specific volume from the swelling line at $\ln(p') = 0$ or $p' = 1$ kPa
w	Water content
ϕ'	Effective angle of internal friction
ϕ^{*}_{cs}	Angle of intrinsic shearing resistance

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ABSTRACT

A considerable development activity within the Coastal Region of Bangladesh has necessitated an understanding of the geotechnical behaviour of soils from this region. With this objective in view a study into strength-deformation, compressibility and intrinsic properties of reconstituted samples of Chittagong coastal soils were undertaken. This thesis presents stress-deformation characteristics of three selected coastal soils and their sampling effects. The soils were collected from Banskhali, Anwara and Chandanaish in Chittagong coastal belt of Bangladesh. The soils are low to medium plasticity (Liquid limit = 34 to 45 and Plasticity index = 10 to 20). Reconstituted samples of the three soils were prepared in the laboratory by K_0-consolidation of slurry in a large cylindrical consolidation cell using a consolidation pressure of 150 kN/m^2. Overconsolidated samples were prepared in the triaxial cell by releasing the maximum isotropic consolidation pressure of 150 kN/m^2 to appropriate values to achieve overconsolidation ratios (OCR) of 1.5, 2, 5, 10, 20 and 30.

The stress-deformation-strength, stiffness and pore pressure characteristics of reconstituted isotropically normally consolidated and overconsolidated "block" samples of the three coastal soils were investigated in the laboratory by performing undrained triaxial compression tests. Models for the prediction of undrained shear strength of normally consolidated and overconsolidated samples have been developed. To develop intrinsic models of compressibility, intrinsic compression lines (ICL) for the three soils under K_0 and isotropic stress conditions have been established which can be used to determine compressibility indices of soils at any depth of known overburden pressure. State boundary surfaces (Roscoe and Hvorslev state boundary surfaces) and critical state lines of the three coastal soils have been established. The critical state parameters of the soils have also been evaluated. Constitutive models relating critical state soil parameters and plasticity index of the soils have been proposed. Applications of these models to undisturbed natural clays will require further investigation. The experimentally observed stress-strain behaviour of reconstituted normally consolidated samples of the three coastal soils have been compared with those predicted using two critical state models, namely, "Cam clay model" and "Modified Cam clay model". It has been found that the results predicted by using "Modified Cam clay model" compared more favourably with the observed experimental results than "Cam clay model" for the three coastal soils.

The present study has also been carried out to investigate the effects of "perfect" sampling disturbance and tube sampling disturbances on engineering properties of reconstituted normally consolidated samples of the three coastal soils. Undrained triaxial compression tests were carried out on "in situ", "perfect" and "tube" samples. "In situ" samples were prepared by consolidating reconstituted specimens of 38 mm diameter by 76 mm high under K_0-condition in the triaxial cell to its in situ stress state. "Perfect" samples were prepared from "in situ" samples by undrained release of the total stresses in the triaxial cell. "Tube" samples were prepared from the large diameter consolidated samples by inserting samplers of different area ratios, external diameter to thickness ratio (D_e/t) but of constant outside cutting edge angle (OCA) and internal diameter (D_i). Area ratio, D_e/t ratio, OCA and D_i of the samplers were 16.4% to 73.1%, 27.3 to 8.3, 5° and 38 mm, respectively. Undrained triaxial compression tests were carried out on reconsolidated "perfect" and "tube" samples of the three coastal soils to assess the suitability of various reconsolidation techniques to minimize sampling disturbance effects.

Experimental results indicate that disturbances due to perfect and tube sampling have significant influence on the mechanical properties of coastal soils. The nature of the effective stress paths and pore pressure responses of both "perfect" and "tube" samples are markedly different from those of the "in situ" samples. The "perfect" and "tube" samples adopted stress paths and showed pore pressure responses which are more typical of overconsolidated clays. Disturbances due to perfect sampling led to reduction in the values of undrained shear strength (s_u), Skempton's pore pressure parameter A at peak deviator stress (A_p), initial tangent modulus (E_i) and secant stiffness at half the peak deviator stress (E_{50}) while axial strain at peak deviator stress (ε_p) increased due to total stress relief. Due to total stress relief, the reduction s_u, E_i and E_{50} increased with the decrease of plasticity while the increase in ε_p increased with the decrease of plasticity of the soils. It is also evident that the decrease in mean effective stress (p′) due to perfect sampling increases with decreasing plasticity of the soils. The initial effective stress (σ'_i) of "tube" samples reduced considerably because of disturbance caused by penetration of tubes. Compared with the "in situ" samples, values of s_u, E_i, E_{50} and A_p of the "tube" samples decreased while ε_p increased. The changes in measured soil parameters between the "in situ" and "tube" samples have been found to depend significantly on the sampler characteristics, i.e., area ratio, D_e/t ratio, used for retrieving the "tube" samples. The values of σ'_i, s_u, E_i and E_{50} were decreased due to increase in area ratio (or reduction in D_e/t ratio). The values of ε_p, however, increased due to increasing area ratio. A quantitative increase in the degree of disturbance (D_d) has been obtained due to increase in area ratio, while the values of D_d increased with the decrease of D_e/t ratio of sampler. Disturbance due to tube sampling has been found to depend on the plasticity of the samples of the three coastal soils. The highest reductions in σ'_i, s_u, E_i and E_{50} occurred in the least plastic samples, whereas the minimum reduction in σ'_i, s_u, E_i and E_{50} occurred in the most plastic samples. Among the samples of the coastal soils, the least plastic sample produced higher degree of disturbance than the most plastic sample.

It appeared from the present investigation that for good quality sampling, a sampler ought to have an area ratio as low as possible, preferably less than 10 %. A correction curve has also been developed from the strength data of "perfect" and "tube" samples for estimating the perfectly undisturbed undrained shear strength of the tube samples retrieved from the coastal region studied for use in geotechnical analyses and designs.

Isotropic reconsolidation to a pressure equal to vertical in situ pressure σ'_{vc} (CIU-$1.0\sigma'_{vc}$) has the effect of producing large overestimation of in situ strength s_u, ε_p, E_i and E_{50} of the "perfect" and "tube" samples. Isotropic reconsolidation to σ'_{vc} also overestimated the values of A_p. However, isotropic reconsolidation to a pressure equal to isotropic effective stress σ'_{ps} (CIU-$1.0\sigma'_{ps}$) of the "perfect" sample underestimated the values of s_u, E_i, E_{50} and A_p, while overestimated the value of ε_p for "perfect" samples. It has been found that compared with SHANSEP procedures, K_0-reconsolidation up to in situ state of stress, i.e., Bjerrum procedure (CK_0U-$1.0\sigma'_{vc}$) produced the best overall estimate of the in situ properties in terms of the undrained strength, strain, stiffness and pore pressure response.

CHAPTER 1

INTRODUCTION

1.1 General

Bangladesh is almost entirely an alluvial, deltaic plain with hills on the north-east, east and south-east margins. The land of Bangladesh slopes gently from the north towards the Bay of Bengal at a rate of 1m in 20 km (Alam et al., 1990). The alluvial plains extend about 400 km south eastward, falling gradually from an elevation of about 90 m in Tetulia in the far northwest to a coastal plain of less than 3 m in elevation south of a line joining Khulna-Chandpur-Noakhali. These low lying areas and the Chittagong coastal plains forms the coastal areas of Bangladesh. Chittagong coastal zone is geologically different from the deltaic arcs of Khulna, Patuakhali, Noakhali. It occupies entire lower reaches of the Karnaphuli and the Sangu rivers in Patya, Anwara, Banskhali and Chandanaish Upazilas. Chittagong coastal tidal flood plain comprises almost level landscape with low ridges, inter-ridge depressions and shallow basins. Most of the ridges are shallowly flooded by rain water in the monsoon season and low ridges inter-ridges depression and shallow basins adjacent to rivers, creeks along the coastal belt are inundated for few hours by tidal waters during high tides. The landscape is underlain by the fine textured deposits that filled up a huge tidal back swamp and was later mostly covered by mainly moderately fine textured sediments. All the sediments are rather silty. They were carried down mainly by the Karnaphuli and the Sangu rivers and were deposited under tidal condition.

It is essential to understand the stress-deformation characteristics of the soil in the coastal belt of Chittagong region in Bangladesh as in recent times considerable development works are in progress in this region. Every year this region is affected by severe cyclone and flood. Several million people live in the coastal high risk area along the Chittagong coastal belt which is prone to cyclone damage. In addition to this, cyclone and storm surge produced extensive damage to livestock, agriculture, power system, telecommunication, housing and other physical infrastructure facilities mainly in and around Chittagong. Flood protection embankments and cyclone shelters are needed to build in large quantities in this coastal region. So it is important to determine the necessary soil properties, particularly stress-deformation-strength, stiffness and compressibility of the soil at shallow depths of this region.

From the above different points of views, soils from the three locations in the coastal belt of Chittagong region were selected for evaluation of their stress-deformation-strength, stiffness and compressibility characteristics. To determine compressibility of natural intact soils, it may be useful to establish a generalized curve or equation using Burland's (1990) concept. A model to predict undrained shear strength of natural soils is to be established from the correlation of shear strength and overburden pressure in case of reconstituted samples of normally consolidated and overconsolidated states. Comparison of experimentally observed stress-strain behaviour with those predicted using two Critical State theories, namely Cam clay and Modified Cam clay, are to be made. To develop the complete state boundary surface of the coastal soils, laboratory tests were performed on reconstituted isotropically normally consolidated and overconsolidated samples. In this research work attempt has also been made to develop some constitutive models relating critical state parameters and the plasticity indices of the soils.

The engineering properties of soils needed for geotechnical analyses and designs are usually carried out on soil samples previously retrieved from the ground using some form of sampling procedure. In the laboratory the stresses, deformations and boundary conditions can be readily and precisely controlled and observed (Jamiolkowski et al., 1985). However, the inherent problem with the sampling approach is that it disturbs the soil sample. This disturbance can be significant, such that the behaviour of the soil in the laboratory differs markedly from its in situ behaviour. The significance of the disturbance of the soil depends on many factors including the type of soil, the method of sampling, sealing, storage, specimen preparation and testing procedure. Soil disturbance is often regarded as a significant problem because it is thought to prevent acquisition of realistic soil parameters. During sampling process, soil is disturbed in two major ways. Firstly, mechanical disturbance is caused by pushing tube samplers into the soil which produces shear distortion and subsequent compression of soil close to the inside wall of the tubes (Schjetne, 1971). This disturbance is termed as tube penetration disturbance. The source of this disturbance is directly associated with sampler design and can be controlled to certain extent. Secondly, the disturbance can be experienced as a result of stress relief due to removal of the sample from the field to zero total stress state in the laboratory. This disturbance is termed as stress relief disturbance or "perfect" sampling disturbance. Numerous investigators attempted to assess the influence of "perfect" and tube sampling disturbance for both intact natural soils and laboratory prepared reconstituted soils (Skempton and Sowa, 1963; Noorany and Seed, 1965; Davis and Poulos, 1967; Kubba, 1981; Kirkptrick and Khan, 1984; Hight et al., 1985; Baligh et al., 1987; Hight and Burland, 1990; Siddique, 1990; Clayton et al., 1992; Wei et al., 1994; Hird and Hajj, 1995; Georgiannou and Hight, 1994; Hight and Georgiannou, 1995; Siddique and Farooq, 1996; Siddique and Sarker, 1995; Siddique and Sarker, 1997; Siddique et al., 1999; Siddique et al., 2000; Siddique and Rahman, 2000; Rahman, 2000).

Since the stress-deformation properties of soils are significantly affected by sample disturbance due to sampling operations, attempt has been made in present research to evaluate the effects of sampling process, e.g. "perfect" sampling and tube sampling on the measured geotechnical parameters of the coastal soils. Comparisons of the results of the present investigation can be made with the available limited results on "perfect" and tube sampling disturbance effects in coastal soils of Bangladesh as reported by Farooq (1995), Siddique and Farooq (1996) and Siddique et al. (2000). This would establish a generalized framework of behaviour of coastal soils of Bangladesh and their sampling effects.

In order to study specific effects of tube and perfect sampling disturbance only on soil behaviour, disturbances due to other sources should be eliminated. This can be achieved in laboratory study on reconstituted soil in which disturbances due to boring and trimming of specimens can be eliminated. Natural intact soils are seldom uniform due to complex geological conditions in the field acted upon them. As such, from the test results on the samples collected from the field, it is rather difficult to generalize the behaviour of soils and to study specific effects on soil properties due to disturbance caused by "tube" sampling and also by "perfect" sampling. Therefore, in this research it has been considered essential to use uniform reconstituted samples prepared under controlled conditions in the laboratory. Reconstituted soils enable a general pattern of behaviour to be established. The major advantages of using data from reconstituted soils are that the ambiguous and substantial effects of sample inhomogeneity can be eliminated while the essential history and composition of in-situ soils can be represented.

Regarding the extent of sample disturbance in clays, one of the most important contributory factors is the design of the sampler. Soil disturbance can be minimized by careful control of the whole sampling process and also to large extent by using properly designed sample tubes. The degree of disturbance varies considerably depending upon the dimensions of the sampler and the precise design of the cutting shoe of the sampler (Hvorslev, 1949; Jakobson, 1954; Kallstenius, 1958; Kubba, 1981; Andresen, 1981; La Rochelle et al. 1981; Baligh et al., 1987; Siddique, 1990; Siddique and Clayton, 1995; Siddique and Sarker, 1996; Tanaka et al., 1996; Siddique and Clayton, 1998; Clayton et al., 1998; Siddique and Farooq, 1998; Clayton and Siddique, 1999; Siddique et al., 2000; Siddique and Rahman, 2000). It is therefore extremely important that geotechnical engineers have a sound understanding of the extent, both qualitative and quantitative, to which the soil parameters being used have been affected by the sampling process as well as by the design of a sampler. In the present works, attempt has also been made to assess the effect of sampler characteristics, namely area ratio and external diameter to thickness ratio of sampler, on the measured undrained soil parameters of the three coastal soils.

A number of investigations were carried out in the past to select the appropriate technique for minimizing the effects of "perfect" and "tube" sampling disturbances on the undrained stress-strain-strength, stiffness, and pore pressure characteristics of soils. Anisotropic K_0-reconsolidation to in situ stress state has been suggested as the best minimizing method by a number of researchers (Davis and Poulos, 1967; Atkinson and Kubba, 1981; Kirkpatrick and Khan, 1984; Graham et al., 1987; Graham and Lau, 1988; Fleming and Duncan, 1990). On the other hand SHANSEP procedure has been suggested as the best minimizing procedure by other researchers (Gens, 1982; Hight et al., 1985; Baligh et al., 1987; Siddique and Sarker, 1996; Siddique and Farooq, 1996). Moreover, Wang et al. (1982) and Ladd et al. (1985)

reported that the values of normalized strength vary considerably from one silt to another and would need to be evaluated specifically for each new silt deposit. So, it is necessary to know by how much the stress level should be raised in order to eliminate or minimize the effects of "perfect" or "tube" sampling disturbance. Therefore, the present investigation has also been aimed to investigate different reconsolidation techniques, particularly isotropic reconsolidation and anisotropic reconsolidation under K_0-conditions including SHANSEP procedures, in order to minimize the effects of both "perfect" sampling and "tube" sampling disturbance in the coastal soils.

1.2 Objectives of the Present Research

The present study has been undertaken to investigate the stress-deformation characteristics and sampling effects of the three soils collected from Chittagong Coastal region in Bangladesh in order to achieve the following objectives:

(i) To evaluate the intrinsic compressibility characteristics of the three coastal soils using Burland's (1990) concept.

(ii) To develop models to predict undrained shear strength of normally consolidated and overconsolidated samples, and compressibility of three soils collected from the Chittagong Coastal region of Bangladesh.

(iii) To evaluate the critical state soil parameters for reconstituted isotropically and K_0 consolidated soils and to establish some constitutive equations between plasticity index and critical state parameters.

(iv) To develop the complete state boundary surface (Roscoe and Hvorslev State Boundary Surfaces) for the three coastal soils.

(v) To predict the strains and the effective stresses using the critical state Cam clay model and Modified Cam clay model, and to compare those with the experimental results of reconstituted isotropically normally consolidated samples.

(vi) To investigate the effects of "perfect" and "tube" sampling disturbance on engineering properties (e.g., undrained shear strength, deformation, stiffness and pore water pressure response) of the three reconstituted soils of varying plasticity.

(vii) To investigate the effects of "tube" sampling disturbance on sampler characteristics (e.g., area ratio and D_e/t ratio) undrained shear characteristics of the three coastal soils.

(viii) To investigate the influence of isotropic reconsolidation and anisotropic reconsolidations using Bjerrum (1973) and SHANSEP procedures (Ladd and Foott, 1974) in order to assess the suitability of reconsolidation of both "perfect" and "tube" samples to establish "in situ" behaviour by minimizing sampling disturbance effects in reconstituted coastal soils.

1.3 The Research Scheme

The use of undisturbed soil samples for testing would be very much desirable in the investigation of their behaviour. However, such samples are seldom uniform due to complex geological conditions acted upon them and as such, from the test results on such samples, it is rather difficult to generalize the behaviour of soils. Therefore, to study any specific effect on the behaviour of soils, reconstituted samples (of uniform density and water content) of coastal soils were prepared. From such samples, it is possible to develop basic frame work for strength and deformation characteristics of coastal soils and their sampling effects. Since reconstituted samples have been used in this research, the pattern of behaviour discussed in this thesis will be taken to represent that of young or unaged resedimented soils where no post depositional process have operated. In order to attain the objectives, the whole programme was carried out according to the following phases:

Phase 1: Index properties of the three soils of Chittagong coastal region were determined for characterization of the soils.

Phase 2: Compressibility and swelling characteristics of the reconstituted normally consolidated samples under isotropic condition and K_0-condition were performed in the triaxial cell to develop a model of compressibility for providing a frame of reference for assessing the in situ state of a natural soil.

Phase 3: Conventional triaxial compression test (CIU-test) were performed on reconstituted normally consolidated and overconsolidated "block" samples of three coastal soils to establish a model for predicting undrained shear strength of normally consolidated and overconsolidated samples and to establish the complete state boundary surface.

Phase 4: Conventional triaxial compression tests (CIU- test) on reconstituted samples of three coastal soils were carried out to determine critical state soil parameters and to establish models relating critical state soil parameters and plasticity index.

Phase 5: Computer programme were developed to predict axial strains and effective stresses using two constitutive models, namely, the critical state Cam clay model and the Modified Cam clay model.

Phase 6: Sampling tubes of different area ratio but of same internal diameter as that of test specimens have been designed and fabricated to study the effects of tube sampling disturbance and sampler geometry on undrained shear properties.

Phase 7: The engineering properties of "in situ" samples of the three reconstituted soils were determined by performing undrained triaxial compression tests to determine the reference "undisturbed" behaviour of the soils.

Phase 8: Behaviour of "perfect" samples was investigated by modelling "perfect" sampling on "in situ" samples in the triaxial cell. Unconsolidated undrained (UU) triaxial compression test was performed on each "perfect" sample of reconstituted soils from three locations.

Phase 9: Behaviour of "tube" samples was investigated by performing unconsolidated undrained (UU) triaxial compression tests on "tube" samples of the three reconstituted soils.

Phase10: Finally, undrained triaxial compression tests were carried out on both isotropically and anisotropically reconsolidated "perfect" and "tube" samples to investigate the suitability of various reconsolidation procedures to minimize the effects of "perfect" and "tube" sampling disturbances.

1.4 Thesis Layout

Chapter 2 presents a review of available literature on geology of coastal region of Bangladesh and engineering properties of soils of coastal region of Bangladesh. Review on Critical State concept, critical state models, stress-strain-strength characteristics of soils, intrinsic models of shear strength and compressibility of reconstituted soils have been reviewed in this chapter. The effects of "perfect" and "tube" sampling disturbances on undrained shear properties of both intact and reconstituted soils, effect of sampler geometry on sample disturbance have been reviewed. The application of different reconsolidation procedures to minimize sample disturbance and the various methods for correcting sample disturbance effects have also been reviewed.

In Chapter 3 the equipment and instrumentation used for the laboratory investigation in order to develop models for strength and deformation of soils and to investigate sample disturbance effects in reconstituted soils from the locations in Chittagong Coastal belt are outlined.

Chapter 4 presents the experimental techniques and procedures used for investigating compression and swelling characteristics, undrained shear characteristics for different stress history and the effects of sample disturbance on reconstituted coastal soils.

Chapter 5 presents the undrained behaviour of reconstituted normally consolidated and overconsolidated "block" samples of coastal soils of Chittagong region. Stress-strain, strength, pore pressure and stiffness characteristics of reconstituted coastal soils, model for prediction of undrained shear strength of soils, compressibility or deformation characteristics of reconstituted soils as a basic frame of reference for assessing the in-situ state of stress are presented in this Chapter.

Determination of critical state soil parameters, establishment of state boundary surfaces, correlations of soil constants with plasticity index are presented in Chapter 6. Comparison of experimental stress-strain and stress path with the predicted values using two critical state models are also presented and discussed in this Chapter.

Chapter 7 presents the undrained shear properties of "in situ" samples, "perfect" samples and reconsolidated "perfect" samples. The influence of "perfect" sampling disturbance on undrained shear properties of the soil have been presented in this Chapter. The effects of reconsolidation of "perfect" samples to restore "in situ" behaviour have also been discussed in this Chapter.

Chapter 8 presents the effects of tube sampling disturbance on undrained shear properties. The influence of sampler geometry on the measured soil properties and the different reconsolidation procedures to minimize tube sampling disturbance effects are discussed in this Chapter. A procedure for correction of undrained shear strength is also presented in this Chapter.

Chapter 9 presents the conclusions of the present investigation and recommendations for further research in this field.

CHAPTER 2

REVIEW OF STRESS-DEFORMATION CHARACTERISTICS OF SOILS AND THEIR SAMPLING EFFECTS

2.1 General

This review deals with the geology of coastal soils and their physical and engineering properties. Stress-deformation behaviour of reconstituted coastal soils and the effects of sample disturbance on the stress deformation behaviour have also been reviewed and discussed in this chapter. Intrinsic properties, undrained stress-strain, strength, pore water pressure, stiffness characteristics of reconstituted normally consolidated and overconsolidated soils have also been discussed. This review also deals with Roscoe and Hvorslev state boundary surfaces, critical state parameters, some critical state models, etc.

The availability of realistic mechanical soil parameters for geotechnical design depends on careful testing. Testing may be performed in the field or in the laboratory, but in both the cases the most significant factor controlling the quality of the results would depend on soil or sample disturbance. The mechanisms of sample disturbance have been well understood since 1940s (Hvorslev, 1940 and 1949; Jakobson, 1954; Kallstenius, 1958). Disturbances to soil in its widest sense occur during drilling, during the process of sampling itself, transportation and storing after sampling. A number of different procedures are adopted for measuring, analyzing and correcting the effects of soil sampling disturbance and, in order to highlight the importance of the present research, it is necessary to review previous investigations on sample disturbance.

There has been a wide range of reported observations on the effects of sampling procedures on different types of soils. Some direct investigations considered the effects of major causes of disturbances on the stress-strain, strength, stiffness and pore pressure properties of soils while other indirect observations were concerned more with the design, use and maintenance of samplers and the development of sampling techniques. In this chapter, the previous investigations performed on topics related to soil sampling disturbance are reviewed. The effects of sampling disturbance on the mechanical properties of soils, particularly regional soil are presented. The influences of the design parameters, dimensions of sampler and sampling methods on the measured soil parameters are reviewed. Methods of correcting sampling disturbance effects are also presented.

2.2 The Coast of Bangladesh

2.2.1 The Coastal Zone

Coasts are dynamic interface zones involving the meeting of atmosphere, land and sea. Within the coastal zone, major movements of sediments and nutrients are powered by waves, tides and currents in water and air. These movements shape the coastal profile, producing erosional and depositional landforms.

Characteristically, the coastal zone is taken to include the area between the tidal limits as well as the continental shelf and coastal plain (Viles and Spencer, 1995). Most of the seas are margined by shallow water zones, called the continental shelves, bordering the continents. Tidal range is an important control on coastal ecology and geomorphology, determining the width of coast subjected to alternate wetting and drying and the impact of waves.

2.2.2 Geological Settings of Coastal Regions of Bangladesh

Bangladesh is almost entirely an alluvial, deltaic plain laid down by the sediments of great rivers, Padma, Meghna and Brahmaputra which covers as much as 90 percent of the total land area. The alluvial plain gradually slopes towards the southeast falling from an elevation of 90 meters at Tetulia in northwest corner of the country and extends for about 400 km up to coastal plain having an elevation of 1.5 meters in a line south of Khulna-Barisal-Lakhmipur. These low-lying areas and the Chittagong coastal plains form the coastal areas of Bangladesh. The geology of the coast area is part of the overall Quaternary geology of the Bengal Basin. Bangladesh is situated at the top of the Bay of Bengal where the Indo-Gangetic plain meets the Indian Ocean.

Bangladesh occupies the largest delta in the world. The delta is located at the vertex of the funnel shaped Bay of Bengal. Tropical cyclones are frequent in this area. Storm surges cause an elevation of sea level above the tidal height, which if they occur at spring maxima may have devastating effects on a low-lying coast such as that of Bangladesh.

Bangladesh Coastal area is situated to the south of the line connecting Khulna, Barisal, Chandpur and Noakhali as shown in Fig. 2.1. Bangladesh has been divided into a number of physiographic units, each having fairly uniform physical characteristics (Brammer, 1971). Coastal area of Bangladesh can be placed under the following physiographic units.

Estuarine Flood Plains:

The estuarine flood plains spreading around the mouth of the old Meghna river contain sediment deposits originating from the Ganges and Brahmaputra rivers. Few undulations are observed and the soil type is silty soil. This area is shown under alluvial deltaic silt deposits in Fig. 2.1. In Fig. 2.1 this plain is known as Meghna delta plain.

Ganges Tidal Flood Plains:

These plains are linked to the Ganges estuarine flood plains upstream but are less undulating. The tidal flood plains differ from the Ganges estuarine flood plains in that a well-developed network of numerous tidal creeks and river channels has been formed in the Ganges tidal flood plains. The sediments are mainly of non-calcareous clay, but become more silty in the east and have a buried peat layer in the west. The area is marked as Ganges tidal plain in Fig. 2.1.

The Sundarbans:

The Sundarbans, the south-western forest area of Bangladesh, are areas covered by mangrove forests which are under the influence of tidal floods of brackish or saline water. This unit comprises two sub-units: The Khulna Sundarbans in the south-west and the Chokoria Sundarbans at the mouth of the Matamuhuri in the south-east.

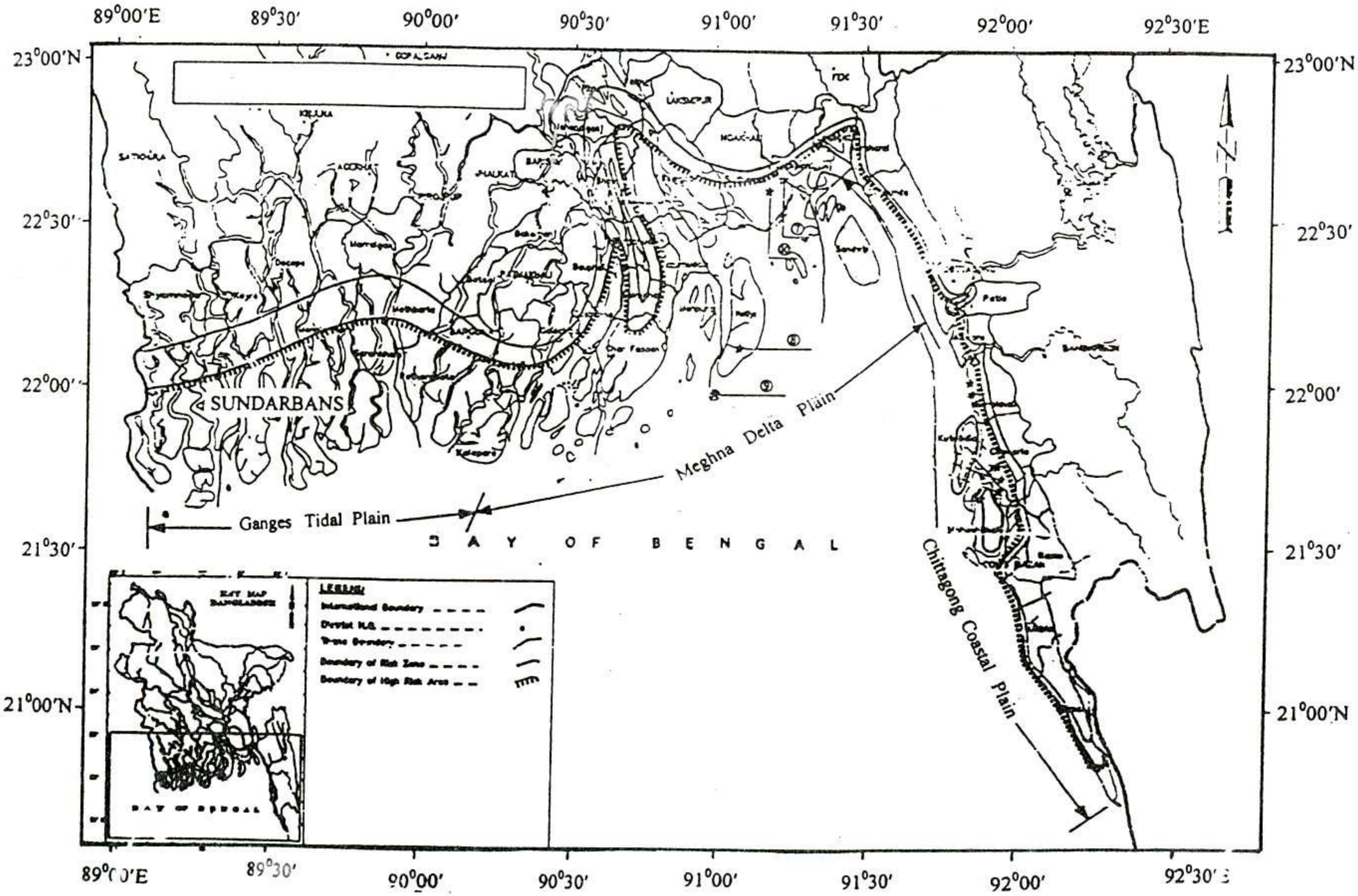

Fig. 2.1 Map of Bangladesh Showing Coastal Zone and Boundary of High Risk Area

The landscape is almost level with innumerable tidal rivers creeks criss-crossing the area. The Sundarbans lies in front of the Ganges Delta and along side of the Early Meghna Delta. The shallow sediments of the Sundarbans consist of flood deposited saline clay and peat layers (BWDB, 1979).

Chittagong Coastal Plain:

This unit comprises the generally narrow strip of land between the Chittagong hills and the sea, together with the Halda, the Karnafuli and the Sangu floodplains and the offshore islands. The landscape is underlain by the fine textured deposits that filled up a huge tidal back swamp and was later mostly covered by mainly moderately fine textured sediments. All the sediments are rather silty. They were carried down mainly by the Karnaphuli and the Sangu rivers and were deposited under tidal condition. Chittagong coastal tidal floodplain comprises almost level tidal landscape with low ridges, inter-ridge depressions and shallow basins. The landscape is traversed by numerous tidal creeks. Most of the ridges are shallowly flooded by rainwater in the monsoon season and low ridges, inter-ridge depression and shallow basins adjacent to rivers, creeks along the coastal belt are inundated for few hours by tidal waters during high tides. The eastern part of this landscape is usually non-saline and the parts adjacent to rivers, creeks and along the coastal belt are locally saline (Soil Survey, 1973). The eastern part of Chittagong coastal zone is also formed of piedmont alluvial deposits transported from the Hill Tracts. The area is shown in Fig. 2.1.

2.2.3 Characteristics of Deposition by Rivers

Rivers play a vital role in the process of erosion, transportation and deposition of sediments. Bangladesh broadly is a drainage basin of the eastern Himalayan region. The Ganges-Brahmaputra-Meghna river systems bring annually about 120 million hectare meter of water into Bangladesh from a catchment area of about 1,65,000 sq. km which is carried to the Bay of Bengal. The tremendous amount of water passing through this river systems lose energy on reaching the plain from hills or adjoining higher areas due to drop of gradient and consequently coarser sediments are deposited in the higher flow regime and progressively finer materials are transported and deposited in the lower flow regime till carried to the sea. On the other hand, vertical accretion leads to lateral variation in the nature of sediments from coarse to fine materials progressively from riverbanks towards the back swamps.

2.2.4 Ground Water in Coastal Area

Ground water in the coastal area is strongly influenced by saline water. Tube wells of more than 200 m deep are dug in and around the Chittagong hills and hills near Moheskhali Island to avoid saline water intrusion. Some flowing artesian wells are also observed in these areas. The well depth in other parts of the coastal area of around 300 m is generally much deeper. Some wells near Noakhali are more than 400 m in depth (MPCSP, Phase II, 1992).

2.2.5 Generalized Geological Cross-Section of Coastal Areas

Most soil studies in the coastal area in the past have dealt with the soil types up to approximately 20 m below the ground surface. Fig. 2.2 shows the generalized geological sections of coastal areas of the three major rivers based on data collected from deep tubewells in the relevant areas. The surface layer mainly consists of silt and clay and has a thickness of some 50 m, except at the mouth of the Meghna river where the thickness is reduced to some 10 m. A more detailed examination reveals that the soil texture of the surface layer differs from one area to another in both the horizontal and vertical directions. The grain size, density and consistency also largely differ from one area to another. These differences reflect the sedimentation environment and are caused by frequent changes of the well-developed river and water channel courses. In general, the deposits of the major rivers are coarser than those of the sea currents.

2.2.6 Physical and Engineering Properties of Coastal Soils of Bangladesh

Available published literature on the geotechnical characteristics of soils are based on soil borings made within the coastal areas for the purpose of building water development projects and low rise buildings such as cyclone shelters. Serajuddin (1969) reported correlation between Dutch Penetrometer cone resistance (q_c) and standard penetration test (SPT) N-value and unconfined compressive strength (q_u) of silty clay materials of the coastal districts of Khulna, Barisal and Chittagong including

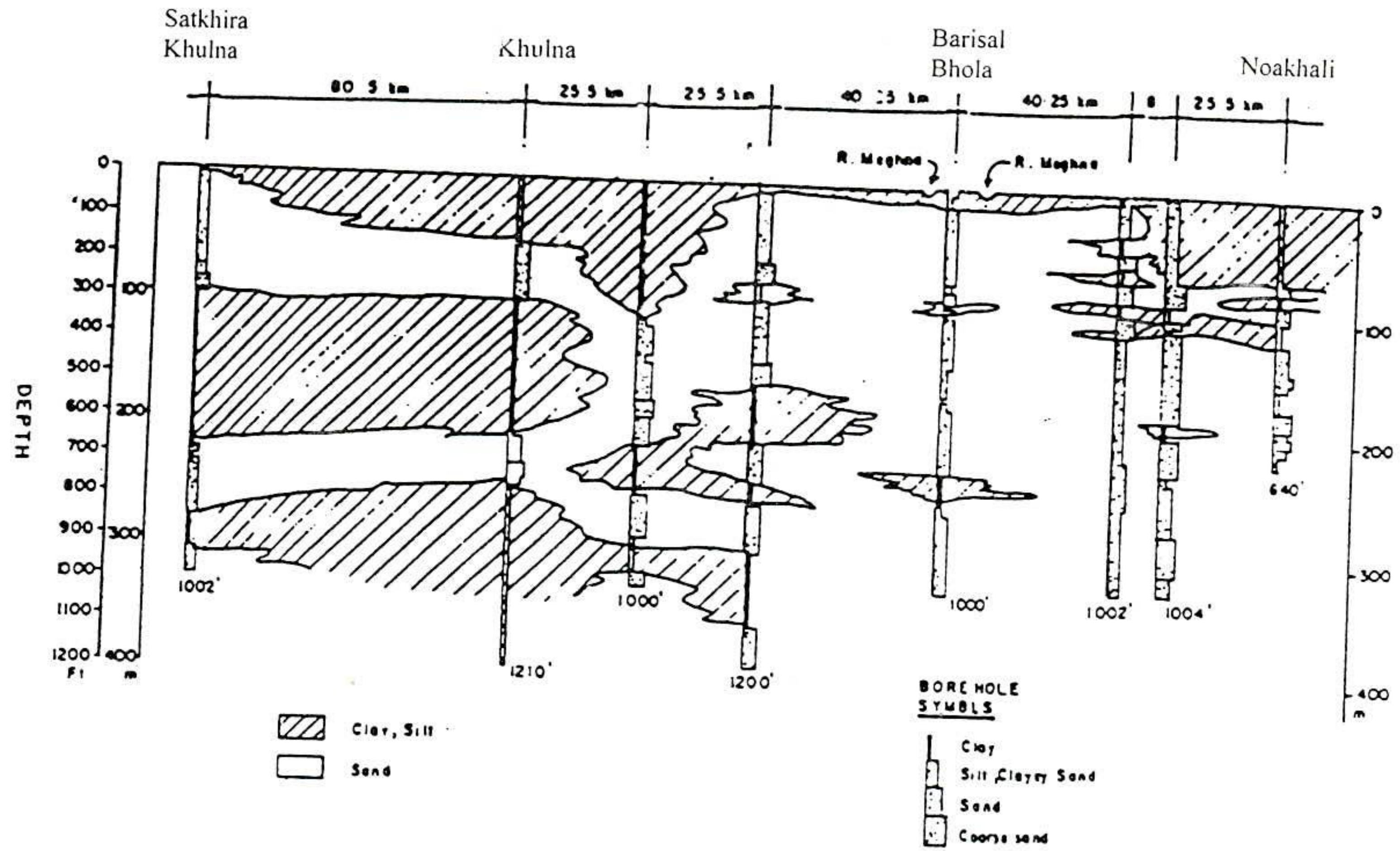

Fig. 2.2 Generalized Geological Cross Section Across Coastal Areas of Bangladesh (after MPCSP, 1992)

islands of Maheskhali and Kutubdia. Dutch cone penetration resistance and disturbed sampling at a large number of sites (163) were done of which probing for 74 sites have been considered. The study also reveals that project area generally has three predominant soil materials, i.e. cohesionless fine sand-silt, peat and cohesive silt-clay. Cohesive silt-clay soils usually exist in the surface and upper layers in varying thickness of about 2 m to about 9 m, particularly in the districts of Khulna and Barisal. In some places the silt-clay is from surface down to depths as great as about 18 m to 24 m, and in other areas it is interbedded with fine sand-silt. The silt-clay materials at the upper layers appear to possess an average cone resistance of about 490 kN/m^2 to 980 kN/m^2. In the non-plastic silty fine sands cone resistance ranged from 1960 kN/m^2 to 8830 kN/m^2 at depths greater than 9 m. However, The SPT N-values in the upper silty clay layers varied widely (from 1 to 17). In some areas, peat layers are found within silt-clay layers.

Amin et al. (1987) reported a comprehensive data on the geotechnical properties of the coastal soils for the districts of Barisal, Bhola, Noakhali and Sandwip from about 200 boreholes. It has been found that the soils from top eight to ten meters of all the zones are predominantly composed of inorganic sandy silts which are saturated, soft to medium stiff in consistency and moderately compressible. These soils are of low plasticity, moderate sensitivity and low activity. Below this silt layer horizons of sand layers exist. Almost all the soils are seen to fall in ML group according to Unified Soil Classification System. A summary of the geotechnical properties of the soils in upper layer studied by Amin et al. (1987) is given in Table 2.1.

Safiullah (1991) reported that geotechnical aspects of soils of Bangladesh is complex and are usually heterogeneous, both in vertical and horizontal directions. Soils consist of wide varieties of material ranging from gravel, poorly graded sand to silt and clay. In general, there is a predominance of silt sized particles. Majority of the soils of Bangladesh falls in two types of deposits, namely terrace deposits and as recent deposits. Finer materials at the surface underlain by coarser materials characterize recent deposits, which consist more than eighty percent of land surface of Bangladesh. Soils of coastal region consist of recent deposits.

Table 2.1 Geotechnical Properties of Soils from Coastal Regions (after Amin et al., 1987)

Properties	Location			
	Barisal	Bhola	Noakhali	Sandwip
Physical Properties				
Liquid limit, LL (%)	24-57	26-45	24-52	25-47
Plastic limit, PL (%)	22-37	23-34	21-31	21-31
Natural moisture content, w (%)	28-57	24-47	27-42	26-46
Plasticity index, Pl (%)	2-25	2-21	2-21	2-24
Clay content (%)	0-18	0-16	0-17	0-26
Engineering Properties				
Compressibility ratio, C_c / (1+e_o)	0.07-0.21	0.06 -0.13	0.05-0.19	0.05 -0.16
Undrained shear strength, s_u (kN/m^2)	8-75	10-90	12-90	12-84

Safiullah (1994) also reported the geotechnical properties of some coastal soils of Bangladesh collecting from ten locations. At each location 5 boreholes were made each within 100 m distance. The details of the soil exploration can be found in MPCSP (1992). The soil profiles are, in general, highly stratified and discontinuous in each direction. The grain size distribution and plasticity chart for soils from above 50 boreholes are presented in Figs. 2.3 and 2.4 respectively. Fig. 2.3 shows that there is an absence of coarse or medium grained sands while Fig. 2.4 shows that finer soils are clays of low plasticity (CL) or silts of low plasticity (ML). Fig. 2.5 shows the plotting of undrained shear strength ratio versus plasticity index for nine coastal soils. Some variation in strength ratio due to variation in individual samples and testing techniques has been observed. Fig. 2.5 also shows that under normally loaded condition the low plasticity soils show a significant increase in strength ratio with decrease in plasticity which is contrary to Skempton's (1957) observation shown in the figure for normally loaded sedimented clay, but converge at PI close to 20%. For minimum number of tests, it might be possible similar to Skempton's (1957) observation. This deviation of trend may also be due to difference in depositional environment or depositional process in soils. The problems of evaluation of in situ undrained shear strength of silty deposits of the coastal areas have also been discussed. It has been demonstrated that traditional correlations of strength and compressibility used for clays may be in error if applied to these soils. Safiullah (1994) also suggested field tests (such as plate load) than laboratory tests for estimation of base stability of embankment in silty clays of Bangladesh. Compressible formations exist within the coastal zones. Analyses of one-dimensional consolidation data for nine locations showed that values of compression index (C_c) ranged between 0.07 for non-plastic silts and 0.41 for plastic clays. Fig. 2.6 shows correlation between liquid limit and $C_c/(1+e_o)$ for the coastal soils. Some scatter at higher values of liquid limit is apparent from Fig. 2.6.

Undrained shear strength and compressibility characteristics of four Coastal soils collected from Chittagong, Patuakhali and Cox's Bazar were reported by Ansary (1993). Tests were carried out on both undisturbed and laboratory prepared reconstituted samples at different stress history and stress conditions. The soils studied consisted of clays of high plasticity (CH), clays of low plasticity (CL) and silts of low plasticity (ML) with clay contents varying from 30 to 41% with little or no

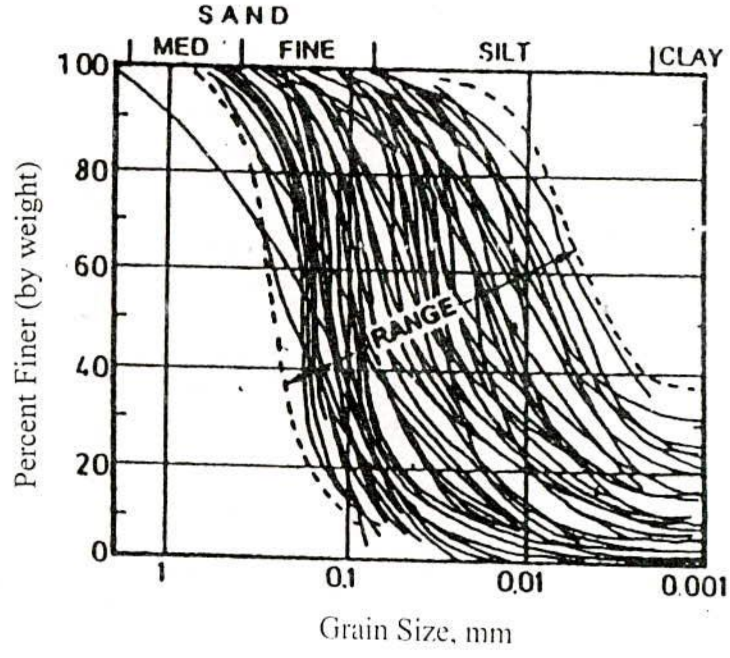

Fig. 2.3 Grain Size Range for Soils from Ten Locations of Coastal Regions of Bangladesh (after Safiullah, 1994)

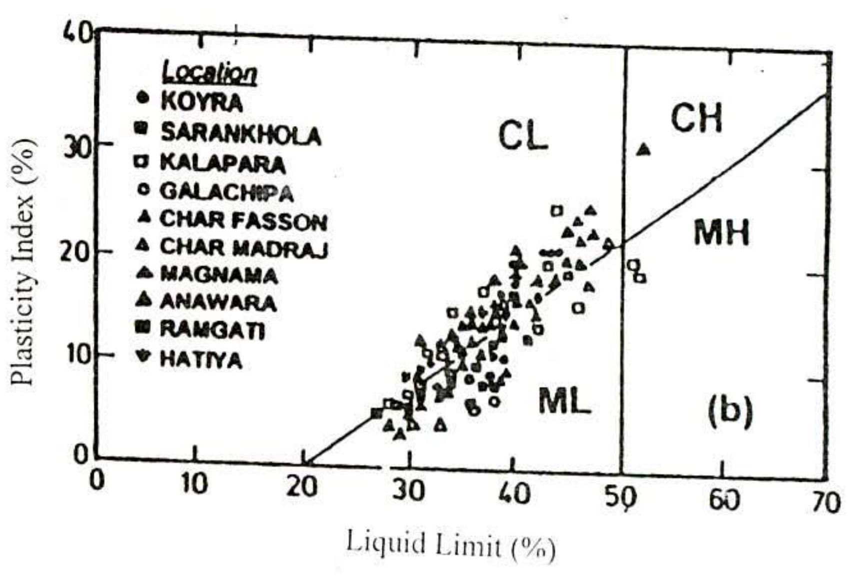

Fig. 2.4 Plasticity Chart for Soils from Ten Locations of Coastal Regions of Bangladesh (after Safiullah, 1994)

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No	Location	w_L (%)	I_p (%)	Soil Type	% Clay
1	KOYRA	44	21	CL	18
2	SARANKHOLA	39	13	ML	15
3	KALAPARA	45	19	CL	16
4	GALACHIPA	36	6	ML	9
5	CHARFASSON	40	14	ML	14
6	CHARMADRAJ	33	4	ML	9
7	MOGNAMA	45	20	CL	15
8	ANWARA	42	15	ML	27
9	HATIA	41	16	CL	16

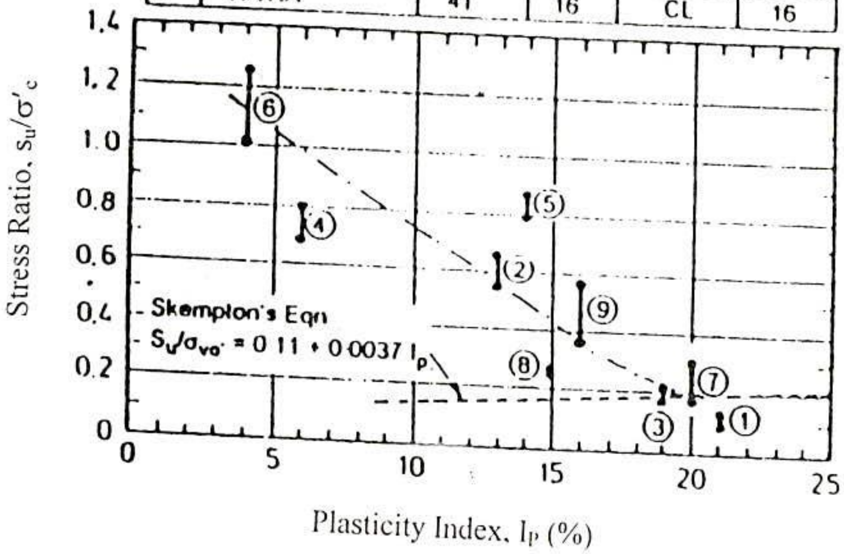


Fig. 2.5 Undrained Shear Strength Ratio (s_u/σ'_{vc}) vs. Plasticity Index for Recompressed Nine Coastal Soils (after Safiullah, 1994)

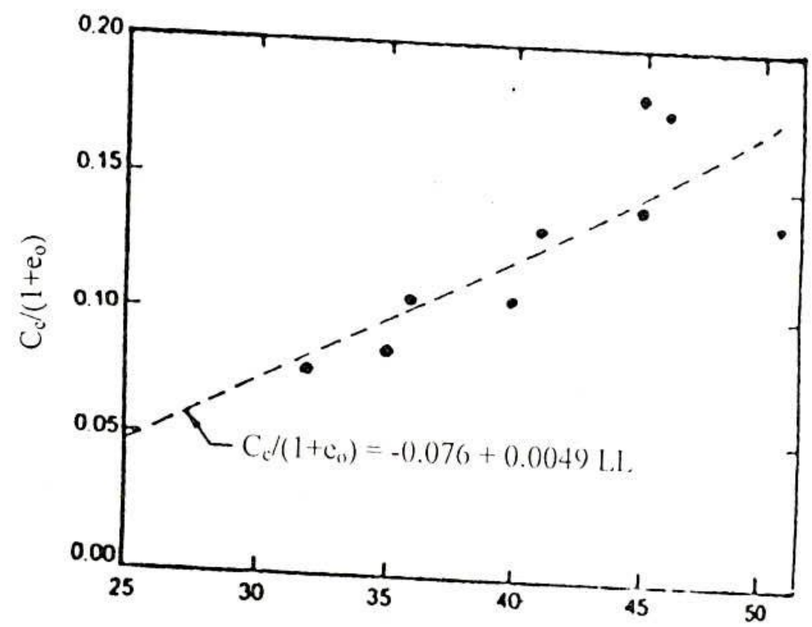


Fig. 2.6 Correlation Between Liquid Limit and $C_c/(1+e_o)$ for Coastal Soils (after Safiullah, 1994)

sand depending on location. For these samples in normally loaded condition, range of s_u/σ'_c was between 0.30 and 0.35 and between 0.14 and 0.31 for undisturbed and reconstituted Coastal soils, respectively. Compared with Skempton's (1957) equation, Ansary (1993) found that for undisturbed soil sample the values of s_u/σ'_c lie well above Skempton's value for the same plasticity index as shown in Fig. 2.7. Ansary (1993) also found that for reconstituted coastal soils, the undrained strength ratio value was high compared to that from Skempton's equation for low plastic clay. However, for high plastic clay the strength ratio is slightly lower than that predicted using Skempton's equation. Siddique and Farooq (1995) studied the strength characteristics of three reconstituted coastal soils and it has been found that strength ratio is increased with the increase of plasticity index. Amin et al. (1987) also reported that s_u/σ'_c increased with the increase of plasticity index for the coastal soils as shown in Fig. 2.8. In Fig. 2.8, zone A, B, C and D means the district of Barisal, Bhola, Noakhali and Sandwip, respectively and their physical properties are shown in Table 2.1.

Siddique and Farooq (1997) investigated the compressibility characteristics of three reconstituted coastal soils from Chittagong belt. They showed that the values of compression index, C_c and the values of swelling index, C_s varied from 0.26 to 0.30 and 0.02 to 0.03, respectively. Ansary et al. (1999) presented the compressibility and swelling characteristics of reconstituted and undisturbed samples of the four coastal soils. They found that the values of C_c and C_s for reconstituted samples varied from 0.29 to 0.40 and 0.028 to 0.046, and for undisturbed samples varied from 0.22 to 0.42 and 0.037 to 0.062, respectively. Ansary et al. (1999) reported that for the samples of high natural water content (38 to 52%), the C_c- values of the undisturbed samples are higher than those of reconstituted samples. For the samples of comparatively low natural water content (26 to 28%), however, the C_c values of the undisturbed samples are lower than those of reconstituted samples. Ansary et al. (1999) also reported that the C_s values of undisturbed samples were always higher than those of reconstituted samples for any water content.

Experiments were also conducted on Rann of Kutch clay (PI = 49) and Kanpur clay (PI = 18) by Varadarajan (1973) to investigate consolidation characteristics. It

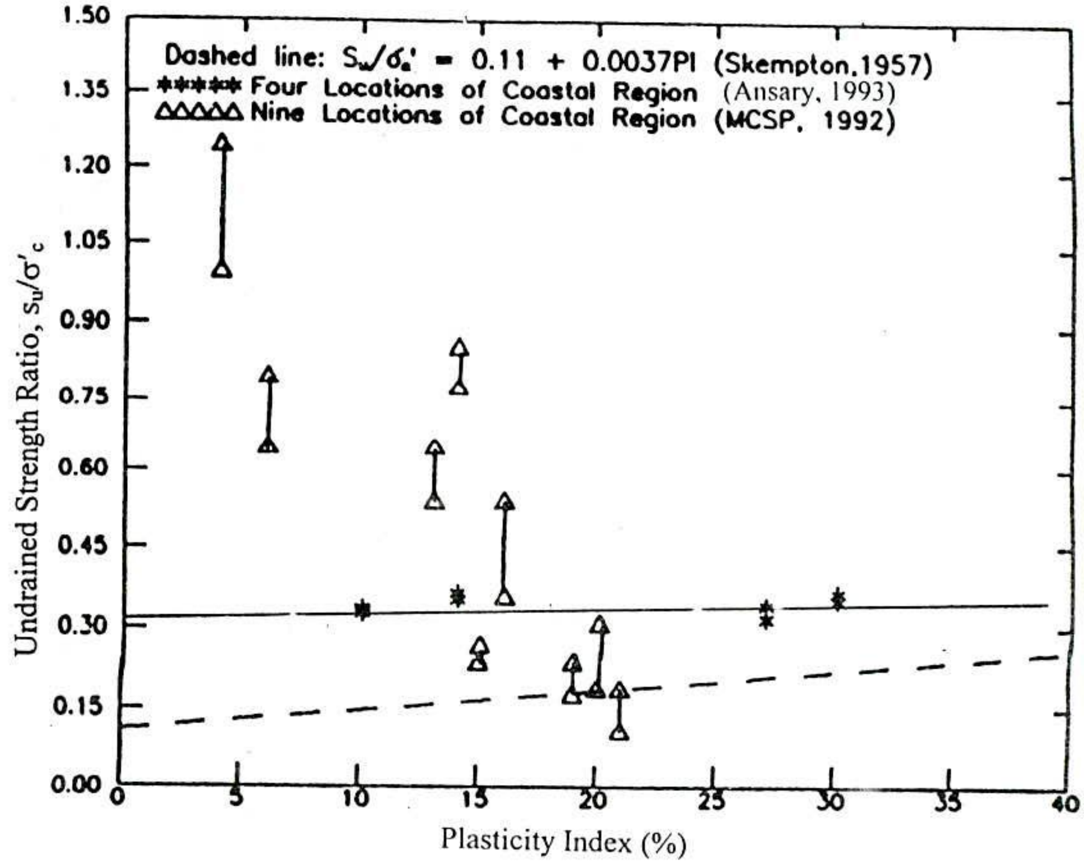
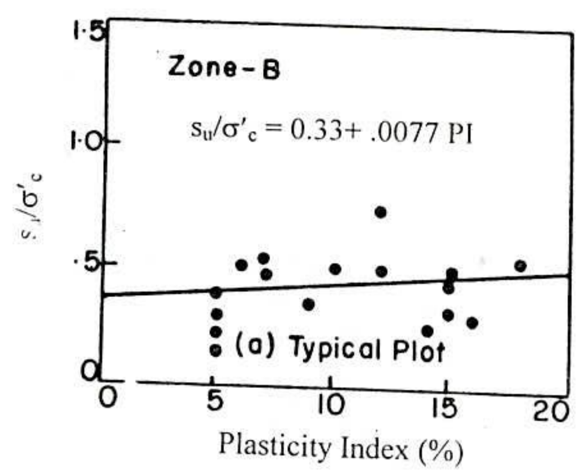


Fig. 2.7 Undrained Strength Ratio Versus Plasticity Index for Recompressed Normally Loaded Undisturbed Coastal Sols (after Ansary, 1993)



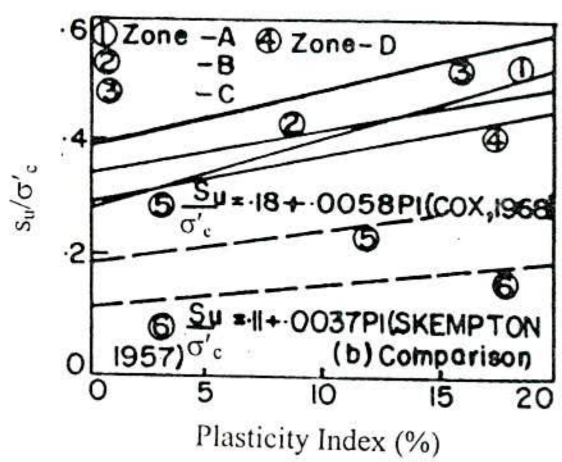


Fig. 2.8 s_u/σ'_c Versus Plasticity Index (after Amin et al., 1987)

appeared that Rann of Kutch clay has more recoverable strain energy than Kanpur clay and this would be expected from the differences in plasticity indices of the two clays. From the swelling behaviour of the two clays, it might be noted that, while the swelling index of Rann of Kutch clay is 56% of the compression index, it was only 21% of the compression index in case of Kanpur clay. This relative difference in swelling behaviour suggests that Rann of Kutch clay is "weakly bonded" while, Kanpur clay is "strongly bonded" (Bjerrum, 1967). The compression indices are 0.61 and 0.34 and the swelling indices are 0.28 and 0.06 for Rann of Kutch clay and Kanpur clay, respectively.

2.3 Use of Intrinsic Compressibility of Soils

Burland (1990) introduced a new normalizing parameter termed as the void index to aid in correlating the compression characteristics of various clays. 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 liquid limit (LL). The term "intrinsic" has been used to describe the properties of clays which have been reconstituted at a water content of between LL and 1.5 LL (preferably 1.25 LL) and then consolidated 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 its natural state. 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. The compressibility characteristics of reconstituted clays were used as a basic frame of reference for interpreting the corresponding characteristics of natural sedimentary clays (Burland, 1990).

Fig. 2.9 shows one-dimensional compression curves for some reconstituted natural clays covering a wide range of plasticity (Burland, 1990). One dimensional compression curve for reconstituted natural clay is normalized by assigning fixed values to e^*_{100} and e^*_{1000}. An asterisk is used to denote an intrinsic property. The

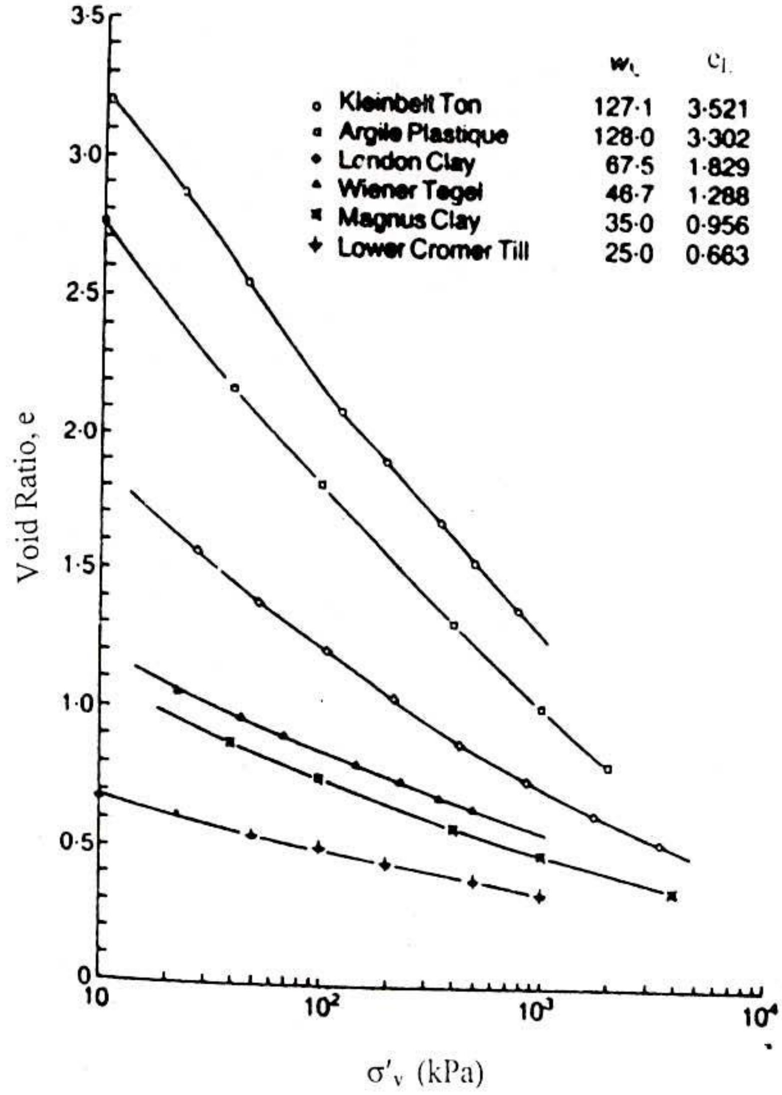


Fig. 2.9 One Dimensional Curves for Various Reconstituted Clays (after Burland, 1990)

parameters e^*_{100} and e^*_{1000} are the intrinsic void ratios corresponding to consolidation pressure, σ'_v = 100 kPa and σ'_v = 1000 kPa, respectively. The normalizing parameter chosen has been defined as the void index, I_V such that

$$I_v = \frac{e - e^*_{100}}{e^*_{100} - e^*_{1000}} = \frac{e - e^*_{100}}{C_c^*} \tag{2.1}$$

where, C_c^* (= e^*_{100} - e^*_{1000}) is called the intrinsic compression index. Fig. 2.10(a) shows the intrinsic compression curve for a given clay. Using normalizing parameter I_V, the compression curve in Fig. 2.10(a) may be transformed to the normalized curve in Fig. 2.10(b). When e = e^*_{100}, I_V = 0 and when e = e^*_{1000}, 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. Following Terzaghi (1925) the parameters e^*_{100} and C_c^* are called constants of intrinsic compressibility.

The intrinsic compression curves shown in Fig. 2.9 covering a wide range of liquid limits and of pressures have been replotted in Fig. 2.11 in terms of void index I_V versus log σ'_v. It has been shown that a reasonably unique line is achieved which is termed as the Intrinsic Compression Line (ICL). The available experimental evidence suggests that the ICL is insensitive to the test conditions and also to load increment ratio in excess of unity. The co-ordinates of the ICL are given in Fig. 2.11 and may be represented with sufficient accuracy by the following cubic equation

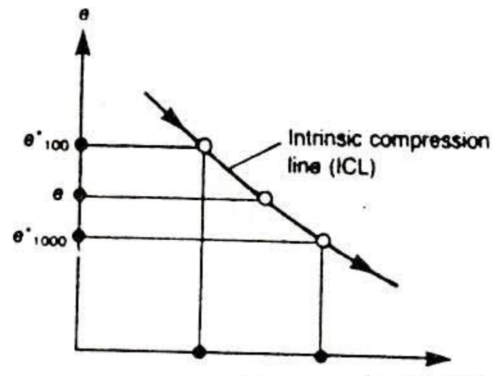
$$I_V = 2.45 - 1.285x + 0.015x^3 \tag{2.2}$$

where, x = log σ'_v in kPa and σ'_v = effective vertical stress.

The intrinsic compression line may either be measured directly for a clay or, if the values of e^*_{100} and C_c^* are known for the clay, the ICL may be constructed using the above equation (2.2). In the latter case, if it is required to plot the ICL in terms of e versus log σ'_v, then the values of e corresponding to various values of log σ'_v may be obtained from Fig. 2.11 or from equation (2.1) using the following expression:

$$e = I_V C_c^* + e^*_{100} \tag{2.3}$$

where, again the values of I_V may be obtained from equation (2.2).



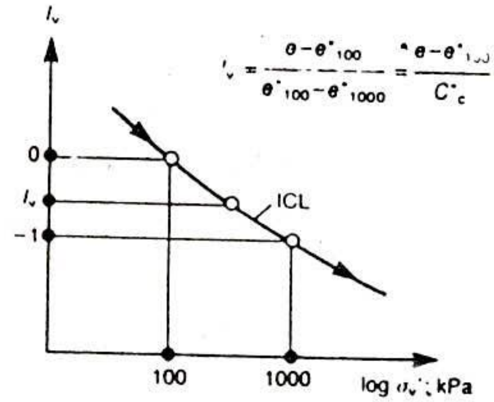


Fig. 2.10 The Use of Void Index, I_V to Normalize Intrinsic Compression Curves (after Burland, 1990)

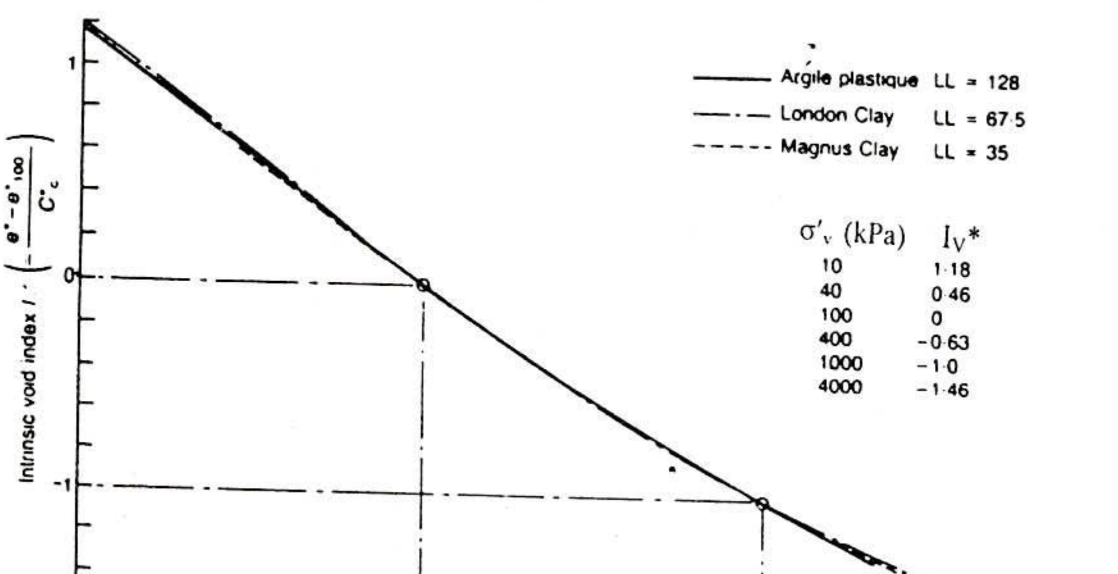


Fig. 2.11 Normalized Intrinsic Compression Curves Giving Intrinsic Compression line (ICL) (after Burland, 1990)

For a normally consolidated natural clay with a void ratio of e under an effective overburden pressure of σ'_v, the void index I_V of the clay element is given by the following equation:

$$I_V = (e - e^*_{100})/C_c^* \tag{2.4}$$

The value of e^*_{100} and C_c^* are preferably measured by means of an oedometer test on the reconstituted soils. Burland (1990) also suggested the following empirical equations for prediction the values of e^*_{100} and C_c^*.

$$e^*_{100} = 0.109 + 0.679\, e_L - 0.089\, e_L^{\,2} + 0.016\, e_L^{\,3} \tag{2.5}$$

and $$C_c^* = 0.256\, e_L - 0.04 \tag{2.6}$$

where e_L is the void ratio at liquid limit.

Using Burland's concept (1990), Ansary et al. (1999) investigated the compressibility characteristics of four coastal soils (PI = 16 to 26) of Bangladesh. The intrinsic compression lines for four coastal soils are shown in Fig. 2.12. It can be seen from Fig. 2.12 that, the intrinsic compression lines for the samples of the coastal soils investigated compared favourably with that proposed by Burland (1990). Intrinsic compression line for Dhaka clay (Kamaluddin, 1999) also agreed with that proposed by Burland (1990). Table 2.2 shows a summary of intrinsic constants of compressibility for some reconstituted natural clays, including the coastal soils of Bangladesh as reported by Ansary et al. (1999). Equations of intrinsic compression lines of Dhaka clay and four coastal soils are as follows:

$$I_V = (e - e^*_{100})/C_c^* = (e - 0.775)/0.25 \quad \text{for Dhaka clay} \tag{2.7a}$$
$$I_V = (e - e^*_{100})/C_c^* = (e - 1.02)/0.405 \quad \text{for Gohira1 (coastal soil)} \tag{2.7b}$$
$$I_V = (e - e^*_{100})/C_c^* = (e - 0.73)/0.242 \quad \text{for Gohira3 (coastal soil)} \tag{2.7c}$$
$$I_V = (e - e^*_{100})/C_c^* = (e - 0.80)/0.295 \quad \text{for Kalapara (coastal soil)} \tag{2.7d}$$
$$I_V = (e - e^*_{100})/C_c^* = (e - 0.73)/0.272 \quad \text{for Mognama (coastal soil)} \tag{2.7e}$$

Intrinsic swelling characteristics of overconsolidated clay can be represented by intrinsic swelling line (ISL) as shown in Fig. 2.13. It is to be noted that the intrinsic

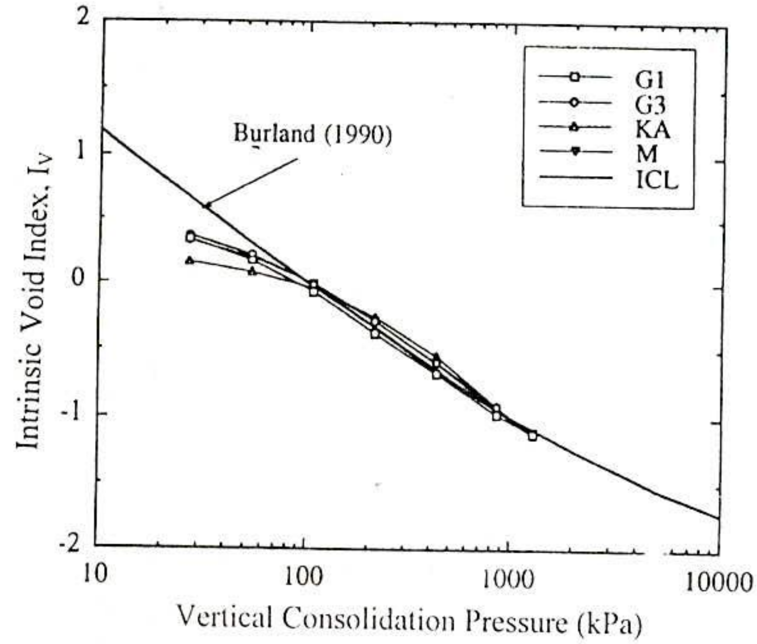


Fig. 2.12 Intrinsic Compression Lines for Samples of Four Coastal Soils (after Ansary et al., 1999)

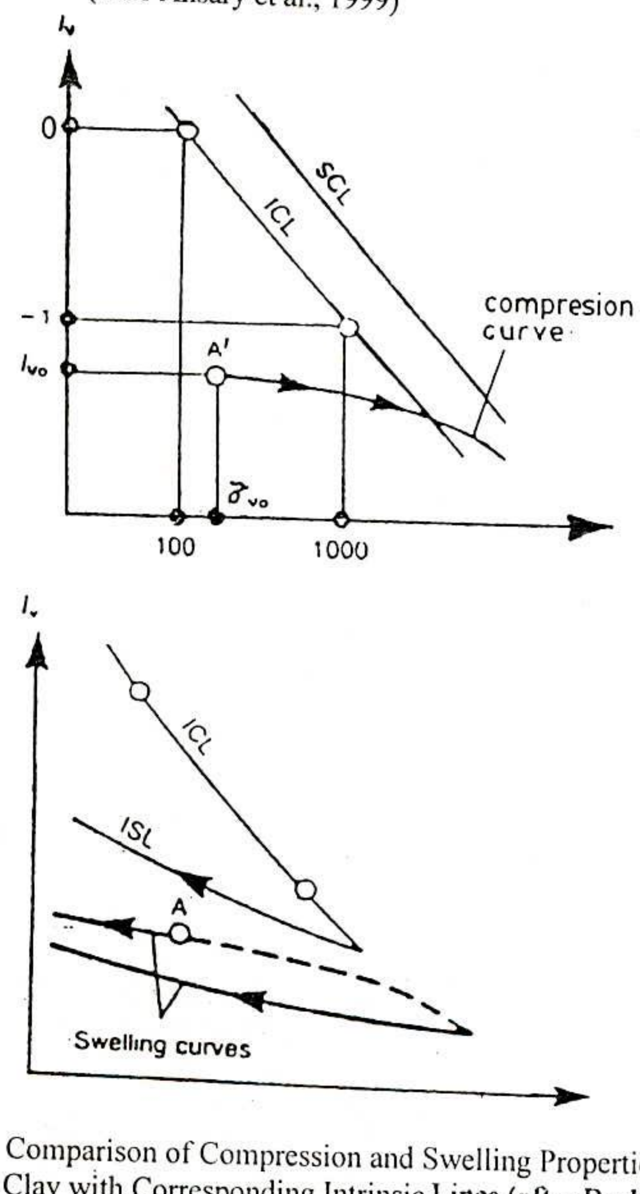


Fig. 2.13 Comparison of Compression and Swelling Properties of Overconsolidated Clay with Corresponding Intrinsic Lines (after Burland, 1990)

swelling index C_S^* is defined as the slope of the ISL at an overconsolidation ratio of 10. For overconsolidated clays, ICL provides a useful means of assessing the degree of overconsolidation of a natural clay particularly when the yield pressure σ'_v is not well defined. Kamaluddin (1999) also developed the intrinsic swelling line (ISL) for reconstituted Dhaka clay. The values of C_S^*/C_C^* varied from 0.14 to 0.10 and the void ratio corresponding to liquid limit (e_L) for Dhaka clay has been found to be 1.17.

Table 2.2 Intrinsic Constants of Compressibility for Reconstituted Natural Clays

Soil	LL	PI	e_L	e^*_{100}	C_C^*	Reference
Lower Cromer Till	25	12	0.663	0.503	0.154	Gens (1982)
Silty clay	28	20	0.762	0.603	0.136	Ramiah (1959)
Weald clay	39	20	1.065	0.77	0.24	Skempton (1944)
Boston Blue clay	39	16	1.084	0.80	0.21	
Oxford clay	53	26	1.362	0.96	0.30	Skempton (1944)
London clay	62	38	1.707	1.20	0.446	Jardine (1985)
Belfast Estuarine clay	67	37	1.782	1.0	0.32	
Ganges delta clay	69	41	1.911	1.22	0.42	Skempton (1944)
London clay	77	49	2.087	1.28	0.49	
Argile plastique	128	97	3.302	1.82	0.81	
Dhaka clay	43	23	1.17	0.775	0.25	Kamaluddin (1999)
Kleinbelt Ton	127	91	3.518	2.18	0.91	Hvorslev (1937)
Whangamarino clay	136	61	3.74	2.44	0.797	Newland and Allely (1956)
Gohira (3.5m)	36	16	1.11	0.73	0.242	
Gohira (1.5m)	48	24	1.65	1.02	0.405	Ansary et al. (1999)
Kalapara (GL)	45	18	1.19	0.80	0.295	
Mognama (1m)	49	26	1.80	0.73	0.272	

2.4 Critical State Models for Prediction of Soil Behaviour

For clays generally the critical state is the condition in which the clay continues to deform at constant volume under constant effective stress. The critical state concept represents idealized behaviour of remoulded clays, but it is assumed to apply also to undisturbed clays in triaxial compression test. The critical state parameters and two Critical state models, namely, Cam clay model and modified Cam clay model are explained in the following sections.

The critical state model in its original form is best described by reference to a three dimensional space (Fig. 2.14) whose three axes define the magnitudes of the variables p′, q′ and v, where

$$p' = \sigma'_{oct} = \frac{\sigma'_1 + \sigma'_2 + \sigma'_3}{3} \tag{2.8a}$$

$$q' = \frac{3}{\sqrt{2}}\tau_{oct},$$

$$\tau_{oct} = \frac{1}{3}\sqrt{(\sigma'_1 - \sigma'_2)^2 + (\sigma'_2 - \sigma'_3)^2 + (\sigma'_1 - \sigma'_3)^2} \tag{2.8b}$$

$$v = 1 + e \tag{2.8c}$$

The specific volume (v) is the total volume of soil containing unit volume of solid particles. In case of triaxial compression test in hydrostatic state, $\sigma'_2 = \sigma'_3$, then the stress parameters are defined as

$$p' = (\sigma'_1 + 2\sigma'_3)/3 \tag{2.9a}$$

$$q' = \sigma'_1 - \sigma'_3 \tag{2.9b}$$

in which σ'_1, σ'_2 and σ'_3 are the principal effective compressive stresses, and p′ and q′ are referred to as the mean normal stress and deviator stress respectively. The stress ratio, q′/p′, is denoted by η. The stress parameter, p_e, is called the mean equivalent pressure and is defined as

$$p_e = p'_o \exp[(e_0 - e)/\lambda \tag{2.9c}$$

in which p'_o and e_o correspond to the preshear consolidation pressure and void ratio on the isotropic consolidation line; and λ is the slope of the isotropic consolidation line in the e, ln p′ plot. During undrained tests, there is no change in voids ratio and, therefore, p_e remains constant during shear at a value of p'_o. However, during a drained test in which the voids ratio, e, decreases, the mean equivalent pressure, p_e, increases from its initial value of p'_o.

In the case of one-dimensional consolidation, there is a nearly linear relationship between specific volume (v) and the logarithm of mean effective stress. This relationship may be expressed as

$$v = N - \lambda \log_e p' \tag{2.9d}$$

where N (capital nu) is defined as the specific volume corresponding to p′ = 1.0. Then

$$p' = \exp[(N - v)/\lambda] \tag{2.9e}$$

Since the curves for one dimensional and spherical consolidation are nearly parallel, $C_c = 2.303\lambda$.

For overconsolidated soil, relationship between specific volume (v) and the logarithm of mean effective stress can be expressed as follows:

$$v = v_k - \kappa \log_e p' \tag{2.9f}$$

where v_k is defined as the value of v of an overconsolidated soil corresponding to p′ = 1.0 on the line BD produced (swelling line). Since, however, the slopes of the lines for one dimensional and spherical consolidation are not exactly the same, C_s is only approximately equal to 2.303k.

The equivalent consolidation pressure, p'_e, as the value of p′ on the normal consolidation line corresponding to any value of e. Then for any value of v,

$$v = N - \lambda \log_e p'_e \tag{2.9g}$$

so that

$$p'_e = \exp[(N - v)/\lambda] \tag{2.9h}$$

For overconsolidated soils, where $p' < p'_e$, the overconsolidation ratio (for spherical consolidation) will be defined as

$$OCR = p'_e / p' \tag{2.9i}$$

2.4.1 State Boundary Surface and Critical State Line

The State Boundary Surface (SBS) is defined as a unique surface as shown in Fig 2.14 which separates the states of an element of the soil from those that are not admissible. This surface is formed by two distinct surfaces, namely the Roscoe surface, on which volumetric yielding takes place, and the Hvorslev failure surface. The Roscoe surface is defined by undrained stress paths of normally consolidated clay while the Hvorslev surface is the locus of failure points for heavily overconsolidated samples. The existence of the State Boundary Surface has been verified by many investigators (Parry, 1960; Wroth and Loudon, 1967; Balasubramaniam, 1969).

The end points of all specimens, when they are sheared to failure, lie on a unique line defined as the Critical State Line (CSL). Its projection on the (q′, p′) plane is a straight line which passes through the origin having a constant slope M (capital mu) and can be described by the following equation:

$$q' = Mp' \tag{2.10a}$$

The projection of the CSL onto the v : p′ plane in Fig. 2.15 is curved. However, if the same data are replotted with axes v : ln p′ or q′: p′. the points fall close to a straight line. It is highly convenient that the gradient of this line turns out to be the same as the gradient of the corresponding normal consolidation line. The critical state line in v : ln p′ space can be described by the following equation:

$$v = \Gamma - \lambda \ln p' \tag{2.10b}$$

Γ (capital gamma) is defined as the value of v corresponding to p′ = 1.0 kN/m^2 on the critical state line; thus Γ locates the critical state line in v : ln p′ plane in the same way that N located the normal compression line. Eqns. (2.10a) and (2.10b) together define the position of the CSL in q′ : p′ : v space; M and Γ like N, λ and κ are regarded as soil constants.

The Critical State Line separates the Roscoe surface which dictates the volumetric yielding from the Hvorslev failure surface. When a state of sample reaches the Critical State, it experiences unlimited distortion while the effective stress and the volume of the soil remains unchanged. The area between the Critical State Line and the normal

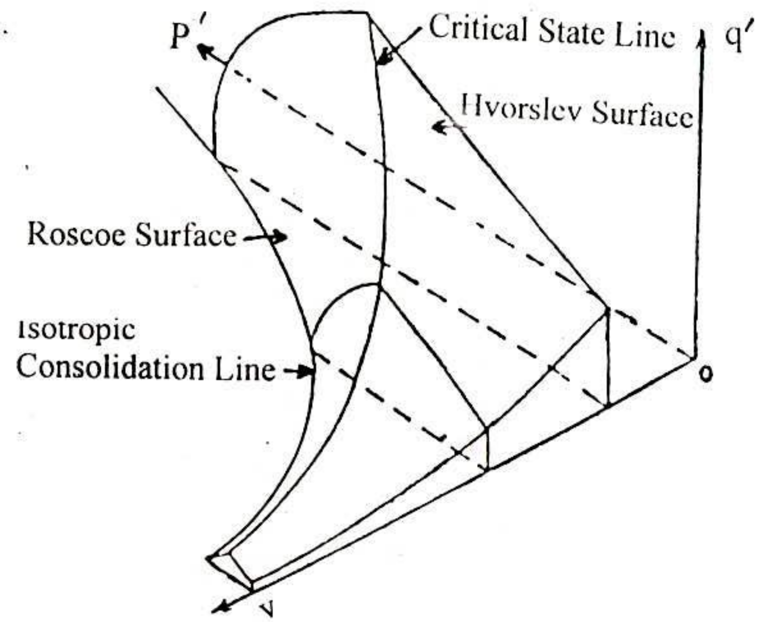


Fig. 2.14 Three Dimensional Presentation of the State Boundary Surface and the Critical State Line

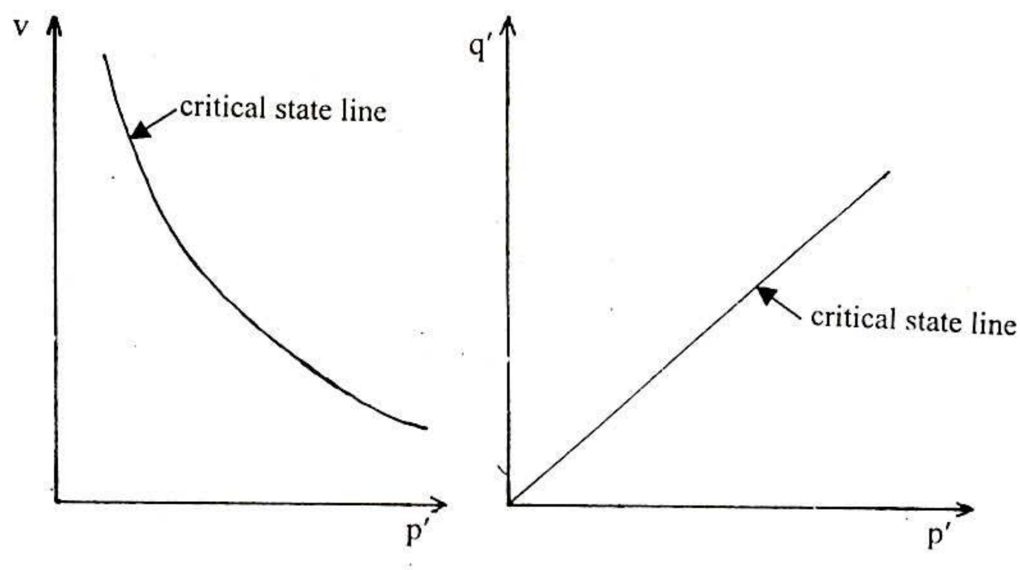


Fig. 2.15 Critical State Line in v-p′ Plane and q′-p′ Plane

consolidation line in the water content-log mean normal stress plot is called 'wet of critical' while the area to the left of the Critical State Line is called 'dry of critical'. The soil when sheared in the 'wet' zone would generate a positive pore pressure response under undrained condition or decrease in volume under drained conditions. On the other hand, a soil in the 'dry' zone would show an increase in volume under drained conditions or tend to develop a negative pore pressure. In its simplified form, the SBS is accepted to be symmetrical about the hydrostatic p-axis provided there is no substantial time effects and anisotropy either from the depositional mode or from the applied stress conditions. Much of the challenge and arguments on SBS and the Critical State Concept seem to be on these aspects, but nevertheless their effects can be incorporated in a primary SBS with appropriate deviations as per the perturbations.

The State Boundary Surface for triaxial compression (isotropic) consists of the following:

(a) Critical State Line (CSL) : $q' = Mp', \quad v = \Gamma - \lambda \ln p'$ (2.11a)

(b) Normal Consolidation Line (NCL) : $v = N - \lambda \ln p'$ (2.11b)

(c) Elastic Walls : $v = v_k - \kappa \ln p'$ (2.11c)

(d) Roscoe Surface : $\dfrac{q'}{Mp'} + \left(\dfrac{\lambda}{\lambda - \kappa}\right) \ln p' - \left(\dfrac{\Gamma - v}{\lambda - \kappa}\right) = 1$ (2.11d)

(e) Hvorslev Surface : $\dfrac{q'}{Hp'} - \left(\dfrac{M - H}{Hp'}\right) \exp\left(\dfrac{\Gamma - v}{\lambda}\right) = 1$ (2.11e)

(f) Tension Cutoff : $q' = 3p'$ (2.11f)

For one dimensional compression :

(a) Normal Consolidation Line (NCL) : $v = N_0 - \lambda \ln p'$ (2.12a)

(b) Elastic Walls (or Overconsolidated clay) : $v = v_{k0} - \kappa \ln p'$ (2.12b)

Typical values of the constants M, N, λ and κ for some clays are listed in Table 2.3.

Table 2.3 Typical Values of Soil Constants for a Wide Range of Clays

Properties	Klein Belt Ton	Wiener Tegel V	London Clay	Weald Clay	Kaolin	Bothkennar
λ	0.356	0.122	0.161	0.093	0.26	0.181
Γ	3.990	2.130	2.448	1.880	3.265	2.78
M	0.845	1.01	0.888	0.95	1.02	1.38
ϕ_c	21.75^0	25.75^0	22.5^0	24.25^0	26^0	---
Average κ	0.184	0.026	0.062	0.035	0.05	---
$\Lambda = (\lambda\text{-}\kappa)/\lambda$	0.483	0.788	0.614	0.628	0.807	---
LL (%)	127	47	78	43	74	67
PI (%)	91	25	52	25	32	38
G_s	2.77	2.76	2.75	2.75	2.61	---
Source of Data	Hvorslev (1949)		Parry (1956)		Loudon (1967)	Allman & Atkinson (1992)

2.4.2 The Critical State Pore Pressure Parameter

The critical state theory can be used to represent the undrained shear strength of overconsolidated soils. Atkinson and Bransby (1978) have 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.

$$[s_u/\sigma'_{vo}]^{oc} = [s_u/\sigma'_{vo}]^{nc} \times (OCR)^{\Lambda} \tag{2.13a}$$

where, Λ = Critical state pore pressure parameter

$$= 1-\left[c_s^{(iso)} / c_c^{(iso)}\right] = \left[1-\kappa^{(iso)} / \lambda^{(iso)}\right] \tag{2.13b}$$

$[s_u/\sigma'_{vo}]^{oc}$ = Ratio of undrained shear strength to overburden pressure of over-consolidated soil

$[s_u/\sigma'_{vo}]^{nc}$ = Ratio of undrained shear strength to overburden pressure of normally consolidated soil

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

$C_c^{(iso)} = 2.303\lambda^{(iso)}$, $C_s^{(iso)} = 2.303\kappa^{(iso)}$

The results of isotropic and anisotropic consolidated undrained shear tests (CIU, CK_0U) can be used to determine the critical state pore pressure parameter. The effects of OCR and initial stress state (K_0) 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_0 and level of shear of failure, thus requiring only two basic soil constants in order to predict undrained shear strength, namely, the effective stress friction angle (ϕ') and critical state pore pressure parameter (Λ).

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_0) has been redefined in terms of overconsolidated state.

$$\Lambda = [\log (s_u/\sigma'_{vo})^{oc} - \log (s_u/\sigma'_{vo})^{nc}] / \log OCR \qquad (2.14)$$

Ladd and Foott (1974) demonstrated that normalized strengths (s_u/σ'_{vo}) are a function of OCR. The "critical state failure" described by Schofield and Wroth (1978) is defined as the stress level at which continuous deformation occurs at no change in volume. The basic proposed by the theory would be valid if applied to undrained strength at maximum deviator stress.

Most soil encountered in nature are overconsolidated to some degrees. The OCR is not often known during testing unless supplementary consolidation testing is conducted or SHANSEP approach is applied. The parameter Λ 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 $(s_u/\sigma'_{vo})^{oc}$ and log OCR as shown in Fig. 2.16.

It is important to note that critical state pore pressure parameter (Λ) has been determined from a total stress approach and it is a soil constant. The parameter Λ can be used to predict undrained shear strengths for various stress histories and initial stress conditions. The application for 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 $(s_u/\sigma'_{vo})^{oc}$ and log OCR as the intercept at OCR=1 as shown in Fig. 2.16

2.4.3 Cam Clay and Modified Cam Clay Models

Stress-strain models to describe the soil behaviour are developed based on several assumptions and hypotheses in conjunction with well-known concepts of plasticity theory. These theories assume that the soil is isotropic, follow the Critical State Concept, and that there is no recoverable shear strain. The state of the sample inside the State Boundary Surface must remain on the elastic wall which is a vertical plane above an isotropic swelling line. The plastic deformation is assumed to occur only when the state of the sample changes on the State Boundary Surface. These theories appeal to only a few well-known soil parameters instead of depending on a large number of empirical constants.

The Cam Clay model was developed for normally consolidated and lightly over consolidated clay. The authors assumed that the energy dissipated at any infinitesimal increment of plastic work is only a function of the plastic shear strain. The proposed expression for energy dissipation with an assumption that the principal axes of stress and plastic strain increment coincide is,

$$dW = pd\varepsilon_{vp} + qd\varepsilon_{sp} \tag{2.15a}$$

where, dW : energy dissipated per unit volume of soil

p, q : mean effective principal stress, deviator stress

$d\varepsilon_{vp}$, $d\varepsilon_{sp}$: increments of plastic strains

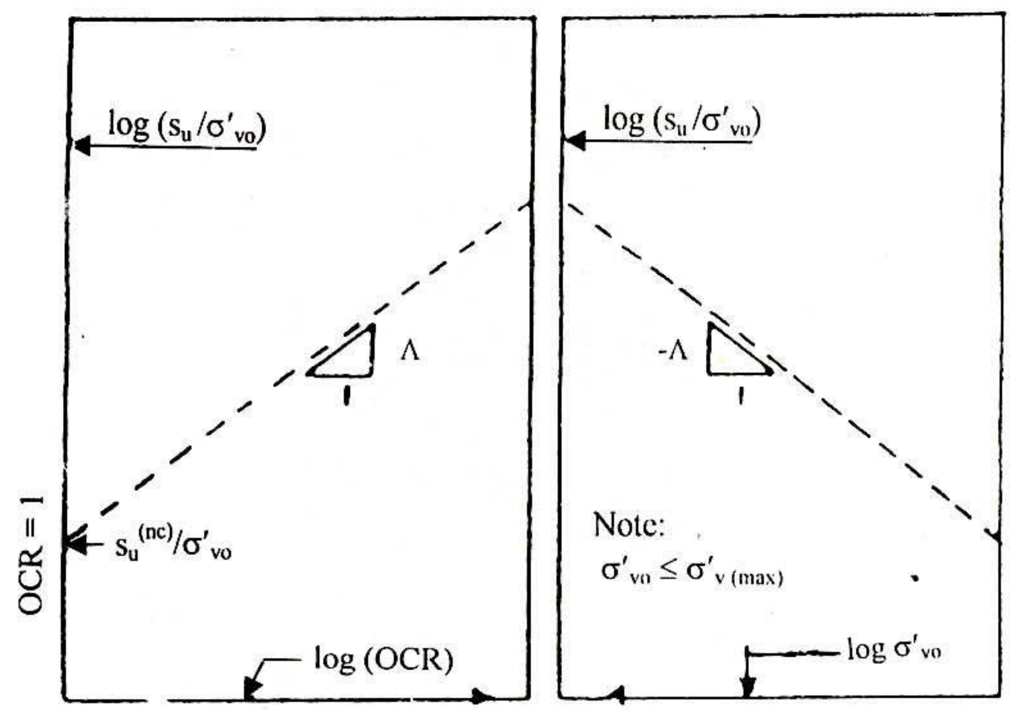


Fig. 2.16 Definitions of Critical State Pore Pressure Parameter from Undrained Strength Data

Eqn. (2.15a) can be expressed as,

$$dW = pd\varepsilon_{vp} + qd\varepsilon_{sp} = Mpd\varepsilon_{sp} \tag{2.15b}$$

where M is the slope of the Critical State Line in (q′, p′) plot. Eqn. (2.15b) leads to the following flow rule,

$$\left(\frac{d\varepsilon_{vp}}{d\varepsilon_{sp}}\right) = M - \eta \cdot \tag{2.15c}$$

where η is the stress ratio, q′/p′.

The normality rule is applied such that the equation of the yield locus is,

$$q' = Mp' \ln\left(\frac{p_o}{p'}\right) \tag{2.15d}$$

where p_0 is the preconsolidation stress. In this theory the shear and volumetric strain increments for states on the State Boundary Surface are given as,

$$d\varepsilon_s = \left(\frac{\lambda - \kappa}{v}\right)\left(\frac{1}{M - \eta}\right)\left[\frac{d\eta}{M} + \frac{dp'}{p'}\right] = d\varepsilon_s^{\,p} \quad \text{As}\left[d\varepsilon_s^{\,e} \longrightarrow 0\right] \tag{2.15e}$$

$$d\varepsilon_v = \frac{1}{v}\left[\frac{(\lambda - \kappa)d\eta}{M} + \lambda\frac{dp'}{p'}\right] \tag{2.15f}$$

The State Boundary Surface can be derived as,

$$\eta = \frac{\lambda M}{(\lambda - \kappa)} \cdot \ln\left(\frac{p_o}{p'}\right) \tag{2.15g}$$

The volumetric yield loci as described by Eqn. (2.15d) is bullet shaped at the p-axis and seems to be more applicable for volumetric yielding inside the SBS when the associated plastic volumetric strain is smaller than the value when the state paths lie on the SBS.

In an attempt to improve the limitations of the Cam Clay model, Burland (1965) proposed a modified work equation which considers the work dissipated in plastic volume change. The energy dissipated in the Modified Cam Clay Model is given by,

$$dW = p\left[\left(d\varepsilon_{vp}\right)^2 + \left(Md\varepsilon_{sp}\right)^2\right]^{1/2} \tag{2.16}$$

The flow rule and yield locus are given by the following equations (2.17) and (2.18), respectively:

$$\frac{d\varepsilon_{sp}}{d\varepsilon_{vp}} = \frac{M^2 - \eta^2}{2\eta} \tag{2.17}$$

$$p' = \frac{p_o M^2}{M^2 + \eta^2} \tag{2.18}$$

Hence, the shape of the volumetric yield locus was changed from the earlier log spiral to an elliptic form.

The incremental shear and volumetric strains are as follows:

$$d\varepsilon_s = \left(\frac{\lambda - \kappa}{v}\right)\left(\frac{2\eta}{M^2 - \eta^2}\right)\left[\frac{2\eta d\eta}{M^2 + \eta^2} + \frac{dp'}{p'}\right] \tag{2.19a}$$

$$d\varepsilon_v = \frac{1}{v}\left[\frac{2\eta(\lambda - \kappa)d\eta}{M^2 + \eta^2} + \lambda\frac{dp'}{p'}\right] \tag{2.19b}$$

The State Boundary Surface described in this theory is,

$$\frac{p_o}{p'} = \left(\frac{M^2 + \eta^2}{M^2}\right)^{\left(1 - \frac{\kappa}{\lambda}\right)} \tag{2.20}$$

2.5 Undrained Behaviour of Normally Consolidated and Overconsolidated Clays

2.5.1 Undrained Stress Paths

Wroth and Loudon (1967) conducted a series of triaxial tests on sedimented and then overconsolidated Kaolin under isotropic pre-shear conditions. They demonstrated that the undrained stress paths for the normally consolidated samples formed a part of the boundary to the possible states of stress that can be experienced by such samples. Also, the undrained stress paths within the State Boundary Surface (SBS) rose nearly vertical until they approached the State Boundary Surface and then reached the Critical State. It is noted that the State Boundary Surface is defined as a unique surface which separates the states of an element of the soil from those that are not admissible and the critical state is the condition in which the clay continues to deform at constant volume under constant effective stress. The Critical State Soil Mechanics, which deals with the behaviour of soil sheared from an initially isotropic stress condition, imply that the undrained stress paths of an overconsolidated clay will reach the same point at the Critical State Line as a normally consolidated sample with the same water content (Schofield and wroth, 1968). This idea was not fully supported by the experimental data on overconsolidated clays. Mitchell (1970) rather observed that the effective stress paths of cemented clay tested from isotropic stress conditions within a yield curve reached a portion of the failure envelope parallel to the p-axis and deviating from the Critical State Line.

Allman and Atkinson (1992) investigated the basic behaviour of one-dimensionally normally consolidated and lightly overconsolidated reconstituted Bothkennar soil (LL = 67%, PL = 29%, clay = 22%, w = 73%) in laboratory triaxial stress path tests. The main programme of tests established a State Boundary Surface containing Critical State Lines, and evaluated the basic material parameters. The one dimensional and isotropic compression lines were identified from the results of continuous loading tests and are given by $\lambda = 0.181$, $N_0 = 2.88$ and $N = 2.91$. During one-dimensional normal compression, the value of K_0 was 0.5. The gradient of compression lines ($\lambda = 0.181$) is approximately the same as that for Kaolin clay ($\lambda = 0.2$) and is larger than the typical value for London clay ($\lambda = 0.1$). The relationship between λ and PI is λ/PI

= 1/210, which is comparable with the value of 1/170 given by Schofield and Wroth (1968). Allman and Atkinson (1992) also showed that most soils reached reasonably well-defined critical states identified from the stress-strain curves. The Critical State Lines were given by $M_c = 1.38$, $\lambda = 0.181$ and $\Gamma = 2.78$.

2.5.2 Undrained Stress-Strain and Stiffness Characteristics

The remoulded clays and normally consolidated clays clearly demonstrate the nonlinearity of clay behaviour. On the contrary, it was reported that most natural clays were often stiffer and showed more linear behaviour in a certain range of loading. Parry and Nadarajah (1973) demonstrated that the stiffness (i.e., deviator stress increment Δq for a given strain ε_1) of remoulded clay specimens with the isotropic consolidation was nearly the same in compression as it was in extension during the early stage of undrained loading.

Crooks and Graham (1976) showed that the stress-strain behaviour of samples consolidated isotropically under pressures less than the maximum past pressure, even at a stress below the overburden pressure, has no sharp break in p'_{max} between the low compressibility and the high compressibility behaviour while those under anisotropic conditions exhibited a distinct change of rate when axial stresses exceeded p'_{max}. It was also found that the samples which has been consolidated isotropically to stresses either above or below p'_{max} showed much higher strains at maximum deviator stresses than those observed from the anisotropically consolidated samples.

Fig. 2.17 (a) and (b) show typical stress-strain curves for consolidated undrained tests (Arora, 1992) and Fig. 2.18 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.

The immediate or undrained settlement is referred to as elastic settlement. The undrained settlement occurs without sufficient dissipation of excess pore water

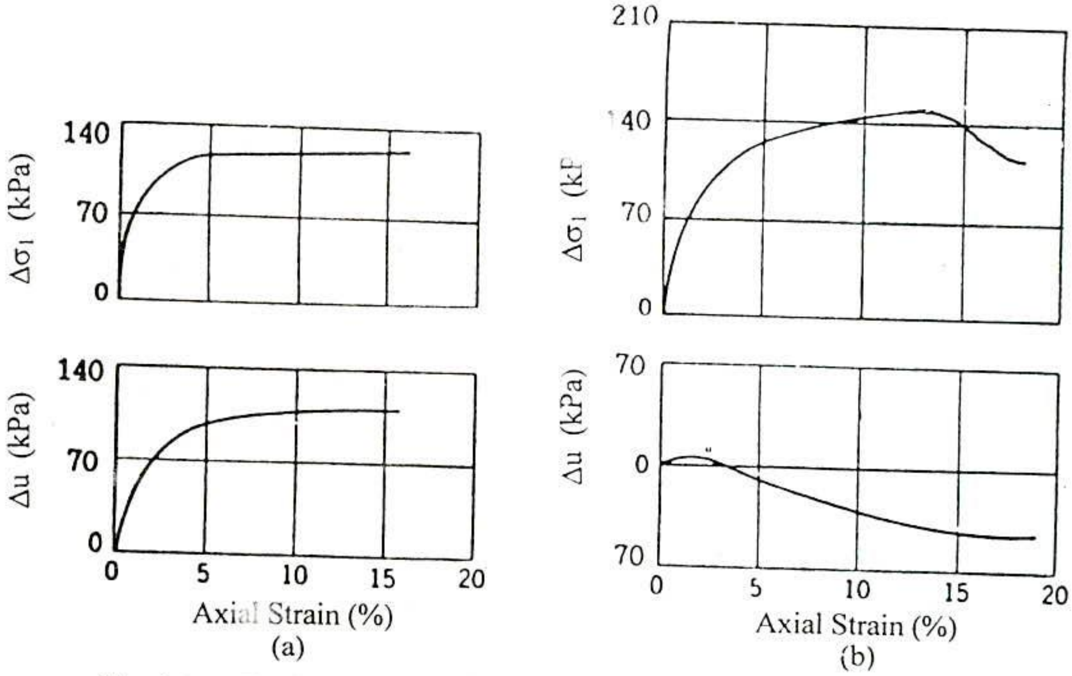
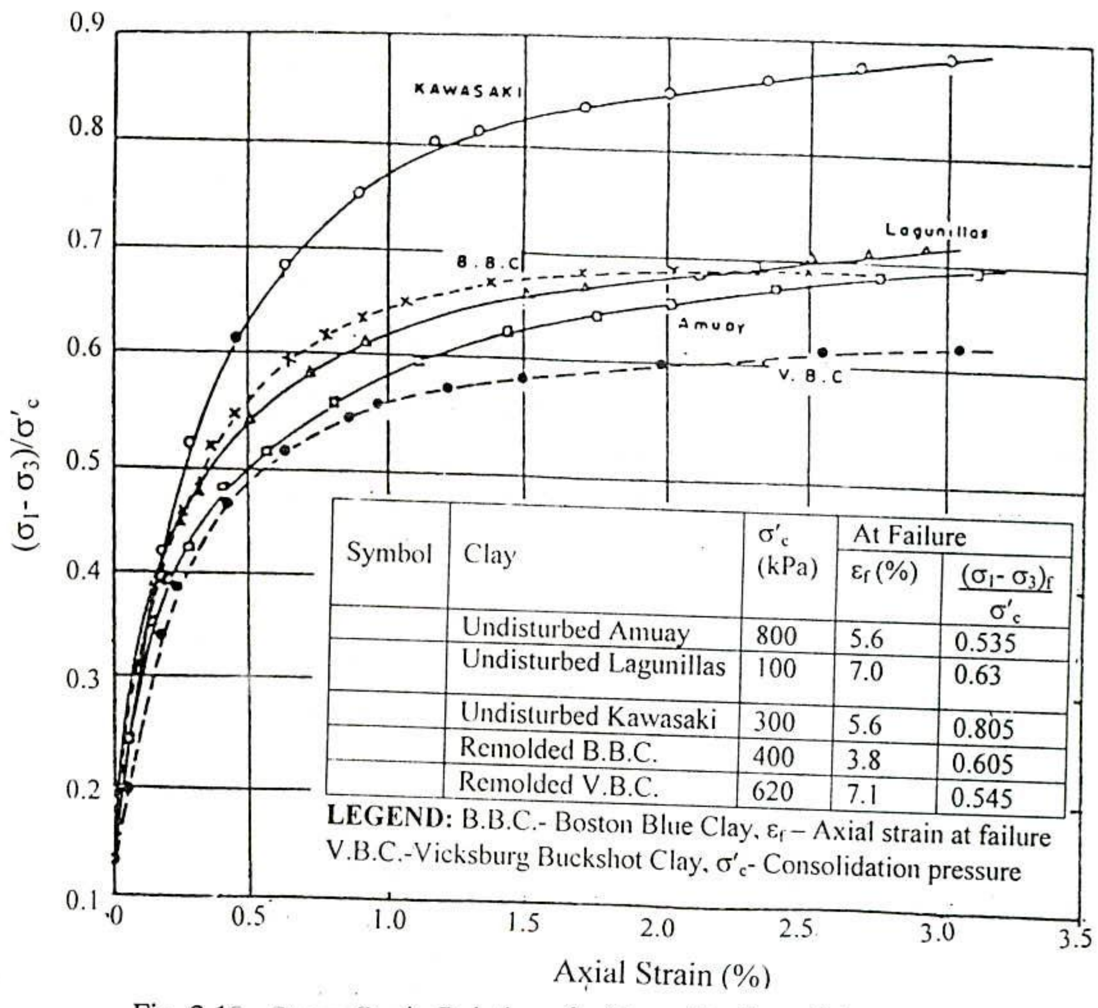


Fig. 2.17 Typical Stress-Strain from CU Tests on Weald Clay, (a) Normally Consolidated State, (b) Overconsolidated State (after Arora, 1992)



Symbol	Clay	σ'_c (kPa)	At Failure ε_f (%)	At Failure $\frac{(\sigma_1 - \sigma_3)_f}{\sigma'_c}$
	Undisturbed Amuay	800	5.6	0.535
	Undisturbed Lagunillas	100	7.0	0.63
	Undisturbed Kawasaki	300	5.6	0.805
	Remolded B.B.C.	400	3.8	0.605
	Remolded V.B.C.	620	7.1	0.545

LEGEND: B.B.C.- Boston Blue Clay, ε_f – Axial strain at failure
V.B.C.-Vicksburg Buckshot Clay, σ'_c- Consolidation pressure

Fig. 2.18 Stress-Strain Relations for Normally Consolidated Clays in Undrained Conditions (after Ladd, 1964) pressure due to sudden application of load. Undrained settlement is closely related to the undrained stability of foundation and is very important in foundation in 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_u for the clay layer involved for undrained loading condition.

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. A significant amount of work on young's modulus of clays has been reported by various investigators such as Ladd (1964), Desai (1971) and Yudhbir et al. (1975).

Due to difficulties in obtaining undisturbed soil samples, attempts have been made in the past to correlate undrained modulus (E_u) to undrained shear strength (s_u) to determine elastic modulus. Cooling and Skempton (1942), Skempton and Henkel (1957) have found that the undrained young's modulus for saturated clay soils can be obtained by the following relation:

$$E_u = k\ s_u \tag{2.21}$$

The value of k suggested by them was equal to 140. But Bjerrum (1964) suggested the value of k as 250 to 500.

D'Appolonia et al. (1971) made the following comments regarding the interrelationship between k, E_u and other soil properties:

(i) The value of k decreases with increase of overconsolidation of the clay. This is shown for three clays in Fig. 2.19.

(ii) The value of k generally decreases with the increase of the plasticity index of the clay.

(iii) The value of E_u determined from unconfined compression tests and unconsolidated undrained triaxial tests are generally low.

(iv) The value of k decreases with the organic content in the soil.

(v) For most cases, CIU or CK0U type of tests on undisturbed samples yield values of E_u that are more representative of field behaviour.

(vi) For highly plastic clays, CU tests yield E_u values generally indicative of field behaviour.

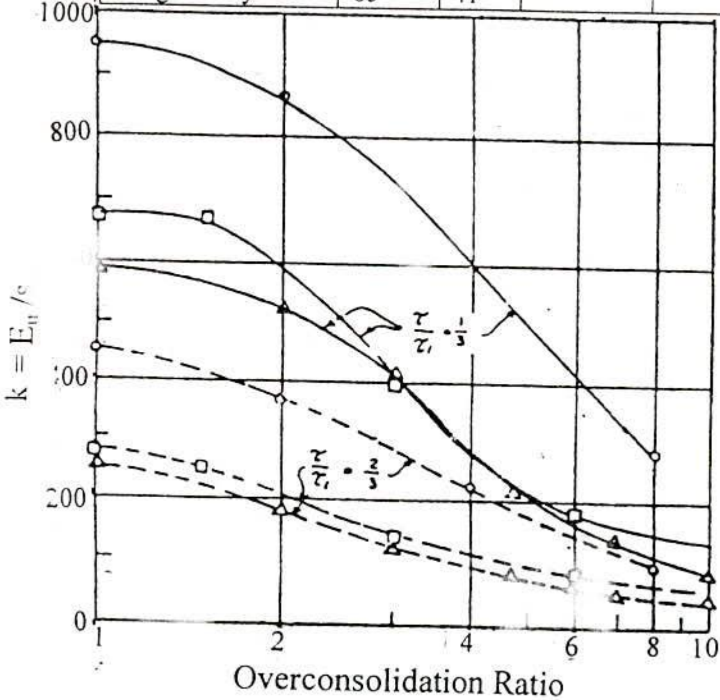
Factors Affecting the Undrained Stress-Strain Modulus of Clays

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 UU or CU, sample disturbance, factor of safety, load cycle and anisotropy. Some of the salient factors are discussed below briefly:

<u>Consolidation Pressure</u>: Yudhbir et al. (1975) found that initial tangent modulus (E_i) increaes with consolidation pressure. This has been shown in Fig. 2.20. The variation as a crude approximation, is linear. The same has also been observed by Janbu (1963), Ladd (1964) and Varadarajan (1973). In Fig. 2.21, Lambe and Whitman (1969) 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 contours are straight radial lines for a group of tests, then a plot of t'/σ'_c (where, $t' = (\sigma'_1-\sigma'_3)/2$ and σ'_c = consolidation pressure) versus strain would be unique. If the plot is unique, the modulus is then proportional to consolidation pressure.

<u>Stress Level</u>: Ladd (1964) found that at a particular consolidation condition, modulus decreases with the increase of stress level. Fig. 2.22 (after Ladd and Varallyay, 1965) presents a plot of normalized secant modulus, E_u/σ'_{vc} (ratio of secant modulus and vertical consolidation pressure) versus applied shear stress ratio, $\Delta q/\Delta_{qf}$ 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 CK_0U test.

Clay	LL (%)	PI (%)	Applied Stress Ratio	
			$\tau/\tau_f = 1/3$	$\tau/\tau_f = 2/3$
Boston Blue clay	41	32		
Maine Organic clay	65±10	33±2		
Bangkok clay	65	41		



Data from $\overline{CK_0U}$

$E_u = 3\tau/\gamma$

$E_u = 3G$

$s_u = \tau_f$

Type: Direct Simple Shear Tests

τ = Applied Horizontal Shear Stress

τ_f = Maximum Value of τ

γ = Shear Strain

Fig. 2.19 Relationship Between E_u/s_u and OCR from CU Tests on Three Clays (after Appolonia, Poulos and Ladd, 1971)

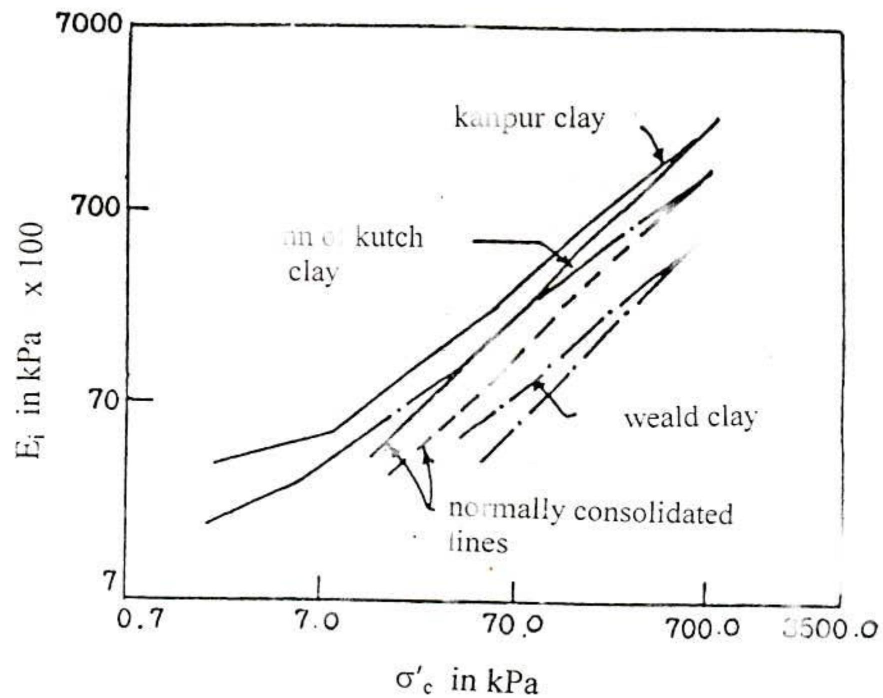


Fig. 2.20 Initial Tangent Modulus (E_i) Versus Consolidation Pressure (σ'_c) for three Clays (after Yudhbir et al., 1975)

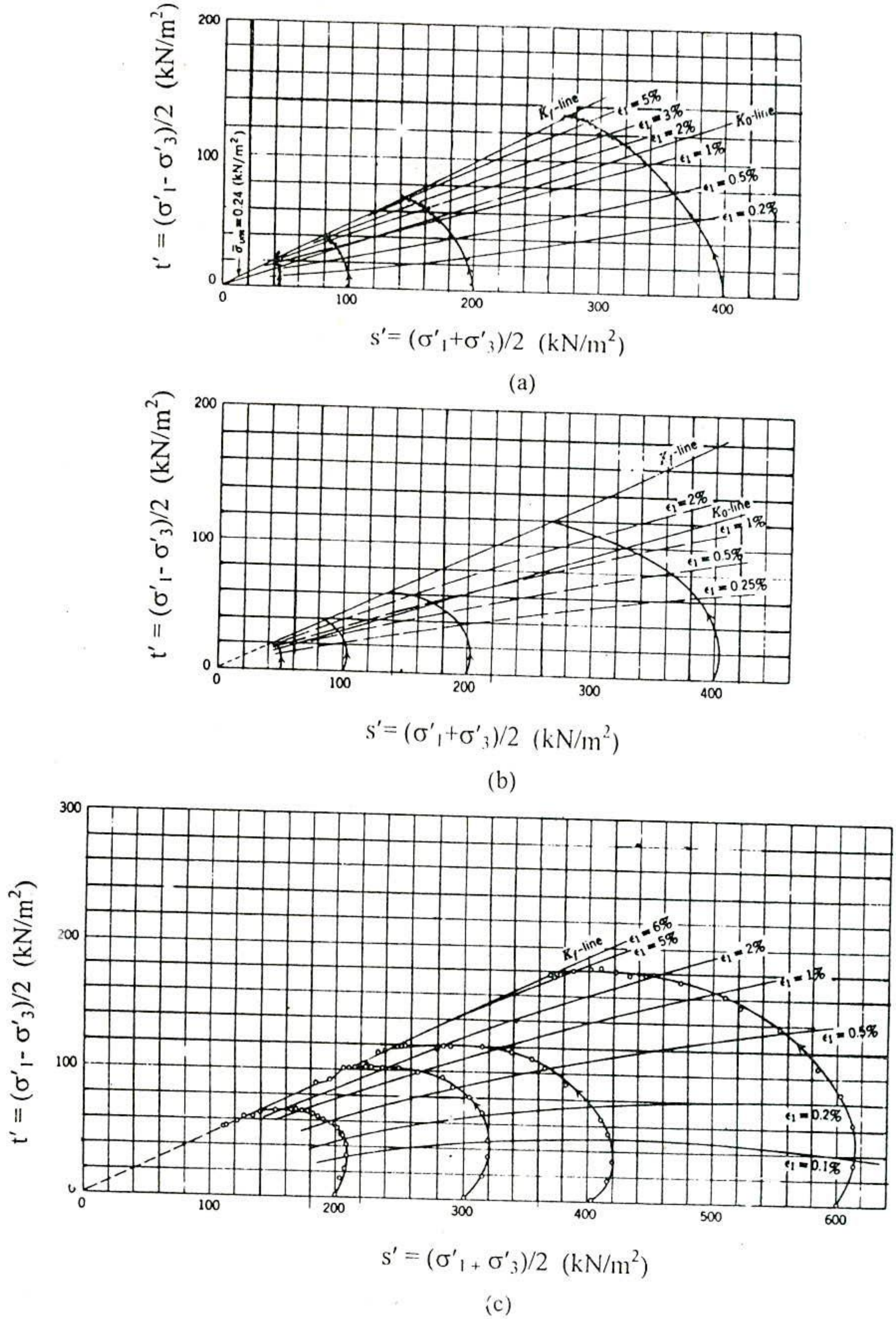


Fig. 2.21 Contours of Equal Strain for Three Normally Consolidated Clays (a) Amuay Clay, (b) Lagunilias Clay and (c) Remolded Boston Blue Clay (after Lambe, 1969)

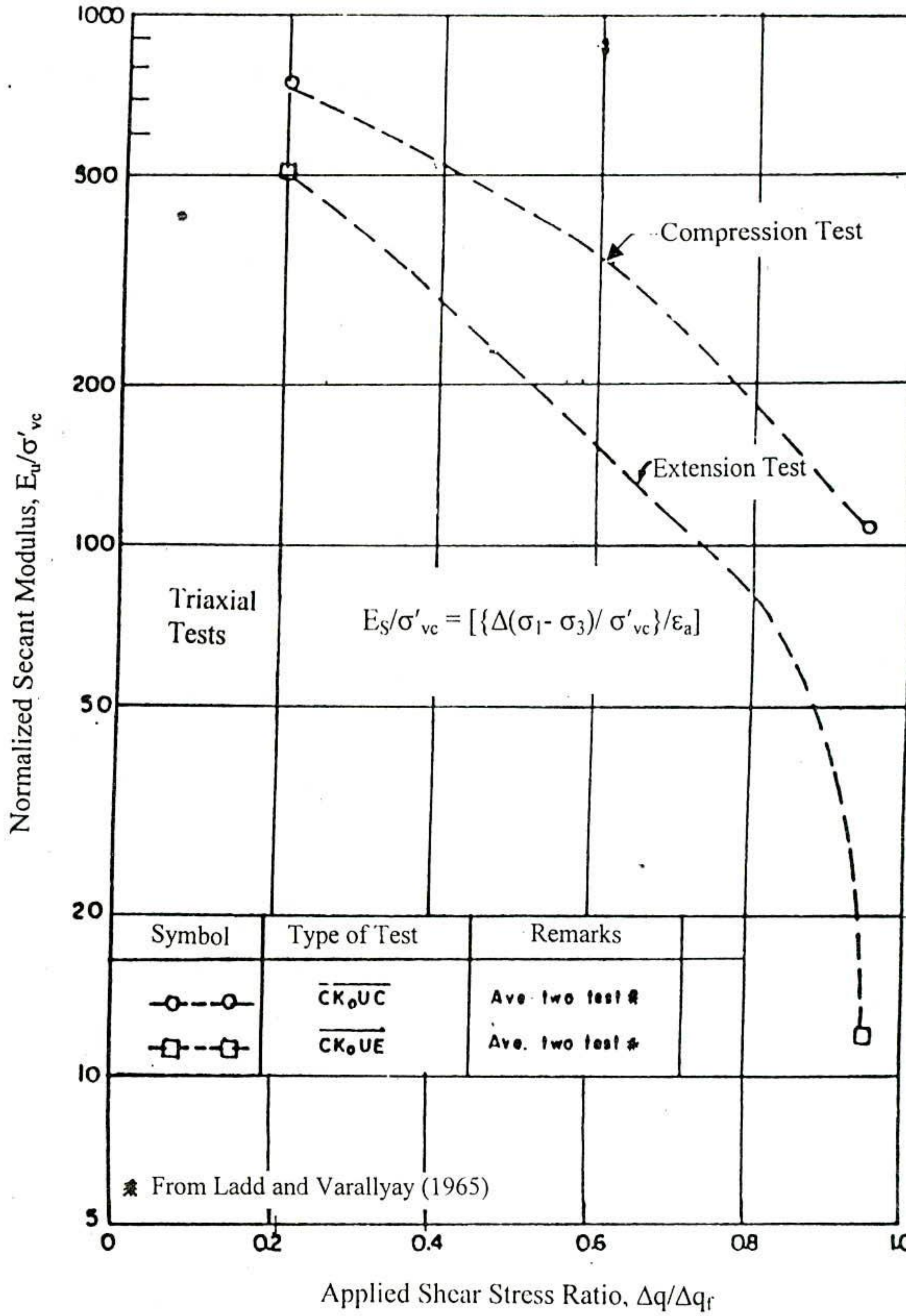


Fig. 2.22 Normalized Secant Modulus from CK_0U Triaxial Tests on Normally Consolidated Boston Blue Clay (after Ladd and Varallyay, 1965)

Overconsolidation: Yudhbir et al. (1975) showed the variation of the ratio of initial tangent modulus to reduced swelling pressure (E_i/σ'_c) with OCR for three clays in Fig. 2.23. The trend is pronounced at high values of OCR. Ladd (1964) and Varadarajan (1973) found similar trend.

Factor of Safety: Fig. 2.24 (Ladd, 1964) shows the ratio of secant modulus to consolidation pressure (σ'_c) for clays of Fig. 2.18 plotted against the factor of safety. The plot shows that E_u/σ'_c increases with the increase of factor of safety.

2.5.3 Undrained Shear Strength Characteristics

In most practical problems, undrained shear occurs wherever the load imposes at a much faster 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. In most practical problems related to clay soils, undrained shear strength is of interest to the geotechnical engineers.

The undrained strength of normally consolidated clays is known to increase with the consolidation pressure. However, the change in the undrained strength was observed to be small for lightly overconsolidated clay with an overconsolidation ratio less than 2.5 (Parry and Nadarajah, 1973). This was particularly true for the K_0-consolidated samples. This indicates that no significant increase in shear strength with increasing consolidation pressure can be expected in lightly overconsolidated samples until they reached the normally consolidated state. Similarly, Hight et al. (1987) reported that the undrained strength ratio was reduced with the increase in overconsolidation ratio, especially on lightly consolidated samples (OCR < 2.0), when those values were normalized with respect to the vertical preconsolidation pressure. On the contrary, the normalized undrained strength ratio with respect to vertical pre-shear consolidation pressure increased with increasing overconsolidation ratio (Yudhbir and Varadarajan, 1974; Ladd et al., 1977; Koutsoftas, 1981; Hight et al., 1987). The increase being pronounced at the higher value of the overconsolidation ratio. This tendency is due to a significant increase in the effective stress during shear of the overconsolidated samples.

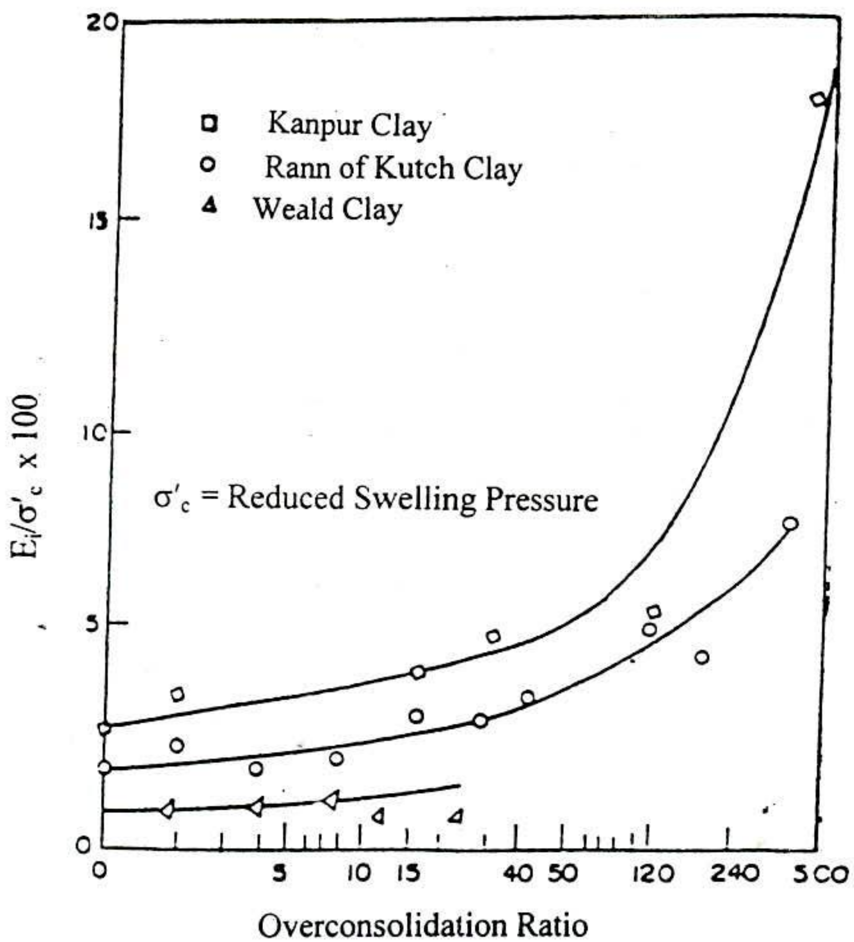


Fig. 2.23 Normalized Initial Tangent Modulus (E_i/σ'_c) as a Function of Overconsolidation Ratio (after Yudhbir et al., 1975)

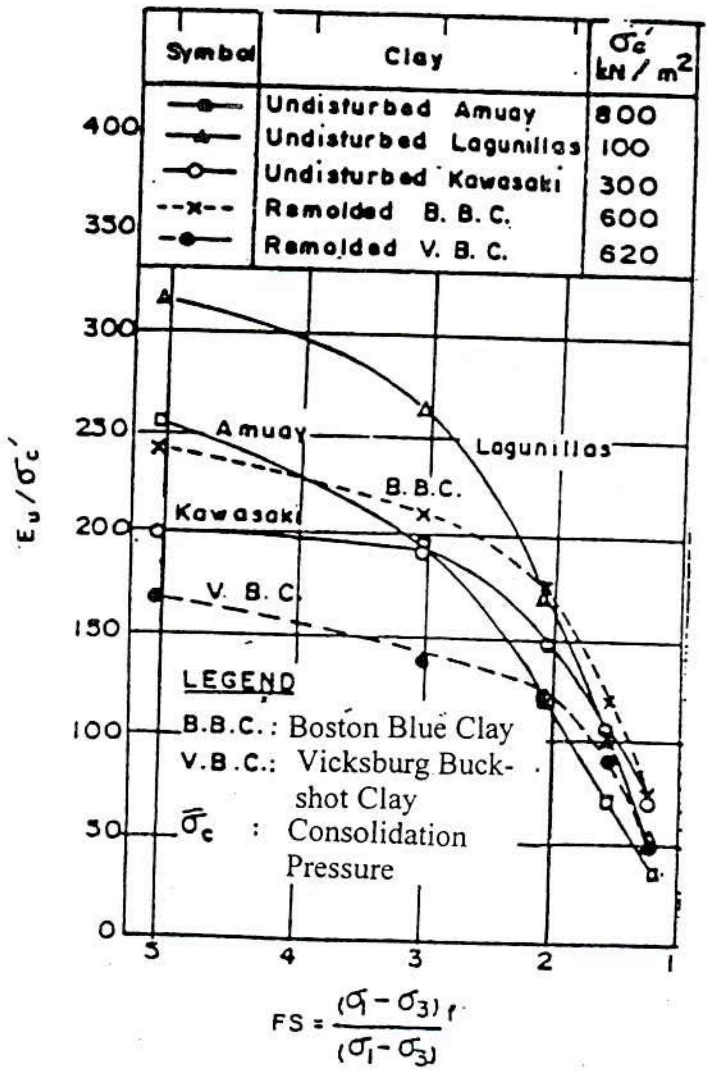


Fig. 2.24 Undrained Modulus for Five Normally Consolidated Clays (after Ladd, 1964)

Clays exhibit normalized behaviour between undrained shear strength, s_u, the in situ overburden pressure, σ'_{vo} and some index properties. Table 2.4 presents a number of relations obtained by various researchers those can be used to obtain undrained shear strength of clay soils. Equations (a) to (e) are for normally consolidated clays. The s_u/σ'_{vc} values are related to plasticity index in equations (a) and (b), to liquidity index I_L in equation (c) and liquid limit in equation (d). In equation (e), ϕ is the angle of internal friction that can be obtained from drained triaxial test and Λ_0 is the critical state pore pressure parameter (Schofield and Wroth, 1968). Equations (f) to (k) are applicable to overconsolidated soils.

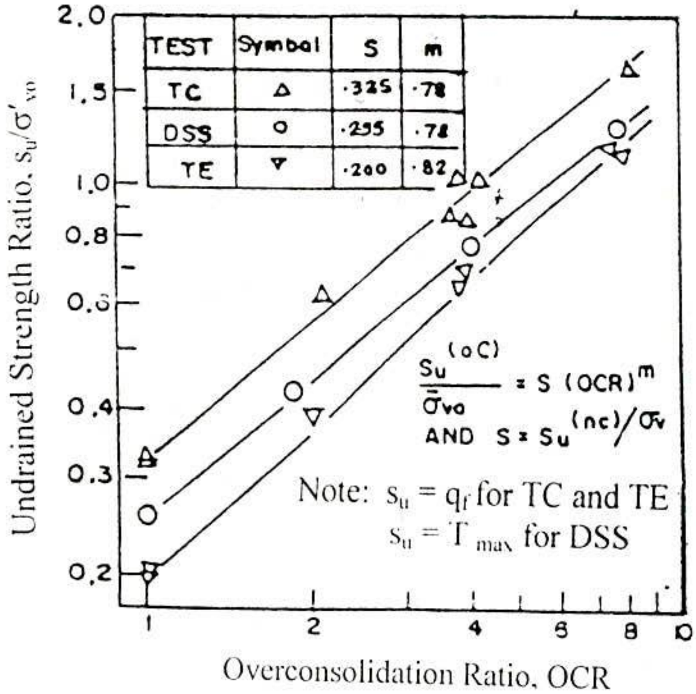
Ladd et al. (1977) gave an extensive discussion on soils to obtain normalized soil parameters (NSP) for design use. Ladd et al. (1977) summarized the effect of OCR on K_o and undrained stress-strain-strength parameters for a variety of clays. Most of those data came from SHANSEP (stress History and Normalized Soil Engineering Properties) test programmes, wherein undisturbed samples were K_0 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. 2.25 shows plots log OCR versus log s_u/σ'_{vo} [where σ'_{vo} = in-situ consolidation pressure] from CKoUTC, TE and DSS tests (Koutsoftas and Ladd, 1984; Lefebvre, 1983). The log-log plot gives essentially straight lines and hence is closely approximated by the relationship:

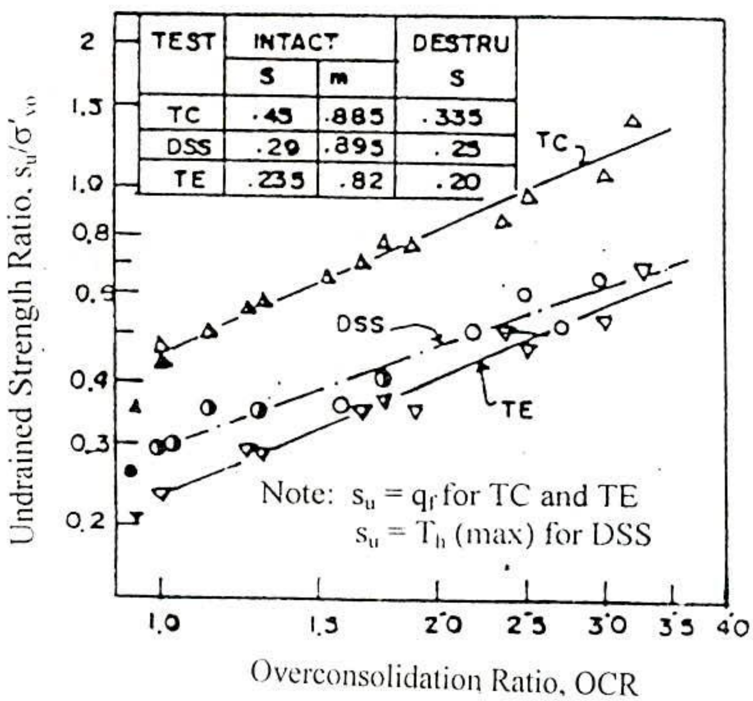
$$[s_u^{(oc)}/\sigma'_{vo}] \,/\, [s_u^{(nc)}/\sigma'_{vo}] = (OCR)^m \qquad (2.22)$$

where m is a dimensionless coefficients.

Ameen and Safiullah (1986) also showed linear relationship for Dhaka clay under isotropic and K_0 stress conditions between undrained shear strength ratio and overconsolidation ratio in log-log scale. Moreover, the slope of this line m is almost same for both isotropic and K_0 stress conditions and its value closely approximate to that $(1-C_s)/C_c$, where C_c and C_s are compression and swelling indices, respectively.



(a)



(b)

Fig. 2.25 Undrained Strength Ratio Versus OCR (a) From 'SHANSEP' CK_0U Tests on AGS CH Marine Clay (after Koutsoftas and Ladd, 1984) (b) From CK_0U Tests on Intact and Destructured Marine Clay (after Lefebvre, 1983)

Therefore m may be replaced by Λ. Ameen and Safiullah (1986) reported that the general equation proposed by Mitachi and Kitago (1976) was valid for Dhaka clay as a model to determine the undrained shear strength of overconsolidated Dhaka clay.

Fig. 2.26 presents the relative increase in undrained shear strength ratios with increasing overconsolidation ratio (Koutsoftas, 1981). In this figure, the normalized shear strength ratio at a particular OCR, $[s_u/\sigma'_{vo}]^{oc}$, is divided by the corresponding normalized shear strength ratio for the normally consolidated soil, $[s_u/\sigma'_{vo}]^{nc}$. The data of the figure are from a comprehensive series of undrained shear strength 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 OCR may be described as suggested by Ladd et al. (1977) by an equation of the form as shown in Eqn. 2.22.

For the data shown in Fig. 2.23, the value of m varies from 0.8 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. For Dhaka clay Ameen and Safiullah (1986) reported that in isotropically consolidated and K_0 consolidated conditions, the values of Λ are 0.746 and 0.83 respectively. Kamaluddin (1999) also showed the values of m are 0.734 and 0.821 for isotropic and K_0 stress conditions respectively for reconstituted Dhaka clay. For four reconstituted coastal soils, Ansary (1993) found that the variation of undrained shear strength ratio with OCR is linear and the values of Λ varied from 0.80 to 0.87. The model, valid for Dhaka clay and coastal soils, as reported by Kamaluddin (1999) and Ansary (1993) are shown in Table 2.4. These results suggest that in order to generalize the value of Λ parameter for coastal soils, further tests are required for other varieties of coastal soils with different plasticity indices and different liquid limits. At the same time further study is also required to obtain more controlled field data with actual field strength tests to verify the applicability of the proposed model for coastal soils.

Ladd and Foot (1974) and Ladd et al. (1977) have shown that the results of laboratory shear tests performed on clay samples with the same overconsolidation ratio, but different consolidation stresses, exhibit very similar stress-strain characteristics when normalized with respect to the vertical consolidation stress. These clays are said to exhibit normalized behaviour. Ladd et al. (1977) gave a means of estimating the undrained shear strength of preloaded soil as illustrated in Fig. 2.27 based on direct simple shear (DSS) tests. The original plot used five soils: three from the N.E. United States; one from Louisiana; and one from Bangkok, Thailand. The liquid limits (for all but the varved clay) ranged from 41 to 95% and with LI from 0.8 to 1.0. These clays were tested in CK_0UDSS at OCR from 1 to large values with the results normalized as follows:

$$Y = (s_u)^{oc} / \sigma'_{vo} \quad \text{and} \quad X = (s_u)^{nc} / \sigma'_{vo}.$$

It is evident that at OCR = 1, Y/X is equal to 1. Also when σ'_{vo} is the in-situ overburden pressure, the ratio $Y/X = (s_u)^{oc}/(s_u)^{nc}$. The more general form of Y/X allows one to use a laboratory value of σ'_{vo} which may be different from the field value. The initial curve had only a modest scatter and would appear useful for almost any clay. Other tests data from Mahar and O'Neill (1983) and Simons (1960) have been plotted by the author onto this curve (codes 2 and 3 of Fig. 2.27). Clays range from inorganic to organic and highly desiccated (code 2). Code 1 covers five clays, Code 2 is same locale but two separate stratums, Code 3 is from Oslo, Norway. The general curve trend is present and it can be suggested that these curves might be used for similar soils and the same local test method.

Mahar and O'Neill (1983) reported that higher values of liquid limit of soil will produce lower value of slope of the curve log s_u/σ'_{vo} versus log OCR. They plotted s_u/σ'_{vo} versus OCR for two different types of soils and found two different curves whereas they found that the plot of s_u. LL $/\sigma'_{vo}$ versus OCR for the two types of soils produced a common curve as shown in Fig. 2.28.

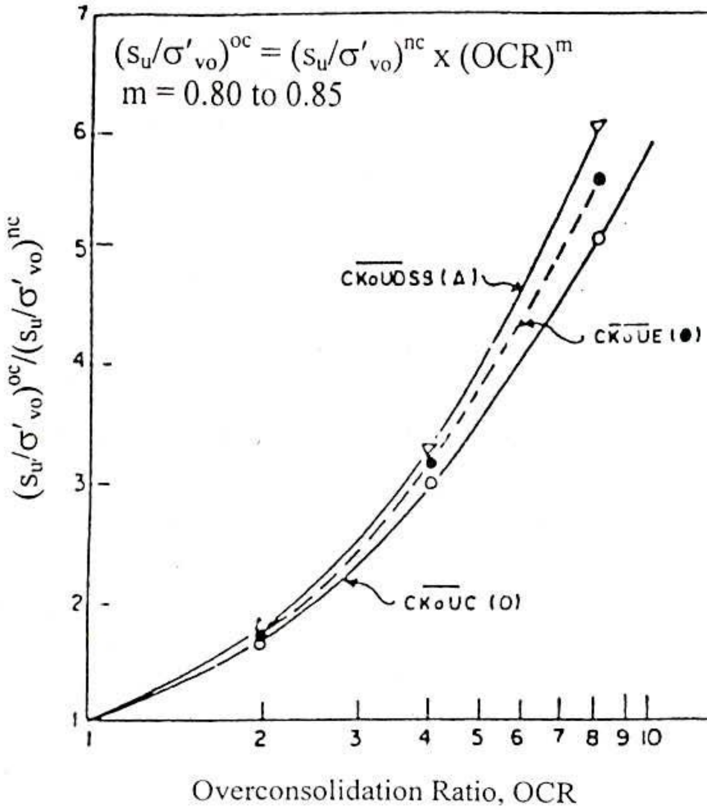


Fig. 2.26 Relative Increase in Undrained Strength Ratio with Increasing OCR (after Koutsoftas, 1981).

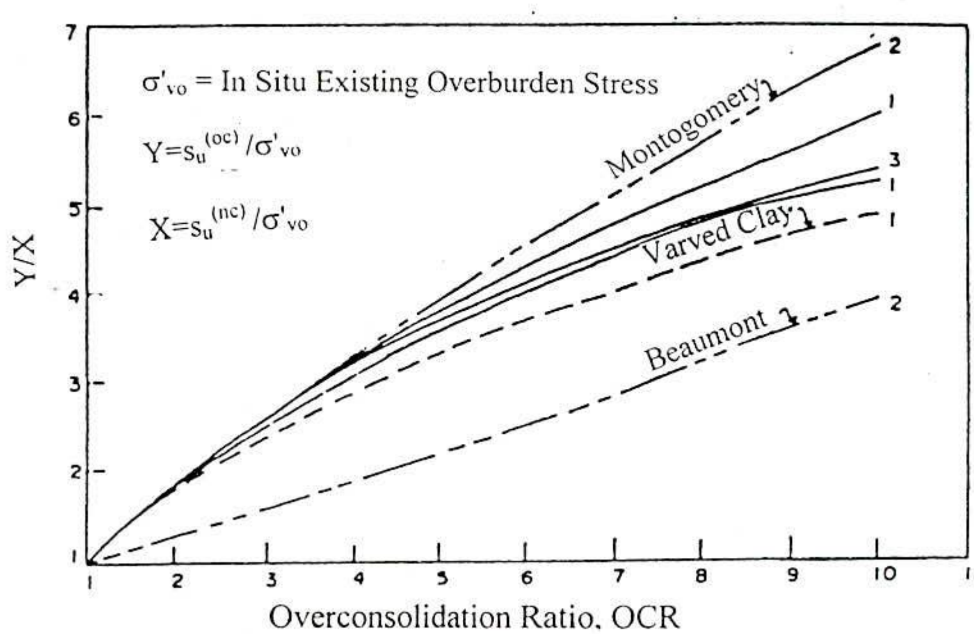


Fig. 2.27 Variation of (Y/X) with OCR (after Ladd et al., 1977)

Table 2.4 Some Models to Represent Undrained Shear Strength of Normally Consolidated and Overconsolidated Clays

Eqn No.	Model Equation	State of Stress	Reference
(a)	$s_u/\sigma'_{vc} = 0.11 + 0.0037\ (PI)$	NC	Skempton (1957)
(b)	$s_u/\sigma'_{vc} = 0.45\ (PI)^{1/2}$, where PI>5%	NC	Bjerrum and Simons (1960)
(c)	$s_u/\sigma'_{vc} = 0.18\ (I_L)^{-1/2}$, where $I_L > 0.5$	NC	
(d)	$s_u/\sigma'_{vc} = 0.5$ LL, where LL > 20%	NC	Karlsson and Viberg (1967)
(e)	$S_u/\sigma'_{vc} = [3 \sin\phi'/(3- \sin\phi')]x[1/\exp(\Lambda)]$	NC	Schofield and Wroth (1968)
(f)	$[s_u/\sigma'_{vo}]^{(oc)}/[s_u/\sigma'_v]^{(nc)} = (OCR)^m$	OC	Ladd and Foot (1974)
(g)	$[s_u^{(oc)}/\sigma'_{vo}] = [s_u^{(nc)}/\sigma'_v]$ x $(OCR)^\Lambda$	OC	Atkinson and Bransby (1978)
(h)	$[s_u^{(oc)}/\sigma'_{vo}] = [s_u^{(nc)}/\sigma'_v]$ x $(OCR)^{1-N}$ where $N = C_S/C_C$	OC	Mitachi and Kitago (1976)
(i)	$s_u^{(oc)}/\sigma'_{vo} = [3\ \mathrm{Sin}\phi'/ (3- \mathrm{Sin}\phi')]$ x $[(OCR)^\Lambda/\exp(\Lambda)]$	OC	Mayne (1980)
(j)	$[s_u^{(oc)}/\sigma'_{vo}] = [s_u^{(nc)}/\sigma'_v]$ x $(OCR)^\Lambda$	OC	Kamaluddin (1999)
(k)	$[s_u^{(oc)}/\sigma'_{vo}] = [s_u^{(nc)}/\sigma'_v]$ x $(OCR)^\Lambda$	OC	Ansary (1993)

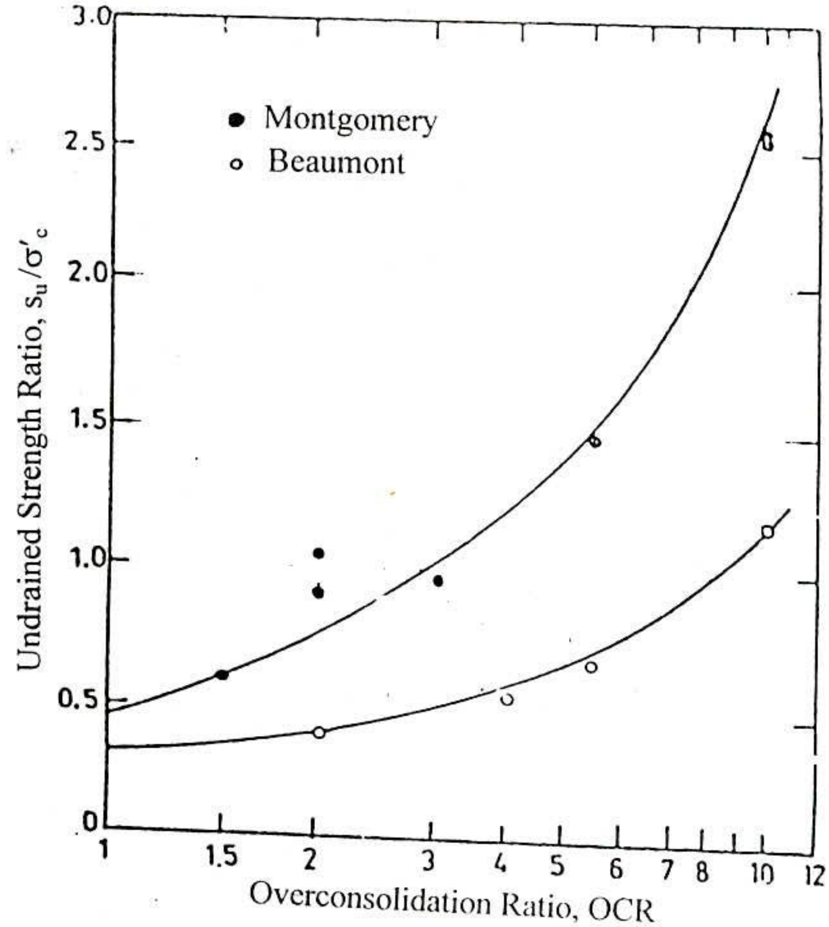


Fig. 2.28 (a) Relationship of Undrained Strength Ratio (s_u/σ'_c) with OCR (after Mahar and O'Neill, 1983)

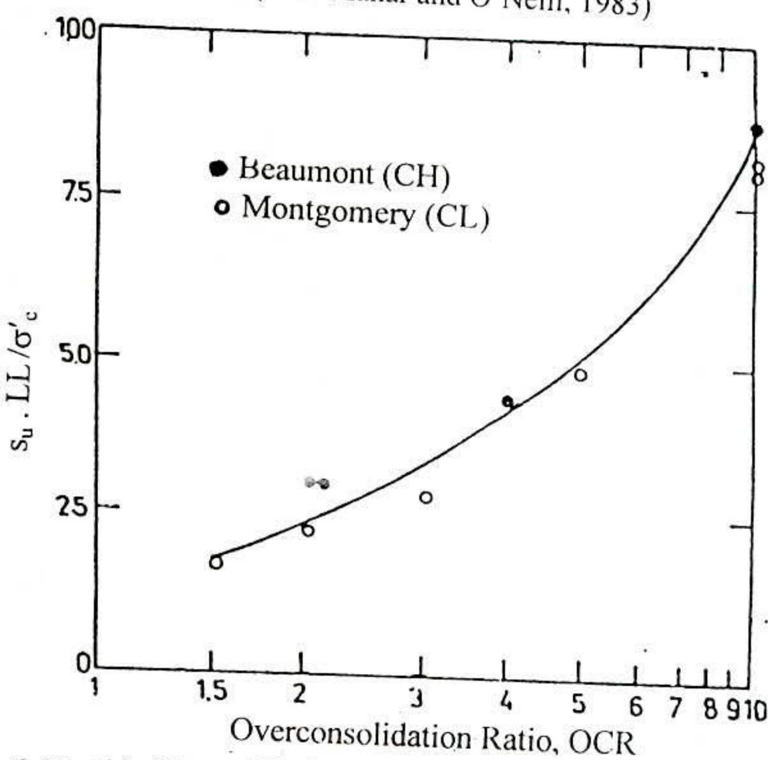


Fig. 2.28 (b) General Undrained Strength Ratio ($s_u w_L/\sigma'_c$) Versus OCR (after Mahar and O'Neill, 1983)

2.5.4 Pore Pressure-Strain Relationships

Previous research has shown that the pore pressure parameter at failure, A_f decreased as the overconsolidation ratio is increased and eventually the variation of A_f is significantly smaller at higher overconsolidation ratios as compared with its variation at low overconsolidation values. It was also noted that the value of A_f was generally dependent not only on the type of clay and its overconsolidation ratio, but also on the applied stress systems.

Parry and Nadarajah (1973) reported the results from a series of triaxial compression and extension tests on normally and lightly overconsolidated kaolin clay sheared both from isotropic and K_0-conditions. The relationship between A_f and the overconsolidation ratio for the isotropically consolidated samples both in compression and in extension was almost identical over the range of overconsolidation ratio close to 2.6. Up to an overconsolidation ratio of 1.25 the A_f values from the K_0-compression tests were much higher than those from the isotropic compression. For overconsolidation ratios of higher than 1.25, the value of A_f from the K_0-compression test drop quickly to a constant value of 0.3. Similarly, Koutsoftas (1981) also observed a rapid decrease in the value of A_f for K_0-overconsolidated samples as the overconsolidation ratio is increased. However, these values remained positive even for overconsolidation ratios up to 10. This observation agreed well with those made by Nakase and Kobayashi (1971).

Yudhbir and Varadarajan (1974) carried out triaxial tests on reconstituted clay under normally consolidated and overconsolidated pre-shear isotropic conditions to study the effect of overconsolidation ratio, OCR on the stress-strain-pore pressure response. A relationship for the shear strain, the overconsolidation ratio and the A parameter for isotropically consolidated clay was suggested by Yudhbir and Varadarajan (1974). The values of A at various levels of strains in the normally consolidated states converged to a lower strain-independent A value at an overconsolidation ratio termed as the "critical overconsolidation ratio". The critical overconsolidation ratio, which depends on the type of clay, separated the overconsolidation ratio into two zones: for overconsolidation ratios less than the critical value, A increases with the strain and for overconsolidation ratio greater than the critical value, A decreases with the strain. Yudhbir and Varadarajan (1974) emphasized the significance of the degree of overconsolidation on the stress-strain relationship.

Mayne and Stewart (1988) presented the overall trend of A_f value from anisotropically and isotropically consolidated samples. The authors have shown that the A_f values of anisotropically consolidated sample tend to be asymptotic to zero at higher overconsolidation ratios, while these A_f values for isotropically consolidated clay became negative for overconsolidation ratios of 4 to 6.

Handali (1986) also normalized all excess pore pressure, u measured from undrained tests of the overconsolidated samples with respect to the pre-shear consolidation pressure of normally consolidated samples instead of normalizing them with respect to their individual pre-shear consolidation stress. In other words, the points of reference for normalization of the excess pore pressure of overconsolidated samples were taken as the same as the pre-shear consolidation pressure of the normally consolidated sample with the same void ratio.

2.6 Stress Deformation Characteristics of Silts

An extensive series of undrained triaxial tests were performed on reconstituted samples of Alaskan silts (LL = 22 to 60, PI = 3 to 28) in both the normally consolidated and overconsolidated state by Fleming and Duncan (1990). The results of the investigation show that the pore water pressure increases to a peak and then gradually and continuously decreases with strain. Comparison of deviator stress and pore pressure behaviour for undisturbed and remoulded samples are shown in Fig. 2.29.

Fleming and Duncan (1990) developed a testing programme to determine whether the Alaskan silt exhibits normalized behaviour. A series of UU, IC-U and AC-U tests were performed at varied consolidation pressures and overconsolidation ratios. The measured range of normalized shear strength at various OCR for each type of test is given in Table 2.5. It was found that the normalized strength varied with the consolidation pressure. The higher the consolidation pressure, the lower was the normalized strength. This may appear to indicate that the undrained strength of Alaskan silts can not be normalized. However, these relatively small variations in normalized strength are believed to result from sample preparation and reconsolidation effects. The no malized strengths (s_u/σ'_{vo}) versus overconsolidation ratios (OCR) are shown in Fig. 2.30.

Fleming and Duncan (1990) compared the results of the test on normally consolidated samples in Table 2.6 to results obtained by Ladd et al. (1985) and Wang and Vivatrat (1982) from tests performed on other silts. It may be seen that there is a considerable variation among the various measured values of normalized strength. It may thus be concluded that values of normalized strength vary considerably from one silt to another, and would need to be evaluated specifically for each new silt deposit. The results have shown that the stress-deformation characteristics of silts are quite different from those of clay. Silt samples are easily disturbed, and their undrained strengths are more likely to be seriously affected by disturbance than those of many clays. Therefore, minimizing sample disturbance is important in order to obtain reliable estimates of the strengths of silts in the ground.

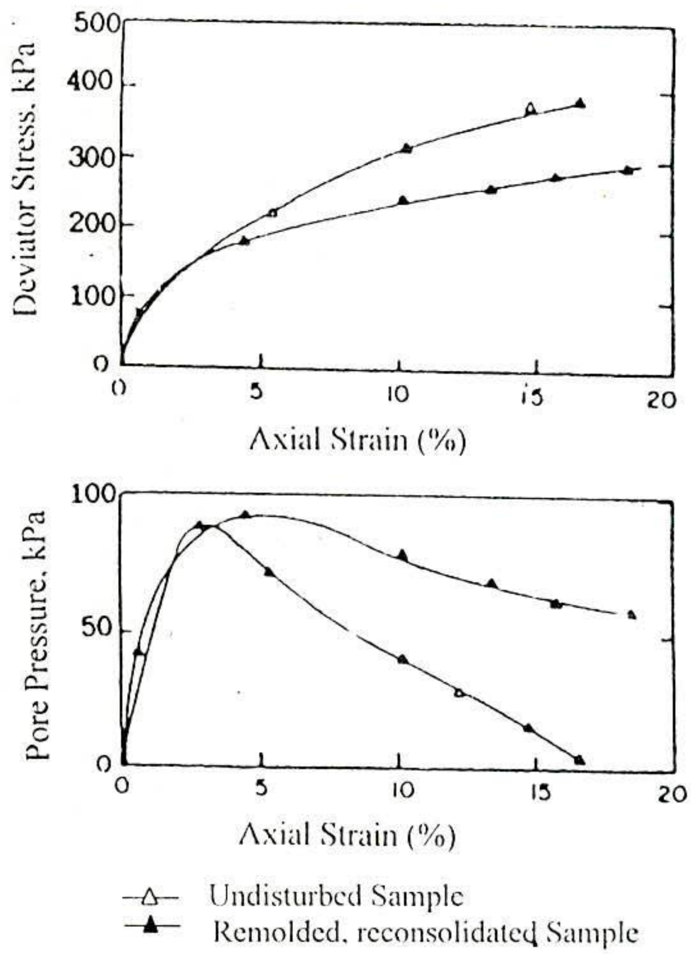


Fig. 2.29 Comparison of Deviator Stress and Pore Pressure behavior for Undisturbed and Remolded Samples (after Fleming and Duncan, 1990)

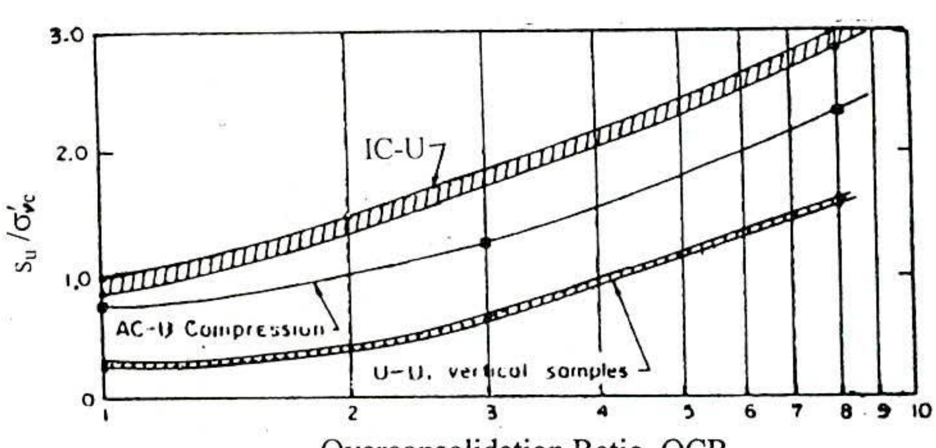


Fig. 2.30 Normalized Undrained Shear Strength (after Fleming and Duncan, 1990)

Table 2.5 Values of s_u/σ'_{vc} for Alaskan Silt (after Fleming and Duncan, 1990)

Type of Test	Overconsolidation Ratio			
	1	2	3	8
UU	0.25-0.30	0.60-0.64	-	1.55-1.60
IC-U	0.85-1.00	1.70-1.85	-	2.85-3.00
AC-U	0.78	-	1.28	2.37

Table 2.6 Normalized Strength Parameters for Normally Consolidated Silts (after Fleming and Duncan, 1990)

Type of Test	s_u/σ'_{vc}	K_0	Reference
UU	0.25-0.30	-	Fleming and Duncan (1990)
UU	0.185	-	Ladd et al. (1985)
IC-U	0.25	-	Ladd et al. (1985)
IC-U	0.30	-	Ladd et al. (1985)
IC-U	0.85-1.0	-	Fleming and Duncan (1990)
IC-U	0.30-0.65	-	Wang et al. (1982)
AC-U	0.32-0.39	0.84; 0.59	Ladd et al. (1985)
AC-U	0.26	0.59	Ladd et al. (1985)
AC-U	0.75	0.50	Fleming and Duncan (1990)

2.7 Use of Critical State Models

2.7.1 Prediction of Strains Using Cam Clay and Modified Cam Clay Models

Balasubramaniam and Chaudhry (1978) investigated experimentally the stress-strain characteristics for Bangkok clay (LL = 97-121%, PL = 32- 46%, w_n = 99-122% and clay = 55-68%). Balasubramaniam and Chaudhry (1978) compared their observed strains with the strains predicted from the critical state theories. Two models are employed and these are the Cam Clay model by Roscoe, Schofield and Thurairajah (1963) and Schofield and Wroth (1968); and the Modified Cam Clay model by Roscoe and Burland (1968). The fundamental soil parameters used in the Critical State Theories were λ, κ and M. Isotropic consolidation and swelling tests were carried out on soft Bangkok clay which indicated the value of λ is 0.51 and that of κ is 0.091. Also, the critical state parameter, M, was taken as 1.0. From the experimental data on Bangkok clay, Balasubramaniam and Chaudhry (1978) compared the observed strains with the predicted strains from Critical State Models and the following conclusions were drawn :

(i) The Cam Clay Model overpredicts the strains in all tests, and

(ii) The Modified Cam Clay Model successfully predicts the strains in all tests.

In the Modified Cam Clay model of Roscoe and Burland, corrections were made for the shear strain from the contribution due to the constant q yield loci. The contributions from the constant q yield loci were approximately the same as the shear strain obtained from undrained tests in the q'/p': ε plot. From their tests, the following conclusions have been drawn: (i) The Cam Clay model overpredicts the volumetric strains and the shear strain; and (ii) the Cam Clay Modified model successfully predicts the strains in all tests.

2.7.2 Relationship of Critical State Parameters with Plasticity Index and Experimental Stress-Strain Values with the Prediction Values

The possibility of establishing correlations between constitutive parameters and simple soil properties such as consistency limits has long attracted researchers, and Nakase et al. (1988) have presented some useful data in this direction. An attempt was made by Nakase et al. (1988) to correlate various parameters often used in specifying the constitutive models, based on data obtained from laboratory tests. For each soil sample, oedometer tests and a series of triaxial tests were performed consisting of CIUC (undrained compression test on an isotropically consolidated specimen), CIUE (undrained extension test on an isotropically consolidated specimen), CK_0UC (undrained compression test on a K_0-consolidated specimen) and CK_0UE (undrained extension test on a K_0-consolidated specimen), with a constant rate of axial strain of 0.07%/min.

The Cam Clay model (Schofield and Wroth, 1968) and the Sekiguchi-Ohta (1977) model were considered by Nakase et al. (1988). The Cam Clay model has been generated largely from the results of triaxial compression tests on soft clay samples; it can be expected to be effective in predicting the behaviour of isotropically consolidated samples, but not to be particularly successful at matching the observed experimental data of anisotropically consolidated samples. The Sekiguchi-Ohta model can, however, incorporate the anisotropical stress history that the soils have more commonly experienced into the model, requiring the same parameters as the Cam Clay model for their in viscid model.

In these constitutive models, the following four basic soil parameters are required to specify the model:

(i) λ = compression index,

(ii) κ = swelling index,

(iii) $M = 6 \sin\phi'/(3 - \sin\phi')$, where, ϕ' = the effective angle of internal friction obtained from the compression test, and

(iv) N = a specific volume of soil isotropically normally consolidated at 98 kPa.

In order to specify the behaviour of the model, three other values are also required to describe the current condition of the soil, namely, initial void ratio (or specific volume), current stress state, and the K_0 value of the soil.

Nakase et al. (1988) estimated soil properties from experimental observations for stress-strain and stress path predictions of models. These are shown in Table 2.7 and their correlations with plasticity indices are shown in Table 2.8.

From above discussions the following conclusions can be drawn:

(i) Correlations between plasticity index and the soil parameters for the constitutive equations were obtained with high values in the coefficient of correlation. There was no direct correlation between M_c and PI, i.e., M_c is independent of plasticity index. Similar observations have been made by the writers (Frydman and Samoocha, 1985: Frydman, 1987) with regards to the effective friction angle, ϕ', of compacted Israeli clays. For a large range of clays, ϕ' was found to be about 25°, regardless of the consistency limits of the clay. Vaughan et al. (1979) also obtained similar results for UK clays, again finding an average ϕ' of about 25°. On the contrary, Hvorslev (1949) and Parry (1960) have been shown that M_c decreases with the increase of plasticity index.

(ii) The correlations may be used together or separately to estimate the parameters, if a better alternative is not available.

Table 2.7 Estimated Soil Properties for Stress-Strain and Stress Path Predictions (after Nakase et al., 1988)

Soil	PI	λ	κ	e_0	K_0	M_c	M_e
M-50	51.1	0.25	0.038	1.49	0.42	1.65	1.13
M-30	29.4	0.16	0.021	1.07	0.42	1.65	1.24
M-10	10.7	0.06	0.005	0.70	0.42	1.65	1.33

Table 2.8 Correlation Between Critical State Parameters and Plasticity Index (after Nakase et al., 1988)

Parameters	Correlation Coefficient, R
$\lambda = 0.02 + 0.0045$ PI	0.98
$\kappa = 0.00084$ (PI-4.6)	0.94
$N = 1.517 + 0.019$ PI	0.95
$M_c = 1.650$	-----
$M_e = 1.385 - 0.00505$ PI	0.85

2.8 Mechanisms of Sample Disturbance

A soil sample will be subjected to disturbance when it is transferred from its position in the ground to the laboratory and prepared for testing. The mechanisms associated with this disturbance can be classified as follows:

(1) Changes in stress conditions;
(2) Mechanical deformation;
(3) Changes in water content and voids ratio; and
(4) Chemical changes

Changes in stress conditions occur as the total stresses being applied to the sample of soil change. In its extreme, this is the relaxation of the total horizontal and vertical stresses from their in-situ value, to zero, on the laboratory bench. Mechanical deformations are shear deformations applied to the soil sample while the sample experiences no change in volume. Changes in water content can be an overall swelling or consolidation of the soil sample, or a redistribution of moisture due to the setting up of pore pressure gradients. A change in voids ratio distinct from the above changes in moisture content, is associated with the expansion of gases in the soil sample as a consequence of relaxation of total stresses. These gases either being free in partially saturated soils or in solution in saturated soils. Chemical changes are associated with the change in chemical properties of the soil particles, inter-particle bonding or pore water.

These mechanisms can occur at different stages during the process of transferring a soil sample from the ground to the laboratory, and during preparation for testing. Some of the mechanisms occur very quickly, while others are more time dependent. Some of the mechanisms are unavoidable while others can be minimized or even eliminated. The magnitude of the mechanisms is not only dependent on the sampling processes being used, but also on the type of soil being sampled. The effect of these mechanisms can also be different for different soil types.

A geotechnical engineer is fundamentally concerned with the physical stress-strain-strength properties of the soil under investigation. If the effective stress, fabric or structural features in a sample of soil are altered during the sampling process, then the soil sample in the laboratory will no longer exhibit the same physical properties as it would in situ. It is therefore important to understand where, in the sampling and testing process, the afore-mentioned mechanisms are occurring, and it is necessary to minimize or even eliminate these mechanisms wherever possible. Where these mechanisms are unavoidable, it is important to know what affect, both qualitatively and quantitatively, they have on the physical properties being measured. In addition, it is important to establish whether the effects of these mechanisms on the physical properties being measured can be assessed and even corrected for.

2.9 Different Stages of Sample Disturbance

The disturbance experienced by a sample of soil due to sampling from the ground is caused by several stages. The principal stages of sampling disturbance can be stated as follows (La Rochelle et al., 1981):

(1) Disturbance of the soil to be sampled before the beginning of sampling as a result of poor drilling operation.
(2) Mechanical distortion during the penetration of the sampling tube into the soil.
(3) Mechanical distortion and suction effects during the retrieval of the sampling tube.
(4) Release of the total in-situ stresses
(5) Disturbance of the soil during transportation, storage and sample preparation.

The first cause can be reduced by sampling with properly cleaned boreholes advanced by using bentonite slurry. The second and third causes are directly associated with sampler design and can be controlled to certain extent. The fourth cause is unavoidable even though its effects may be different depending on the depth of sampling and soil properties. The fifth cause can be reduced by storing samples for minimum time in controlled atmosphere and careful handling of samples during transportation and preparation.

Mechanisms and causes of sampling disturbance have been summarized by Clayton (1986). Detail descriptions of the disturbances caused during boring, excavating, sampling, transportation, storage and sample preparation have been reported by a number of researchers (Hvorslev, 1949; Kallstenius, 1971; Schjetne, 1971; Bozozuk, 1971; Shackel, 1971; Sone et al., 1971; Bjerrum, 1973; Brand, 1975; Arman and McManis, 1976; La Rochelle et al., 1976; Kimura and Saitoh, 1982; Kirkpatrick and Khan, 1984; Baligh, 1985; Clayton, 1986; Chin, 1986; Baligh et al., 1987; Graham et al., 1987; Baldi et al., 1988; Siddique, 1990; Hajj, 1990; Hight and Burland, 1990; Hopper, 1992; Chandler et al., 1993; Sarker, 1994; Rahman, 2000).

Formation of a hole in the ground modifies the stresses, can impose strains, and even lead to failure of the soil at the base of the hole. The disturbances are dependent on both the type of boring or excavating technique, and the type of soil. Hvorslev (1949), ISSMFE (1965) and Broms (1980) stated that the borehole should always be cleaned out before sampling is commenced. BS:5930 (1981) Clayton (1986) commented on the importance of ensuring good maintenance of equipment, good drilling technique and expert and detailed supervision.

Another principal cause of sample disturbance during drilling is the reduction in total vertical and total lateral stresses due to removal of soil from the borehole. Swelling at the base of borehole occurs as a consequence of stress relief. The process is fast and unavoidable in granular soil; in cohesive soils, however, swelling can be reduced by sampling as quickly as possible following boring. The amount of swelling that occurs is proportional to the change of total stress occurring at the base of a borehole. Thus if the borehole is substantially empty of water there is likely to be more swelling than if the borehole is kept full of mud or water. Other severe effects of stress relief during drilling on soil are base heave, piping and caving (Clayton et al., 1982). Base heave can be thought of as foundation failure under decreased vertical stress. When the total stress relief at the base of a borehole is very great compared with its undrained shear strength, plastic flow of soil may take place upwards into the borehole. Failure in a borehole by base heave can occur in very soft soils if the water level is kept too low (Begemann, 1977). When a borehole is inducing total stress relief, and water balance is insufficient to prevent high seepage pressure gradients in the soil at the base of the hole, large volumes of fine granular soil may move up into the casing. Soil below the bottom of the casing will be brought to a very loose state. This phenomenon is called piping. Both base heave and piping can be reduced by keeping the hole full of water. Caving typically occurs when boreholes are advanced into soft, loose or fissured soils. Material from the sides of the borehole collapses into the bottom of the hole and must be cleaned out before sampling can take place.

During sampling the change of volume resulting from the intrusion of a sampling tube into a soil mass produces appreciable distortions. Hvorslev (1949) described the forces acting on an element of soil while it is being tube sampled. There are two main forces associated with sampling. The first is that occurring as the soil is displaced by the advancing cutting edge. This can cause quite considerable shear strains, and possibly large forces. This disturbing effect is reduced by decreasing the cross-sectional area of the cutting edge. The second disturbing force in the soil during tube sampling is that caused by friction or adhesion between the soil and walls of the sampler. Hvorslev (1949) considered that friction on the internal wall would be more significant than that on the outside wall, causing the structure of the sample to be altered. Bjerrum (1973) also reported that due to friction between the clay and the sampling tube, the outer zone of the sample becomes remoulded. The volume of these zones of badly disturbed clay and the degree to which the original structure of the clay in these zones is destroyed is, however, not the same in all types of clay. The greatest amount of disturbance is, for instance, experienced in clays of low plasticity. Clays with pronounced cohesive properties will undergo less disturbance. The same is the case with highly sensitive or quick clays, the remoulded strength being so low that the friction between clay and sampling tube is practically eliminated.

During sampling another important contributory factor to disturbance is due to release of in-situ total stresses. In response to the reduction of applied total stresses, the pore pressures in a sample will reduce and may normally be expected to become negative. If the sample is coarse-grained, it will have a high coefficient of permeability and a large average pore size and water or air will rapidly penetrate it and dissipate the negative pore pressures. Thus, with total and effective stresses reduced to zero, a granular soil has little strength and is very difficult to sample or prepare for laboratory testing. In a cohesive soil, however, a small average pore size normally precludes the penetration of air. Because of low permeability a considerable period of time may be required for water to penetrate and dissipate the negative pore pressures set up in the sample. A sample which has received no disturbance other than that involved with the release of in-situ total stresses is termed "perfect" sample and the disturbance caused due to the release of in-situ total stresses is called stress release or "perfect" sampling disturbance. Several workers have investigated the effects of "perfect" sampling disturbance on the undrained stress-strain, stiffness and strength properties of clays. These have been discussed in detail in section 2.10.1.

When transporting the samples from the field to the laboratory the sample can be disturbed due to inadequate sealing, vibration and shock, thermal variations, pore pressure equalisation and chemical effects. Samples are usually sealed and stored for some period of time before testing and this delay may cause further alternations to the clay structure. Migration of water within the sample may still lead to significant changes of properties such as compressibility and undrained strength. Two types of effects have been noted. Firstly, water migrates from one type of soil to another (Kimball, 1936; Rowe, 1972) and secondly, differential residual pore pressures in the samples equalize with time (Kallstenius, 1971; Schjetne, 1971; Bjerrum, 1973).

A major contributory factor to disturbance is the extrusion of the sample from the sampling tube. When the specimen is being prepared for testing, the soil can be disturbed mainly due to the (i) forces and friction during extrusion and (ii) moisture changes. The force required to extrude a soil sample from a sampling tube was investigated by Sone et al. (1971) for a clayey silt. It was much larger than the unconfined compressive strength of the soil. The undrained shear strength was reduced 10% to 20% by the extraction up to 10 cm to 20 cm from the bottom of the sample. Arman and McManis (1976) also examined the extrusion stress for tube samples of very stiff clay. Soil cores were extruded using hydraulically operated pistons. During core extrusion, the end of the sample in contact with the piston began to show measurable displacements before the opposite end. Thus internal displacements were occurring within the tube. The maximum strain at the piston end varied from 0.001 to 0.005. In all cases, the applied stress exceeded the unconfined compressive strength of the soil to a maximum of 900%. X-ray radiography was also carried out to determine the extent of disturbances in the extruded soil cores. Radiographs showed two distinct distortion effects caused by extrusion process. The first type of distortion, observed in all cores, was a gradual bending of the soil layers, with a maximum at the tube surface and decreasing toward the center.

2.10 Effects of Sample Disturbance

The effects of sampling disturbance on stress-strain characteristics can be considered by dealing separately with the following:

(i) In the "Perfect" sampling which is usually simulated by consolidating specimens anisotropically in the triaxial apparatus and then releasing firstly the in-situ shear stress and secondly releasing the total isotropic stress to zero under undrained conditions. The isotropic effective stress left in the removed sample, so called residual effective stress, σ'_{ps}, for a "perfect" saturated sample of clay which had in-situ vertical and horizontal effective stresses of σ'_{vo} and σ'_{ho} ($\sigma'_{ho} = K_0\sigma'_{vo}$) respectively, is given by the following expression (Ladd and Lambe, 1963; Ladd and Varallyay, 1965):

$$\sigma'_{ps} = \sigma'_{vo}\,[K_0 + A_u\,(1 - K_0)] \tag{2.23a}$$

Where K_0 is the coefficient of earth pressure at rest and A_u is the Skempton's pore pressure parameter for the undrained release of the in-situ stresses which existed at the K_0 conditions. The parameter A_u for a saturated clay (i.e., Skempton's B parameter is equal to unity) is given by

$$A_u = (\Delta u - \Delta\sigma_h) / (\Delta\sigma_v - \Delta\sigma_h) \tag{2.23b}$$

Where, Δu is the pore pressure change; and $\Delta\sigma_v$ and $\Delta\sigma_h$ are the changes of vertical and horizontal total stresses.

For normally consolidated clays, $A_u = 0.1 \pm 0.2$, $K_0 = 0.55 \pm 0.1$. For heavily overconsolidated clays, where $K_0 = 2 \pm 0.5$, A_u is approximately equal to 0.4 ± 0.1. Typical values of A_u, K_0 and stress ratios $\sigma'_{ps}/\sigma'_{vo}$ are shown in Table 2.9. What are thought to be typical values of K_0, A_u and $\sigma'_{ps}/\sigma'_{vo}$ are suggested in Table 2.9 based on the limited data by Bishop and Henkel (1962) and Skempton

(1961). As shown in Table 2.9, the effective stress of "perfect" samples will be only 35 to 80 percent of the in-situ vertical effective stress σ'_{vo} for normally consolidated clays, but may be double σ'_{vo} for a highly overconsolidated plastic clay.

(ii) Imperfect sampling, in which some arbitrary stress path is assumed to be applied before undrained shearing to failure. Imperfect sampling has been further subdivided into tube sampling and block sampling.

(iii) "Ideal" sampling (Baligh et al., 1987), which can be modelled in the laboratory by consolidating samples anisotropically in the triaxial apparatus and then imposing predicted tube penetration strains, followed by undrained stress relief simulating "perfect" sampling.

Table 2.9 Typical Values of K_0, A_u and Stress Ratios for "Perfect" Sampling (after Bishop and Henkel, 1961)

Types of Specimen	K_0	A_u	$\sigma'_{ps}/\sigma'_{vo}$
Normally Consolidated			
Clayey Silt	0.4 to 0.5	-0.1 to 0.0	0.35 to 0.5
Lean Clay	0.5 to 0.6	0.1 to 0.2	0.55 to 0.7
Plastic Clay	0.6 to 0.7	0.2 to 0.3	0.65 to 0.8
Heavily Overconsolidated			≅2.0
Plastic Clay	1.5 to 2.5	0.3 to 0.5	

2.10.1 "Perfect" Sampling

In order to understanding the influence of "perfect" sampling on the undrained shear characteristics of soils, a number of investigators (Skempton and Sowa, 1963; Ladd and Lambe, 1963; Hight et al., 1985) have idealized the process of stress release in the laboratory either by undrained release of the total deviator stress to zero from an in situ anisotropic condition, but maintaining an isotropic total stress state. Others, however, simulated stress relief by unloading both the deviator stress and isotropic stress to zero, i.e., by reducing the total stresses to zero (Noorany and Seed, 1965; Davis and Poulos, 1967; Kirkpatrick and Khan, 1984; Kirkpatrick et al., 1986; Graham and Lau, 1988; Sarker, 1994; Siddique and Farooq, 1996; Rahman, 2000).

Ladd and Lambe (1963) investigated the effect of "perfect" sampling on undrained behaviour of Kawasaki clay (LL = 48-106, PI = 16-46). "Perfect" sampling produced completely different stress paths as compared with in-situ sample. Ladd and Lambe (1963) also determined the values of isotropic effective stress, σ'_{ps} and pore pressure parameter, A_u of "perfect" specimens of Kawasaki clay and Boston Blue clay. The resulting values of the ratio, $\sigma'_{ps}/\sigma'_{vo}$ were 0.56 ± 0.05 with corresponding A_u values of 0.17 ± 0.10. Similar test data on normally consolidated Boston Blue clay yielded $\sigma'_{ps}/\sigma'_{vo} = 0.59$ and $A_u = 0.11$. Skempton and Sowa (1963) reported values of the ratio, $\sigma'_{ps}/\sigma'_{vo}$ = 0.57 and 0.67 with corresponding A_u values of -0.02 and -0.10 for overconsolidated clays of Weald (OCR = 2) and Weald (OCR = 14) respectively. Ladd and Varallyay (1965) also reported values of A_u, 0.12 to 0.24 and $\sigma'_{ps}/\sigma'_{vo}$, 0.57 to 0.67 for remoulded Boston Blue clay. Kirkpatrick et al. (1986) reported values of the ratio $\sigma'_{ps}/\sigma'_{vo}$ were 0.48, 0.38 and 0.20 with corresponding A_u values of 0.25, 0.20 and 0.20 for overconsolidated clays of Kaolin (OCR = 2), Illite (OCR = 2.7) and Illite (OCR = 5), respectively.

Siddique and Farooq (1996) reported values of the ratio, $\sigma'_{ps}/\sigma'_{vo}$ were 0.55 to 0.58 with A_u values of 0.10 to 0.13 for normally consolidated soft Chittagong coastal soils (LL = 43 to 57, PI = 18 to 33). Siddique and Sarker (1997) reported value of the ratio, $\sigma'_{ps}/\sigma'_{vo}$ were 0.65 with corresponding A_u value of 0.13 for reconstituted normally consolidated soft Dhaka clay (LL = 45, PI = 23). The values of A_u and the ratio $\sigma'_{ps}/\sigma'_{vo}$ for "perfect" sampling obtained by different investigators are summarized in Table 2.10.

Table 2.10 A_u-Values and Stress Ratios ($\sigma'_{ps}/\sigma'_{vo}$) for "Perfect" Sampling of Normally Consolidated and Overconsolidated Clays

Clay Type	Index Properties	K_0	A_u	$\sigma'_{ps}/\sigma'_{vo}$	Reference
Undisturbed Kawasaki clay	LL= 48-106 PI = 16- 46	.47	.07 to .28	.51 to .61	Ladd and Lambe (1963)
Undisturbed Boston Blue clay	LL = 33 PI = 14	.54	.11	.59	Ladd and Lambe (1963)
Remoulded Boston Blue clay, $S_t = 7 \pm 2$	LL = 33 PI = 15	.54	.12 to .24	.57 to .67	Ladd and Varallyay (1965)
Remoulded Weald clay, $S_t = 20$	LL = 46 PI = 24	.59	-.02 to -.1	.57 to.61	Skempton and Sowa (1963)
Undisturbed San Francisco Bay Mud, S_t ~ 10	LL = 88 PI = 45	.50	.16 to .24	.58 to .62	Seed et al. (1964)
Kaolin, OCR = 2	PI = 30	.85	.25	.48	Kirkpatrick et al. (1986)
Illite, OCR = 2.67	PI = 40	1.0	.20	.38	
Illite, OCR = 5	PI = 40	1.0	.20	.20	
Kaolin, OCR = 1	PI = 30	.56	.25	1.0	
Illite, OCR = 1	PI = 40	.67	.20	1.0	
Reconstituted Dhaka Clay	LL = 45 PI = 23	.60	.13	.65	Siddique and Sarker (1997)
Reconstituted Patenga clay	LL = 44 PI = 18	.49	.133	.56	Siddique and Farooq (1996)
Reconstituted Fakirhat clay	LL = 43 PI = 22	.50	0.10	0.55	
Reconstituted Kumira clay	LL = 57 PI = 33	.52	0.117	0.58	
Reconstituted Dhaka clay, OCR=1	LL = 47 PI = 26	.50	0.182	0.59	Rahman (2000)
Reconstituted Dhaka clay, OCR=2	LL = 47 PI = 26	.50	0.077	0.54	
Reconstituted Dhaka clay, OCR=5	LL = 47 PI = 26	.50	0.0053	0.503	

Skempton and Sowa (1963) examined the effect of "perfect" sampling in remoulded Weald Clay (LL = 46, PI = 24) which has a low sensitivity (S_t = 2). Skempton and Sowa (1963) found that the undrained strength of normally consolidated "perfect" samples were only 2% less than that of the "ground" samples although the stress paths were entirely different. They also found that failure strain of "perfect" samples were increased.

Most likely, clays with higher sensitivity will be affected more by "perfect" sampling; since Noorany and Seed (1965) observed a 5% reduction of the undrained strength for San Francisco Bay Mud with a sensitivity 8 to 10. Noorany and Seed (1965) also observed a 5% increase in strain at peak strength and 10% reduction of the initial stiffness for normally consolidated "perfect" samples of soft clay (LL = 88, PI = 45). Ladd and Varallyay (1965) found a 7% decrease in undrained strength and 150% increase (highly) in the strain at peak strength for normally consolidated Boston Blue clay (LL = 33, PI = 15) due to "perfect" sampling. Davis and Poulos (1967) reported a 19% decrease in undrained strength of a remoulded "perfect" kaolin (LL = 55, PI = 22) specimen tested unconfined. However, the undrained strength of the reconsolidated "perfect" specimen was only 5% less than that of the "field" element. Kubba (1981) reported a decrease in undrained strength of 5 to 11% due to "perfect" sampling for normally consolidated samples of kaolin.

"Perfect" sampling has a marked influence on pore pressure responses as reported by Seed et al. (1964), Noorany and Seed (1965), and Ladd and Varallyay (1965). The pore pressure parameter A at failure was found to decrease by as much as 50% for specimens subjected to "perfect" sampling. Ladd and Varallyay (1965) also observed a slight reduction in stiffness and a large increase in axial strain required to mobilize the peak shearing resistance. Atkinson and Kubba (1981) also reported considerably lower stiffness for anisotropically consolidated "perfect" specimens than that for the "in situ" specimens.

Kirkpatrick and Khan (1984) investigated the influence of stress release caused by "perfect" sampling on the undrained stress-strain behaviour of normally consolidated Kaolin (PI = 30) and Illite (PI = 40). The tests on both clays showed that, compared to "in situ" soil, "perfect" samples suffered considerable loss in strength, increase in failure strain, and produced appreciably different effective stress paths to failure. Kirkpatrick and Khan (1984) reported that 56% and 38% reduction of the undrained strength, 175% and 250% increase in strain at peak strength, 24% and 22% decrease in the initial stiffness obtained for normally consolidated clays of Kaolin and Illite respectively due to "perfect" sampling. The strength losses were more acute in the less plastic Kaolin compared with the more plastic and less permeable Illite. Kirkpatrick et al. (1986) also reported that due to perfect sampling, undrained strength decreased to 48%, 38% and 14%, strain at peak strength increased to 75%, 150% and 10%, and initial stiffness decreased to 68%, 73%, 6% for the overconsolidated clays of Kaolin (OCR = 2), Illite (OCR = 2.7) and Illite (OCR = 5), respectively. Kirkpatrick et al. (1986) reported from Fig. 2.31 that the undrained strength (s_u) increased with increasing OCR for "perfect" samples. In Fig. 2.31 s_{up} and s_{ui} are undrained strength of "perfect" and "in situ" samples respectively. This finding contrast with that reported by Rahman (2000). Rahman (2000) found the decrease of s_u with increasing OCR for "perfect" samples as shown in Fig. 2.32.

The effects of "perfect" sampling on low plasticity clays (LL = 32, PI = 17) have been discussed by Hight et al. (1985). Fig. 2.33 presents the undrained behaviour of a young K_0-consolidated low plasticity clay from North Sea when sheared at two OCRs (=1 and 7) from either in situ conditions or those resulting from perfect sampling. It is evident from Fig. 2.33 that perfect sampling greatly reduces the initial mean effective stresses. Peak undrained strength of both normally and overconsolidated samples were reduced due to "perfect" sampling. The ultimate strength is little affected but the overall stress-strain behaviour is modified considerably. It is apparent from Fig. 2.33 that the effective stress changes during "perfect" sampling are completely different from the two stress histories considered. The effect of stress history on the "perfect" sampling stress path and on the changes in effective stress was reported by Hight and Burland (1990) for the case of a low plasticity clay. This is shown in Fig. 2.34. It can be seen from Fig. 2.34 that the effective stress changes reduce as the OCR increases; for an OCR of 4, there is no change in effective stress; for the heavily overconsolidated clay, however, there is a slight increase in average effective stress.

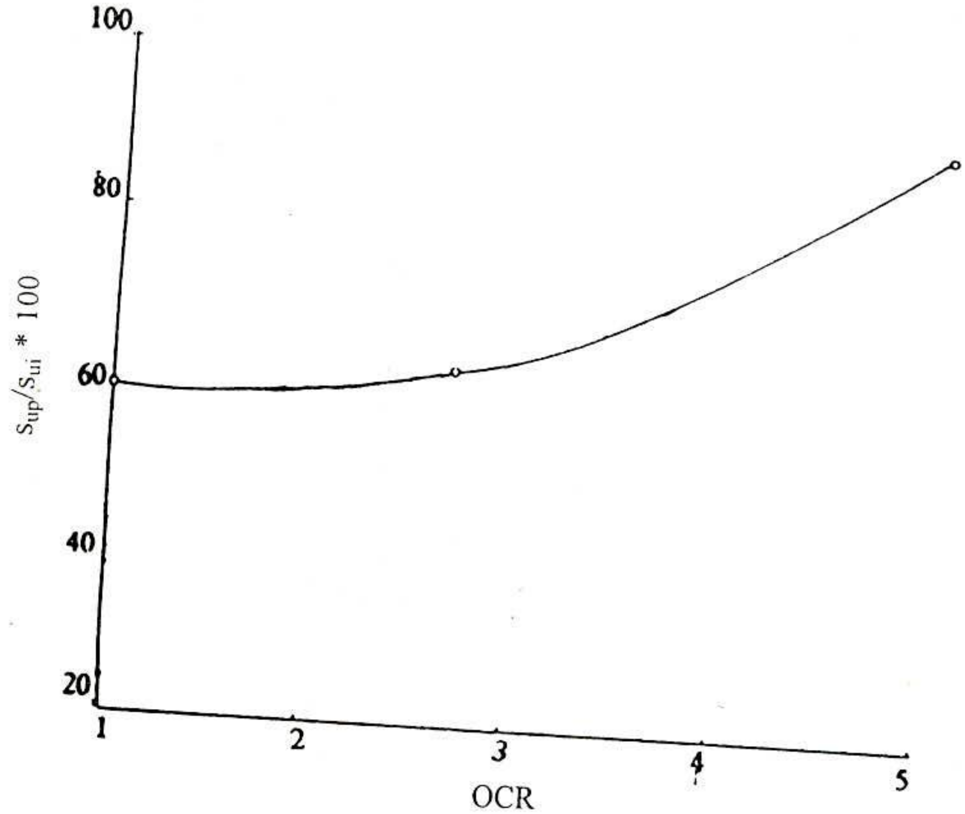


Fig. 2.31 Variation of the Ratio of s_{up}/s_{ui} with OCR of "Perfect" Samples of Illite (after Kirkpatrick et al., 1986)

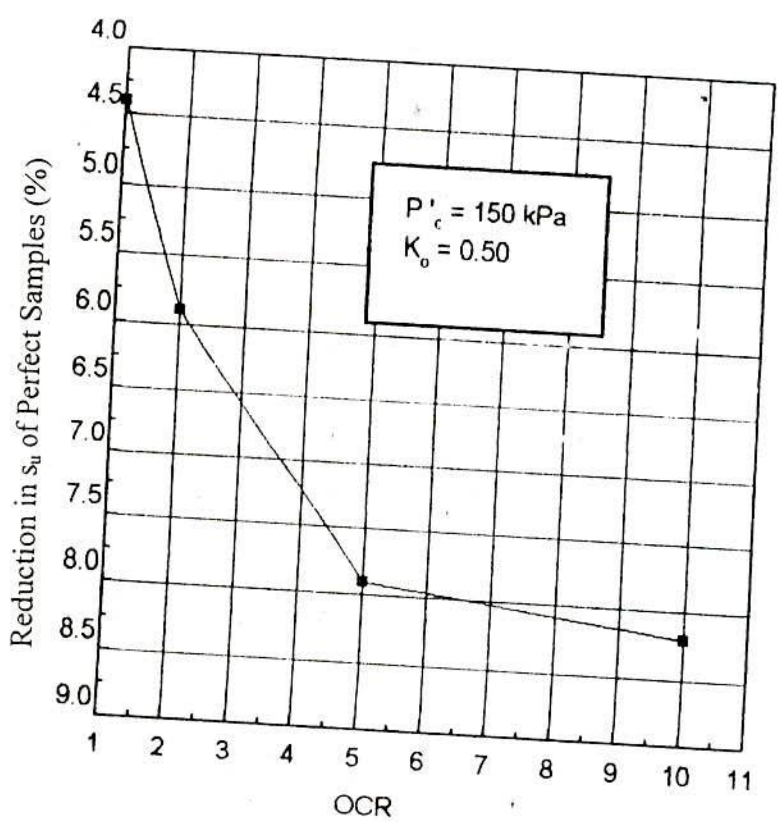
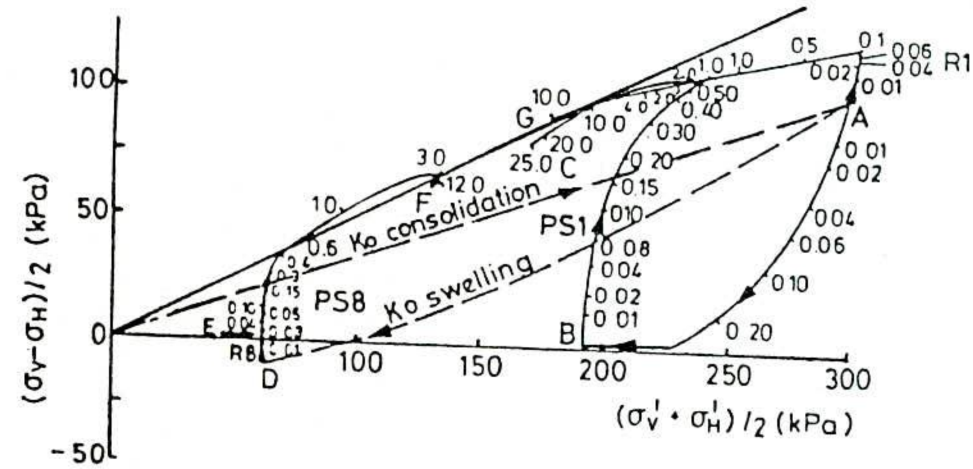
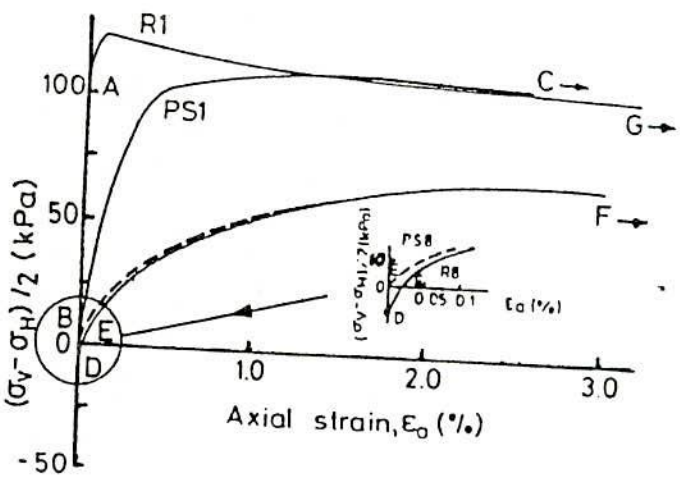


Fig. 2.32 Variation of % Reduction in Shear Strength With OCR for Perfect Samples Reduction in s_u of Perfect Samples (%) (after Rahman, 2000)



(a)



(b)

Normally consolidated soil:
AG - "in-situ" path, AB - "perfect" sampling path
BC - triaxial compression after "perfect" sampling

Overconsolidated soil (OCR = 7.4):
DEF - "in-situ" path, DE - "perfect" sampling path
EF - triaxial compression after "perfect" sampling

Fig. 2.33 "Perfect" Sampling Behavior of Normally Consolidated and Heavily Overconsolidated North Sea Clays: (a) Stress Paths (b) Stress-Strain Curves (after Hight et al, 1985)

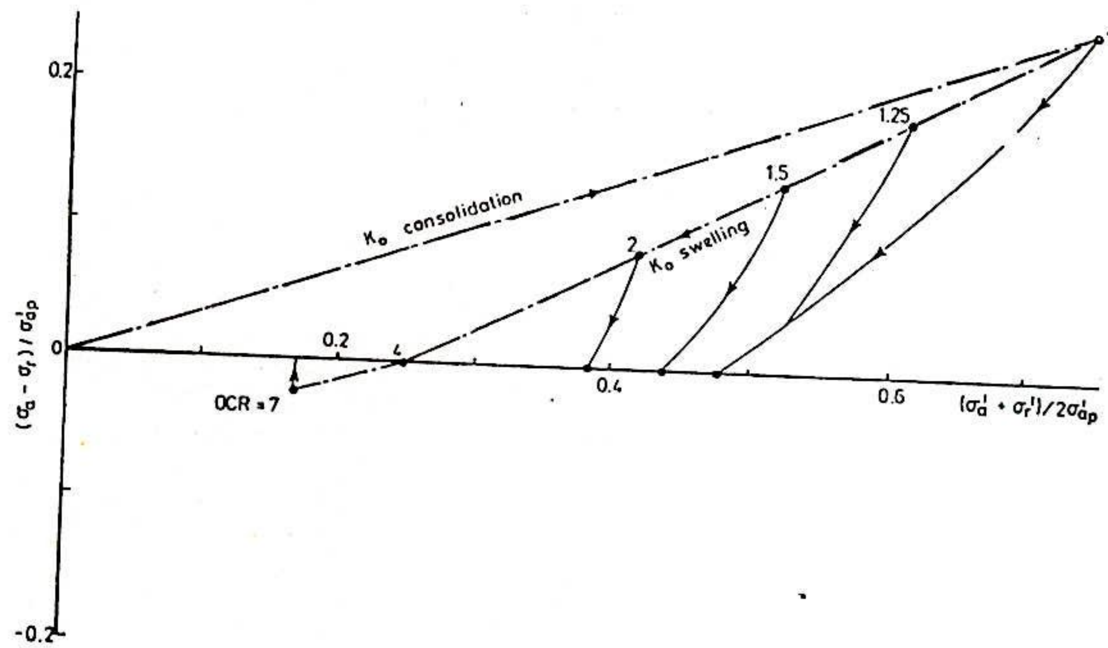


Fig. 2.34 Effect of Stress History on "Perfect" Sampling Stress Path for Low Plasticity Clay (after Hight and Burland, 1990)

The effect of "perfect" sampling disturbance on overconsolidated (OCR = 2.5) plastic Drammen clay (PI = 27) has been reported by Lacasse and Berre (1988). Lacasse and Berre (1988) reported about 11% decrease in undrained shear resistance in compression. "Perfect" samples, however, when consolidated to maximum vertical stress of the undisturbed specimen and then unloaded to the appropriate OCR of 2.5 provided 3% increase in shear resistance in compression. Kubba (1981) reported a decrease in undrained strength of 5 to 11% due to "perfect" sampling for normally consolidated samples of Kaolin.

Apart from leading to a decrease in strength, "perfect" sampling has a marked influence on pore pressure responses as reported by Seed et al. (1964), Noorany and Seed (1965) and Ladd and Varallyay (1965). The pore pressure parameter A at failure was found to decrease by as much as 50% for specimens subjected to "perfect" sampling. Ladd and Varallyay (1965) also observed a slight reduction in stiffness and a large increase in axial strain required to mobilize the peak shearing resistance. Atkinson and Kubba (1981) also reported considerably lower stiffness for anisotropically consolidated "perfect" specimens than that for the "in situ" specimens.

Siddique and Farooq (1996) investigated the effects of "perfect" sampling disturbance on undrained shear properties of reconstituted normally consolidated coastal soils. Reductions in undrained strength (s_u) and pore pressure parameter A at peak deviator stress, A_p while increase in axial strain at peak deviator stress (ε_p), initial stiffness (E_i) and secant stiffness at half the peak deviator stress (E_{50}) have been reported due to "perfect" sampling. Siddique and Farooq (1996) reported that because of the relief of total stress, undrained strength of the samples from Patenga (LL = 44, PI = 18) and Kumira (LL = 57, PI = 33) decreased by 13% and 7% respectively while ε_p increased by 32% and 24% respectively for the samples of Patenga and Kumira. The normalized stiffness E_i/σ'_{vc} have been increased by about 40% and 47% in samples from Patenga and Kumira respectively. The value of A_p reduced considerably by about 68% and 83% for Patenga and Kumira respectively because of disturbance due to total stress relief. Fig. 2.35 and Fig. 2.36 show the comparison of pore pressure response and the stress paths of "perfect" and "in situ" samples of two reconstituted coastal soils investigated by Siddique and Farooq (1996). It can be seen from Fig. 2.36 that the "perfect" samples adopted stress paths completely different from the "in situ" samples. Effective stress paths of "perfect" samples are similar to those of overconsolidated samples. In Fig, 2.35 and Fig. 2.36, PI and KI means "in situ" samples of Patenga and Kumira, respectively, and PP and KP means "perfect" samples of Patenga and Kumira, respectively.

The effect of "perfect" sampling disturbance on undrained shear properties of reconstituted normally consolidated soft Dhaka clay (LL = 45, PI = 23) has been investigated by Siddique and Sarker (1997). Siddique and Sarker (1997) reported deviator stress of the "perfect" sample is lower than that of the "in situ" sample resulting in reduction in undrained strength. Siddique and Rahman (2000) also reported that ε_p, initial stiffness and secant stiffness increased due to "perfect" sampling. The value of A_p, however, reduced considerably because of "perfect" sampling in Dhaka clay. Fig. 2.37 and Fig. 2.38 show the variation of deviator stress and pore pressure response with the axial strain for "in situ" and "perfect" samples of normally consolidated and overconsolidated Dhaka clay. From Figs. 2.37 and 2.38 it can be concluded that the reduction in s_u increase, increase in value of ε_p increases, increase in the value of E_i and E_{50} reduces, and reduction in the value of A_p reduces with the increase of OCR.

A summary of the effects of "perfect" sampling on some engineering properties of the soils as reported by a number of investigators is presented in Table 2.11.

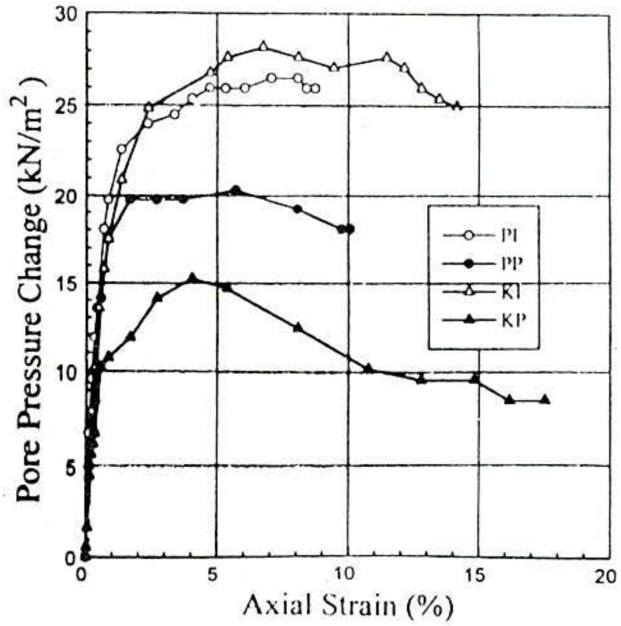


Fig. 2.35 Comparison of Pore Pressure Response of In Situ and Perfect Samples from Patenga and Kumira (after Siddique and Farooq, 1996)

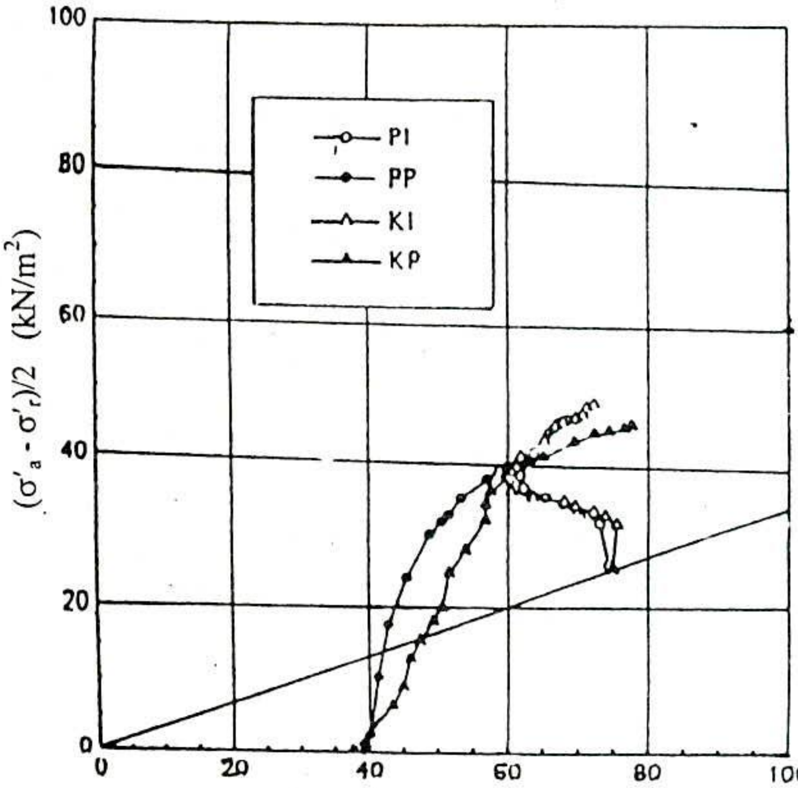


Fig. 2.36 Effective Stress Paths of "Perfect" and "In Situ" Samples from Patenga and Kumira (after Siddique and Farooq, 1996).

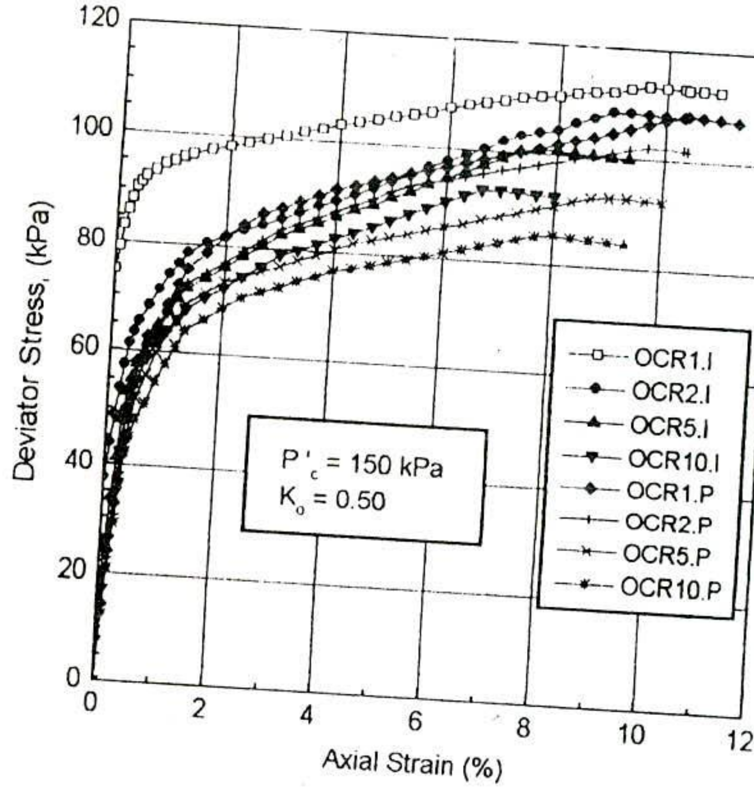


Fig. 2.37 Daviator Stress vs. Axial Strain (%) Plots for "in Situ" and "Perfect" Samples of Normally Consolidated and Overconsolidated Dhaka Clay (after Rahman, 2000)

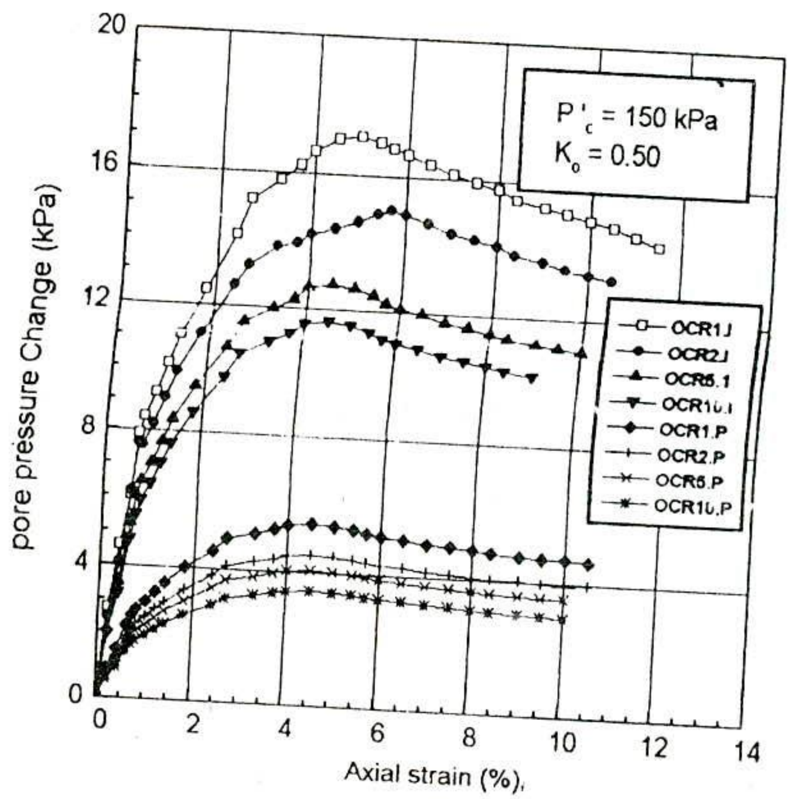


Fig. 2.38 Pore Pressure Change vs. Axial Strain (%) Plots for "in Situ" and "Perfect" Samples of Normally Consolidated and Overconsolidated Dhaka Clay (after Rahman, 2000)

Table 2.11 Summary of the Effects of "Perfect" Sampling on Engineering Properties of Normally Consolidated and Overconsolidated Soils

Soil	Index Values	OCR	Ratio of s_u	Ratio of ε_p	Ratio of E_i	Ratio of E_{50}	Ratio of A_p	Reference
Weald clay	LL = 46 PI = 24	1.0	0.98	1.29	-	-	-	Skempton and Sowa (1963)
		2.0	1.03	0.88	-	-	-	
		14.0	1.08	-	-	-	-	
Soft clay	LL = 88 PI = 45	1.0	0.95	1.05	0.9	-	-	Noorany and Seed (1965)
Boston clay	LL = 33 PI = 15	1.0	0.93	2.50	-	-	-	Ladd and Varallyay (1965)
Kaolin	LL = 55 PI = 22	1.0	0.81	-	--	-	-	Davis and Poulos (1967)
Kaolin	PI = 30	1.0	0.44	2.75	0.76	-	-	Kirkpatrick and Khan (1984)
Illite	PI = 30	1.0	0.58	3.50	0.78	-	-	
North Sea clay	LL = 32 PI = 17	1.0	0.72	8.00	1.19	-	-	Hight et al. (1985)
		7.4	0.96	1.00	0.47	-	-	
Patenga soil+	LL = 44 PI = 18	1.0	0.87	1.32	1.40		0.32	Siddique and Farooq (1996)
Kumira soil+	LL = 57 PI = 33	1.0	0.93	1.24	1.47		0.17	
Dhaka clay	LL = 45 PI =23	1.0	0.97	1.16	1.67	1.40	0.36	Siddique and Sarker (1997)
Dhaka clay	LL = 47 PI = 26	1.0	0.96	1.07	1.14	1.19	0.10	Rahman (2000)
		2.0	0.94	1.09	1.09	1.11	0.21	
		5.0	0.92	1.2	1.08	1.11	0.25	

All ratios are of ("perfect"/"in situ") samples

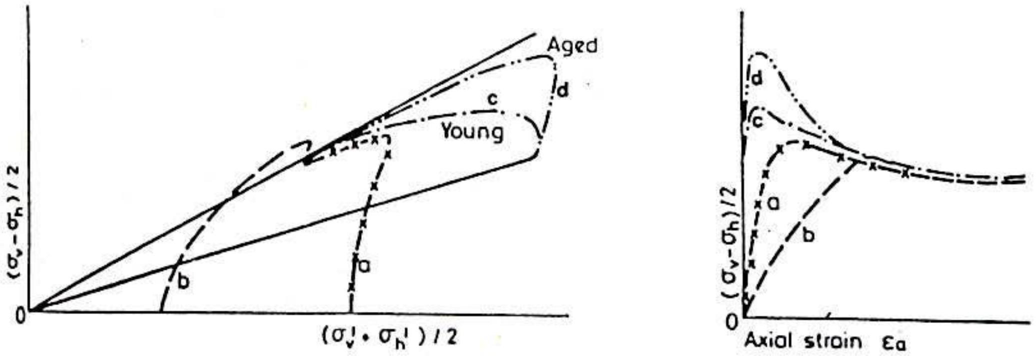
+ *Reconstituted Coastal soils*

2.10.2 Tube Sampling

The response that could be anticipated in normally consolidated soil after tube sampling and extrusion has been investigated by Hight et al. (1987) for young low to medium plastic clays. It has been found that the undrained stress path and stress-strain curves of "tube" sample are markedly different from those of "perfect" and "in situ" samples. Hight et al. (1987) also reported the behaviour of three "tube" samples taken from the sea bed in the North Sea. The estimated OCR's of the first two samples were 1.1 and the OCR of the third sample was greater than 50. The initial mean effective stresses of the normally consolidated samples were below those estimated in situ, but the heavily overconsolidated sample showed a large overall increase in initial mean effective stress. None of the three intact tests provided a satisfactory model for the in situ behaviour. Fig. 2.39 shows the unconsolidated undrained stress paths and stress-strain curves for "perfect", tube and "in situ" samples. From Fig. 2.39 it can be seen that the undrained stress path and stress-strain curve of tube sample are markedly different from those of "perfect" and "in situ" samples.

The effects of tube sampling disturbance on undrained shear properties of reconstituted normally consolidated samples of Dhaka clay (Siddique and Sarker, 1995, Siddique and Rahman, 2000) and three coastal soils (Siddique et al., 2000) are summarized in Table 2.12. It can be seen from Table 2.12 that disturbances due to tube sampling cause the following effects:

- reduction in initial effective stress (σ'_i);
- reduction in undrained shear strength (s_u);
- reduction in initial stiffness (E_i) and secant stiffness at half the peak deviator stress (E_{50});
- reduction in and Skempton's pore pressure parameter A at peak deviator stress (A_p); and
- increase in axial strain at peak deviator stress (ε_p).



a = UU stress path and stress-strain curve for "perfect" sample
b = UU stress path and stress-strain curve for tube sample
c = UU stress path and stress-strain curve for "in-situ" young sample
d = UU stress path and stress-strain curve for "in-situ" aged sample

Fig. 2.39 Unconsolidated Undrained (UU) Stress Paths and Stress-Strain Curves for "Perfect", Tube and "in Situ" Samples (Height, et al., 1987)

Table 2.12 Effects of Tube Sampling Disturbance on Mechanical Properties of Reconstituted Normally Consolidated Regional Soils of Bangladesh

Soil location	Reduction in σ'_i (%)	Change in properties compared with "in situ" sample					Reference
		Reduction in s_u (%)	Increase in ε_p (%)	Reduction in E_i (%)	Reduction in E_{50}(%)	Reduction in A_p (%)	
Dhaka LL = 45 PI = 23	18.5-33.8	17 - 35	35 - 81	11 - 49	1 - 34	106 -111	Siddique and Sarker (1997)
Patenga LL = 44 PI = 18	8.3-33.5	42 - 55	19 - 78	34 - 74	---	115 -123	Siddique et al. (2000)
Fakirhat LL = 43 PI = 22	7.3-30.0	34 - 55	4 - 32	31 - 70	---	101 -115	
Kumira LL = 57 PI = 33	5.7-22.7	34 - 56	4 - 13	31 - 76	---	102 -117	
Dhaka LL = 47 PI = 26	9.0-26.2	21 – 41	13– 58	32– 62	33– 65	146- 147	Siddique and Rahman (2000)

Figs. 2.40 (a) and (b) show the effective stress paths of samples of reconstituted normally consolidated Dhaka clay and coastal soil from Kumira respectively. It can be seen from Figs. 2.40 (a) and (b) that, "tube" samples adopted stress paths completely different from the normally consolidated "in situ" samples. Effective stress paths of "tube" samples are similar to those for overconsolidated samples. Fig. 2.41(a) and (b) show the variation of pore pressure change with axial strain for "tube" and "in situ" samples of a coastal soil and Dhaka clay, respectively. It can be seen from Fig. 2.41 that compared with the "in situ" sample, the values of pore pressure changes of the "tube" samples are considerably less.

Apart from stress-strain behaviour, tube sampling also affects the compressibility characteristics of clays. The effect of tube sampling disturbance on consolidation parameters of soft clay samples was examined by Bromham (1971), The major effect of sampling disturbance was to produce low values of coefficient of volume compressibility, m_v, especially near the overburden pressure. Values of m_v calculated from the reconstructed field curve were considerably higher than those obtained directly from the laboratory consolidation test. Coefficient of consolidation, c_v for the least disturbed specimens, with disturbance factor of 15 to 20, were less than the extrapolated field values by a factor of 2 to 5. Hight et al. (1987) also reported higher volumetric strains for tube samples of lightly overconsolidated Magnus clay (OCR = 1.15) than those for "in situ" samples. For the tube samples the values of m_v were considerably smaller than the "in situ" sample. Compression indices, C_c were, however, the same for tube and "in situ" samples.

2.10.3 Block Sampling

Block sampling can be modelled in the laboratory by releasing and trimming blocks of soil from large oedometer samples. Hight et al. (1985) demonstrated the behaviour of specimens of Lower Cromer Till, another low plasticity clay, due to block sampling. The results of unconsolidated undrained triaxial compression tests are presented in Fig. 2.42. The specimens were cut from the blocks having different stress histories (OCRs of 1, 2, 4, 7 and 80). It can be seen that the effect of block sampling largely obliterates the important effects of stress history on in situ behaviour. The specimens tend towards similar initial mean effective stress levels and, as a consequence, show similar behaviour. As could have been anticipated from the results of "perfect" sampling, peak strengths and undrained brittleness are reduced in the normally and lightly overconsolidated soil. The effect of block sampling on the stress-strain behaviour was also assessed by Hight et al. (1985). They reported results from two similar unconsolidated undrained tests on specimens of North Sea clay cut from reconstituted blocks (OCR = 2). Both the initial stiffness and degree of non-linearity were reduced.

The quality of block samples has been compared with that of tube samples by several workers. Raymond et al. (1971) studied the behaviour of sensitive Leda clay sampled by six different sampling methods to assess the significance of the different features in the design of samplers. As an example is shown in Fig. 2.43, demonstrating qualitatively the differences in stress-strain relationships of block and tube samples and the qualitative similarities between different tube samplers. Of the five different tube samplers used, the samplers causing least disturbance were, in order: (a) the 125 mm Osterberg hydraulic piston sampler; (b) the SGI 50 mm standard piston sampler; (c) the 50 mm thin-walled Shelby tube piston sampler with sharp outside cutting edge; (d) the 50 mm thin-walled Shelby tube piston sampler with normal cutting edge; and (e) the 50 mm thin-walled open-drive Shelby tube.

The influence of sampling methods on some soil properties for two sensitive slightly overconsolidated clays was reported by Milovic (1971a). Clay samples were obtained by Shelby tubes and Norwegian piston sampler. The area ratio and inside clearance ratio for both Shelby tube and piston sampler were respectively 12 ± 1.5% and 0.8 ± 0.1%. Cubic blocks were cut by hand. The unconfined compressive strength, the secant modulus, shear strength parameters and consolidation parameters of these sensitive clays, determined on Shelby and Piston specimens, were systematically lower than those obtained for Blocks.

La Rochelle and Lefebvre (1971) reported that for sensitive Champlain Clay, undrained shear strengths measured on samples obtained by NGI 54 sampler (AR = 10%, ICR = 1%) were 50 to 60% of the value measured on block samples.

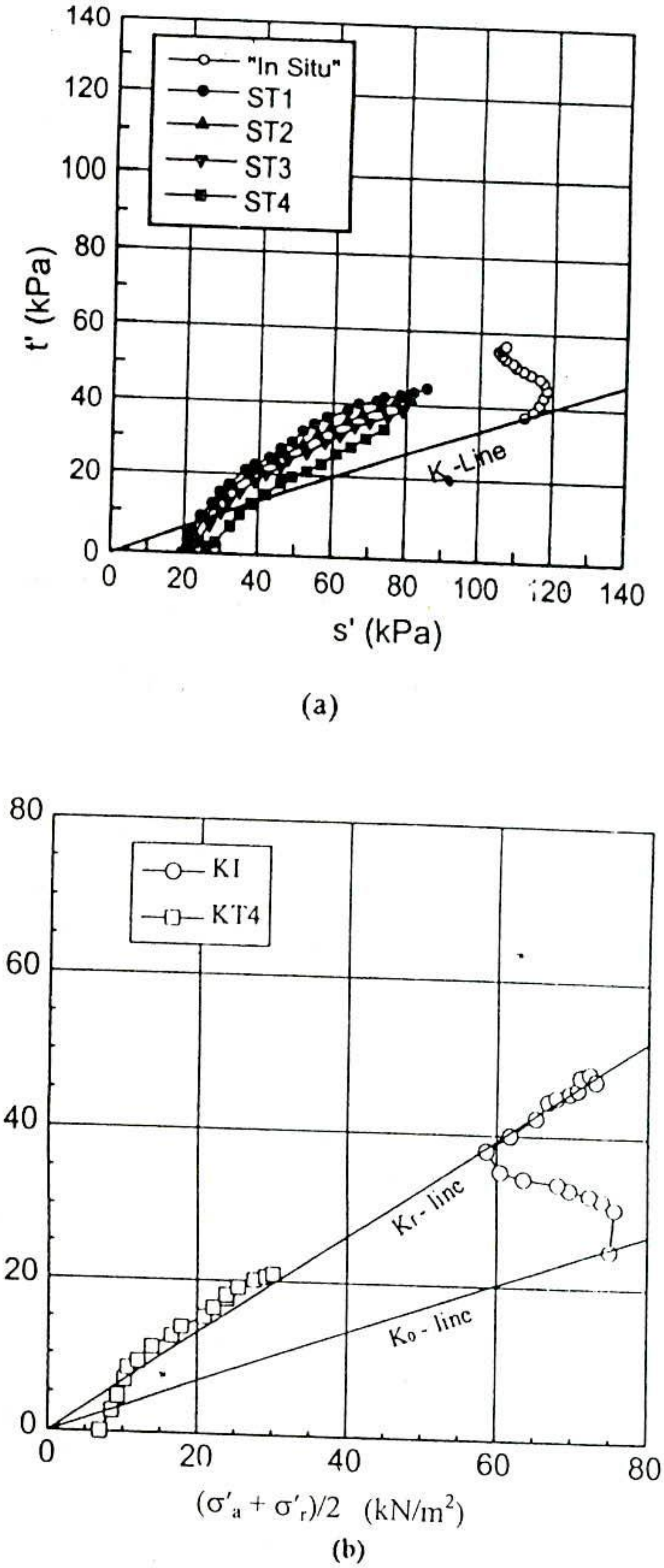
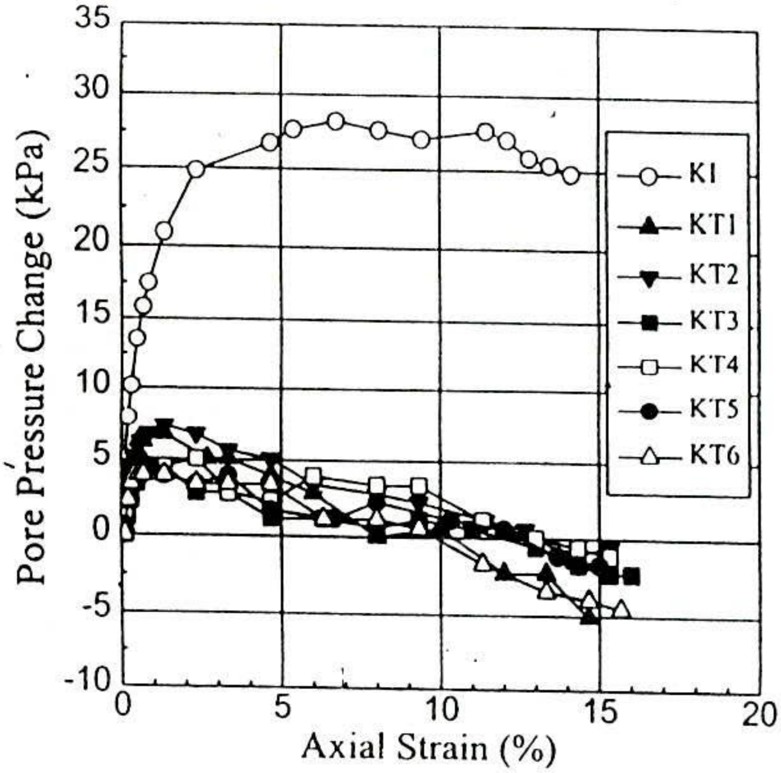
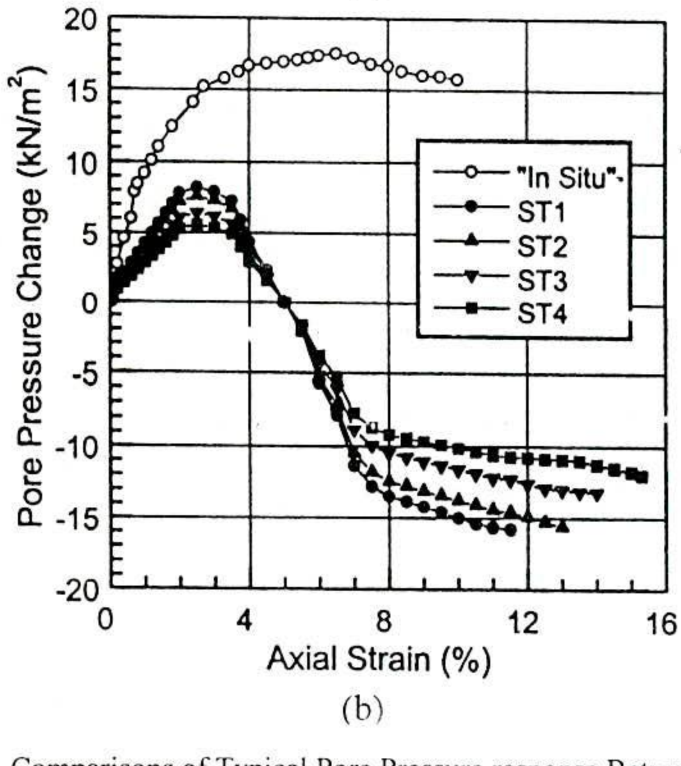


Fig. 2.40 Comparison of Typical Effective Stress Paths between "In Situ" and "Tube" Samples: (a) Dhaka Clay (After Siddique and Rahman, 2000), (b) Kumira Soil (after Siddique et al., 2000)



(a)



(b)

Fig. 2.41 Comparisons of Typical Pore Pressure response Between the "in Situ" and "Tube" Samples of (a) Kumira Soil (after Siddique et al, 2000) (b) Dhaka Clay (after Siddique and Rahman, 2000)

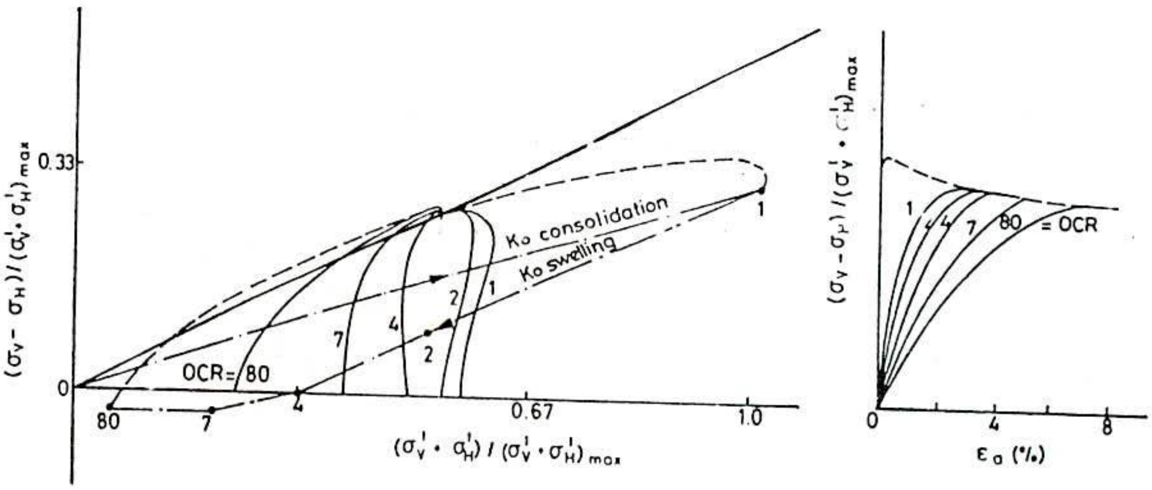


Fig. 2.42 Stress-Strain Characteristics of Block Samples of reconstituted Lower Cromer Till in Unconsolidated Undrained Triaxial Compression Tests. (after Hight et al., 1985)

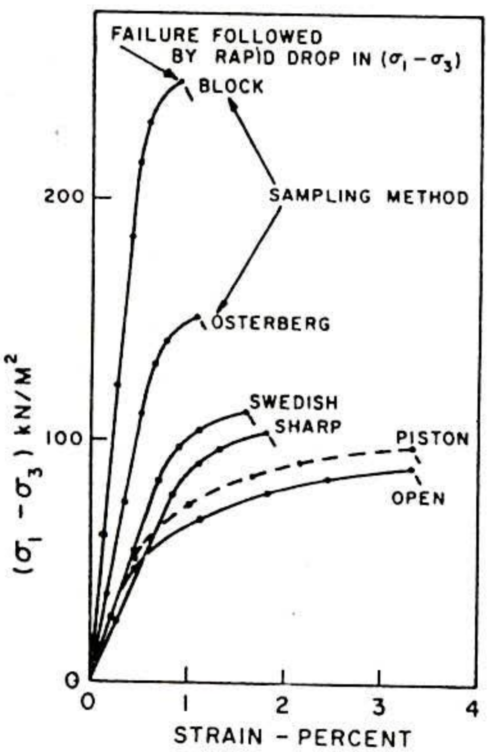


Fig. 2.43 Stress-Strain Curves for Tube and Block Samples (after Raymond, 1971)

Milovic (1971b) also studied the effect of sampling methods on some loses properties. Loess samples (LL = 41, PI = 18) were obtained by Shelby tubes (AR = 12%, ICR = 0.8%) and also from blocks, cut by hand. The unconfined compression tests and consolidation tests were carried out on both types of specimens. The unconfined compressive strength, Young's modulus, compressibility modulus and preconsolidation pressure obtained on Shelby specimens were considerably higher than those obtained on Block specimens. This was attributed to higher initial density of Shelby specimens. It is well known that the initial density affects the elastic, shear and consolidation properties of loess.

McManis and Arman (1979) investigated the effect of sampling on the properties of undisturbed soil specimens. The soil types studied were soft organic silty clays and stiff, fissured Pleistocene clays. Sampling was performed using 76 mm and 127 mm thin-walled open-drive tubes and by hand cutting of block samples. They also observed that specimens cut from block provided higher undrained strengths than the tube specimens.

Lacasse et al (1985) compared the behaviour of block samples of Norwegian marine clays with the behaviour of 95 mm tube samplers. The block samples were taken with the University of Sherbrooke cylindrical block sampler for soft sensitive clays (Lefebvre and Poulin, 1979). Using a series of rotating blades, this sampler carves out a block of soil, 300 mm dia. by 350 mm high, at the base of a mud-filled hole. On completion of the carving, blades fan out to slice through the base of the block and these blades support the sample as it is raised from the borehole. With this sampler block samples can be obtained at much greater depths than in an open trench. During sampling with the Sherbrooke sampler, the borehole is kept full of bentonite mud to reduce drastically the stress relief. In addition to allowing block sampling from the surface, the method provides samples of equivalent or better quality than conventional block samples (Lefebvre and Poulin, 1979). The tube samples were obtained with the NGI 95 mm fixed piston sampler (AR = 14%, ICR = 1.4%, outside cutting edge taper angle = 10^{o}). Two quick clays of low plasticity and one sensitive clay of high plasticity were sampled. The laboratory test results were compared in terms of preconsolidation pressure, oedometer curves, and stress-strain-strength behaviour from unconfined compression, triaxial and direct simple shear tests. The quality of the block samples was superior to the quality of the samples obtained by 95 mm piston sampler. However, the degree of disturbance due to tube sampling varied for different types of clays. In case of lean quick clays, block sampling resulted in 30% higher undrained strength and 4 times higher Young's modulus. In case of the plastic sensitive clay, the block and 95 mm samples had similar characteristics. Only small differences were observed in the preconsolidation stress profiles derived from tests on both types of samples. The effect of sampling disturbance on the test result also varied with the type of test. The disturbance effect appeared smaller in tests offered large confinement. The effect of sampling disturbance was indeed the least in the oedometer test, intermediate in consolidated triaxial test and the largest in unconfined compression tests. The experience in the Norwegian clays demonsurated the ability of the University of Sherbrooke cylindrical block sampler to obtain samples of excellent quality, even at large depths (>10 m).

2.10.4 "Ideal" Sampling

"Ideal" sampling (Baligh et al., 1987) are modelled in the laboratory by consolidating samples anisotropically in the triaxial apparatus and then imposing predicted tube penetration strains, followed by undrained stress relief simulating "perfect" sampling. The effects of ideal tube sampling have been studied by different-researchers (Baligh et al., 1987; Hajj, 1990, Siddique, 1990; and Clayton, Hight and Hopper, 1992). In general, the effect of ideal sampling disturbances causes significant reduction in initial mean effective stress. It also found that undrained shear strength and stiffness decreases and axial strain at peak deviator stress increases due to ideal sampling disturbance.

New insights into tube sampling disturbance have been made possible using the Strain Path Method (Baligh 1985). Baligh (1985) used the Strain Path Method to predict the strains that would be set up by a "simple sampler" with external diameter (D_e) to thickness (t) ratio, i,e., D_e/t ratios varying from 10 to 40. Fig. 2.44 shows predicted strains for B/t values of 10, 20 and 40. For this particular tube geometry, Baligh (1985) predicted that strains excursions on the centerline of the sample would have maximum values in axial compression and extension of between 0.75% and 4.0%.

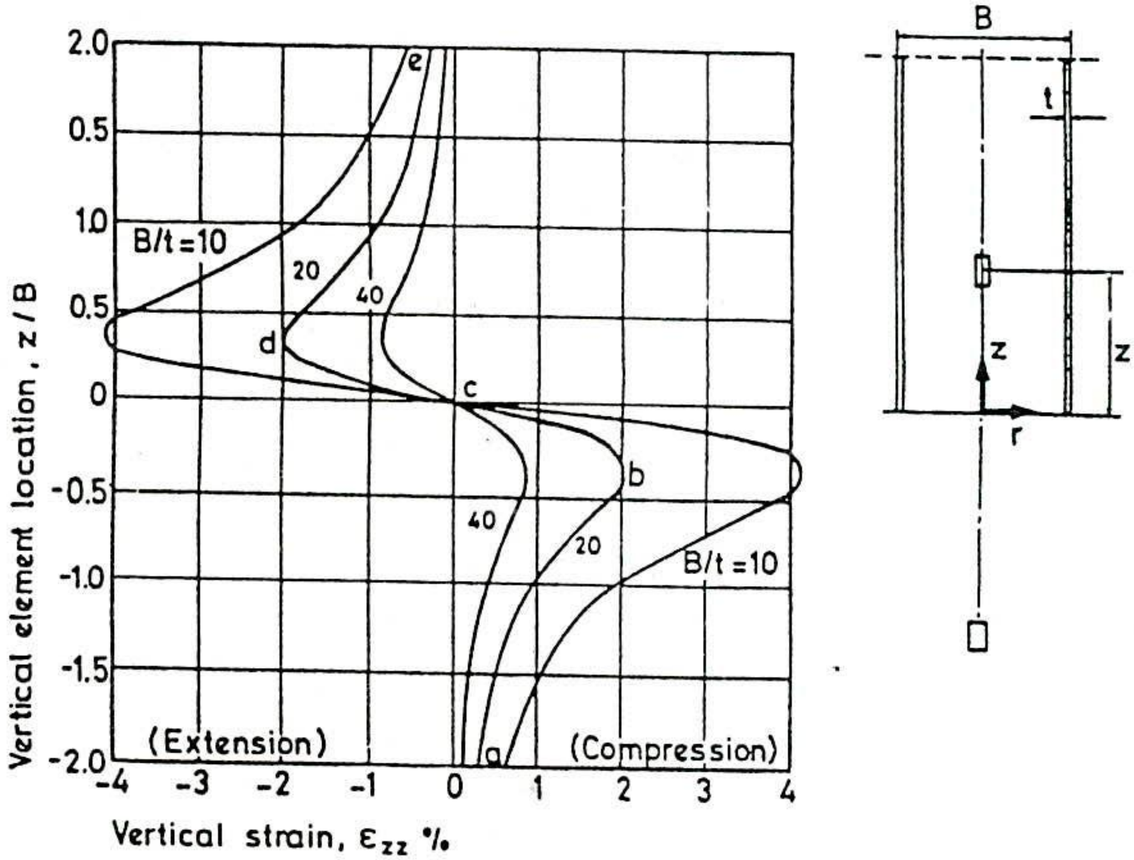


Fig. 2.44 Strain Paths for an Element on the Centerline of a Tube Sampler (after Baligh, 1985)

Baligh et al. (1987) proposed ideal sampling approach (ISA) as an extension to "perfect" sampling. Ideal sampling approach denotes an idealised method of incorporating the effects of tube penetration, sample retrieval to the surface and extrusion from the tube, but neglects all other types of disturbances, including operator dependent disturbances and water content changes in the soil. The proposed method for implementing ISA consists of the following steps:

(i) Estimation of tube penetration disturbances at the centerline of sampler using the Strain Path Method (Baligh, 1985).

(ii) Estimating the effects of sample retrieval and extrusion by assuming an idealised process of undrained stress relief from the (generally) anisotropic stress conditions in the tube to the final isotropic stress state of the sample before testing.

Step (ii) adopts the same simplification adopted by "perfect" sampling regarding sample retrieval and extrusion simulation. Therefore, the only difference between the proposed ISA and "perfect" sampling is the incorporation of tube penetration disturbances, i.e., step (i), and hence ISA is equivalent to "perfect" sampling when tube penetration disturbances are insignificant.

Hight (1986) pointed out the following effects due to ideal sampling:

(i) in the normally consolidated soil, the effective stresses are reduced;

(ii) in the heavily overconsolidated soil, the effective stresses are increased;

(iii) changes in pore pressure are different on the centerline and around the periphery so that a process of equalisation takes place.

The level of distortion which occurs around the periphery of tube samples is often apparent when such a sample is split to expose its fabric. Although the strain paths followed in this outer zone have been modelled in triaxial tests, it can be reasonably anticipated that:

(a) soil in an initially normally consolidated or lightly overconsolidated state will develop positive pore pressure increments.

(b) soil in a heavily overconsolidated state will develop negative pore pressure increments. Extrusion involves additional distortion.

Because of axial symmetry, the effects of the predicted tube sampling strains on soil properties can be examined by applying them to a triaxial specimen in the form of compression, followed by extension, test phases. In most of these studies, strain path excursions of amplitude ±1% have been applied to reconstituted clays. This is equivalent to imposing the strains predicted along the centerline of a simple sampler with $D_e/t = 40$ and an inside clearance ratio of about 1%. The reported results show marked changes in mean effective stress, peak undrained shear strength, strain at failure, and undrained stiffness between the "undisturbed" samples and samples to which tube penetration disturbances or ideal sampling disturbances have applied: Progressive destructuring and changes in the yield surface have been observed in natural (bonded) clays (Clayton et al., 1992).

Table 2.13 shows a summary of previous results on the effects of ideal sampling disturbance on undrained shear strength properties of normally consolidated and overconsolidated clays. It can be seen that although very large decreases in mean effective stress (p'_o) have been observed, particularly for normally consolidated reconstituted clays, the associated reductions in undrained shear strength (s_u) have not been particularly great. Indeed, it seems likely that if samples are reconsolidated to their effective stress before sampling, the decrease in void ratio may lead to an increase in s_u, as found by Hajj (1990). But the decreases in stiffness caused by a reduction in mean effective stress are likely to be high. Baligh et al. (1987) found 59% reduction in p'_o in normally consolidated reconstituted Boston Blue clay (LL = 42, PI = 20) while in slightly overconsolidated natural Bothkennar clay (LL = 76, PI = 42), Clayton et al., (1992) found that p'_o reduced by 43% due application of tube sampling strains of amplitude ±1%. In the natural overconsolidated Vallericca clay (LL = 53, PI = 31), and London clay (LL = 60, PI = 32), Georgiannou and Hight (1994) found that p'_o reduced by 10%. In reconstituted normally consolidated Speswhite kaolin (LL = 72, PI = 32). Hird and Hajj (1995) reported 50% to 60% reduction in p'_o while in the reconstituted normally consolidated London clay (LL = 69, PI = 45), Siddique et al. (1999) found 10% to 37% reduction in p'_o. Baligh et al. (1987) have reported a 21% reduction in undrained strength ratio (s_u/σ'_{vc}) for reconstituted Boston Blue clay due to application of tube sampling strains of amplitude ±1%. Wei et al. (1994) found a reduction in s_u of about 14% for normally consolidated reconstituted mixture of kaolin

(80%) and silty sand (20%). Siddique et al. (1999) found a reduction of 2% to 6% in s_u in reconstituted London clay. In Vallericca and London clays, Georgiannou and Hight (1994) found that s_u reduced by less than 5% while in Bothkennar clay, Clayton et al.(1992) reported that s_u reduced by 5%. Siddique et al. (1999) found that ε_p increased up to 127% due to application of tube sampling strains of amplitude ±1% in reconstituted London clay. Baligh et al. (1987) and Wei et al. (1994) also reported significant increase in ε_p, 27 times and 10 times, respectively. For strain path excursion of amplitude ±1%, Baligh et al. (1987) reported decrease in undrained modulus ratio, E_{50}/σ'_{vc} (E_{50} is the secant stiffness at half the peak deviator stress) of as much as 95%.

Lacasse and Berre (1988) also reported reductions in initial moduli for normally consolidated and overconsolidated specimens of Drammen clay due to application of equivalent tube sampling strains. For Bothkennar clay, Clayton et al. (1992) reported a reduction in normalized secant stiffness at 0.1% axial strain of between 30% and 61%, when the amplitude of the strain cycle was greater than ±0.5%. However, an increase in stiffness of 32% was found following a strain cycle of amplitude ±0.5%, which was attributed to reduction in water content during reconsolidation more than compensating for any damage to the structure due to disturbances during path cycles. In overconsolidated reconstituted Vallericca clay and London clays, Georgiannou and Hight (1994) have reported reductions of stiffness at 0.01% axial strain of 35% and 25%, respectively.

In reconstituted normally consolidated clayey sand, Hight and Georgiannou (1995) found minor effects on small stiffness due to application of tube sampling strains of amplitudes ±0.5% and ±1%. Siddique et al. (1999) reported that values of E_i , E_{50} and A_p reduced by 77%, 65% and 78% respectively, provided in reconstituted London clay due application of tube sampling strains of amplitude ±1%. For overconsolidated London clay (OCR = 3.7), Siddique et al. (1999) reported a reduction in mean effective stress (p'_o) 10.5% and reduction in undrained shear strength 6% while increase in strain at peak strength 56%.

Table 2.13 Effects of Ideal Sampling on Undrained Shear Properties of Normally Consolidated and Overconsolidated Clays

Soil type	Atterberg limits (%)		OCR	Change in properties (%)			Reference
	LL	PI		Reduction in p'_o	Reduction in s_u	Increase in ε_p	
Boston Blue clay	42	20	1.3	59	21	5 - 18	Baligh et al. (1987)
Lightly Drammen clay	-	27	2.5	-	0	-	Lacasse and Berre (1988)
Speswhite Kaolin	72	32	4	11	16	-	Hajj (1990)
Bothkennar Clay	76	42	1.4-1.6	43	½ - 10	35	Clayton et al. (1992)
OC Vallericca and London clays	53 60	31 32	- -	10	< 5	20	Georgiannou and Hight (1994)
Sandy Kaolin	-	-	1.0	-	14	7 - 10	Wei et al. (1994)
Speswhite Kaolin	72	32	1.0	50 - 60	-	10	Hird and Hajj (1995)
London Clay	69	45	1.0	10 - 37	2 - 7	30-313	Siddique et al. (1999)
London Clay	86	61	3.7	10.5	6	56	

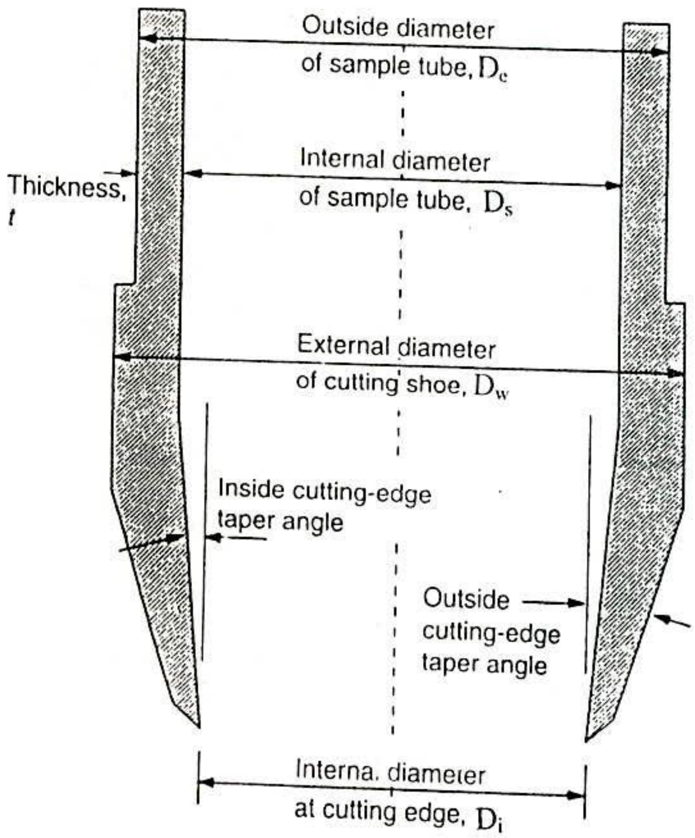
2.11 Sampler Design and its Effect on Sample Disturbance

The design of a sampler is one of the most important factors that should be considered for quality sampling. The amount of disturbance varies considerably depending upon the dimensions of the sampler and the precise geometry of the cutting shoe of the sampler (Hvorslev, 1949; Jakobson, 1954; Kallstenius, 1958; Kubba, 1981; Andresen, 1981; La Rochelle et al. 1981; Baligh et al., 1987; Siddique, 1990; Siddique and Clayton, 1995; Siddique and Sarker, 1996; Tanaka et al., 1996; Siddique and Clayton, 1998; Clayton et al., 1998; Siddique and Farooq, 1998; Clayton and Siddique, 1999, Siddique et al., 2000; Siddique and Rahman, 2000).

Hvorslev (1949) defined the geometry of a sampling tube in terms of its area ratio, length/diameter ratio, and inside clearance ratio, and the International Society for Soil Mechanics and Foundation Engineering (ISSMFE). Working party on Soil Sampling (1965) recognized the very significant importance of cutting-edge taper angle. More recently, Baligh (1985) has preferred to work in terms of diameter/thickness (D_e/t) ratio, rather than area ratio. Dimensions and terms used to define cutting shoe geometry a tube is shown in Fig. 2.45. Traditionally, when developing a new sampling device, a single (more or less) uniform soil would be sampled using a range of samplers, and performance would be judged by reference to the average and scatter of some index parameter such as unconsolidated undrained strength. Investigations of these sorts showed the importance of the details of cutting-shoe design, and subsequently led to the recommendations of the ISSMFE. Experience suggests that the most important factor governing sample disturbance is the combination of area ratio and cutting-edge taper angle.

2.11.1 Effect of Area Ratio and Cutting Edge Taper Angles

Area ratio is considered one of the critical parameters affecting the disturbance of soil during sampling. Increasing area ratio gives increased soil disturbance and remoulding. The penetration resistance of the sampler and the possibility of the entrance of excess soil also increase with increasing area ratio. For soft clays, area



Area Ratio, $AR = (D_w{}^2 - D_i{}^2) / D_i{}^2$
Inside Clearance Ratio, $ICR = (D_s - D_i) / D_i$
Outside Clearance Ratio, $OCr = (D_w - D_e) / D_e$

Fig. 2.45 Dimensions and Terms Used to Define Cutting Shoe Geometry of a Tube Sampler (after Hvorslev, 1949) ratio is kept to a minimum by employing thin-walled tubes. For composite samplers, the area ratio, however, is considerably higher. In these cases, sample disturbance is reduced by tapering the outside of the sampler tube very gradually from a sharp cutting edge (Hvorslev, 1949), recommended a maximum 10°, so that the full wall thickness is far removed from the point where the sample enters the tube.

Jakobson (1954) investigated the effect of sampler type on the shear strength of clay samples. Samples were collected using nine different types of samplers. These types differ from one another in area ratio, edge angle, inside clearance, drive velocity and other factors. Shear strength of samples was determined by carrying out the unconfined compression tests, the cone test and the laboratory vane test. It was found that an extremely small area ratio offers no special advantages and that the cutting edge taper angle does not seem to have any great influence. However, a very large area ratio or cutting edge taper angle is not recommendable. Kallstenius (1958) also studied the effect of area ratio and cutting edge taper angles on the shear strength of Swedish clays. He carried out tests similar to those reported by Jakobson (1954) on samples obtained using six types of piston samplers. Kallstenius (1958) recommended that a sampler ought to have a sharp edge and a small outside cutting edge taper angle (preferably less than 5°). Very large OCA has also been not recommended by Jakobson (1954) and Andresen (1981). The combined requirements for area ratio and cutting edge taper angle to cause low degrees of disturbance were proposed by the International Society for Soil Mechanics and Foundation Engineering's Sub-Committee on Problems and Practices of Soil Sampling (1965). For samplers of about 75 mm diameter, they suggested the following combinations of area ratio and cutting edge taper:

Area Ratio, AR	Outside Cutting Edge Angle (OCA) (°)
5	15
10	12
20	9
40	5
80	4

Clayton and Siddique (1999) reported that sampling tubes having good sampler geometries are available, which are capable of reducing tube sampling strains to acceptably low levels. Siddique and Clayton (1995) reported that the higher the tube sampling strains, the greater is the changes in the undrained soil parameters.

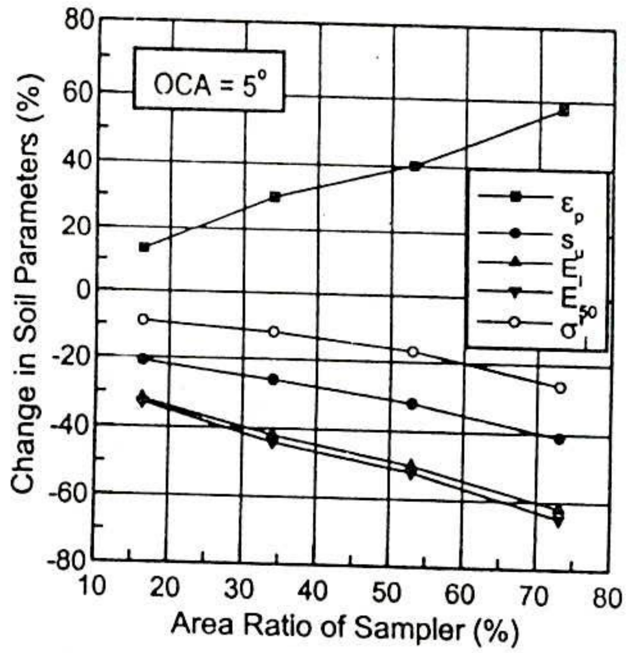
Siddique and Sarker (1996) investigated the effect of area ratio and outside cutting edge angle on undrained soil parameters of reconstituted Dhaka clay by carrying out undrained triaxial compression tests and one-dimensional consolidation tests on tube samples collected with samplers of varying area ratio and outside cutting edge angle. Siddique and Sarker (1996) reported that, for Dhaka clay, initial effective stress (σ'_i), undrained strength (s_u), initial stiffness (E_i) and secant stiffness (E_{50}) were reduced up to 41.5%, 35%, 49% and 34%, respectively, while axial strain at peak strength (ε_p) was increased up to 81% due to increase in area ratio from 10.8 to 55.2%. Siddique and Sarker (1996) also reported that σ'_i, s_u, E_i and E_{50} were reduced up to 36.9%, 32%, 41% and 31%, respectively while ε_p was increased up to 81% due to increase in OCA from 4^0 to 15^0 for Dhaka clay. They found that Skempton's pore pressure parameter, A at peak deviator stress, A_p reduced considerably as area ratio increased and the values of A_p of the "tube" samples of different area ratios are negative.

Siddique et al. (2000) reported that σ'_i, s_u, E_i reduced while ε_p increased due to increase in area ratio and OCA for three Chittagong coastal soils. Siddique et al. (2000) also found that A_p reduced considerably due to increase in area ratio and OCA. Siddique and Rahman (2000) also reported that increase in area ratio of sampler caused increasing reductions in σ'_i, s_u, E_i, E_{50} and increasing the area ratio of the sampler, however, caused an increase in ε_p. The results are shown in Table 2.14. Compared with "in situ" samples, it has been found that the values of A_p are decreased significantly with the increase of area ratio.

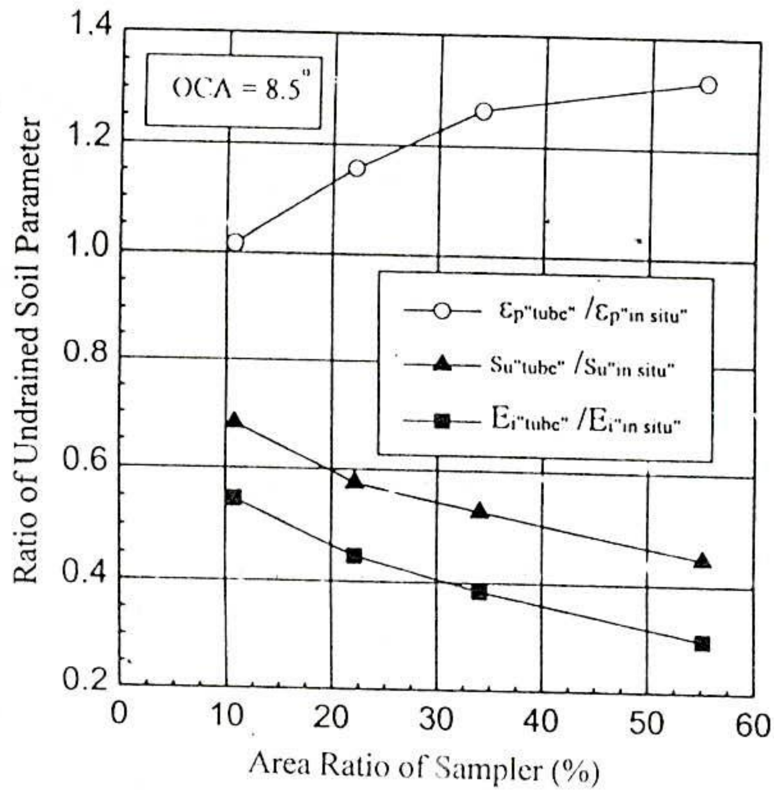
The effects of area ratio of samplers on undrained soil parameters for samples of Dhaka clay (Siddique and Rahman, 2000) and a coastal soil (Siddique et al., 2000) are presented in Figs. 2.46 (a) and 2.46 (b), respectively. It can be seen from Figs. 2.46 (a) and (b) that strength and stiffnesses decrease with the increase in area ratio while strain at peak strength increases with the increase in area ratio of "tube" samples. Increase in the degree of disturbance due to increasing area ratio and outside cutting edge angle has been reported by Kallstenius (1958), Andresen (1981) and has also been predicted numerically by Siddique (1990).

Table 2.14 Effects of Area Ratio (AR) and OCA on Sampling of Samples of Normally Consolidated Reconstituted Regional Soils of Bangladesh

Location of soil	Sampler dimensions			% Change in properties compared with "in situ" sample					Reference
	t mm	AR (%)	OCA (°)	Reduction of p'_o	Reduction of s_u	Increased of ε_p	Reduction of E_i	Reduction of E_{50}	
Dhaka LL = 45 PI = 23	1.5	10.8	8.5	18.5	17	35	11	1	Siddique and Sarker (1996)
	3.0	22.2	8.5	26.2	23	54	28	14	
	4.5	34.1	8.5	33.8	28	62	36	25	
	7.0	55.2	8.5	41.5	35	81	49	34	
	4.5	34.1	4	21.5	21	54	15	8	
	4.5	34.1	15	36.9	32	81	41	31	
Patenga LL = 44 PI = 18	1.5	10.8	8.5	8.3	32	19	34	--	Siddique et al. (2000)
	3.0	22.2	8.5	10.1	38	46	42	--	
	4.5	34.1	8.5	17.3	43	67	50	--	
	7.0	55.2	8.5	33.5	55	78	74	--	
	4.5	34.1	4	13.7	42	62	47	--	
	4.5	34.1	15	23.6	46	70	61	--	
Fakirhat LL = 43 PI = 22	1.5	10.8	8.5	7.3	34	4	31	--	
	3.0	22.2	8.5	11.8	47	6	42	--	
	4.5	34.1	8.5	16.4	47	26	62	--	
	7.0	55.2	8.5	30.0	55	32	70	--	
	4.5	34.1	4	14.5	46	21	56	--	
	4.5	34.1	15	20.9	48	27	69	--	
Kumira LL = 57 PI = 33	1.5	10.8	8.5	5.7	34	4	31	--	
	3.0	22.2	8.5	10.0	47	6	42	--	
	4.5	34.1	8.5	12.6	51	8	52	--	
	7.0	55.2	8.5	22.7	56	13	76	--	
	4.5	34.1	4	11.8	50	8	50	--	
	4.5	34.1	15	18.7	51	11	62	--	
Dhaka LL = 47 PI = 26	1.5	16.4	5	9.0	22	13.4	32	33	Siddique and Rahman (2000)
	3.0	34.1	5	11.8	26	29.9	42	45	
	4.5	53.0	5	16.8	32	40.2	50	52	
	6.0	73.1	5	26.2	43	57.7	62	65	



(a)



(b)

Fig. 2.46 Influence of Area Ratio of Sampler on Undrained Soil Parameters for Samples: (a) Dhaka Clay (after Siddique and Rahman, 2000), (b) Fakirhat Clay (after Siddique et al., 2000)

Siddique and Rahman (2000) and Siddique et al. (2000) investigated the effects of outside cutting edge angles (OCA) of samplers on undrained soil parameters for samples of Dhaka clay and a coastal soil. Fig. 2.47 shows the influence of OCA on undrained soil parameters. It can be seen from Figs. 2.47 (a) and (b) that strength and stiffnesses decrease with the increase in OCA while strain at peak strength increases with the increase in OCA of "tube" samples.

The effects of area ratio and outside cutting edge angles (OCA) on soil properties due to tube sampling for the regional clays of Bangladesh are also summarized in Table 2.14.

Clayton et al. (1998) implemented a method via a finite element approach to assess the influence of cutting shoe geometry (AR, OCA, ICR, cutting edge taper angles) on tube sampling disturbance. Degree of disturbance has been assessed in terms of predicted tube sampling strains in compression and extension at the centreline of soil sample. Figs. 2.48 and 2.49 show the variation of peak axial strain in compression with area ratio and outside cutting edge angle of sampler, respectively. It can be seen from Figs. 2.48 and 2.49 that the peak axial strains in compression increase with increasing area ratio and outside cutting edge angle of sampler. It can be seen from Fig. 2.48 that the imposed tube sampling strains predicted numerically and the predicted strains increased with increasing area ratio of the samplers. It can also be seen from Fig. 2.49 that the predicted strain increased with increasing outside cutting edge angle of the samplers.

Clayton et al. (1998) concluded that in order to restrict the degree of disturbance (peak axial strain in compression) to less than 1%, a sampler should have the following values of design parameters:

(i) The sampler should have a low area ratio, preferably not more than 10%.

(ii) The sampler should have a moderate inside cutting edge taper angle of 1 to 1.5°.

(iii) The sampler should have a small outside cutting edge taper angle, preferably not more 5°.

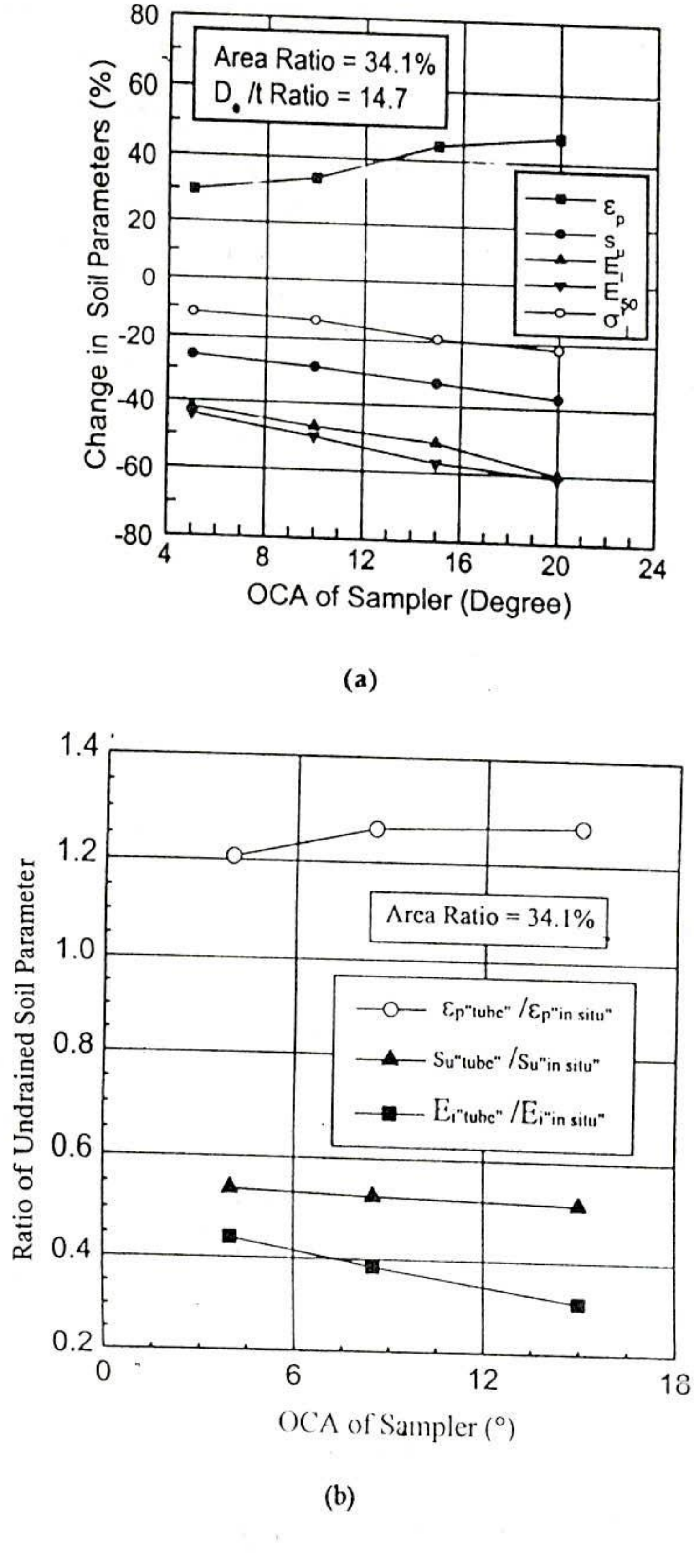


Fig. 2.47 Influence of Outside Cutting Edge Angle (OCA) of Sampler on Undrained Soil Parameters for Samples: (a) Dhaka Clay (after Siddique and Rahman, 2000), (b) Fakirhat Soil (after Siddique et al., 2000).

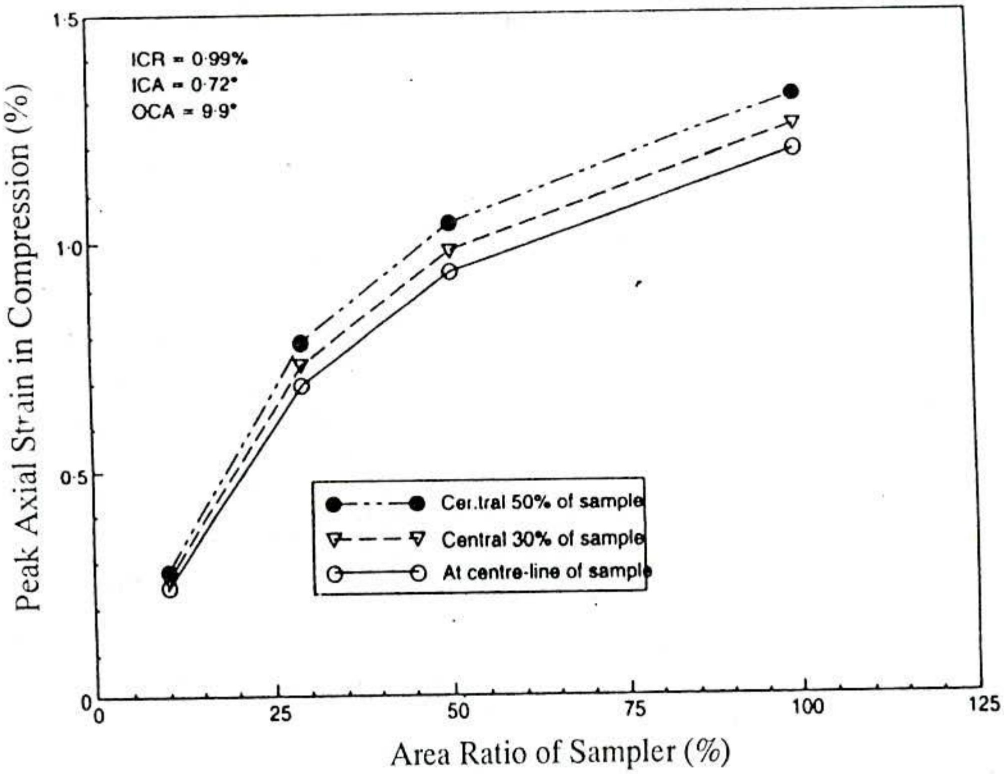


Fig. 2.48 Variation of Peak Axial Strain in Compression with Area Ratio of Samplers (after Clayton et al., 1998)

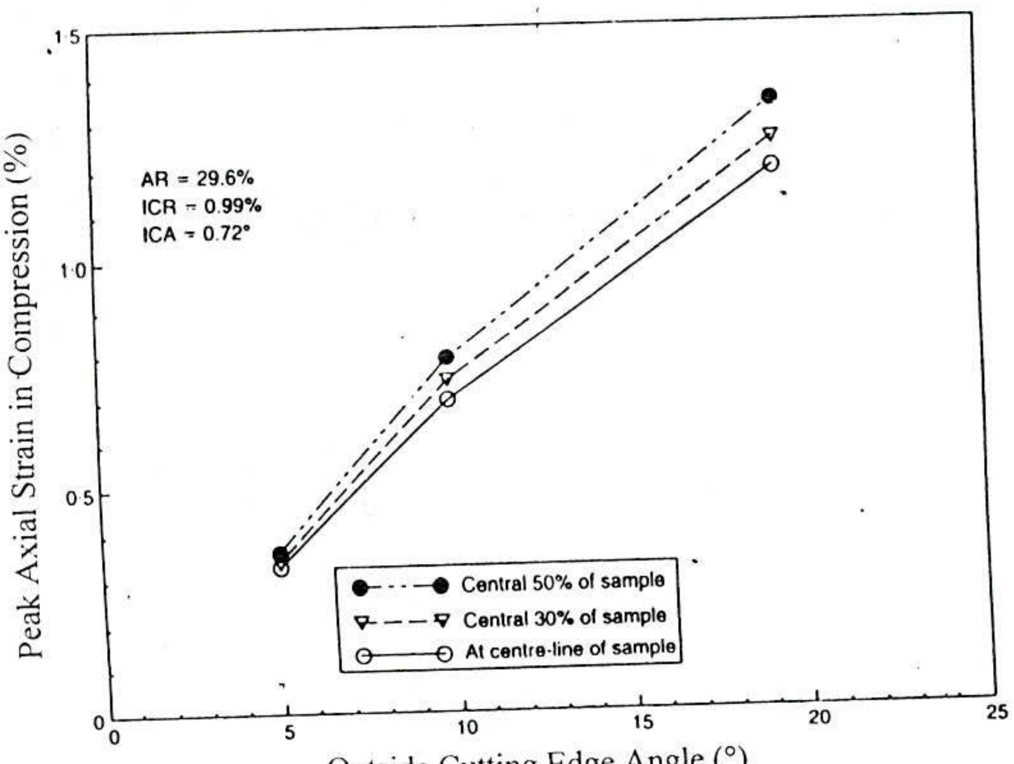


Fig. 2.49 Peak Axial Strain in Compression vs. Outside Cutting Edge Angle of Samplers (after Clayton et al., 1998).

2.11.2 Effect of Inside and Outside Clearance

Inside wall friction is one of the principal causes of disturbance of the sample (Hvorslev, 1949). One of the methods of reducing or eliminating wall friction between the soil and sampler is to provide inside clearance by making the inside diameter of the cutting edge, D_i, slightly smaller than the inside diameter of the sampler tube, D_s.

Inside clearance should be large enough to allow partial swelling and lateral stress reduction but it should not allow excessive soil swelling or loss of the sample when withdrawing from the sample tube. Hvorslev (1949) suggests an inside clearance ratio of 0.75 to 1.5% for long samplers and 0 to 0.5% for very short samplers. Kallstenius (1958) on the basis of Swedish clays sampled by six different piston samplers, also recommends that a sampler ought to have a moderate inside clearance. The clearance reduces the wall friction and probably counteracts to a certain extent the disturbance from displacement of soil caused by the edge and sampler wall during the driving operation. If the inside clearance and the edge angle are moderate, the above positive effects outweigh the disturbance caused by deformation when the sample tends to fill the clearance. The existence of inside clearance may have detrimental effects on sample disturbance as pointed out by La Rochelle et al. (1981). They reported from the work of Sarrailh (1975) that, in general, a "reshaped" 54 mm sampler without inside clearance seemed to give better results than a 54 mm sampler piston tube sampler with inside clearance. The improvement in strength was of the order of 20% or more and the tangent moduli were higher by 50-100%. Based on these observations, La Rochelle et al. (1981) developed a new sampler with no inside clearance for sampling in soft sensitive soils. This sampler, called the Laval Sampler, is of large diameter (208 mm inside diameter and 218 mm outside diameter) and also without a piston. The area ratio, D_e/t ratio and outside cutting edge taper angle of this sampler are 10%, 43.6 and 5° respectively. Clayton et al. (1998) reported that an increase in the inside clearance ratio causes an increase in extensive strain and a slight decrease in compressive strain ahead of the sample tube. Clayton et al. (1998) suggested that in order to restrict the degree of disturbance to less than 1%, a sampler should have a low inside clearance ratio of not more than 0.5%.

In order to reduce outside wall friction, samplers are often provided with outside clearance. An outside clearance ratio of a few percent may decrease the penetration resistance of samplers in cohesive soils. Although outside clearance increases the area ratio, a clearance of 2 to 3% can be advantageous in clay (Hvorslev, 1949).

2.11.3 Effect of Diameter and Length

Hvorslev (1949) stated that the amount of disturbance would be decreased with increasing diameter of the sample. Berre et al. (1969) observed that larger tube samples showed more constant behaviour than those from small tube samples. Oedometer tests carried out on samples of soft marine clay in Norway indicated that a 95 mm piston sampler (area ratio, AR = 14%, inside clearance ratio, ICR = 1.4%) gave less disturbance than a 54 mm piston sampler (AR = 12%, ICR = 1.3%).

An investigation of the difference in quality of samples taken with large diameter fixed piston samples and the 50 mm diameter Swedish Standard piston sampler (AR = 21%, ICR = 0.4%, outside cutting edge taper angle = 5°) was carried out by Holm and Holtz (1977). The large diameter piston samplers used were the 95 mm NGI (Norwegian Geotechnical Institute) research sampler (AR = 14%, ICR = 1.4%, outside cutting edge taper angle = 10°), the 127 mm Osterberg sampler (AR = 18%, ICR = 0.4%, outside cutting edge taper angle = 7°) and the 124 mm SGI (Swedish Geotechnical Institute) research sampler (AR = 27%, ICR = 1.2%, outside cutting edge taper angle = 5°). The investigation has shown that the results of oedometer tests on 50 mm samples are more scattered, supporting findings of Berre et al. (1969). The undrained modulus obtained from 50 mm samples have been found to be lower.

Bozozuk (1971) performed undrained triaxial tests on 1.4 inch diameter samples of soft marine clay. Samples were obtained by the 54 mm NGI piston sampler (AR = 11%, ICR =1%) and the 127 mm Osterberg piston sampler (AR = 6%, ICR = 0.42%). Test results showed that the undrained strengths of samples cut from 127 mm tube sample were higher than those cut from 54 mm tube samples. Samples cut from 54 mm tube samples showed lower stiffness and pore pressure responses.

McManis and Arman (1979) investigated the effect of sampler diameter on the properties of undisturbed soil specimens. The soil types studies were soft organic silty clays and stiff, fissured pleistocene clays. For stiff fissured clay, the strength of the 76 mm diameter tube sample exceeded that of 127 mm diameter specimen. This was attributed to stress release and migration of moisture toward and along the fissure planes. Maguire (1975) also found that for stiff fissured overconsolidated clay the undrained strength increased with decreasing diameter of sample. However, for soft silty clay, McManis and Arman (1979) found that 127 mm tube specimens exhibited strengths greater than that of 76 tube specimens.

Sample quality is also related to the length to diameter ratio of the sampler. One of the major factors controlling sample jamming is the length to diameter ratio of the sampler. The optimum length to diameter ratios suggested for clays of different sensitivities are as follows (the Report of the Sub-Committee on Problems and Practices in Soil Sampling, 1965).

Sensitivity, S_t	Length / diameter ratio
>30	20
5 to 30	12
<5	10

Conlon and Isaacs (1971) carried out unconsolidated undrained triaxial compression tests on specimens of sensitive lacustrine clay of medium to high plasticity. The clay was sampled using 73 mm outside diameter thin-walled Shelby tube (AR = 9.3%) and 127 mm outside diameter fixed-rod thin-walled piston sampler (AR = 10.8%. Some 51 mm thin-walled tube samples were obtained in wash borings and auger holes. Block samples were also collected. Conlon and Isaacs (1971) observed that disturbance increased as the size of the tube sample decreased.

An investigation of the difference in quality of samples taken with large diameter fixed piston samplers and the 50 mm diameter Swedish Standard piston sampler (AR = 21%, ICR = 0.4%, outside cutting edge taper angle = 5^0) was carried out by Holm and Holtz (1977). The large diameter piston samplers used were the 95 mm NGI research sampler (AR = 14%, ICR = 1.4%, outside cutting edge taper angle = 10^0) the 127 mm Osterberg sampler (AR = 18%, ICR = 0.4%, outside cutting edge taper angle = 7^0) and the 124 mm SGI research sampler (AR = 27%, ICR = 1.2% and angle of cutting edge = 5^0). The investigation has shown that in general no significant differences between either the ratio (preconsolidation pressure / in-situ vertical stress) or undrained shear strength derived from laboratory tests on specimens obtained by the various devices, but there are indications that results of oedometer tests on 50 mm samples are more scattered, supporting findings of Berre et al. (1969). The undrained modulus obtained from 50 mm samples has been found to be lower. Holm and Holtz (1977), however, concluded that for routine investigations in soft Swedish clays, there seems to be no need to perform sampling with large diameter piston samplers.

2.11.4 Effect of External Diameter to Thickness Ratio (D_e / t Ratio) of Sampler

Kubba (1981) investigated the effect of thickness of tube on sampling disturbance for a reconstituted Spestone Kaolin (LL = 51, PI = 30). Tube samples were obtained by inserting 38 mm diameter tubes of different wall thicknesses into a 102 mm diameter "perfect" sample. Three tubes of thickness to diameter ratios of 0.039, 0.072 and 0.105 were used for sampling. Kubba (1981) found that increasing the ratio of wall thickness to diameter (t/D_i) of the tube caused a qualitative increase in the degree of disturbance. Kubba (1981) also reported a qualitative increase in the degree of disturbance due to increase in the ratio of thickness to diameter of the samplers.

Marked increase in degree of disturbance (measured in terms of tube sampling strains), with decreasing D_e/t ratio of sampler has also been analytically predicted (Baligh, 1985; Baligh et al., 1987). The levels of straining resulting from penetration of "simple sampler" have been predicted analytically by Baligh et al. (1987). Baligh et al. (1987) found that the peak axial strains are very much dependent on the aspect ratio (D_e/t), as can be seen from Fig. 2.44. Baligh et al. (1987) found from analytical study that the peak axial strain in compression and extension decreases with increasing D_e/t ratio of the samplers. Clayton et al. (1998) also investigated the effect of D_e/t ratio on tube sampling disturbance. Variation of peak axial strain with D_e/t

ratio of sampler is shown in Fig. 2.50. Clayton et al. (1998) found reduction in peak axial strain in compression and extension with increasing D_e/t ratio.

Chin (1986) showed that, for thin-walled simple samplers ($D_e/t >>1$), both maximum axial strain in compression and extension at the centerline of sampler is approximately given by the following expression;

$$\varepsilon_{max} = 0.385\ t/D_e \tag{2.24}$$

Siddique and Clayton (1998) and, Clayton and Siddique (1999) also reported that both the peak axial strain in compression ahead of the sampler and the maximum axial strain in extension inside the sampler are dependent on the external diameter (D_e) to thickness (t) ratio of the sampler. From a numerical study on the effect of cutting shoe geometry of a number of realistic samplers on tube sampling strains, it has been observed that peak axial centre line strain in compression and extension decrease with increasing D_e/t ratio.

Siddique and Sarker (1996) investigated the effect of D_e/t ratio on undrained soil parameters of reconstituted Dhaka clay by carrying out undrained triaxial compression tests on "tube" samples collected with samplers of varying diameter to thickness (D_e/t) ratio. Siddique and Sarker (1996) reported that strength, stiffnesses and pore pressure parameter decreased while strain at peak strength increased with the decrease in D_e/t ratio of sampler. Siddique et al. (2000), and Siddique and Rahman (2000) obtained the similar effects of D_e/t ratio on undrained soil parameters for three Chittagong coastal soils and a Dhaka clay, respectively.

2.12 Assessment of Sample Disturbance

The mechanical properties of soils are modified by sampling disturbance and hence, they can be used to calculate the amount of disturbance quantitatively. The properties of in-situ soils are required as references in calculating disturbance. However, there is no way of obtaining a soil sample so as to maintain exactly the in-situ conditions. This is because its removal involves a change in the in-situ state of stress and usually some disturbance due to sampling and handling. So, degree of disturbance can be assessed by

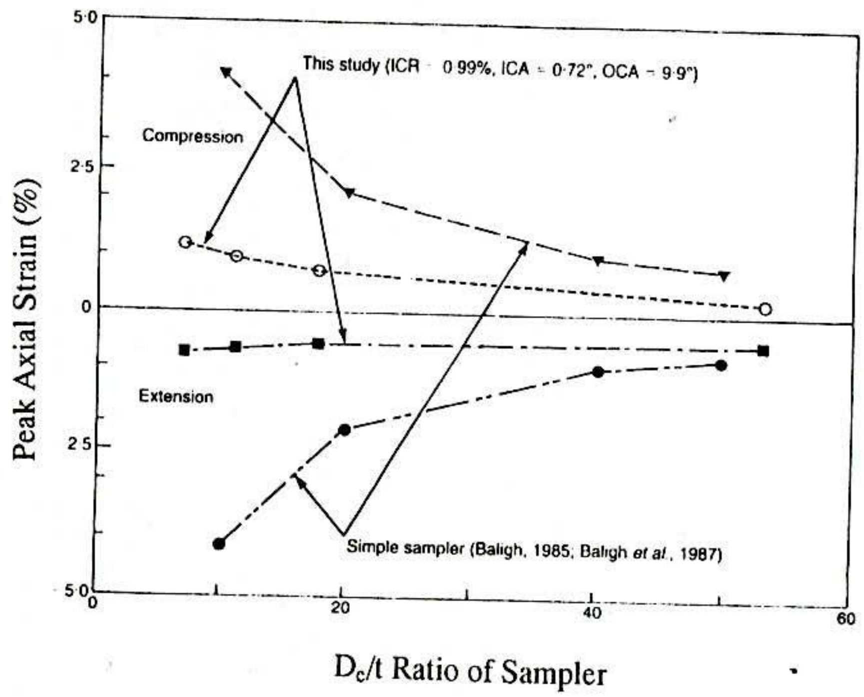


Fig. 2.50 Comparison of Centre-Line Axial Strains for the Sample Sampler with those from Samplers having More Realistic Cutting Shoe Geometries (after Clayton et al., 1998).

investigating the behaviour of the least disturbed sample which is usually a laboratory simulated "perfect" sample.

Because of additional disturbances other than that occurred due to total stress release, the residual effective stress of a disturbed sample, σ'_i is usually less than the effective stress, σ'_{ps} of a "perfect" sample. Perfect sampling, which is usually simulated in the laboratory by consolidating specimens anisotropically in the triaxial apparatus and then releasing firstly the in situ shear stress and secondly releasing the total isotropic stress to zero under undrained conditions. The residual stress σ'_i is the initial effective stress because of additional disturbance other than that occurred due to total stress release at the time of "perfect" sampling.

A number of investigators have defined the degree of disturbance (D_d) in terms of σ'_{ps} and σ'_i . These are as follows:

(a) Ladd and Lambe (1963) proposed that disturbance could be defined as

$$D_d = \sigma'_i/\sigma'_{ps} \tag{2.25a}$$

(b) Noorany and Seed (1965) regarded the difference between σ'_{ps} and σ'_i as a measure of disturbance, i.e.,

$$D_d = \sigma'_{ps} - \sigma'_i \tag{2.25b}$$

(c) Okumura (1971) and Nelson et al. (1971) defined the degree of disturbance as follows:

$$D_d = 1 - (\sigma'_i/\sigma'_{ps}) \tag{2.25c}$$

The value of D_d varies between zero (no disturbance) and unity (maximum disturbance).

Direct and indirect methods of measuring the residual effective stress of a disturbed sample were proposed by Skempton (1961) and Lambe (1961). Different methods of measuring the residual or initial effective stress in clays have been summarized by Baldi et al. (1988) and Hight and Burland (1990).

Siddique and Sarker (1997), Siddique et al. (2000) and Siddique and Rahman (2000), investigated the degree of disturbance in clays of tube sampling at selected regional soils of Bangladesh. Siddique and Sarker (1996) reported that the values of D_d (measured by using eqn. 2.25c) increased from 0.19 to 0.42 due to increase in area ratio from 10.8 to 55.2 (decrease in D_e/t ratio from 40.0 to 10.1) and also the values of D_d increased from 0.22 to 0.37 due to increase in OCA from 4° to 15° for reconstituted Dhaka clay. Siddique and Rahman (2000) investigated the variation of degree of disturbance, D_d with the variation of area ratio or D_e/t ratio for another reconstituted Dhaka clay and found similar results as Siddique and Sarker (1996). Siddique et al. (2000) also reported that similar effect of area ratio, OCA and D_e/t ratio on degree of disturbance obtained for reconstituted three coastal soils.

Fig. 2.51 and Fig. 2.52 show the variations of degree of disturbance (D_d) with area ratio (AR) and outside cutting edge angle (OCA) of samplers respectively for Dhaka clay and three Chittagong coastal soils. Siddique and Sarker (1996), Siddique et al. (2000) and Siddique and Rahman (2000) reported that degree of disturbance increased with the increase in area ratio and OCA of sampler.

2.13 Methods Used for Correcting Undrained Strength of Disturbed Samples

Due to sampling disturbances, it is necessary to correct the undrained strength in order that it is representative of the in situ soil. A number of methods have been proposed for correcting the undrained strength of disturbed sample. Ladd and Lambe (1963) considered the difference between measured residual effective stress, σ'_i and the residual effective stress expected with "perfect" sampling, σ'_{ps} as being similar to an overconsolidation phenomenon which influences the measured strength. For each particular soil they established a relationship between the overconsolidation ratio, OCR and undrained shear strength, s_u. Then by considering the OCR as being equal to σ'_{ps}/σ'_i, they corrected the strength measured at an effective stress of σ'_i to the value that would have existed if the sample had been tested at a stress of σ'_{ps}.

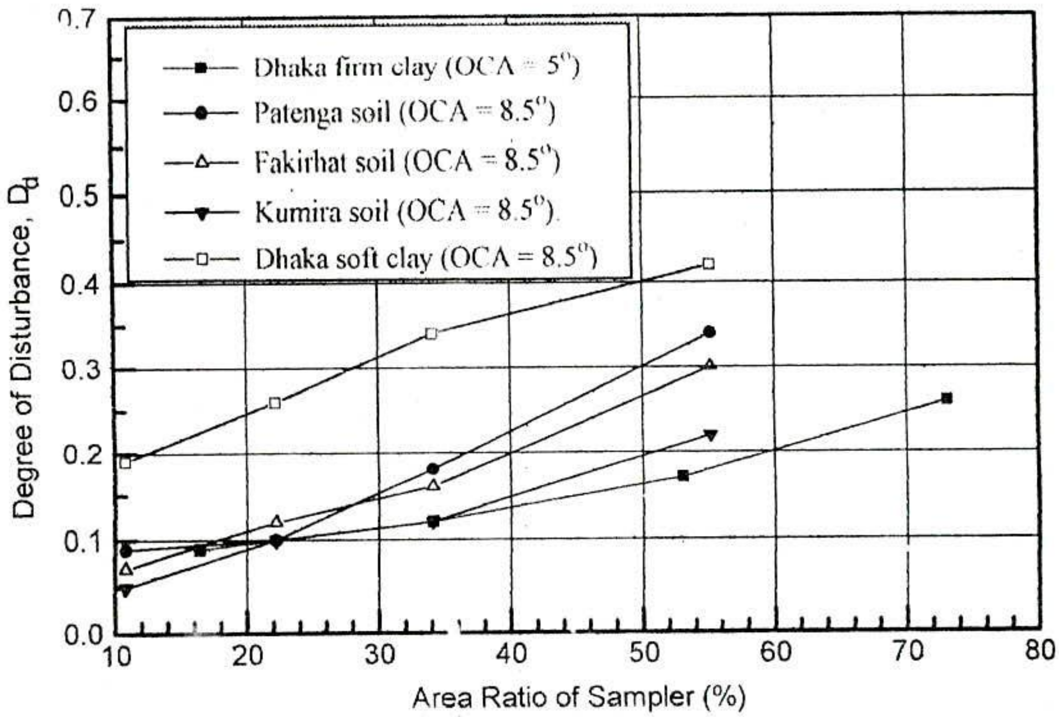


Fig. 2.51 Variation of Degree of Disturbance with Area Ratio of Sampler for Samples: Dhaka firm clay (after Siddique and Rahman, 2000); three Coastal Soils (after Siddique et al., 2000); Dhaka soft clay (after Siddique and Sarker, 1996)

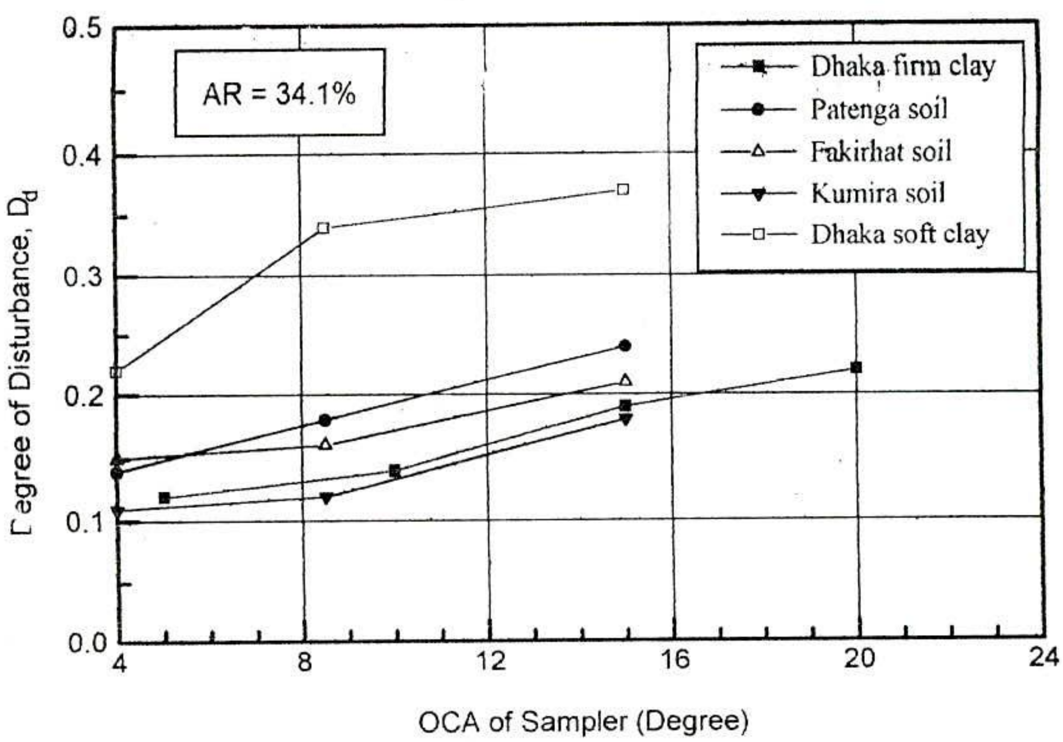


Fig. 2.52 Variation of Degree of Disturbance with OCA of Sampler for Samples: Dhaka firm clay (after Siddique and Rahman, 2000), Three Coastal Soils (after Siddique et al., 2000); Dhaka soft clay (after Siddique and Sarker, 1996)

Nelson et al. (1971) also reported the difference between measured residual effective stress, σ'_i (σ'_c) and the residual effective stress expected with "perfect" sampling, σ'_{ps} (σ'_{cm}) as being similar to an overconsolidation phenomenon which influences the measured strength of Bangkok clay (LL = 42 to 85%, PL = 5 to 35%). Fig. 2.53 shows the effect of OCR on undrained shear strength for various clays. It is found from Fig. 2.53 that undrained shear strength ratio decreases with increasing OCR.

A method similar to Ladd and Lambe (1963) was proposed by Okumura (1971) to correct for a disturbed measured strength. In order to obtain the base for correction, a triaxial compression test, loaded repeatedly up to failure, is performed on a representative specimen consolidated under K_0-conditions and with its deviator stress released in an undrained condition ("perfect" sample). Test results are plotted as disturbed strength ratio (s_{ur}/s_{up}) against disturbance ratio (σ'_p/σ'_i), where s_{ur} is the undrained strength after each cycle, s_{up} is the undrained strength of the "perfect" sample, σ'_i is the residual effective stress after each cycle and, σ'_p is the residual effective stress of the "perfect" sample and also presents the results from repeated loading shear tests plotted as disturbed strength ratio and disturbance ratio. The sample is then sheared to find its disturbed strength. The correction curve obtained by the above process gives the perfectly undisturbed strength of each sample.

Siddique et al. (2000) developed a correction curve by plotting disturbed strength ratio (s_{ut}/s_{up}) versus degree of disturbance [1- (σ'_i/σ'_{ps})], where s_{ut} is the undrained strength of the "tube" sample, s_{up} is the undrained strength of the "perfect" sample, σ'_i is the residual effective stress of the "tube" sample, and σ'_{ps} is the isotropic effective stress of the "perfect" sample for three coastal soils (Patenga, Fakirhat and Kumira) as shown in Fig. 2.54. This strength correction curve is a wide range of band curves for coastal soil samples. It is found from Fig. 2.54 that undrained shear strength ratio decreases with increasing degree of disturbance.

A comprehensive way of correcting the measured value of the undrained shear strength for sample disturbance has been reported by Nakase et al. (1985). An expression has been proposed to evaluate the disturbance ratio (a ratio of the undrained strength of the "perfect" sample to the undrained strength of the actual sample) of a soil sample from

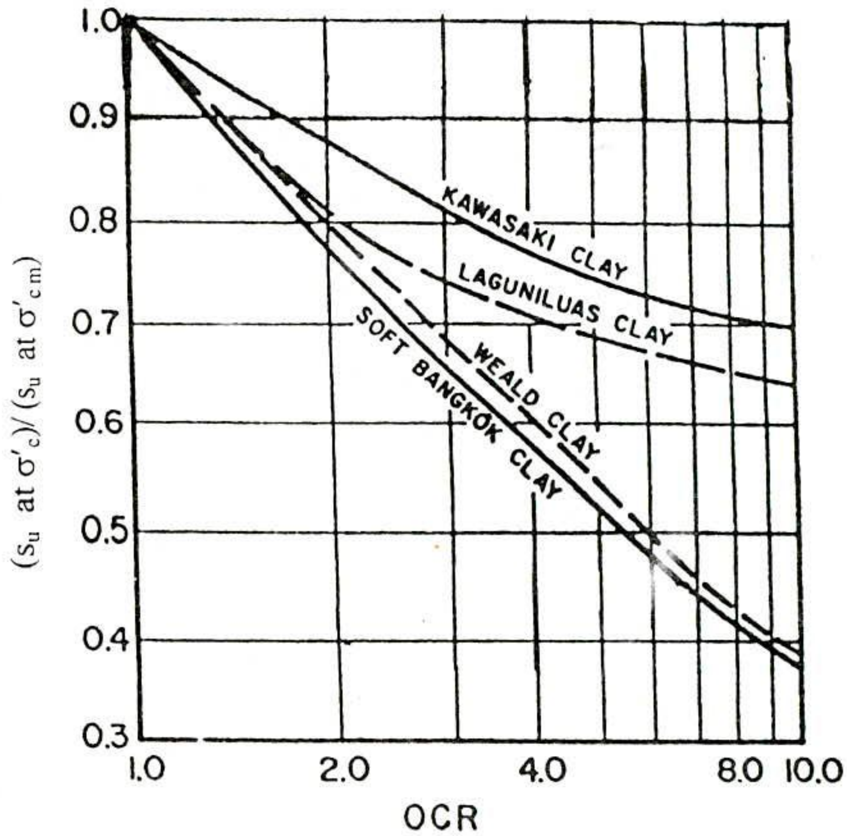


Fig. 2.53 Effect of OCR on Undrained Strength for Various Clays (Nelson et al., 1971).

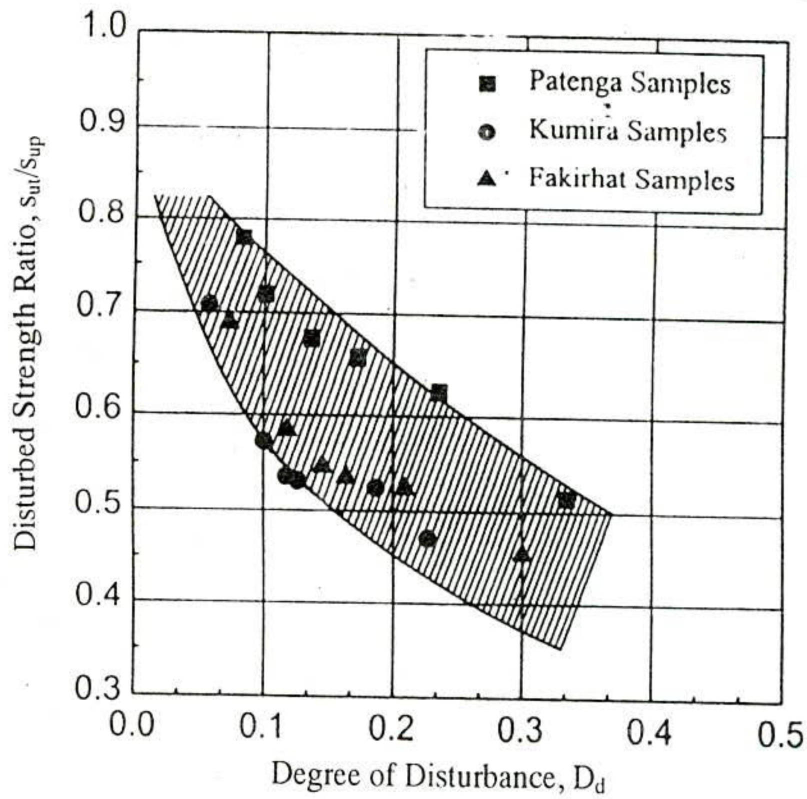


Fig. 2.54 Disturbed Strength Ratio vs. Degree of Disturbance Plot for Samples of Three Coastal Soils (after Siddique et al., 2000)

the measured values of plasticity index, secant modulus, E_{50} and the in-situ effective overburden pressure. The disturbance ratio could then be used to correct the measured undrained strength value. The proposed method of correction is applicable to soils of wide range of plasticity.

Two types of disturbance for unconfined compression strength in soft clay, namely, a remoulding type disturbance and a crack type disturbance has also been reported by Tsuchida (1993). Tsuchida (1993) recommended that the correction methods proposed by Okumura (1971) and Nelson et al. (1971) are valid only for the remoulding type disturbance and not for the crack type disturbance. Tsuchida (1993) also showed that for the crack type disturbance, the reduction in strength obtained from undrained triaxial compression test is much less than that obtained from unconfined compression test.

2.14 Methods Used for Minimizing Sample Disturbance Effects

It is possible to reduce the effects of sampling disturbance on the undrained behaviour of clays by reconsolidating the sample to a more appropriate stress level prior to shearing. The effects of isotropic and anisotropic reconsolidation procedures have been investigated by a number of researchers for minimizing the sampling disturbance. The influence of different types of reconsolidation techniques on the undrained shear properties of "perfect" samples are described as follows.

2.14.1 Isotropic Reconsolidation

Raymond et al. (1971) applied hydrostatic isotropic consolidation pressures to samples of sensitive Leda clay. The ratio of undrained stiffness to undrained strength was close to undisturbed behaviour, when disturbed samples were consolidated to 50 - 75% of their preconsolidation pressure. When the consolidation pressures exceeded the preconsolidation pressure, there was a dramatic decrease of the stiffness-strength ratio, indicating a beak-down in the structure of the sensitive clay.

Kirkpatrick and Khan (1984) adopted two methods of isotropic reconsolidation to examine whether the "in situ" undrained behaviour could be reproduced. Hydrostatic reconsolidations to pressures equal to σ'_{ps} and "in situ" vertical effective pressure, σ'_{vc}, were applied to samples of Kaolin (PI = 30) and Illite (PI = 40). It was found that, reconsolidation to σ'_{ps} resulted in underestimation of "in situ" strength by as much as 14%. However, hydrostatic reconsolidation to σ'_{vc} had the effect of producing fairly large overestimation of "in situ" strength of 16 % or more. They also reported that the undrained strength of the reconsolidated "perfect" samples for normally consolidated Kaolin and Illite clay increase up to 10.7 % and 5.77 % respectively as compared with those of the "in situ" samples due to isotropic reconsolidation. Failure strains and pore water pressures were heavily overestimated by both the methods of reconsolidation.

Graham et al. (1987) reported that in both normally consolidated and overconsolidated samples of Kaolin, isotropic reconsolidation to σ'_{vc} overestimated the strength of "in situ" specimens while isotropic reconsolidation to $0.6\sigma'_{vc}$ underestimated it. In both cases the strains to failure and pore pressure parameter at failure were higher than the "in situ" specimens. These findings agree with those reported by Kirkpatrick and Khan (1984). Similar results have also been reported by Graham and Lau (1988) for normally consolidated kaolin.

2.14.2 Anisotropic Reconsolidation

Instead of isotropic reconsolidation, anisotropic reconsolidation has been proposed by several investigators as an effective method of reducing sampling disturbance effects. K_0-consolidation to the in situ stresses has been suggested by Davis and Poulos (1967) and Bjerrum (1973). Ladd and Foott (1974) proposed the SHANSEP (Stress History and Normalized Soil Engineering Properties) method for reducing the effects of sample disturbance. This method is based on two concepts. The first is that the soil exhibits normalised stress-strain and strength behaviour. The second is that anisotropic reconsolidation of the soil samples that should be reconsolidated anisotropically to a pressure at least equal to 1.5 to 2 times the in-situ vertical effective stress, σ'_{vc}, eliminates the effects of any sample disturbance. The effect of anisotropic reconsolidation in recovering the in-situ behaviour has been studied by many research workers.

reconsolidating the soil and could, in effect, be virtually eliminated by the SHANSEP method.

2.15 Isotropic and Anisotropic Reconsolidation of Reconstituted Normally Consolidated and Overconsolidated Samples of Selected Regional Soils of Bangladesh

2.15.1 Reconsolidation of "Perfect" Samples

Isotropic and anisotropic reconsolidation of "perfect" samples of Dhaka clay and coastal soils in Bangladesh were carried out to investigate the suitability of different reconsolidation procedures to restore again the in situ behaviour. Siddique and Farooq (1996) reported that undrained strength ratio (s_u/σ'_{vc}) increased by 49% and 70% and stiffness ratio (E_i/σ'_{vc}) increased by 42% and 38% due to isotropic reconsolidation for reconstituted normally consolidated "perfect" samples of two coastal soils (Patenga and Kumira respectively). Siddique and Farooq (1996) also reported that strain at peak strength (ε_p) increased by 56% and 5% while, pore pressure parameter at peak strength (A_p) decreased by 34% and 32% due to isotropic reconsolidation. Siddique and Farooq (1996) reported that the values of s_u/σ'_{vc} reduced by 13% and 16% (Patenga), 6% and 13% (Kumira) while the values of ε_p increased by 96% and 65% (Patenga), 10% and 2% (Kumira), due to reconsolidation procedures SHANSEP-1.5 and SHANSEP-2.5, respectively, for "perfect" samples. Siddique and Farooq (1996) also reported that the values of E_i/σ'_{vc} reduced by 4% and 19% (Patenga), 20% and 38% (Kumira), while the values of A_p increased by 26% and 62% (Patenga), 14% and 34% (Kumira), due to reconsolidation procedures SHANSEP-1.5 and SHANSEP-2.5 respectively for "perfect" samples. Siddique and Farooq (1996) found that K_0-reconsolidation of "perfect" sample in SHANSEP-1.5 produced the best agreement between the "perfect" and "in situ" samples in terms of undrained strength ratio, stiffness ratio and A_p - values for two coastal soils.

Fig. 2.55 shows a comparison of variations of deviator stress with axial strain of "in situ" and reconsolidated "perfect" samples of Dhaka clay. Siddique and Sarker (1998) reported that strengths due to isotropic and anisotropic reconsolidation of "perfect" samples are greater than that of "in situ" sample of Dhaka clay. Siddique and Sarker (1998) reported that the values of s_u/σ'_{vc} and E_i/σ'_{vc} increased by 26% and 139% respectively due to isotropic reconsolidation for reconstituted normally consolidated "perfect" samples of Dhaka clay (LL = 45, PI = 23). Siddique and Sarker (1998) also reported that the value of ε_p increased by 62% while A_p decreased by 26% due to isotropic reconsolidation. Siddique and Sarker (1998) reported that the values of s_u/σ'_{vc} reduced by 21% and 15% while the values of ε_p increased by 62% and 81% due to procedures SHANSEP-1.5 and SHANSEP-2.5 respectively, for "perfect" samples of Dhaka clay. Siddique and Sarker (1998) also reported that the values of E_i/σ'_{vc} reduced by 26% and 54% while the values of A_p increased by 39% and 55% due to procedures SHANSEP-1.5 and SHANSEP-2.5 respectively for "perfect" samples. Siddique and Sarker (1998) found that reconsolidation of "perfect" specimens using SHANSEP procedures could not restore the characteristics of the "in situ" specimen in terms of its strength, strain, stiffness and pore pressure response for normally consolidated Dhaka clay.

Rahman (2000) found that isotropic reconsolidation has the effect of marked overestimation of "in situ" undrained shear strength (s_u), axial strain at peak deviator stress (ε_p), initial stiffness (E_i), secant stiffness (E_{50}) and pore pressure parameter at peak deviator stress (A_p) for reconstituted overconsolidated samples (OCR = 2 and 10) of Dhaka clay (LL = 47, PI = 26). Rahman (2000) reported that compared with SHANSEP procedures, the soil parameters of "perfect" samples reconsolidated using Bjerrum procedure (CK_0U-$1.0\sigma'_{vc}$) agrees more closely with those of the respective "in situ" samples than those of the samples reconsolidated using SHANSEP procedures.

A comparison of undrained shear characteristics of "perfect" samples due to isotropic and anisotropic reconsolidation of regional soils of Bangladesh is shown in Table 2.15.

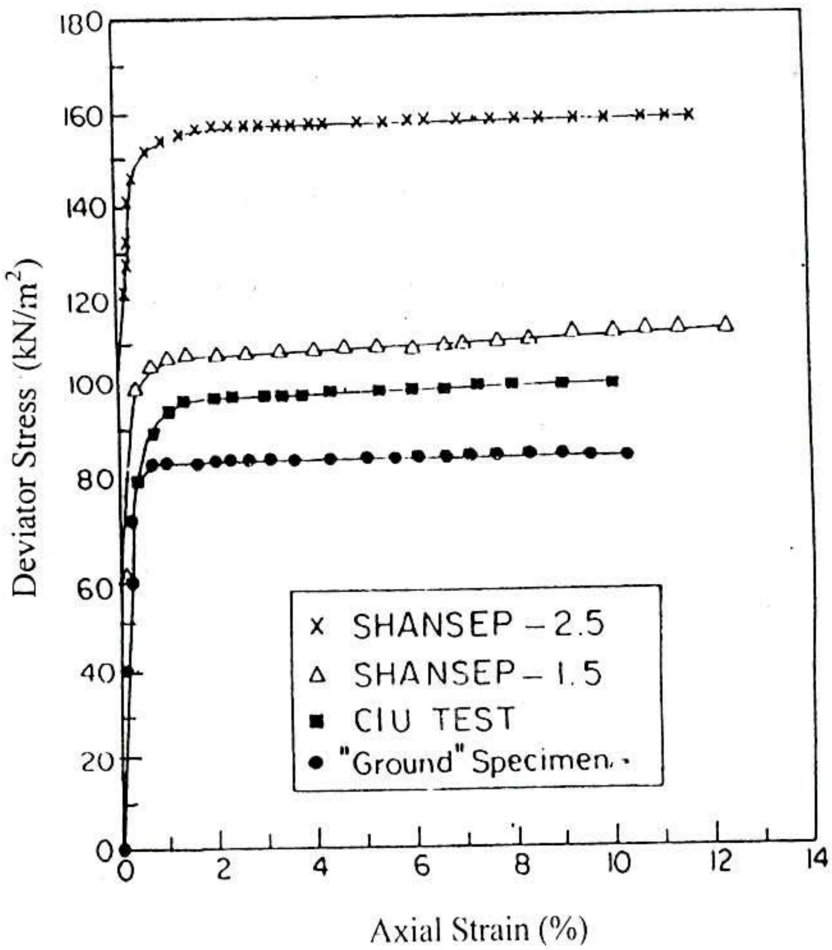


Fig. 2.55 Deviator Stress vs. Axial Strain Plots for "In Situ" and Reconsolidated "Perfect" Samples for Normally Consolidated Dhaka Clay (after Siddique and Sarker, 1998).

Table 2.15 Comparison of Undrained Shear Characteristics of "In Situ" and Reconsolidated Normally Consolidated (NC) and Overconsolidated "Perfect" Samples of Regional Soils of Bangladesh

Location	Test type	s_u / σ'_{vc}	ε_p (%)	E_i/σ'_{vc}	E_{50}/σ'_{vc}	A_p	Reference
Dhaka (NC)	CIU-1.0σ'_{vc}	0.49	6.0	460		0.76	Siddique and Sarker (1998)
	SHANSEP-1.5	0.31	6.0	141		1.43	
	SHANSEP-2.5	0.33	6.7	88.4		1.60	
	"In Situ"	0.39	3.7	192		1.03	
Patenga (NC)	CIU-1.0 p'_0	0.67	13.1	255.9		0.43	Siddique and Farooq (1996)
	SHANSEP-1.5	0.39	16.5	173.9		0.82	
	SHANSEP-2.5	0.38	13.9	145.8		1.05	
	"In Situ"	0.45	8.4	180.3		0.65	
Kumira (NC)	CIU-1.0 p'_0	0.80	14.8	306.3		0.40	
	SHANSEP-1.5	0.44	15.5	176.7		0.51	
	SHANSEP-2.5	0.41	13.9	136.6		0.79	
	"In Situ"	0.47	14.1	221.2		0.59	
Dhaka (OCR=2)	CIU-1.0σ'_{vc}	0.87	11.3	567.2	400.5	0.26	Rahman (2000)
	CK_0U-1.0σ'_{vc}	0.70	9.3	332.6	274.2	0.22	
	SHANSEP-1.5	0.63	9.5	269.2	236.7	0.37	
	SHANSEP-2.5	0.62	10.7	215.3	189.4	0.41	
	"In Situ"	0.72	9.0	353.8	290.0	0.19	
Dhaka (OCR=10)	CIU-1.0σ'_{vc}	3.56	10.0	2216.0	1557.6	0.19	Rahman (2000)
	CK_0U-1.0σ'_{vc}	3.02	7.3	1298.6	1095.0	0.17	
	SHANSEP-1.5	2.53	8.5	1032.2	944.0	0.31	
	SHANSEP-2.5	2.38	9.3	790.8	729.7	0.34	
	"In Situ"	3.10	6.7	1482.0	1263.0	0.13	

2.15.2 Reconsolidation of "Tube" Samples

Isotropic and anisotropic reconsolidation of "tube" samples of Dhaka clay and coastal soils in Bangladesh were carried out to investigate the suitability of different reconsolidation procedures to restore again the in-situ behaviour. A comparison of undrained shear properties of "tube" samples due to isotropic and anisotropic reconsolidation of regional soils in Bangladesh is shown in Table 2.16.

The effect of reconsolidation of "tube" sample of reconstituted Dhaka Clay was investigated by Sarker (1994) by carrying out isotropic and K_o-reconsolidation on a typical "tube" sample. Sarker (1994) reported that the values of s_u/σ'_{vc} and E_i/σ'_{vc} decreased by 8% and 31% respectively due to isotropic reconsolidation for reconstituted normally consolidated "tube" samples of Dhaka clay. Sarker (1994) reported that the value of ε_p increased by 27% while the value of A_p reduced by 2% due to isotropic reconsolidation. Sarker (1994) also found that the values of s_u/σ'_{vc} reduced by 21% and 18% while the values of ε_p increased by 62 % and 81% due to reconsolidation procedures SHANSEP-1.5 and SHANSEP-2.5 respectively for "tube" samples of Dhaka clay. Sarker (1994) also reported that the values of E_i/σ'_{vc} reduced by 27% and 56% while the values of A_p increased by 16% and 61% due to procedures SHANSEP-1.5 and SHANSEP-2.5 respectively. Sarker (1994) concluded that K_0-reconsolidation of the "tube" samples beyond in situ stresses could not recover the "in situ" behaviour and the result of K_0-reconsolidation of "tube" samples using the SHANSEP procedures of reconsolidation to restore "in situ" behaviour may not be applicable to Dhaka clay samples.

A comparison of variations of normalised deviator stress with axial strain of "in situ" and reconsolidated "tube" samples of Patenga clay is shown in Fig. 2.56. Farooq (1995) reported from Fig. 2.56 that normalised strengths due to reconsolidation using SHANSEP-1.5 and SHANSEP-2.5 procedures are lower, while normalised strength due to isotropic reconsolidation is greater of "tube" samples than that of "in situ" sample.

The variation of pore pressure change with axial strain of "in situ" and reconsolidated "tube" samples of Patenga is shown in Fig. 2.57. Farooq (1995) reported from Fig. 2.57 that pore pressure changes due to isotropic, SHANSEP-1.5σ'_{vc} and SHANSEP-2.5σ'_{vc} reconsolidation procedures of "tube" samples are greater than that of "in situ" sample.

Table 2.16 Comparison of Undrained Shear Properties of "In Situ" and Reconstituted Normally Consolidated and Overconsolidated "Tube" Samples of Regional Soils of Bangladesh

Location	Test type	s_u/σ'_{vc}	ε_p (%)	E_i/σ'_{vc}	E_{50}/σ'_{vc}	A_p	Reference
Dhaka (NC)	CIU-1.0σ'_{vc}	0.36	4.7	132.8		1.01	Sarker (1994)
	SHANSEP-1.5	0.31	6.0	139.3		1.16	
	SHANSEP-2.5	0.32	6.7	84.0		1.66	
	"In Situ"	0.39	3.7	192.0		1.03	
Patenga (NC)	CIU-1.0 p'_0	0.60	14.4	218.7		0.49	Siddique et al. (2000)
	SHANSEP-1.5	0.41	14.1	160.8		0.56	
	SHANSEP-2.5	0.43	16.6	158.0		0.56	
	"In Situ"	0.45	8.4	180.3		0.65	
Kumira (NC)	CIU-1.0 p'_0	0.81	13.4	247.1		0.17	
	SHANSEP-1.5	0.47	17.9	173.5		0.28	
	SHANSEP-2.5	0.48	14.3	167.5		0.46	
	"In Situ"	0.47	14.1	221.2		0.59	
Dhaka (OCR=2)	CIU-1.0σ'_{vc}	0.84	12.0	483.4	334.5	0.23	Rahman (2000)
	CK_0U-1.0σ'_{vc}	0.69	9.3	286.5	217.2	0.21	
	SHANSEP-1.5	0.59	11.3	226.1	176.6	0.33	
	SHANSEP-2.5	0.57	13.0	180.5	149.7	0.36	
	"In Situ"	0.72	9.0	353.8	290.0	0.19	
Dhaka (OCR=10)	CIU-1.0σ'_{vc}	3.46	10.0	1653.3	1511.9	0.17	Rahman (2000)
	CK_0U-1.0σ'_{vc}	2.95	8.0	1050.6	853.0	0.15	
	SHANSEP-1.5	2.34	8.7	904.0	728.8	0.30	
	SHANSEP-2.5	2.17	10.6	701.8	611.8	0.32	
	"In Situ"	3.10	6.7	1482.0	1263.0	0.13	

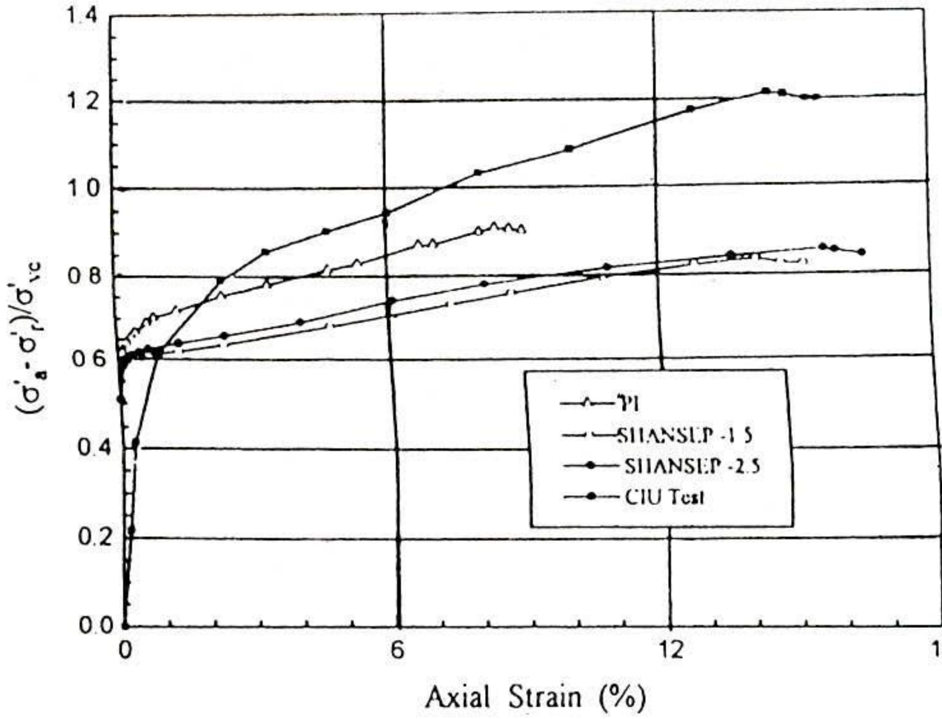


Fig. 2.56 Normalized Deviator Stress vs. Axial Strain Plots for "In Situ" and Reconsolidated "Tube" Samples for a Normally Consolidated Coastal Soil (after Farooq, 1995)

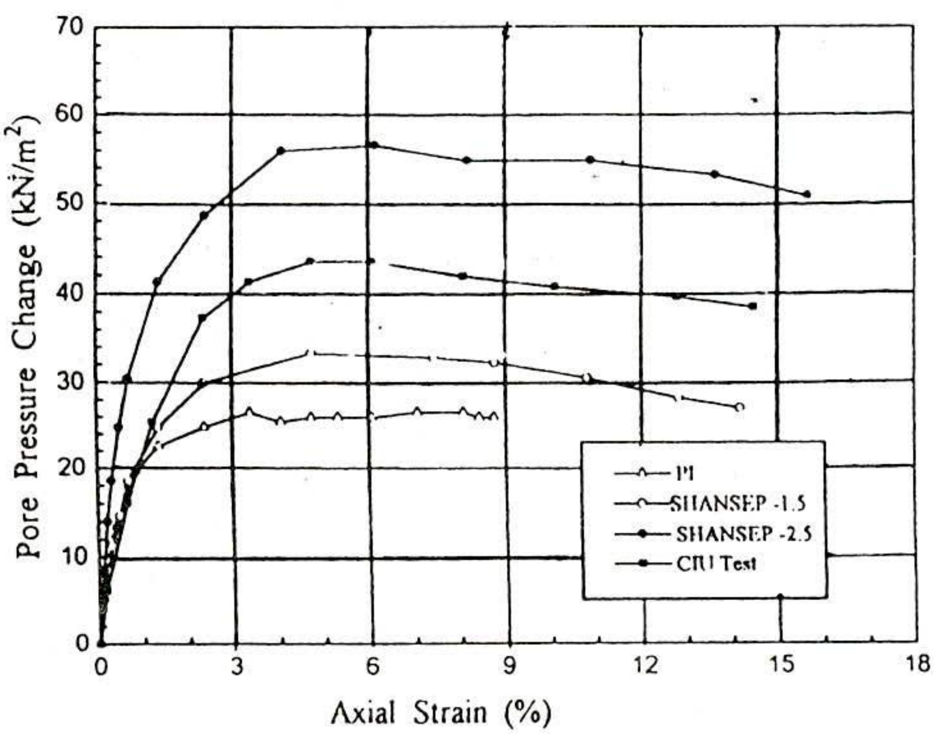


Fig. 2.57 Pore Pressure Change vs. Axial Strain Plots for "In Situ" and Reconsolidated "Tube" Samples for a Normally Consolidated Coastal Soil (after Farooq, 1995)

Siddique et al. (2000) reported that the values of s_u/σ'_{vc}, E_i/σ'_{vc} and ε_p increased while A_p decreased due to isotropic reconsolidation for reconstituted normally consolidated "tube" samples of two coastal soils (Patenga and Kumira). Siddique et al. (2000) also reported that the values of s_u/σ'_{vc}, E_i/σ'_{vc} and A_p reduced while the values of ε_p increased due to the reconsolidation using SHANSEP-1.5σ'_{vc} and SHANSEP-2.5σ'_{vc} procedures of "tube" samples. Compared with isotropic reconsolidation, it can be seen in Table 2.16 that the undrained strength ratio s_u/σ'_{vc} of the "tube" samples reconsolidated using SHANSEP-1.5σ'_{vc} and SHANSEP-2.5σ'_{vc} compares favourably with those of the "in situ" samples. The undrained stiffness ratio E_i/σ'_{vc}, however, decreased by 11% and 12% for samples from Patenga reconsolidated using SHANSEP-1.5σ'_{vc} and SHANSEP-2.5σ'_{vc} procedures, respectively. For the Kumira soil, E_i/σ'_{vc} decreased by 22% and 24% for samples reconsolidated using SHANSEP-1.5σ'_{vc} and SHANSEP-2.5σ'_{vc} procedures, respectively. Siddique et al. (2000) concluded that strength ratio, stiffness ratio, A_p values and ε_p of the samples reconsolidated using SHANSEP-1.5σ'_{vc} and SHANSEP-2.5σ'_{vc} procedures compared more closely with the respective "in situ" sample than the samples reconsolidated isotropically using an effective consolidation pressure equal to p'_0. Rahman (2000) also reported that K_0-reconsolidation of overconsolidated "tube" samples using the SHANSEP procedures could not restore "in situ" behaviour and the samples reconsolidated using Bjerrum (1973) procedure agreed most favourably with the "in situ" sample.

3.2.1 The Rotary Laboratory Mixer

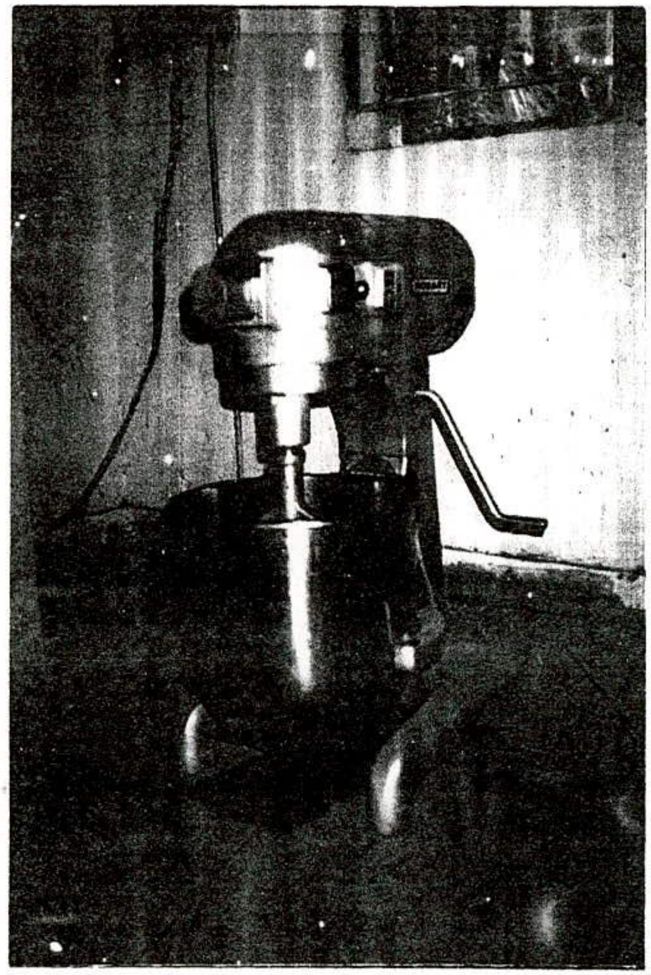
For producing uniform soil slurry, a Hobart mixer machine was used. The rotary blades of this machine ensured proper mixing of soil particles with water over a short period of time at the required moisture content. The mixer machine used has a dimension of 738 mm x 406 mm x 489 mm and includes a three-speed gearbox driven by a fully enclosed and ventilated motor. The shift handle is mechanically interlocked with the switch, giving definite gear location and making necessary to switch off the motor before changing 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 for preparing slurry was 113 revolutions / min for attachment and 198 revolutions per minute for beater. The mixing time was approximately 30 minutes. A photograph of the rotary mixer machine, bowl and attachment used is shown in Fig. 3.1(a) and Fig. 3.1(b).

3.2.2 Apparatus for K_0-Consolidation of Slurry

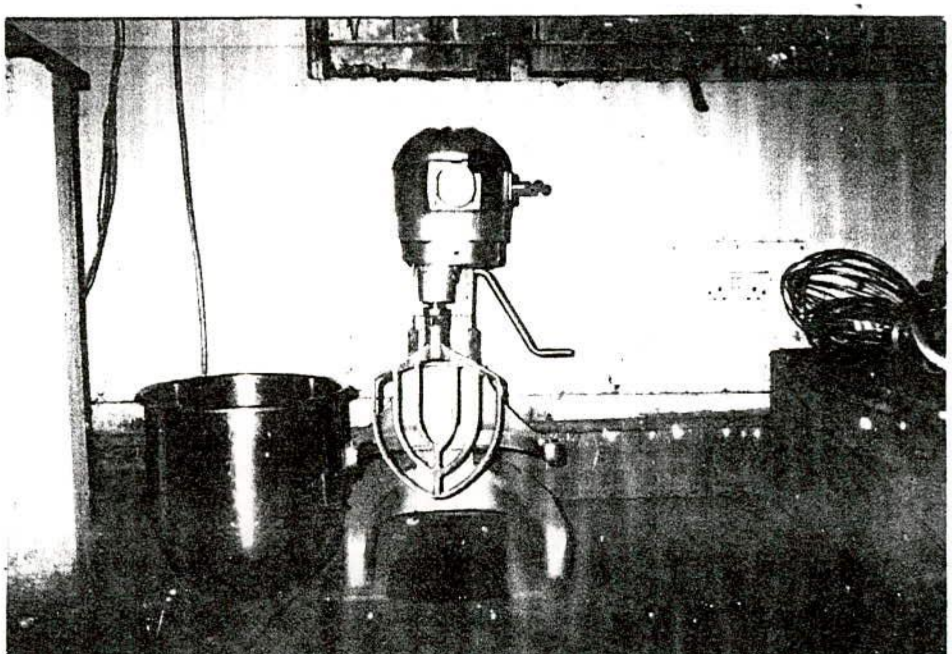
The type of loading frame shown in Fig. 3.2 has been used for K_0-consolidation of slurry. The dimension of the large cylindrical consolidation cell in the loading frame was 210 mm internal diameter and 180 mm in height. The consolidation cell, containing the soil slurry is placed on a rigid platform. The platform is raised manually by rotating a wheel and thus loading the soil sample is achieved 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 the sample. The deformation of the proving ring is measured by a dial gauge that gives the load imposed to sample at any stage of consolidation.

3.2.3 Triaxial Apparatus

Standard triaxial cell manufactured by Wykeham Farrance Engineering Co. which can accommodate 38 mm diameter samples were used. The cell consists of three main components, namely the cell base, the removable perspex cylinder and the top head assembly. The cell base consists of the pedestal for the set up of specimen and three



(a)



(b)

Fig. 3.1 The Hobert Laboratory Mixer Machine (a) Photograph of the Machine (b) Photograph of Attachment and Bowl

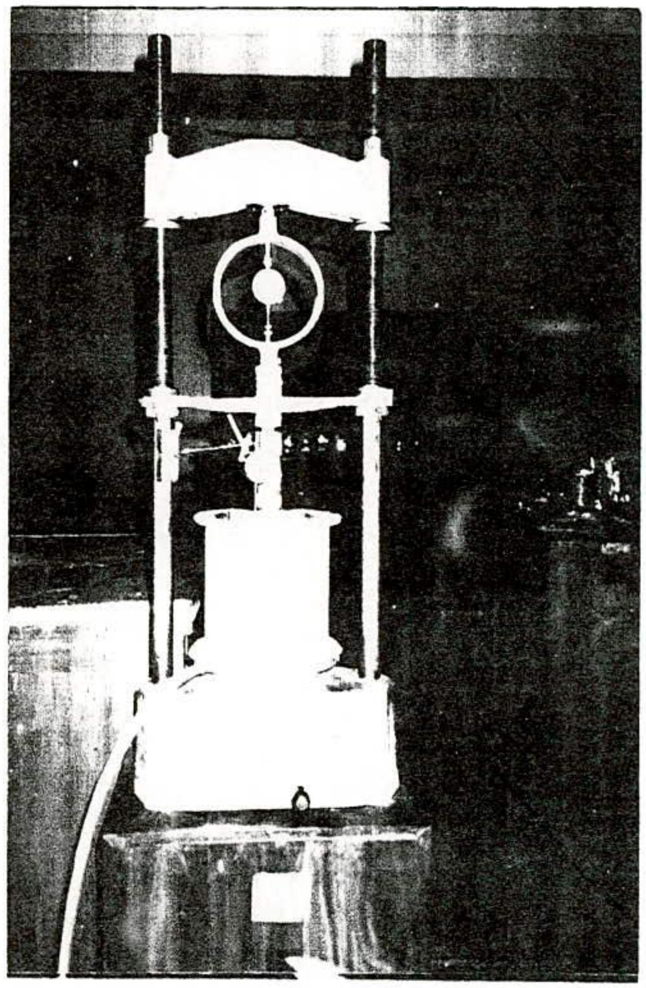


Fig. 3.2 Apparatus for K_0-Consolidation of Soil Slurry

water passages - two are drainage lines and the other is a cell pressure line. Among two drainage lines, one can be connected to the top and another to the bottom of the specimen. The bottom drainage line connected to the cell pedestal is used together with a pore pressure transducer and a burette to measure the pore pressure response during undrained loading and a volume change for drained tests. The back pressurizing for the saturation of specimen is applied through the bottom drainage line. The cell pressure line is used to fill the cell chamber with the drained and distilled water, and through which pressure is applied to the soil specimen.

The cell is provided with a motorized drive unit. The rate of strain during undrained shear test can be controlled by selecting proper driver and driven numbers and gear position. Deformation rates between 0.00064 mm/min and 1.50 mm/min can be applied to sample. A schematic diagram of the triaxial cell is shown in Fig. 3.3. In the triaxial cell, a standard proving ring of capacity 2.8 kN was used to measure the axial load where the resolution of the proving ring was 0.4077 lb. Axial deformation during consolidation and undrained shearing of sample. was measured by a strain gauge whose resolution is 0.0254 mm and maximum travel range is 25 mm. Cell pressure to sample was applied using a standard pressure gauge of operating range from 0 to 1700 kN/m^2. Back pressure was applied to sample using dash pot and control cylinder system. Back pressure up to 1200 kN/m^2 can be applied which is monitored by standard Budenberg test gauge. A photograph of the triaxial machine together with other measuring devices is shown in Fig. 3.4.

3.2.4 Volume Change Measurement Device

Volume change can be measure in triaxial testing by means of three methods (Bishop and Henkel, 1962). The first method measures the volume of fluid entering or leaving the triaxial cell to compensate for the change in volume of the sample. This method is used for the partially saturated soils. Appropriate corrections are required for cell and tubing expansion and piston rod penetration into the chamber during shearing. The second method measures the volume of fluid entering or leaving the pore space of the soil. This method is used only for saturated specimens. The third permits calculation of volume from direct measurement of the change in length and diameter of the

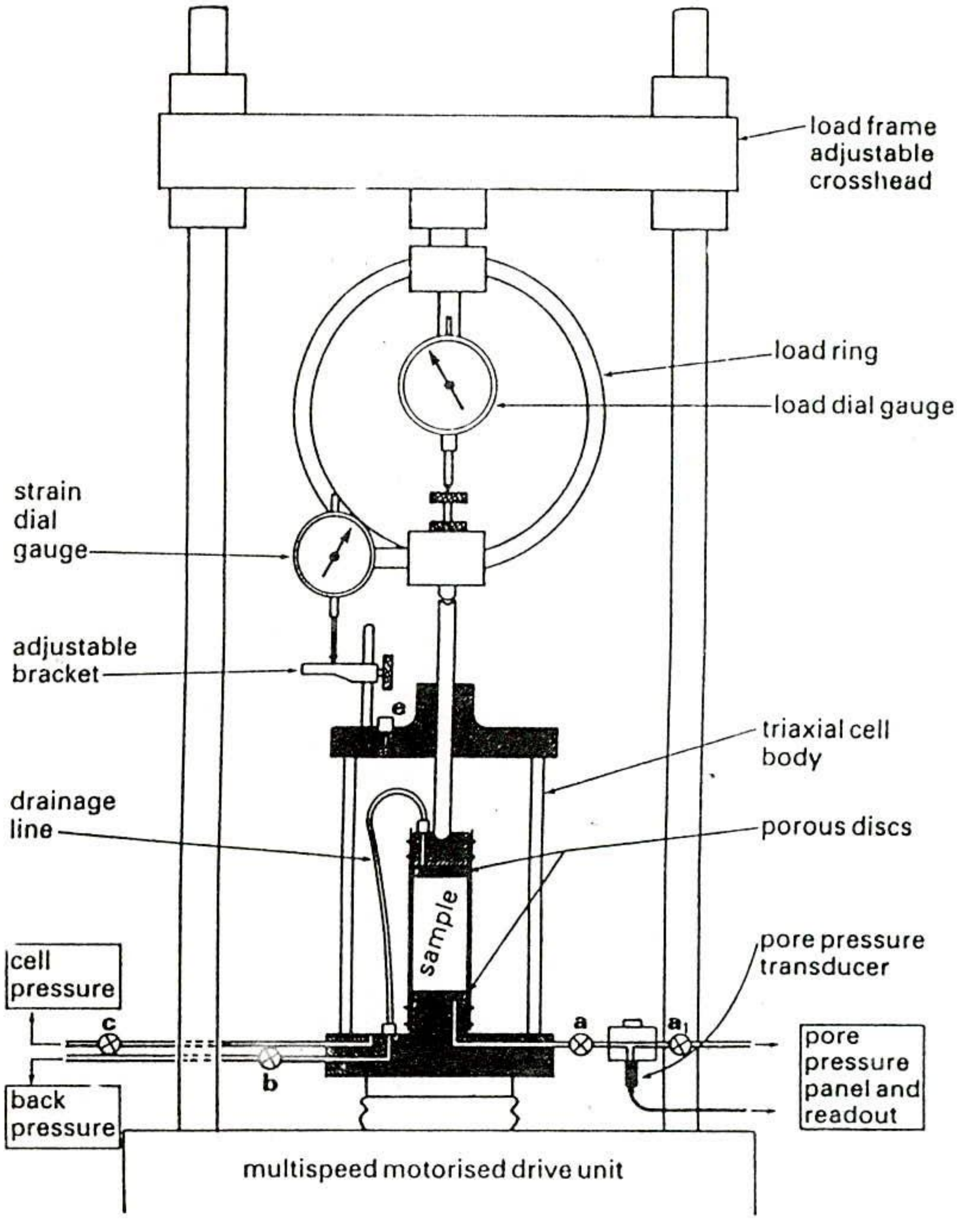


Fig. 3.3 A Schematic Diagram of Triaxial Cell

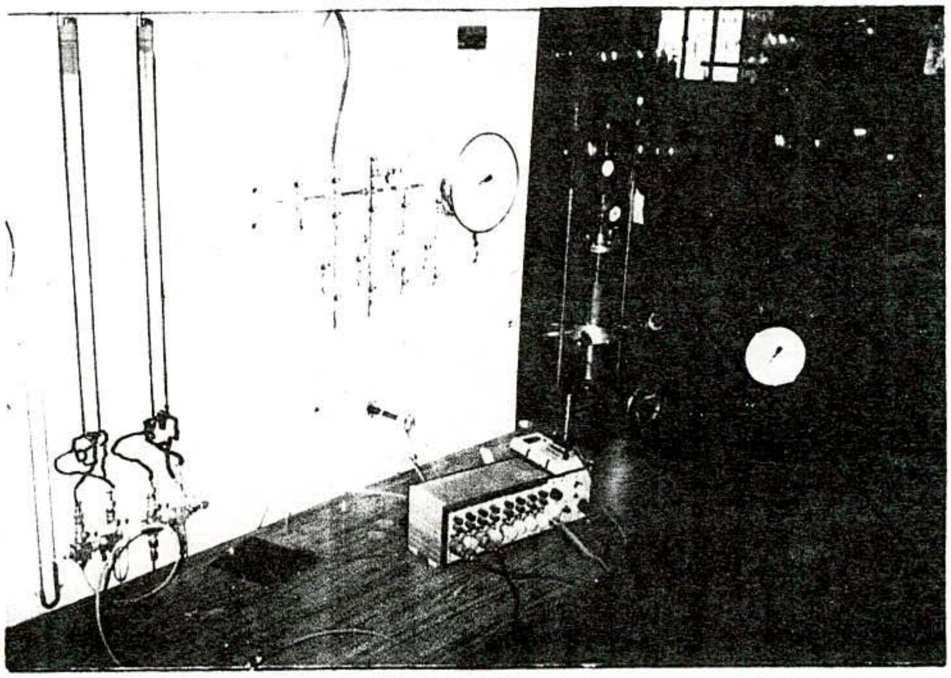


Fig. 3.4 General View Showing the Triaxial Machine

specimen using local axial and radial strain measuring devices (Clayton and Khatrush, 1986; Clayton, Khatrush, Bica and Siddique, 1989). This method may be used for both saturated and unsaturated specimens.

With a fully saturated sample a volume change can only occur under the action of cell pressure, if water is permitted to drain from the sample. In the laboratory, burette system (Bishop and Donald, 1961) is available. The burette is of 10 ml volume. It is necessary to measure volume change by the displacement of the surface between two liquids having different densities. Paraffin has been used as the second liquid. Details of the volume change apparatus have been reported by Bishop and Henkel (1962). A red dye is added to the paraffin, and a silicon water repellant is applied to the glass to maintain a meniscus of uniform shape. Volume changes in 10 m^1 graduated tube can be read to a minimum of 0.02 ml.

3.2.5 Pore Pressure Transducer

In undrained triaxial tests, pore water pressures are normally measured at the base of the samples. In tests carried out with fixed end samples, the shearing rate has to be selected to ensure that pore pressure nonuniformities arising from the restraint have equalized throughout the height of the sample either at failure, if only effective stress shear strength parameters are required, or at an early stage of the test if the effective stress path has to be derived.

Measurement of pore water pressure has been greatly simplified by using a pressure transducer. The transducer mounting block as shown in Fig. 3.5 is fitted directly to the triaxial cell at the outlet part (valve A) from the sample base, ensuring that the transducer is as close to the sample as possible. A Bell and Howell pore pressure transducer of operating range of 0 to 150 psi (0 to 1034 kN/m^2) has been used to record pore pressure generated on samples. Calibration of the transducer has been done by applying known increments of cell pressure and recording the corresponding value from the transducer in millivolt. A typical calibration curve of the pore pressure transducer is shown in Fig. 3.6.

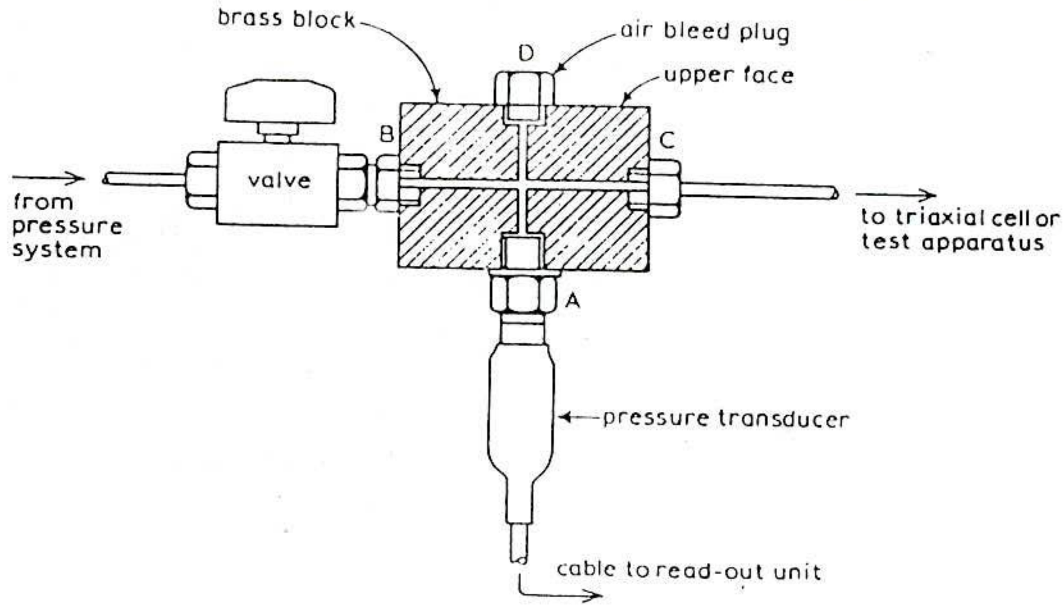


Fig. 3.5 Mounting Block for Pore Pressure Transducer

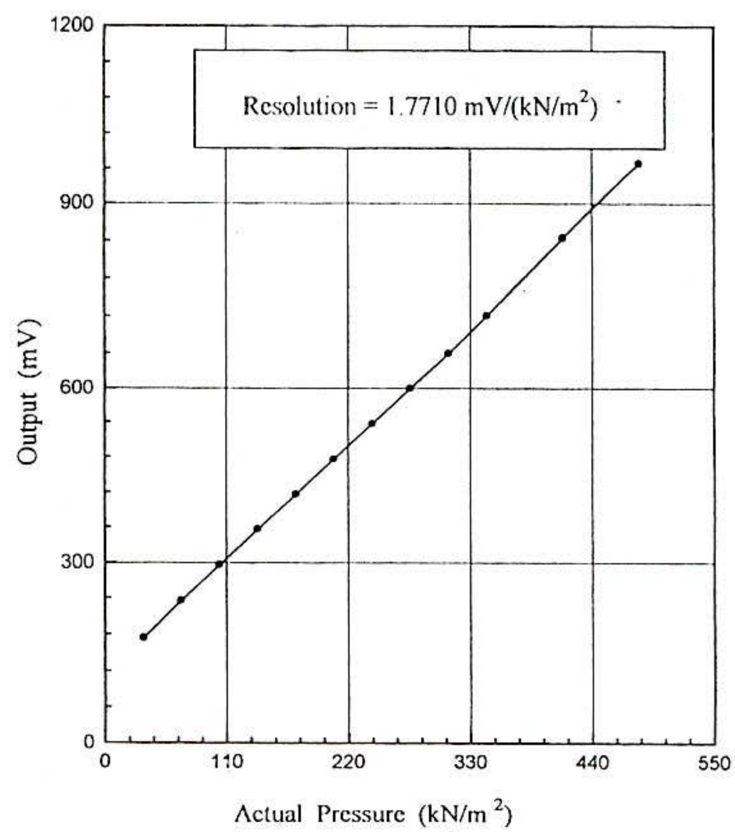


Fig. 3.6 Pore Pressure Transducer Calibration Curve

3.3 Tube Samplers

3.3.1 Fabrication of Samplers

Three samplers of different area ratios were fabricated from locally available mild steel tubes. The internal diameter and outside cutting edge angle of all samplers are constant which are equal to 38 mm and 5° respectively. The thickness (t) of the sample tubes are 1.5 mm, 3 mm and 6 mm.

3.3.2 Dimensions and Characteristics of Tube Samplers

Three samplers of different area ratio were fabricated. The area ratio of these samplers was changed by varying the thickness (t) of the sample tubes and hence the external diameter of the sampler tube (D_e) while keeping the internal diameter (D_s) of the samplers unchanged.

Sampler designations and the dimensions and characteristics of the samplers are presented in Table 3.1. The sampler designations T, M and H have been used to indicate sampler tubes of thickness thin, medium and high, respectively. In Table 3.1 the outside cutting edge angle (OCA) has been defined as the angle which the outside edge of the cutting shoe makes with a vertical plane. Length of each sampler was 127 mm. Internal diameter of the sample tube (D_s) and internal diameter at cutting shoe (D_i) of each sampler was equal, as such each sampler had no inside clearance (i.e., inside clearance ratio = 0%). External diameter of the sample tube (D_e) and external diameter at cutting shoe (D_w) of each sampler was equal, as such each sampler had no outside clearance (i.e., outside clearance ratio = 0%). The area ratio mentioned in Table 3.1, therefore, has been defined by the following equation (Hvorslev, 1949):

$$\text{Area Ratio} = (D^2_e - D^2_i)/ D^2_i \qquad (3.1)$$

Table 3.1 Dimensions and Characteristics of the Tube Samplers Used

Sampler Designation	t (mm)	D_e (mm)	D_i (mm)	D_e/t Ratio	Area Ratio (%)	OCA (°)
T	1.5	41	38	27.33	16.4	5
M	3.0	44	38	14.67	34.1	5
H	6.0	50	38	8.33	73.1	5

CHAPTER 4

LABORATORY INVESTIGATIONS

4.1 General

The use of undisturbed samples of soils for testing would be very desirable in the investigation of their behaviour. Such samples are seldom uniform due to the complex geological conditions acted upon them and as such, from the test results on such samples, it is rather difficult to generalize the behaviour of soils. Therefore, to study any specific effect on the behaviour of soils, it is considered essential to use uniform reconstituted samples prepared under controlled conditions in the laboratory (Hvorslev, 1960). In the investigations reported here, three selected reconstituted soils of Chittagong coastal region in Bangladesh were chosen for the tests and the physical properties of them have been presented. The laboratory investigations made on the soil samples have been described in details in this chapter. A Flow chart of the laboratory investigations is presented in Fig. 4.1.

4.2 Soils Used

The three reconstituted soils used in the tests are Banskhali soil, Anwara soil and Chandanaish soil. These sites are shown in Fig. 4.2.

For the present study disturbed soils were collected from the selected locations. The soils were taken by excavating up to depth of about 2.5 m to 3 m using hand shovels. Proper care was taken to remove any loose material, debris, coarse aggregates and vegetation from the bottom of the excavated pit. Disturbed samples were collected from the bottom of the borrow pit through excavation by hand shovels. All samples were packed in large polythene bags and were eventually transported to the Geotechnical Engineering Laboratory of Bangladesh University of Engineering and Technology (BUET), Dhaka.

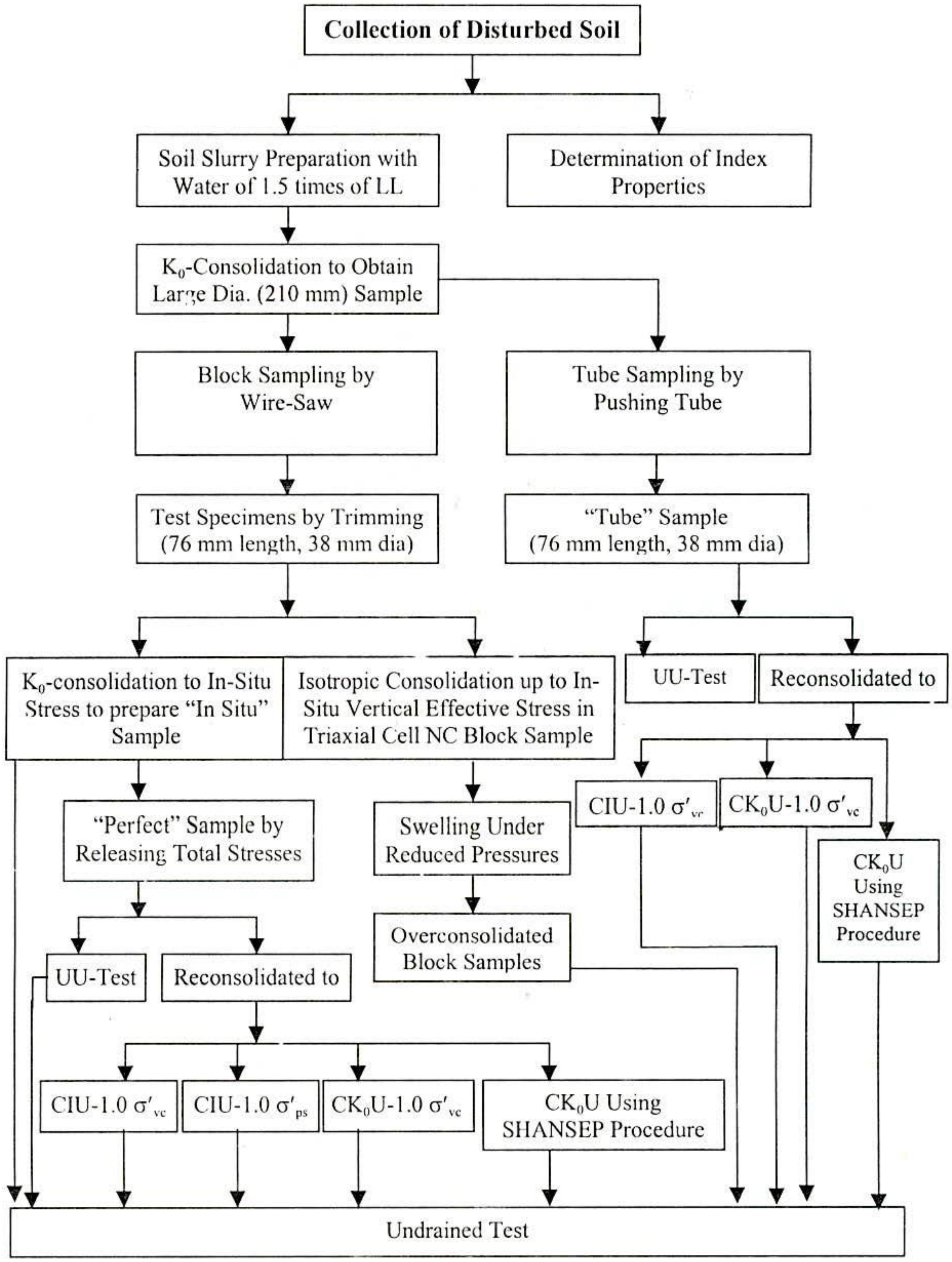


Fig. 4.1 Flow Chart for Laboratory Testing Programme on Reconstituted Soils from Chittagong Coastal Region in Bangladesh

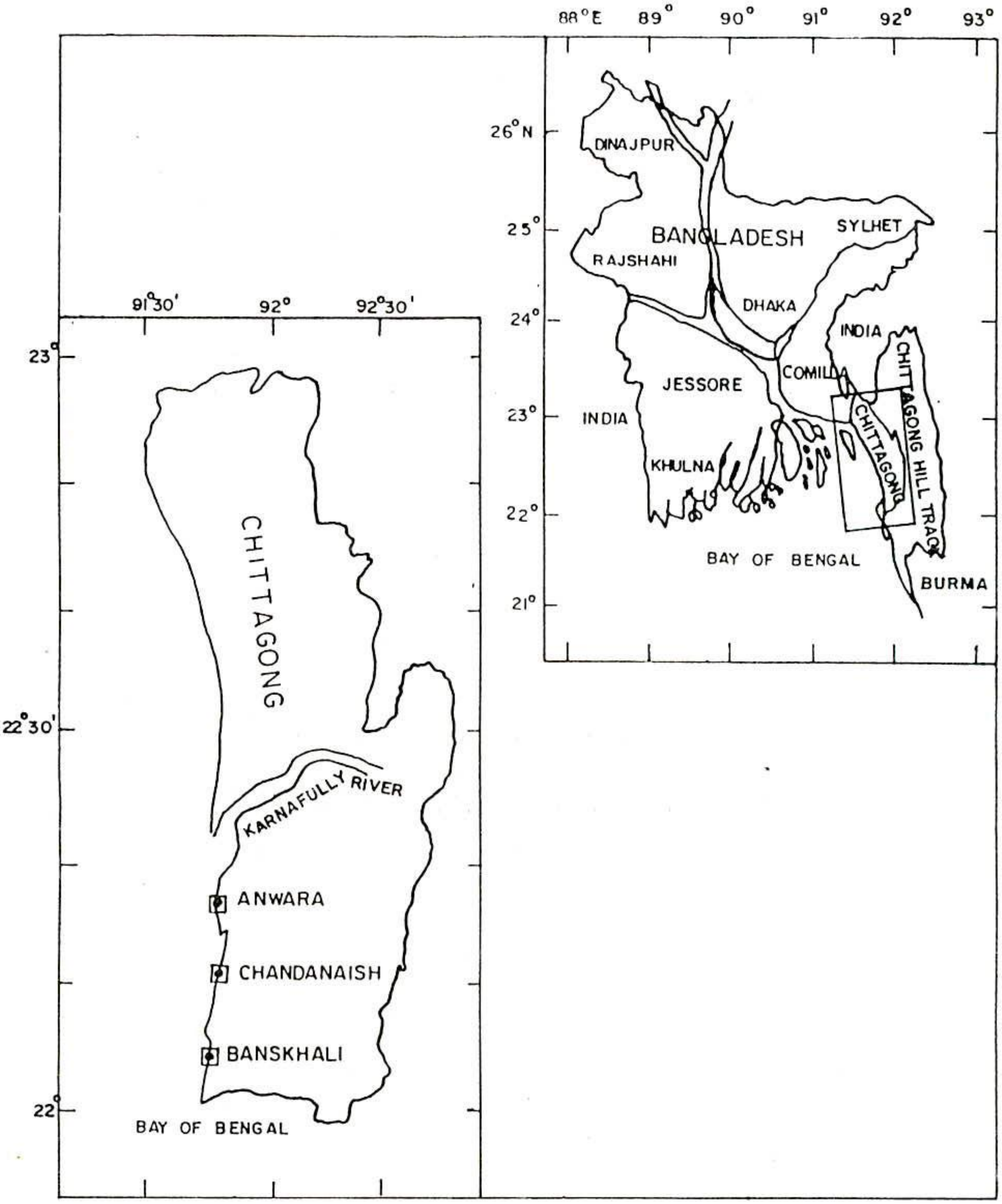


Fig. 4.2 Map of Bangladesh Showing the Locations of Coastal Sites for Soil Collection

4.3 Physical and Index Properties of Soils

Physical and index properties of the soils were determined in order to characterize the soils. The samples collected from the field were disturbed samples. These samples were then air-dried and the soil lumps were broken carefully with a wooden hammer so as to avoid breakage of soil particle. The following tests were performed to determine index properties of soil:

(i) Specific gravity

(ii) Atterberg limits

(iii) Grain size distribution

The specific gravity, liquid limit, plastic limit and plasticity index, and grain size distribution of all the soil samples were determined following the procedure specified in ASTM D854, BS1377, ASTM D424, ASTM D422 respectively. The percentage of sand, silt and clay were determined according to MIT Classification System (1931). The soils were also classified according to Unified Soil Classification System (Casagrande, 1948). Table 4.1 shows the index properties and classification of the soils used. The grain size distribution curves are shown in Fig. 4.3.

Table 4.1 Index Properties and Classification of the Coastal Soils Used

Index Properties and Classification	Location		
	Banskhali	Anwara	Chandanaish
Specific Gravity	2.69	2.70	2.72
Liquid Limit, LL	34	40	45
Plasticity Index, PI	10	16	20
% Sand	4	3	1
% Silt	80	75	67
% Clay	16	22	32
Activity	0.63	0.73	0.63
USCS Symbol	ML	CL	CL

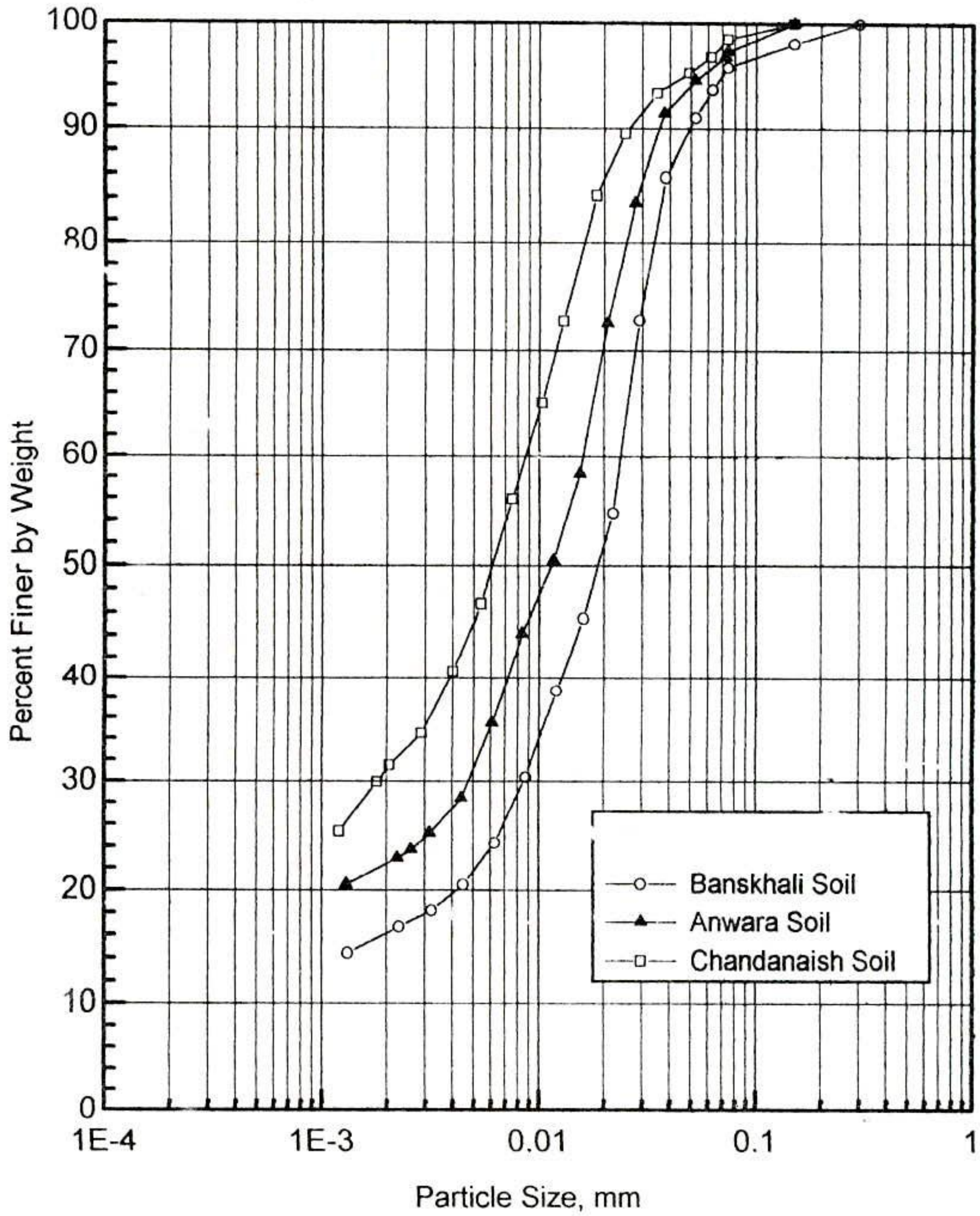


Fig. 4.3 Particle Size Distribution Curves for Three Coastal Soils

4.4 Preparation of Reconstituted Soil

4.4.1 General

Reconstituted soils are those which are prepared by breaking down natural soils, mixing them as slurry and reconsolidating them. Reconstituted soils are distinguished from both remoulded soils and from resedimented soils which are mixed as a suspension and allowed to settle from that state. Jardine (1985) discussed the difficulties of implementing detailed investigations of general stress-strain and strength properties using intact samples and it was found that the most comprehensive studies invariably employed reconstituted soil. Reconstituted soil enables a general pattern of behaviour to be established and comparisons with the response of intact samples may be used to identify any special features associated with fabric, stress history or bonding. The major advantages of using data from reconstituted soils are that the ambiguous and substantial effects of sampling of natural soils and inhomogeneity can be eliminated, while the essential history and composition of in-situ soils can be represented. The disadvantages are that the important effect of post-depositional process, such as ageing, leaching, etc. and of variations of composition and fabric are not included. So the pattern of behaviour for reconstituted soils discussed in the following chapters will be taken to represent that of young or unaged soils where no post-depositional processes have operated.

4.4.2 Preparation of Soil Slurry

Clay slurry with an initial water content well beyond the liquid limit has been commonly used as an initial state for sample preparation (Siddique, 1990, Hopper, 1992). Higher initial water contents provide higher degrees of saturation and higher freedom of particle orientation but require larger initial volumes and longer consolidation periods. Since a large volume of clay was required for preparing enough samples and also in order to reduce the consolidation time, it was essential to use an initial water content which was sufficient to yield a uniform and homogeneous slurry.

The samples were first air dried and powdered with the help of a motorized grinding machine. The powdered samples were then sieved through No. 40 sieve and the sieved samples were mixed with water at approximately 1.5 times the liquid limit to form soil slurry. The soil and water were thoroughly mixed by hand kneading to form a slurry to ensure full saturation. The product was then further remixed uniformly by using rotary laboratory mixer for about 30 minutes.

4.4.3 Consolidation of Slurry

For K_0-consolidation of slurry to form a uniform soil cake, a cylindrical consolidation cell of 210 mm diameter and 180 mm in height was used. A wire net and a 6 mm thick perforated steel disc were placed at the bottom of the mould of the cell. The wall of the cell was coated with a thin layer of silicon grease to minimize side friction and two filter papers were placed over the disc at the bottom of the cell. The slurry was then poured into the K_0-consolidation cell and stirred with steel rod to remove the entrapped air from the slurry. After removing air bubble, the top surface of the soil sample was levelled properly. At the top of the slurry, two filter papers followed by two perforated discs were placed to permit drainage. A wire net was used between the two discs for easy flow of water in the horizontal direction. A clearance of a few millimeter in between the perforated discs and inside edge of the cell was provided to eliminate side friction. Arrangements for preparation of reconsolidated sample in K_0-consolidation cell is shown in Fig. 4.4.

The required axial load of 150 kN/m^2 was gradually applied to the sample using a loading frame with proving ring. Initially the slurry was allowed to consolidate by the self weight of the sample and the weight of the porous discs for about 24 hours. Then a pressure of 14 kN/m^2 was applied to the sample for the next 24 hours. Similarly, pressure was increased gradually to the required value 150 kN/m^2. This pressure (150 kN/m^2) was maintained until the end of primary consolidation, which was indicated by the constant reading of compression dial gauge. It took about seven to eight days for the completion of primary consolidation. Rate of compression was very fast at initial stage of consolidation and then it gradually decreased with time. After the completion of consolidation, the top and bottom part of the cell were separated and

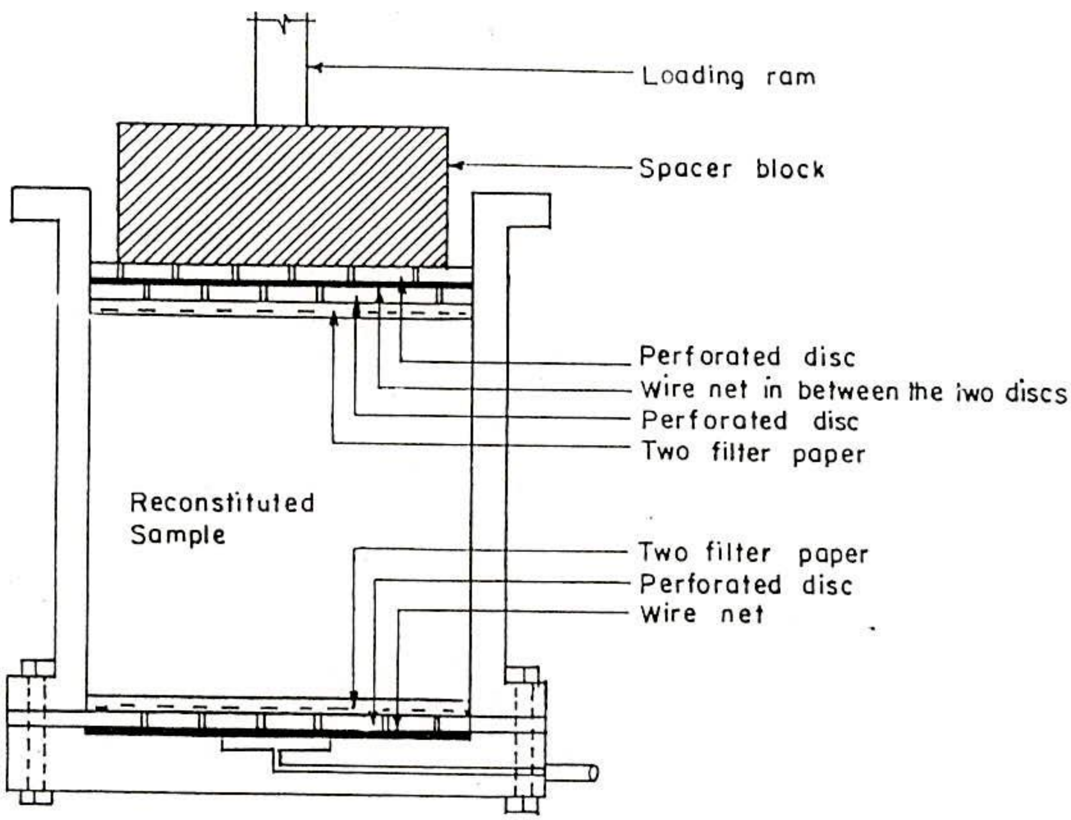


Fig. 4.4 Arrangements for Preparation of Reconstituted Sample in K_0-Consolidation Cell

the soil cake was extruded by using a mechanical extruder. A soil cake of about 114 mm to 127 mm (4.5 to 5 inch) thickness was obtained by the above procedure. The uniformity in density and moisture content of the consolidated soil cake was checked from moisture and density of specimens from two to three locations within the cake. The average water contents of the reconstituted normally consolidated soil samples from Banskhaii, Anwara and Chandanaish were 30.5 ± 0.5%, 31.2 ± 0.5% and 32 ± 0.75%, respectively and the respective average values of bulk density were 19.4 ± 0.07 kN/m^3, 19.7 ± 0.15 kN/m^3 and 19.5 ± 0.2 kN/m^3, respectively.

4.4.4 Selection of Overburden Pressure

Early in the research it was considered that a consolidation pressure (σ'_{vc}) of 276 kN/m^2 was about the minimum value which could make the clay soil just stiff enough to allow setting up specimens (Kirkpatrick and Khan, 1984). Latter on skill in testing improved and it was found possible to handle samples removed at σ'_{vc} = 150 kN/m^2 (after Kirkpatrick and Khan, 1984) but this produced insignificant difference in undrained stress-strain behaviour. So in the present study the reconstituted samples were prepared in a large oedometer cell by consolidation pressure equal to 150 kN/m^2.

4.5 K_0 of Soil Samples

The value of K_0 was determined from a series of anisotropic continuous loading consolidation tests with different stress ratios (σ'_r/σ'_a). The stress ratio for which the axial strain (ε_a) during consolidation is approximately equal to volumetric strain (ε_v) has been taken to be the value of K_0. The approximate values of K_0 of the reconstituted normally loaded soils from Banskhali, Anwara and Chandanaish have been found to be 0.47, 0.49 and 0.50, respectively.

4.6 Types of Test Samples

4.6.1 Normally Consolidated and Overconsolidated "Block" Samples

After extruding the reconstituted soil block from consolidation cell, the large soil block was sliced into small blocks by wire and knife. Each small block was trimmed by using piano wire, soil lathe and a split mould to prepare a sample of nominal dimensions of 38 mm diameter by 76 mm high. Then isotropic consolidation of the sample was carried out in the triaxial cell by applying allround consolidation pressure equal to 150 kN/m^2. These samples have been termed as normally consolidated "block" samples. After completion of consolidation under cell pressure of 150 kN/m^2, the cell pressure was reduced to allow swelling of the sample completely under different swelling pressures. The maximum consolidation pressure of 150 kN/m^2 was reduced to 100 kN/m^2, 75 kN/m^2, 30 kN/m^2, 15 kN/m^2, 7.5 kN/m^2 and 5 kN/m^2 to prepare overconsolidated samples of OCR values of 1.5, 2, 5, 10, 20 and 30, respectively. These types of samples have been termed as overconsolidated "block" samples. A back pressure of 270 kN/m^2 was used during isotropic consolidation and swelling of the samples.

4.6.2 "In Situ" Sample

Samples of nominal dimensions of 38 mm diameter by 76 mm high were prepared from large soil block by adopting the procedure as mention in Article 4.6.1. These samples were consolidated under K_0-condition in the triaxial cell to its "in situ" vertical effective stress, σ'_{vc} (i.e., 150 kN/m^2). A back pressure of 270 kN/m^2 was used during K_0-consolidation of the samples. These samples have been termed as "in situ" samples. The "in situ" samples prepared from the soils from Banskhali, Anwara and Chandanaish, have been designated as BI, AI and CI, respectively.

4.6.3 "Perfect" Sample

"Perfect" samples were prepared from "in situ" samples in the triaxial cell. The "in situ" shear stress, i.e., deviatoric stress of the "in situ" sample was first released from its "in situ" anisotropic stress condition. At this stage, the sample was subjected to an allround isotropic stress (i.e., cell pressure). The cell pressure was then reduced to zero and thereby the sample was subjected to zero total stress. This sample has been termed as "perfect" sample obtained from the complete release of the total "in situ" stresses. The "perfect" samples prepared from the soils from Banskhali, Anwara and Chandanaish, have been designated as BP, AP and CP, respectively.

4.6.4 "Tube" Sample

At first the reconstituted soil cakes were prepared from the disturbed samples in a large consolidation cell as described in Art. 4.4.3. Then the porous discs and filter papers were removed from the top of the consolidated soil cake in the large consolidation cell. Sample tubes of 38 mm inner diameter each but of different area ratios (AR) as mentioned in Table 3.1 were steadily pushed into the reconstituted soil cake. The samples were then extruded manually from the tubes by pushing a steel solid shaft of diameter slightly less than the tube samplers into the sample tubes. These samples have been termed as "tube" samples. The samples have been designated as T, M and H as shown in Table 4.2, and the tube samples prepared from the soils from Banskhali, Anwara and Chandanaish, have been designated as BT, BM, BH, AT, AM, AH, CT, CM and CH as shown in Table 4.2. T, M and H are the samplers of wall thicknesses of 1.5 mm (thin), 3 mm (medium) and 6 mm (high), respectively.

Table 4.2 Designation of "In Situ", "Perfect" and "Tube" Samples

Type of Specimen	Sampler Used for Sampling (wall thickness)	Sample Designation		
		Banskhali Soil	Anwara Soil	Chandanaish Soil
"In situ"	-	BI	AI	CI
"Perfect"	-	BP	AP	CP
"Tube"	T (thin)	BT	AT	CT
"Tube"	M (medium)	BM	AM	CM
"Tube"	H (high)	BH	AH	CH

4.7 Laboratory Testing Programme

The test programme consisted of carrying out the following types of tests:

(i) The compressibility and swelling characteristics of the three reconstituted coastal soils were performed by consolidating the samples under isotropic condition and K_0-condition in the triaxial cell.

(ii) Undrained triaxial compression tests were carried out on three isotropically normally consolidated "block" and eighteen overconsolidated "block" samples of reconstituted soils, seven tests on sample from each location. Among seven tests on each sample, one test for normally consolidated "block" sample and six tests on overconsolidated "block" samples of different overconsolidation ratios were carried out. These "block" samples were prepared by trimming from a large reconstituted block. In these tests, after the completion of consolidation, the samples were sheared at a deformation rate of 0.025 mm/minute in compression under undrained condition.

(iii) Undrained triaxial compression test was performed on three "in situ" samples of reconstituted soils from three locations in order to determine the reference "undisturbed" behaviour of the soils. In these tests after the completion of K_0-consolidation, each sample was sheared in compression under undrained condition at a deformation rate of 0.025 mm/minute.

(iv) Unconsolidated undrained (UU) triaxial compression test on "perfect" samples of reconstituted soils from three locations were carried out. In these tests, soon after simulation of the release of the total "in situ" stresses, each sample was subjected to a total isotropic stress (i.e., allround cell pressure) equal to "in situ" effective vertical stress under undrained condition. When the pore water pressure became steady, each sample was then sheared in compression under undrained condition at a deformation rate of 0.025 mm/minute.

(v) Undrained triaxial compression tests were run on fifteen reconsolidated "perfect" samples of reconstituted soils from three locations, five tests on sample from each location. In these tests, after the completion of reconsolidation, the samples were sheared at a deformation rate of 0.025 mm/minute in compression under undrained condition. The reconsolidation techniques will be discussed latter in this page.

(vi) Unconsolidated undrained (UU) triaxial compression tests were carried out on nine "tube" samples of three reconstituted soils. In these tests, soon after saturation (P-158), the samples were sheared at a deformation rate of 0.025 mm/minute in compression under undrained condition.

(vii) Undrained triaxial compression tests were carried out on twelve reconsolidated "tube" samples of reconstituted soils, four tests on sample from each location. In these tests, after the completion of reconsolidation, the samples were sheared at a deformation rate of 0.025 mm/minute in compression under undrained condition. The reconsolidation techniques will be discussed latter.

Reconsolidation Procedures

In the present investigation five different reconsolidation procedures were adopted for each reconstituted soil in order to assess their applicability to minimize "perfect" and "tube" sampling [as mentioned in steps (v) and (vii) above] disturbance effects. The following reconsolidation techniques were adopted:

(i) "Perfect" and "tube" samples were reconsolidated isotropically using hydrostatic stress equal to the in situ vertical effective stress, σ'_{vc} (i.e., 150 kN/m^2). "Perfect" or "tube" samples reconsolidated using this procedure and sheared in compression under undrained condition have been designated as CIU-1.0σ'_{vc}.

(ii) "Perfect" samples were reconsolidated isotropically using hydrostatic stress equal to isotropic effective stress, σ'_{ps} (i.e., 81.0 kN/m^2, 83.06 kN/m^2 and 85.5 kN/m^2 for the samples from Banskhali, Anwara and Chandanaish, respectively) of the "perfect" samples. "Perfect" samples reconsolidated using this procedure and sheared in compression under undrained condition have been designated as CIU-1.0σ'_{ps}.

(iii) Reconsolidation using Bjerrum (1973) procedure, i.e., K_0-consolidation using vertical effective stress equal to in situ vertical effective stress, σ'_{vc} was followed. In this procedure, "perfect" and "tube" samples were reconsolidated in the triaxial cell under K_0-conditions to vertical effective stress equal to in situ vertical effective stress, σ'_{vc} (i.e., 150 kN/m^2). The stresses were applied in two steps. First the total hydrostatic component was applied in one step. After the porewater pressure had dissipated, the deviator part of the in situ stress (σ'_a - σ'_r) was applied in stages. "Perfect" samples reconsolidated using this procedure and sheared in compression under undrained condition have been designated as CK_0U-1.0σ'_{vc}.

(iv) "Perfect" and "tube" samples were reconsolidated under K_0-condition using SHANSEP (Stress History and Normalized Soil Engineering Properties) procedure (Ladd and Foott, 1974) to vertical effective stress equal to 1.5 times σ'_{vc}. During reconsolidation three increments of the hydrostatic stress component were applied. Each increment of hydrostatic stress was followed by an increment of the deviatoric stress component. Drainage was allowed throughout and an increment of hydrostatic and deviatoric stresses was applied when the excess pore pressure resulting from the previous step had dissipated. "Perfect" or "tube" samples reconsolidated using this procedure and sheared in compression under undrained condition have been designated as SHANSEP-1.5.

(v) "Perfect" and "tube" samples were reconsolidated under K_0-condition using SHANSEP procedure (Ladd and Foott, 1974) to vertical effective stress equal to 2.5 times σ'_{vc}. During reconsolidation, the same procedure as that for sample SHANSEP-1.5 was followed, but instead of three increments hydrostatic and deviatoric stress components four increments of the stresses (hydrostatic and deviatoric) were applied during reconsolidation. "Perfect" or "tube" samples reconsolidated using this procedure and sheared in compression under undrained condition have been designated as SHANSEP-2.5.

4.8 Test Procedures

4.8.1 Undrained Triaxial Compression Test on Normally Consolidated and Overconsolidated "Block" Samples

Conventional undrained triaxial compression tests were performed on "block" samples as prepared in Laboratory.

Preparation and Setting-up of "Block" Sample

As the testing programme progressed, the samples of nominal dimensions of 38 mm diameter by 76 mm high were prepared by trimming from reconstituted samples using piano wire, a soil lathe and a split mould. The initial water content of the sample was determined from the trimmings. The dimensions and the weight of sample were measured with a scale and the balance. After that, to prepare the "in situ" sample total laboratory works were divided into three phases, namely, placing of sample in the triaxial cell, saturation of the sample and finally Isotropic consolidation and swelling in the cell. These phases were discussed as follow:

Placing of Sample in the Cell

(i) Prior to placing the trimming sample and cylindrical cell cap, the base of the triaxial cell was connected with the pressure lines and a pore pressure transducer. Then the entire pressure lines with both bottom and top drainage lines, were flushed with deaired and distilled water to get rid of any entrapped air bubbles. To avoid entrapped air in the line, water was allowed to run slowly. After flushing the drainage control valves were closed.

(ii) Porous stones which were deaired and cleaned by boiling in distilled water, filter papers, and a membrane were placed for the prompt sealing of the sample after trimming.

(iii) One filter paper followed by one saturated porous stone was placed at each end of the soil sample and vertical strip of filter paper were used along the periphery of the sample to permit double drainage and drainage from radial boundary respectively during consolidation. Both ends of the filter paper strip were extended to the porous stones. Each sample was enclosed in a rubber membrane with the help of membrane stretcher and was placed on the pedestal of the cell base. The top cap was placed properly and the top and bottom rubber "O" rings were placed over the rubber membrane.

(iv) Care was taken to keep the sample concentric with both the pedestal and the top loading cap.

(v) By raising the top nut and upper assembly, the cell cap was placed and three clamps were fixed properly so that the apex of the loading ram was just in contact with the centrally placed ball of the top cap.

(vi) Then the cell was filled with water by opening the bleed valve which was at the top of upper assembly. After filling the cell chamber, the bleed valve was closed.

Saturation of Samples

(i) Even though the sample itself was fully saturated, there was always a certain amount of air entrapped between the rubber membrane and the sample, and perhaps, in the previously saturated porous stones and soaked filter papers. Initially the sample was kept under a cell pressure of 34.5 kN/m^2 (5.0 psi) using a dashpot and control cylinder system and back pressure of 24.1 kN/m^2 (3.5 psi) using an overhead water bottle for overnight saturation to remove the entrapped air bubbles within the sample.

(ii) Both bottom and top drainage lines were kept open. The bottom drainage line was connected to the back pressure system and the top drainage line was subjected to atmospheric pressure.

(iii) The proving ring and strain gauge were placed. The base of the cell was raised slowly by operating the wheel handle manually until the proving ring was just in contact with the top of the loading ram. This was showed by the movement of the proving ring dial gauge indicator.

(iv) At this stage, The cell pressure and back pressure using the dash pot and control cylinder system were gradually increased to saturate the sample with the pressure increment of 10 psi until the B-value of the sample reached 0.97 or above. During the application of pressures, the cell pressure was always maintained 6.89 kN/m^2 (1.0 psi) higher than the back pressure.

(v) The volume change of the sample and the corresponding axial strain were also measured. Immediately after saturation, effective cell pressure applied to the sample was 6.89 kN/m^2 (1.0 psi).

Isotropic Consolidation of Sample

(i) After full saturation was achieved (Skempton's B-value was greater than 0.97) the consolidation phase was commenced. At this stage, isotropic consolidation was carried out by applying allround consolidation pressure equal to 150 kN/m^2. Consolidation was generated by using the dashpot and control cylinder system. The drainage valves were opened.

(ii) During this operation excess pore water pressure was generated within the sample. The sample was then allowed to consolidate until the excess pore water pressure completely dissipated.

(iii) During isotropic consolidation the volume change and the compression of the sample were measured. After completion of consolidation, the sample thus obtained in the triaxial cell has been termed as normally consolidated "block" sample.

(iv) After isotropic consolidation the samples were attempted to swell back with the required overconsolidation ratios. These types of samples were known as overconsolidated "block" samples. The maximum isotropic consolidation stress of 150 kN/m^2 was reduced to 100 kN/m^2, 75 kN/m^2, 30 kN/m^2, 15 kN/m^2, 7.5 kN/m^2 and 5 kN/m^2 to prepare samples of overconsolidated "block" samples of OCR values of 1.5, 2, 5, 10, 20 and 30.

Shearing of "Block" Samples

The shearing of "block" samples were carried out on the prepared normally consolidated and overconsolidated "block" samples in the triaxial cell in the following way:

(i) The drainage valves were closed and the strain gauge was set at zero position. The proving ring dial gauge reading and the corresponding pore pressure transducer reading were recorded.

(ii) The sample was then sheared and the proving dial gauge readings and the corresponding pore pressure transducer readings were recorded at specified deformation of the sample. A deformation rate of 0.025 mm/minute was used. Shearing was continued until the proving ring dial reading remained constant or decreased for a few readings of strain dial gauge.

(iii) At the end of the test, cell base was lowered and cell pressure was released. The cell was disassembled and sample was carefully removed from the cell. The weight of the sample was taken and its water content was determined. The qualitative effective stress paths for normally consolidated and overconsolidated "block" samples are shown in Fig. 4.5.

4.8.2 Undrained Triaxial Compression Test on "In Situ" Samples

Conventional undrained triaxial compression tests were performed on laboratory simulated "in situ" samples of three reconstituted samples.

K_0-Consolidation of Sample

(i) After full saturation was achieved as described above in "block" sample, the consolidation phase was commenced. At this stage, the required effective radial stress required for K_0-consolidation was generated by using the dashpot and control cylinder system. The drainage valves were opened and total deviatoric load was applied in small increments to attain K_0-condition.

(ii) During this operation excess pore water pressure was generated within the sample. The sample was then allowed to consolidate until the excess pore water pressure completely dissipated.

(iii) During K_0-consolidation the volume change and the compression of the sample were measured. After completion of consolidation, the sample thus obtained in the triaxial cell has been termed as "in situ" sample.

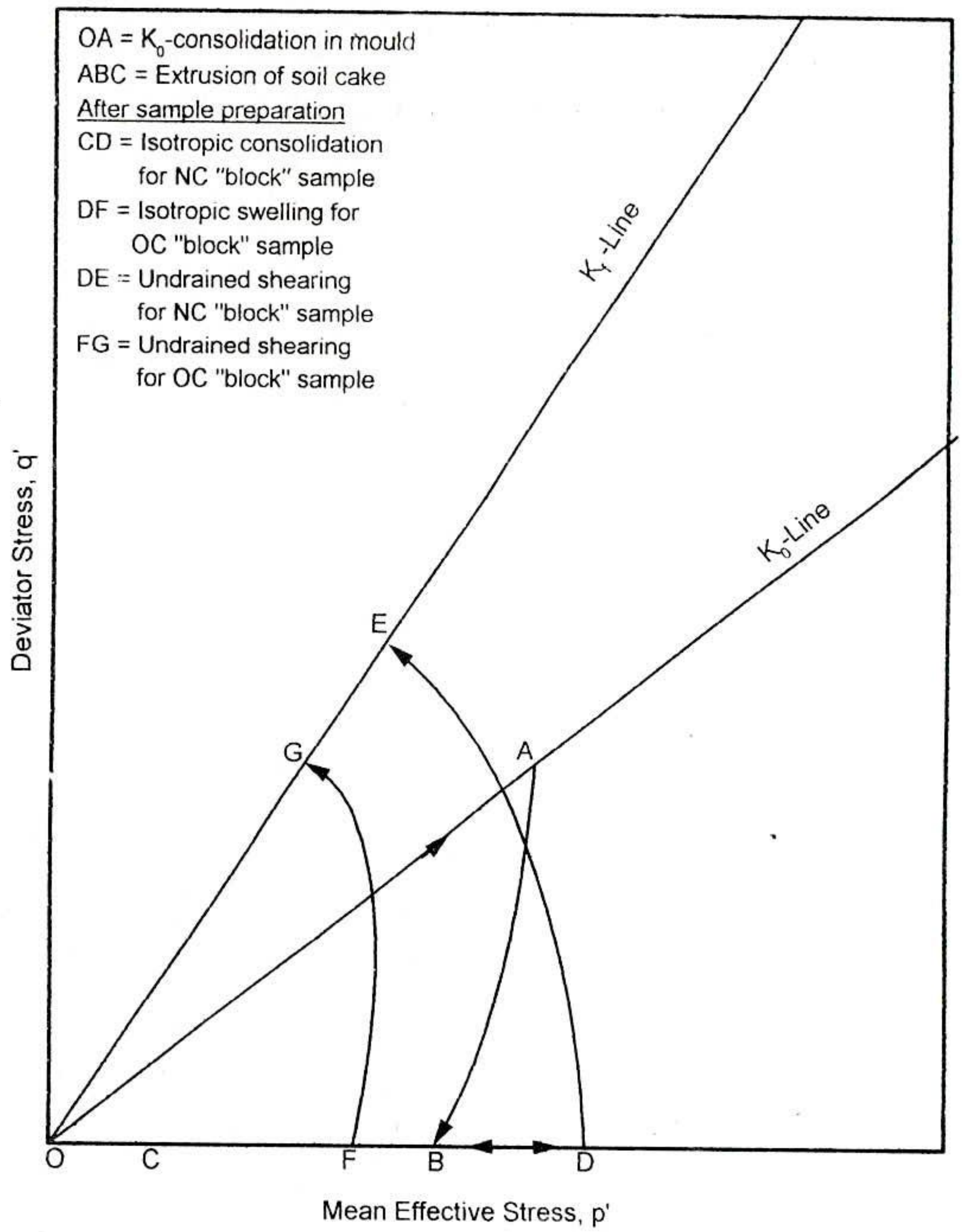


Fig. 4.5 Qualitative Effective Stress Path for Isotropically Normally Consolidated and Overconsolidated Block Samples

Shearing of "In Situ" Sample

The shearing of "in situ" samples were carried out on the prepared "in situ" sample in the triaxial cell as described above in "block" sample. The weight of the sample was taken and its water content was determined. The qualitative effective stress path for "in situ" sample is shown in Fig. 4.6.

4.8.3 Undrained Triaxial Compression Test on "Perfect" Sample

The undrained triaxial compression tests were carried out on each "perfect" sample of three locations. At first K_0-consolidation performed on each sample under in situ stress. K_0-consolidation was carried out following the procedure as mentioned in Article 4.8.2. After completion of K_0-consolidation the following procedure was adopted.

(1) Top and bottom drainage valves were closed.

(2) Deviator stress, i.e., in situ shear stress was unloaded and the pore pressure reading was recorded. At this stage the sample was subjected to only an isotropic effective stress.

(3) The sample was then subjected to zero total stress by reducing the cell pressure to zero.

Following the above procedure "perfect" sampling was simulated in the triaxial cell. At this stage, a cell pressure equal to that required for isotropic consolidation was applied to the sample. The pore pressure transducer reading was recorded at the steady state. At the same time, the proving ring dial gauge reading was recorded and strain dial gauge was set at zero position. Then the sample was sheared in compression under undrained condition at a constant deformation rate of 0.025 mm/min. The qualitative effective stress path for "perfect" sample is shown in Fig. 4.7.

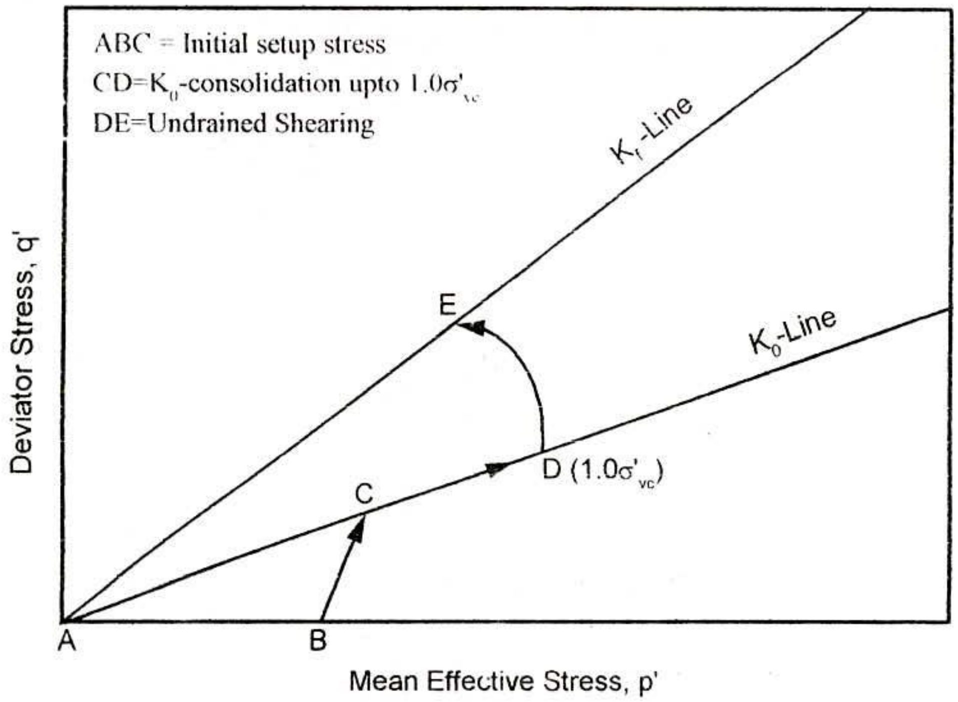


Fig. 4.6 Qualitiative Effective Stress Path for Undrained Triaxial Test on "In Situ" Sample

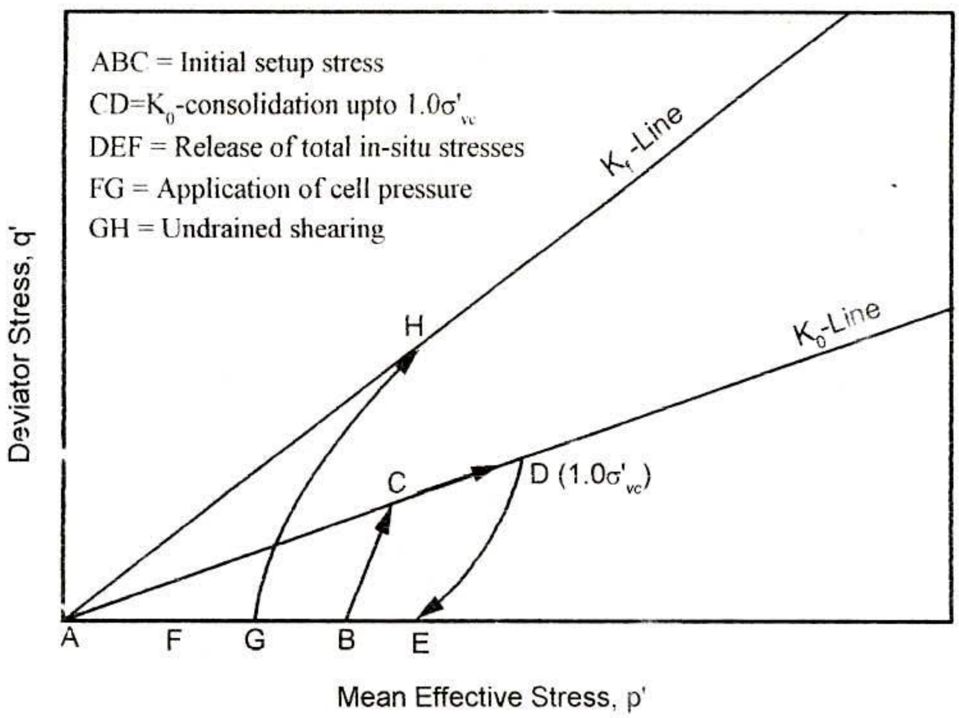


Fig. 4.7 Qualitative Effective Stress Path for Undrained Triaxial Test on "Perfect" Sample

4.8.4 Undrained Triaxial Compression Test on "Tube" Samples

The test was performed at two stages. Firstly, the sample was saturated as mentioned in Article 4.8.1. Finally, each sample was sheared in compression under undrained condition at a constant deformation rate of 0.025 mm/min. The qualitative effective stress path for UU-test on "tube" sample is shown in Fig. 4.8.

4.8.5 Undrained Triaxial Compression Test on Reconsolidated "Perfect" and "Tube" Samples Using Bjerrum and SHANSEP Procedures

Undrained triaxial compression tests were performed on "perfect" and "tube" samples from three different locations. In these tests "tube" and "perfect" samples were consolidated in the triaxial cell equal to and beyond the in situ stress (σ'_{vc} = 150 kN/m^2). The sample tube of area ratio = 34.1% (t = 3 mm) and outside cutting edge angle of 5° was used to collect "tube" samples. The test procedures were similar to those described in Article 4.8.2. In this test, the sample was reconsolidated up to vertical effective stress of 150 kN/m^2, 225 kN/m^2 and 375 kN/m^2 which are respectively 1.0 times, 1.5 times and 2.5 times the effective in situ vertical stress, σ'_{vc} (i.e. 150 kN/m^2). The qualitative effective stress paths for reconsolidated "perfect" samples up to above three stresses are shown in Fig. 4.9, Fig. 4.10(a) and Fig. 4.10(b), respectively, and the qualitative effective stress paths for reconsolidated "tube" samples up to above three stresses are shown in Fig. 4.11, Fig. 4.12(a), Fig. 4.12(b), respectively.

4.8.6 Undrained Triaxial Compression Test on Isotropically Reconsolidated "Perfect" and "Tube" Samples

This test was carried out on each "perfect" and "tube" samples of the three locations. In this test, initially the procedures mentioned as in Article 4.8.2 were followed for "tube" samples and the procedures mentioned in Article 4.8.3 to simulate "perfect" sample in the triaxial cell were followed for "perfect" sample. The samples were then allowed to reconsolidate under isotropic stress condition. For "perfect" samples CIU-

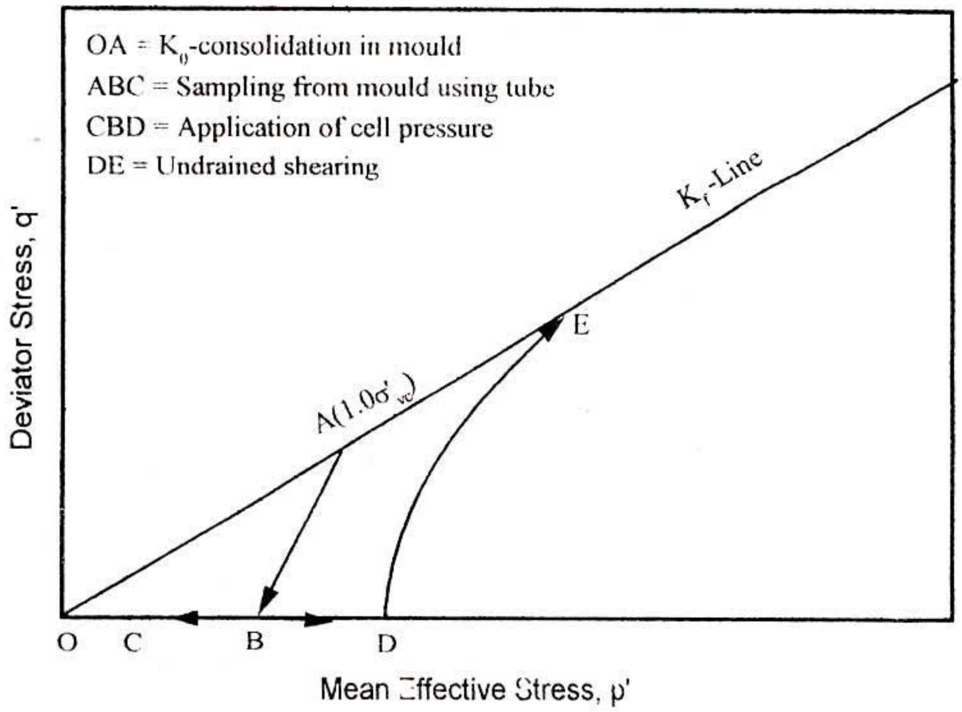


Fig. 4.8 Qualitative Effective Stress Path for Undrained Test on Tube Samples

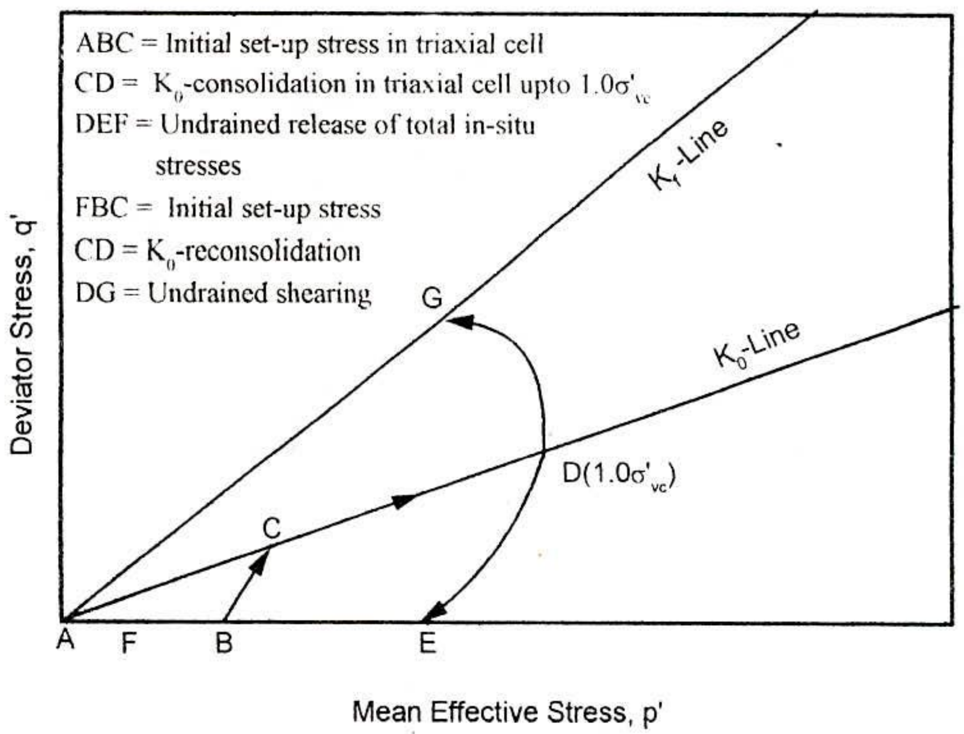
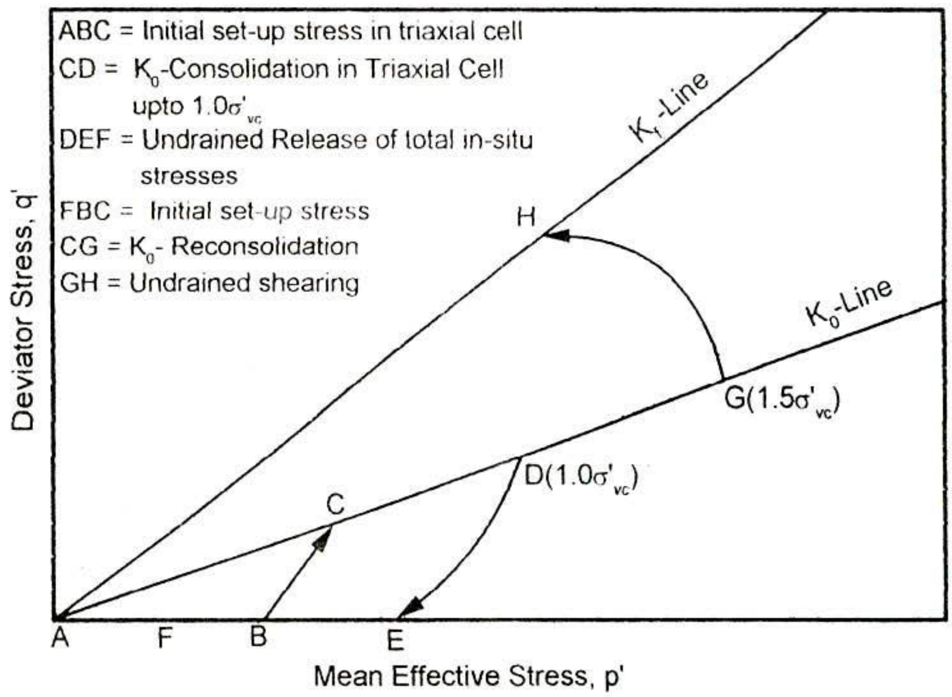
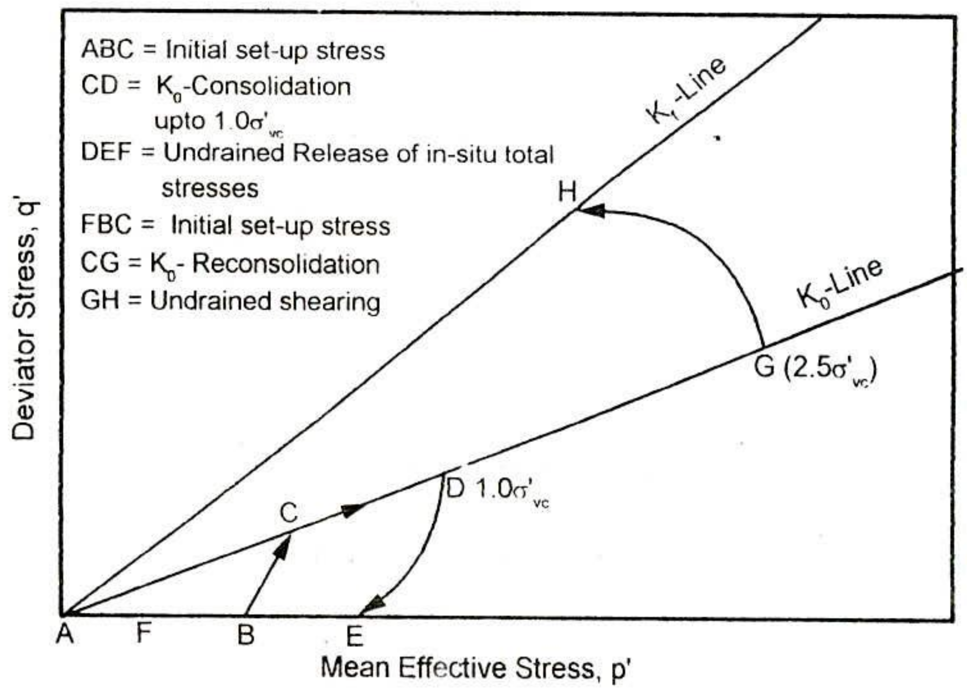


Fig. 4.9 Qualitative Effective Stress Path for Reonsolidated "Perfect" Sample up to In-Situ State ($1.0\sigma'_{vc}$)



(a)



(b)

Fig. 4.10 Qualitative Effective Stress Path for Reconsolidated "Perfect" Sample using SHANSEP Procedures (a) Reconsolidation up to $1.5\sigma'_{vc}$ (b) Reconsolidation to $2.5\sigma'_{vc}$

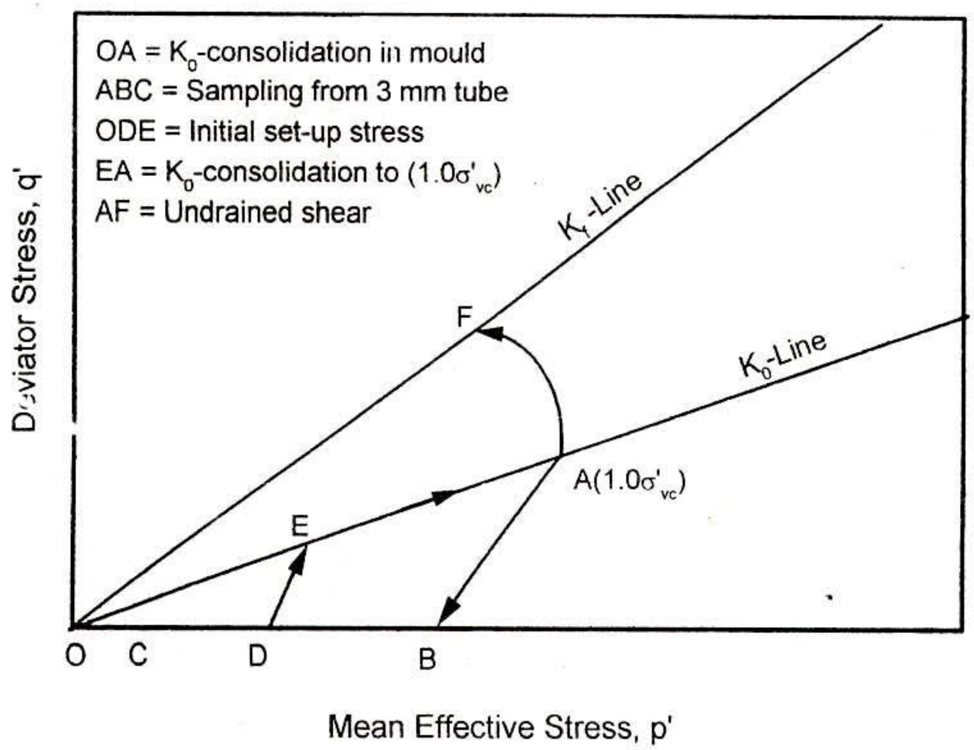
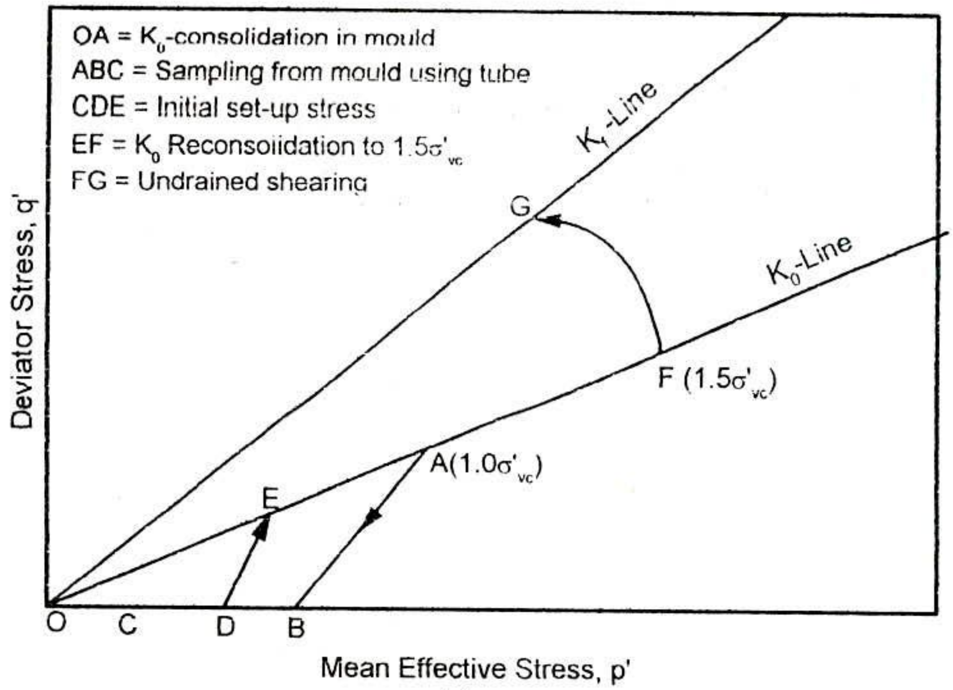
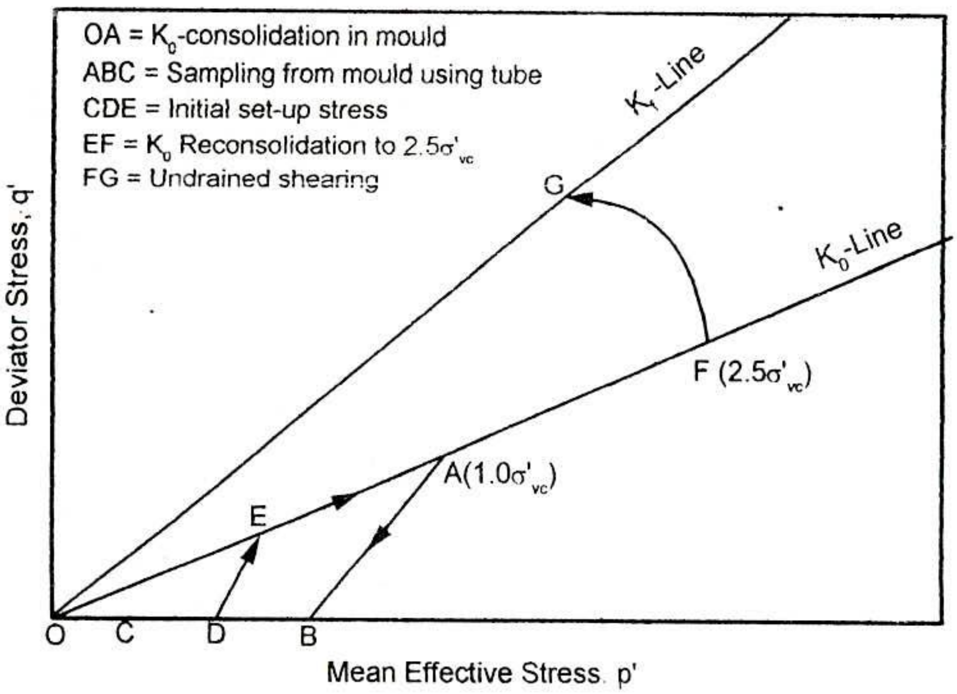


Fig. 4.11 Qualitative Effective Stress Path for Reconsolidated "Tube" Samples Using Bjerrum Procedure



(a)



(b)

Fig. 4.12 Qualitative Effective Stress Path for Undrained Test on Reconsolidated "Tube" Samples using SHANSEP Procedures

(a) Reconsolidation upto $1.5\sigma'_{vc}$

(b) Reconsolidation upto $2.5\sigma'_{vc}$

$1.0\sigma'_{vc}$ and CIU-σ'_{ps} tests were followed, while for "tube" samples CIU-$1.0\sigma'_{vc}$ test was followed. A back pressure of 270 kN/m2 was maintained during reconsolidation. The volume change and consolidation of the sample were monitored. Reconsolidation was continued until the volume change indicator showed constant reading. After completion of reconsolidation, the samples were sheared in compression under undrained condition at a constant deformation rate of 0.025 mm/min. The qualitative effective stress paths for isotropically reconsolidated "perfect" and "tube" samples are shown in Fig. 4.13 and Fig. 4.14, respectively.

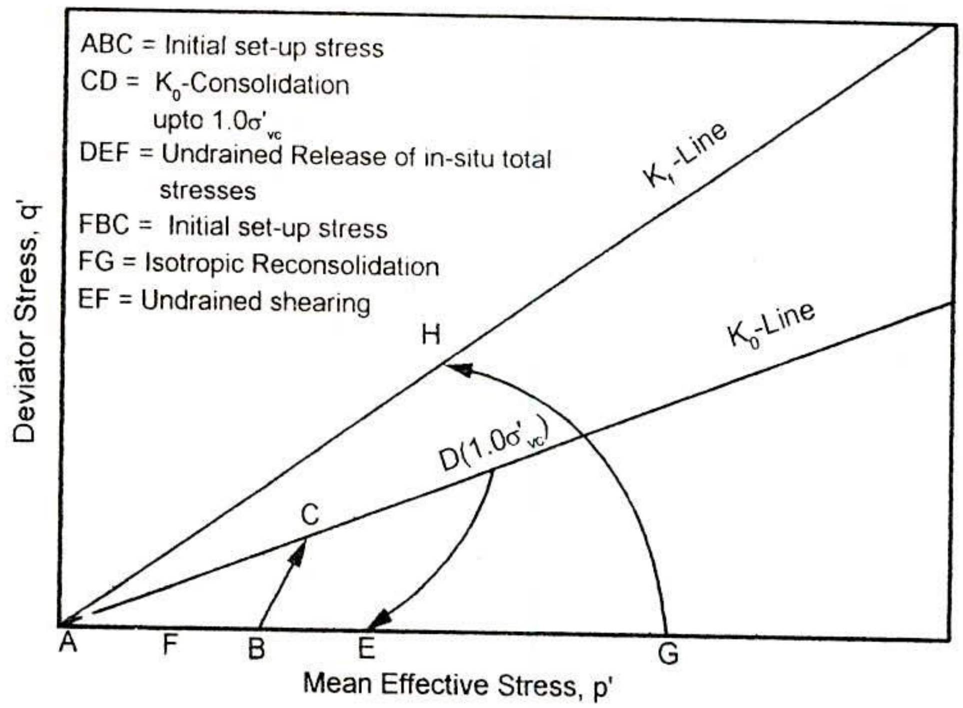


Fig. 4.13 Qualitative Effective Stress Path for Reconsolidated "Perfect" Sample up to $1.0\sigma'_{vc}$ or $1.0\sigma'_{ps}$ using Isotropic Loading

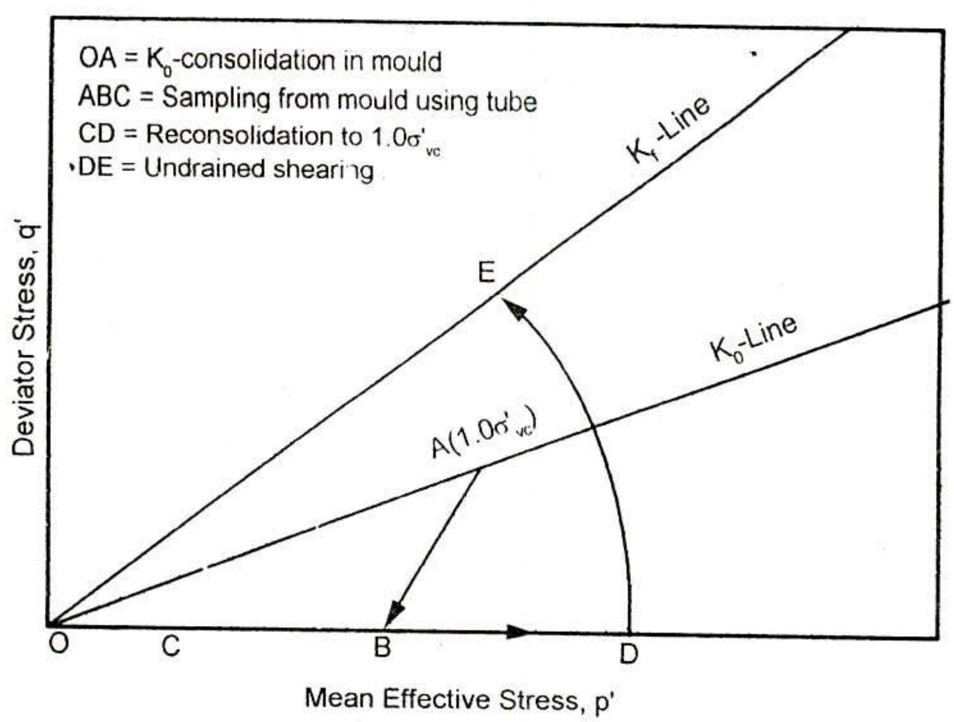


Fig. 4.14 Qualitative Effective Stress Path for Isotropic Reconsolidated "Tube" Samples

CHAPTER 5

UNDRAINED BEHAVIOUR OF ISOTROPICALLY NORMALLY CONSOLIDATED AND OVERCONSOLIDATED SAMPLES

5.1 General

To study the behaviour of isotropic normally consolidated and overconsolidated soil during shear, undrained triaxial compression tests were conducted on the normally consolidated and overconsolidated "block" samples of reconstituted soils from Banskhali, Anwara and Chandanaish as indicated in Fig 4.5 of Chapter 4. The maximum consolidation pressure, σ'_{cm} used was 150 kN/m^2 and the minimum consolidation pressure used for overconsolidated samples was of the order of 5.0 kN/m^2. In all these tests, the samples were sheared to failure by increasing the vertical stress, σ_1 and keeping the cell pressure, σ_3 constant. A total of six overconsolidated samples for each reconstituted soil were sheared from different pre-shear stresses. The overconsolidation ratios of the samples were 1.5, 2, 5, 10, 20 and 30. Each sample was initially subjected to a different pre-shear consolidation stress corresponding to the overconsolidation ratio. The test results have been presented and the effect of overconsolidation on different aspects of the behaviour of the soils has been discussed.

The geotechnical properties of these reconstituted coastal soils such as undrained initial tangent modulus (E_i), secant modulus at half the half the peak deviator stress (E_{50}), K_0 (one- dimensional) and isotropic compression and swelling indices (C_c and C_s), critical state pore pressure parameter (Λ), effective stress friction angle (ϕ'), undrained shear strength (s_u) and other important soil parameters and characteristics have been determined for different stress history. Evaluation of some normalized parameters for these reconstituted soils is presented. Roscoe and Hvorslev state boundary surfaces with critical state soil parameters have been established. Correlations of some soil constants with plasticity index have also been established.

5.2 Consolidation and Swelling Characteristics

The consolidation characteristics of reconstituted normally consolidated samples of three coastal soils from Banskhali, Anwara and Chandanaish were investigated by carrying out incremental loading consolidation tests in triaxial cell. Void ratio versus pressure relationships for isotropic consolidation and swelling of the soils are shown in Fig. 5.1. The compression indices ($C_c^{(iso)}$) are 0.265, 0.295 and 0.313 and the swelling indices ($C_s^{(iso)}$) are 0.044, 0.053 and 0.068 respectively for the three soils from Banskhali, Anwara and Chandanaish, respectively. The same relationships for K_0-consolidation and swelling of Banskhali, Anwara and Chandanaish soils are shown in Fig. 5.2. Fig. 5.2 shows the compression indices ($C_c^{(Ko)}$) of 0.256, 0.288 and 0.309 and the swelling indices of ($C_s^{(Ko)}$) are 0.039, 0.048 and 0.06, respectively for the three soils. In any consolidation test the swelling index is the slope of the line which is the average of swelling and recompression lines. The values of C_c and C_s for the three soils are listed in Table 5.1.

From the consolidation characteristics of the soils, it is clear that the recoverable strain energy increased with the increase of plasticity of the soils. In case of isotropic consolidation and swelling, $C_s^{(iso)}$ of Banskhali, Anwara and Chandanaish soils are 16.6%, 18% and 21.7%, respectively of their compression indices. While in case of K_0-consolidation and swelling, $C_s^{(Ko)}$ of Banskhali, Anwara and Chandanaish soils are 15.2%, 16.7% and 19.4%, respectively of their compression indices. From this swelling behaviour of the three soils, it may be noted that the percentage of swelling is increased with the increase of plasticity for both isotropic and K_0 conditions.

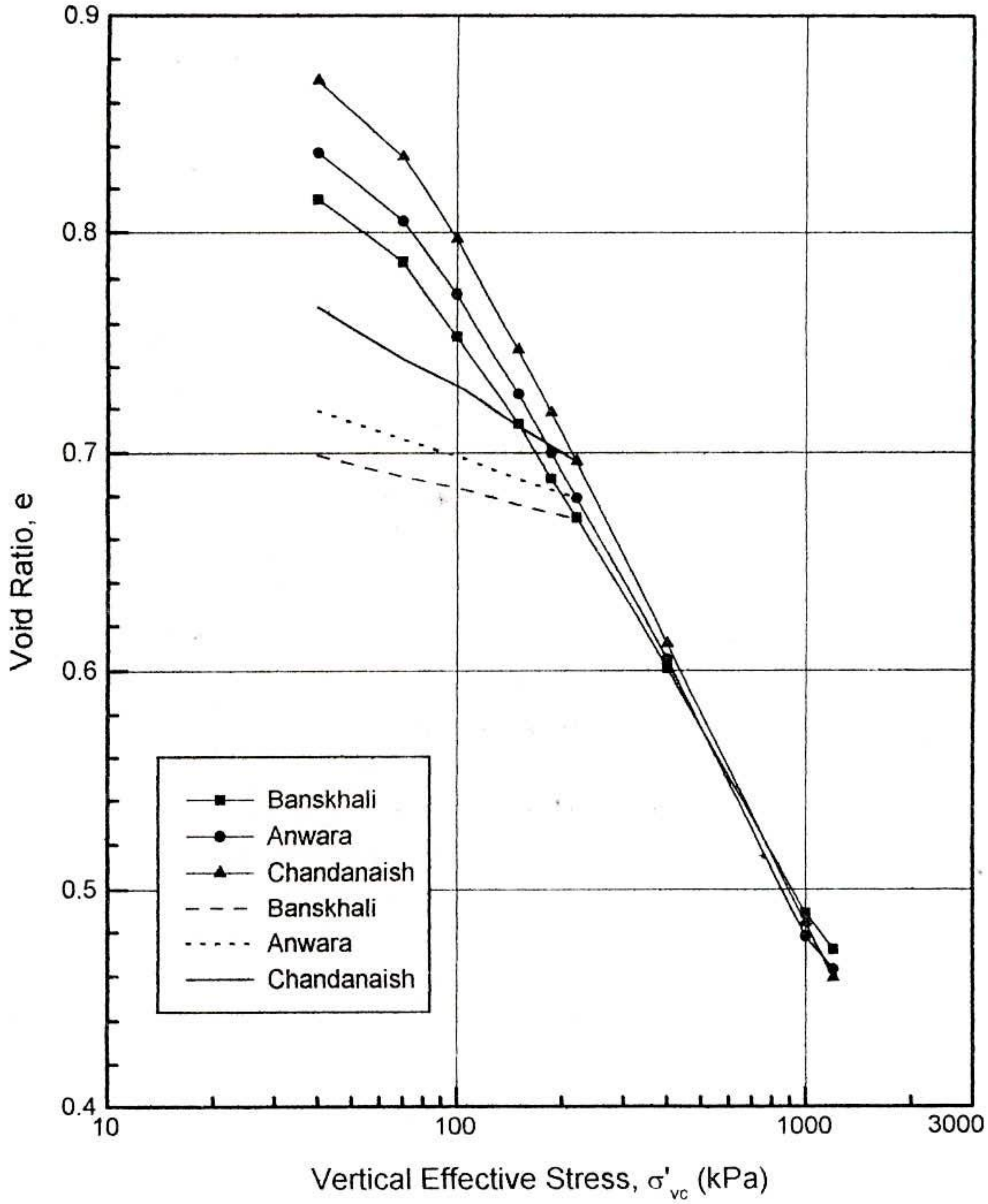


Fig. 5.1 Isotropic Compression and Swelling Curves for Reconstituted "Block" Samples of Three Coastal Soils

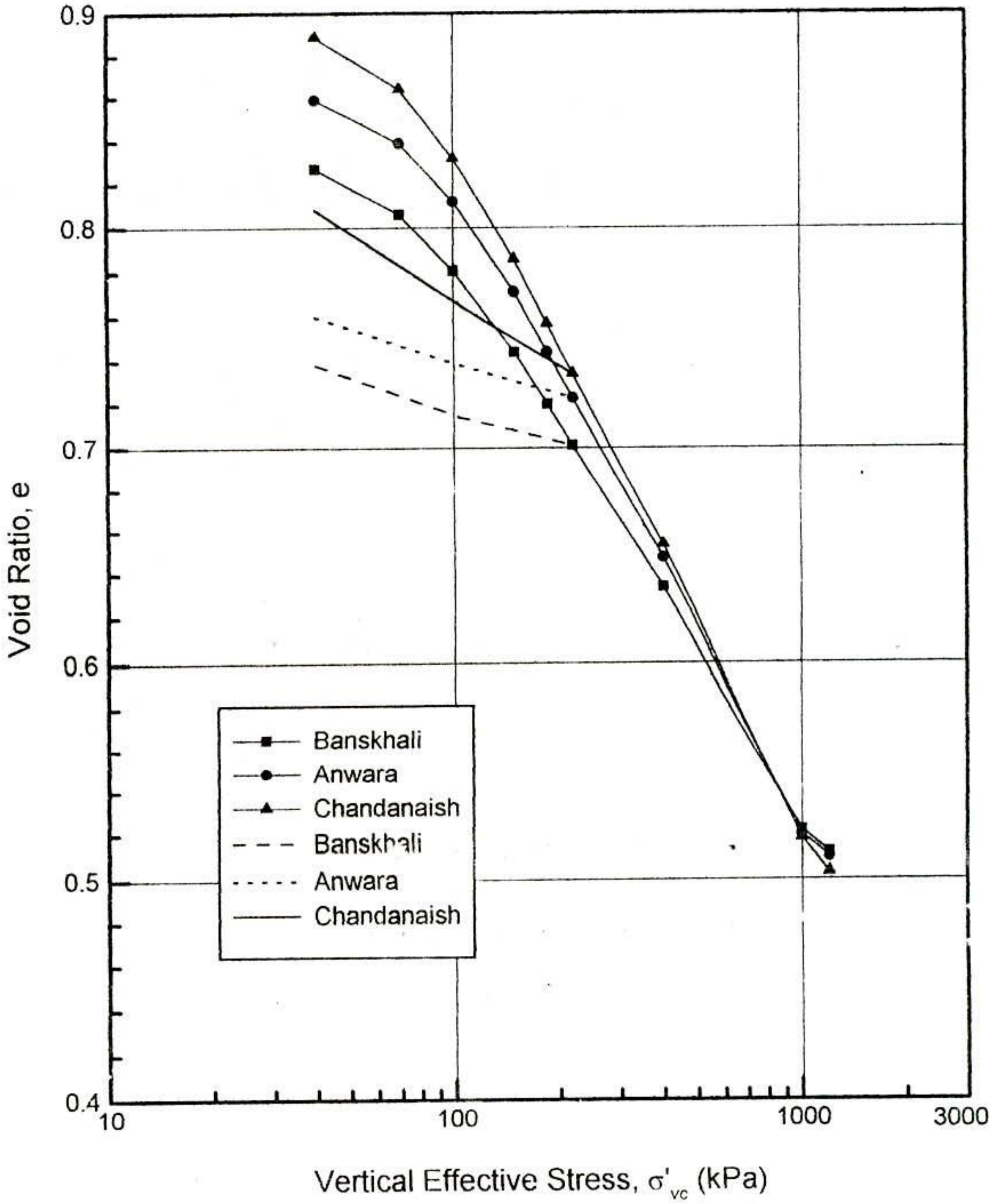


Fig. 5.2 K_0-Compression and Swelling Curves for Reconstituted Block Samples of Three Coastal Soils

Table 5.1 Comparison of Compressibility Parameters of the Reconstituted Samples of the Three Coastal Soils

Location of Soil	LL (%)	P! (%)	From K_0-Consolidation		From Isotropic Consolidation	
			$C_c^{(Ko)}$	$C_s^{(Ko)}$	$C_c^{(iso)}$	$C_s^{(iso)}$
Banskhali	34	10	0.256	0.039	0.265	0.044
Anwara	40	16	0.288	0.048	0.295	0.053
Chandanaish	45	20	0.309	0.060	0.313	0.068

5.3 Undrained Behaviour of Normally Consolidated "Block" Samples

5.3.1 Effective Stress Paths

The effective stress paths of the samples in q′- p′ space are shown in Fig. 5.3 in the plot. The effective stress paths for the three soils are found to be somewhat similar in this plot. The end points of the samples also seem to lie on a straight line which may pass through the origin for average line.

5.3.2 Stress-Strain Relationship

Fig. 5.4 presents the variation of axial strain with deviator stress for reconstituted isotropically normally consolidated samples of Banskhali, Anwara and Chandanaish soils. From the Fig. 5.4, it can be observed that the stress-strain diagram is nonlinear. The elastic modulus is higher at small strains and gradually decreases with increasing of strain. It can be seen from the figure that the stress-strain curves for the three normally consolidated samples show mild peak and a slight strain softening beyond the peak. At larger strain, the stress of normally consolidated samples increases very slowly and the three coastal soils exhibit similar behaviour. The undrained strength (s_u), E_i, and E_{50} were determined from stress-strain data. The undrained shear properties for reconstituted isotropic normally consolidated soils are presented in Table 5.2. The undrained shear strength is determined from half of the peak deviator

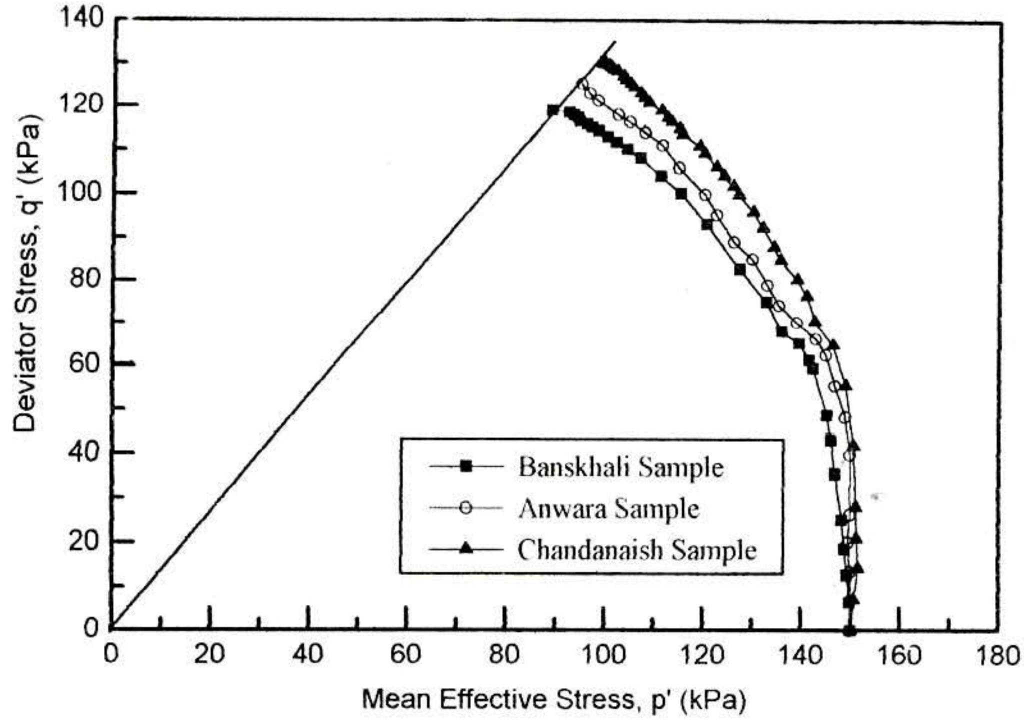


Fig. 5.3 Effective Stress Path for Three Reconstituted Normally Consolidated Samples ("Block" Samples)

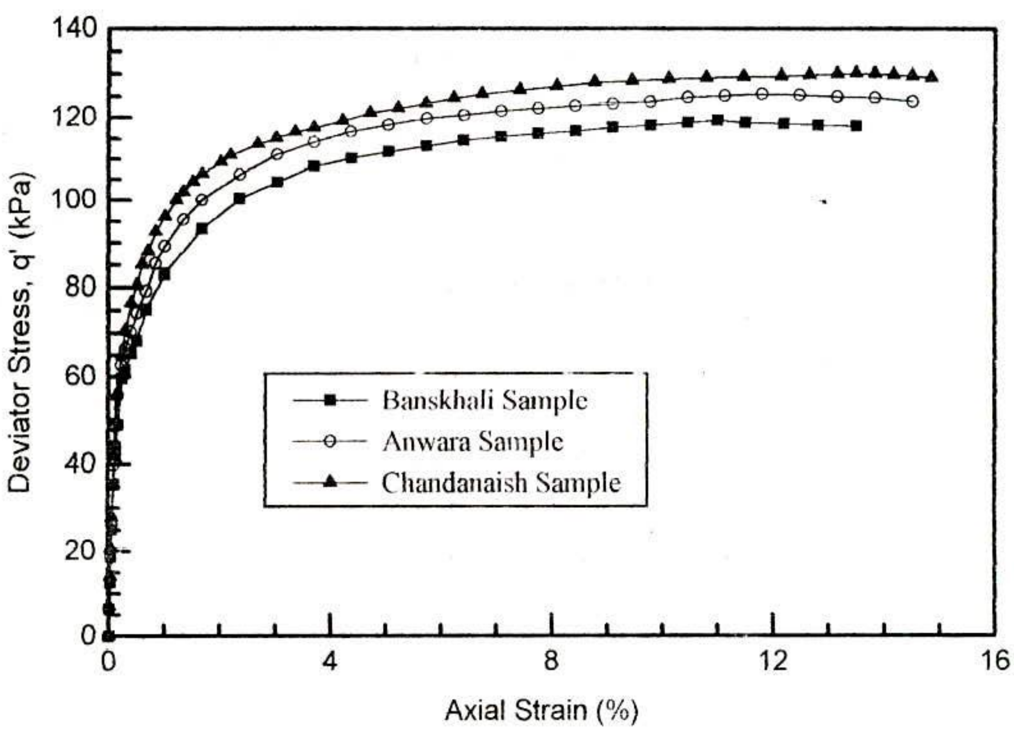


Fig. 5.4 Undrained Stress-Strain Curves of Three Reconstituted Normally Consolidated Samples

stress. The initial tangent modulus was determined by approximating stress-strain curves to hyperbolae as indicated by Kondner and Zelasko (1963) in the form,

$$(\sigma'_1 - \sigma'_3) = \varepsilon / (a + b\,\varepsilon) \qquad (5.1)$$

where, $(\sigma'_1 - \sigma'_3)$ = deviator stress

ε = axial strain

a, b = constants

The equation 5.1when rewritten as $\varepsilon /(\sigma'_1 - \sigma'_3) = a + b\,\varepsilon$ is a straight line in ε versus $\varepsilon /(\sigma'_1 - \sigma'_3)$ or ε versus q' graphical plot. Then the values of a and b was determined from the plot as shown in Fig. 5.5. a is the ordinate at $\varepsilon = 0$ and b is the slope of the line. 1/a gives the initial tangent modulus, E_i. The secant modulus (E_{50}) was determined from the slope of the line joining the point at half of the peak deviator stress on the curve with zero axial strain.

From Table 5.2 it can also be concluded that undrained strength (s_u), axial strain at peak deviator stress (ε_p), initial tangent modulus (E_i) and secant modulus (E_{50}) increased with the increase of plasticity.

Table 5.2 Undrained Shear Properties for Reconstituted Isotropic Normally Consolidated Soils

Location of the Soil	PI %	σ'_{vc} (kN/m^2)	s_u (kN/m^2)	ε_p (%)	A_p	E_i (kN/m^2)	E_{50} (kN/m^2)
Banskhali	10	150	59.6	11.0	0.82	37095	24525
Anwara	16	150	62.7	11.8	0.79	39345	26145
Chandanaish	20	150	65.0	13.5	0.74	41355	27555

5.3.3 Pore Pressure Response

Fig. 5.6 presents the relationship between pore pressure change and axial strain for the isotropically normally consolidated samples from Banskhali, Anwara and Chandanaish. It can be seen from the Fig. 5.6 that pore pressure response decreases with the increase of plasticity. Skempton's pore pressure parameter A at peak deviator stress (A_p) of the samples were determined which have already been presented in Table 5.2. It can be observed from Table 5.2 that the values of A_p varied between 0.74 and 0.82. It can also be found that the value of A_p increased with the decrease of plasticity. Similar results were obtained by Varadarajan (1973). This may be due to present of higher percentage of silt with the decrease of plasticity.

5.4 Undrained Behaviour of Overconsolidated "Block" Samples

5.4.1 Effective Stress Path for Overconsolidated "Block" Sample

Fig. 5.7 shows the undrained effective stress paths for overconsolidated "block" sample in p′: q′ space. In Fig. 5.7 it is shown that at low overconsolidation ratio (up to 2) the sample behaves like a normally consolidated soil and for higher consolidation ratio the failure points lie on the dry side of the critical which exhibits dilation of the samples. Similar results were also observed for chandanaish soil

5.4.2 Stress-Strain-Strength and Stiffness Characteristics

Fig. 5.8 shows the typical variation of axial strain with deviator stress for reconstituted isotropically overconsolidated samples of Banskhali and Anwara soils. From the Fig. 5.8, it can be observed that the stress-strain diagram is nonlinear and there was no sharp break between the low compressibility and high compressibility behaviour. Crooks and Graham (1976) also showed that the stress-strain behaviour of samples consolidated isotropically under pressures less than the maximum past pressure, even at a stress below the overburden pressure, has no sharp break in p'_{max} between the low compressibility and high compressibility behaviour. Similar behaviour was also shown by the sample from Chandanaish. Tables 5.3, 5.4 and 5.5 present the values of σ'_{vo}, s_u, ε_p, s_u/σ'_{vo}, E_i/σ'_{vo}, E_{50}/σ'_{vo}, E_i/s_u, E_{50}/s_u, for the normally and overconsolidated "block" samples of Banskhali, Anwara and Chandanaish soils,

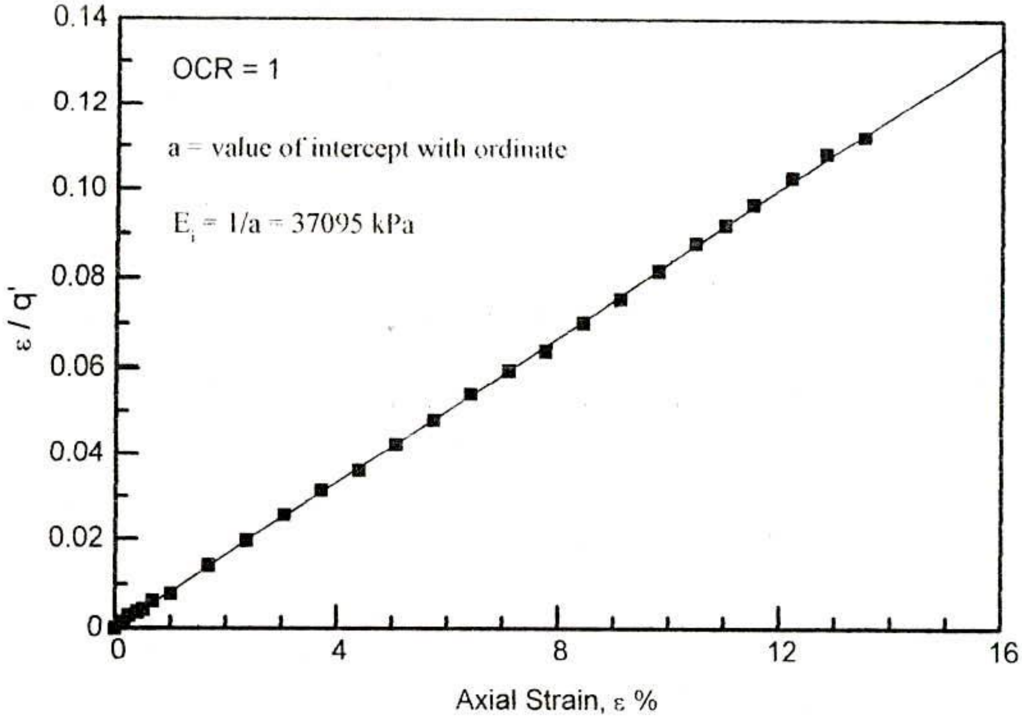


Fig. 5.5 Typical Transformed Hyperbolic Representation of Stress-Strain Relationship for Banskhali Soil

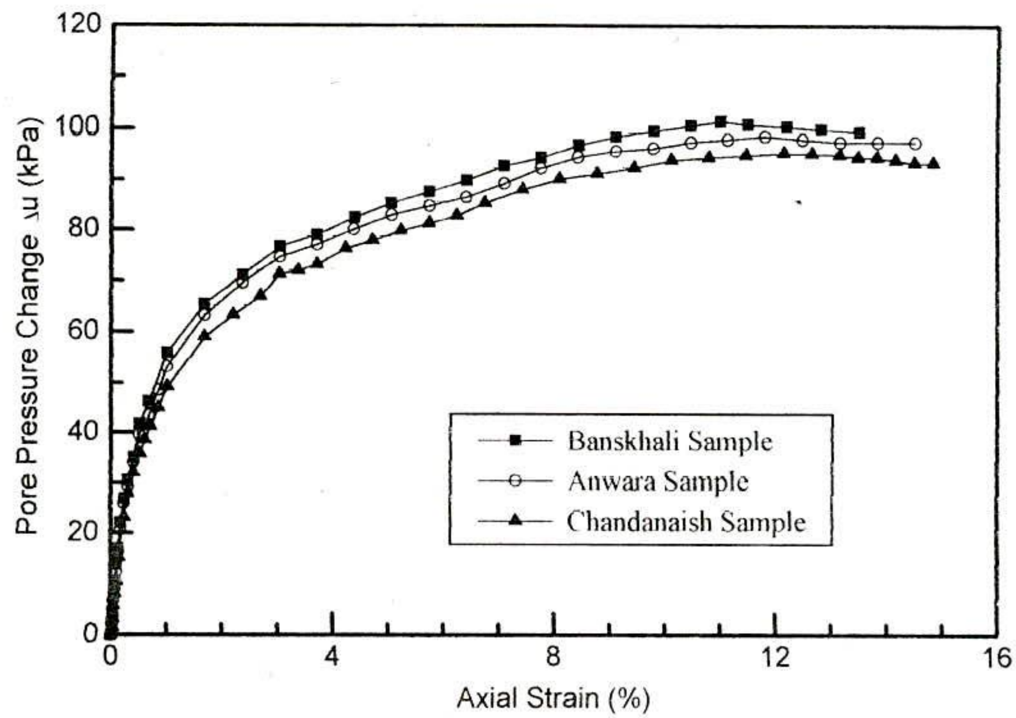
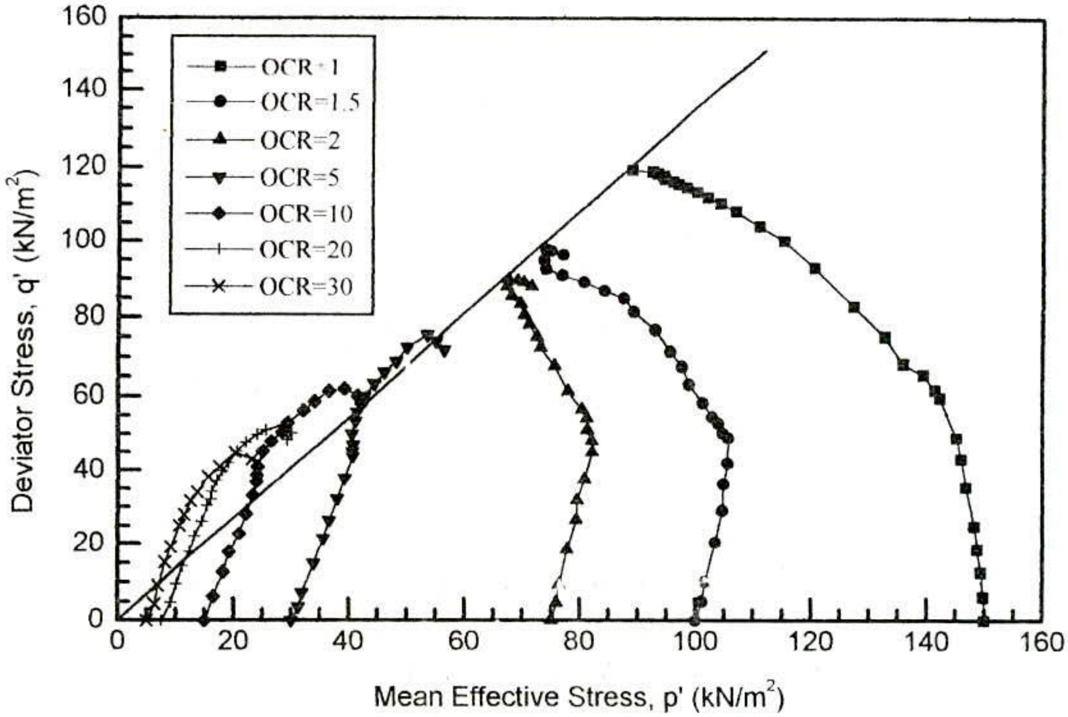
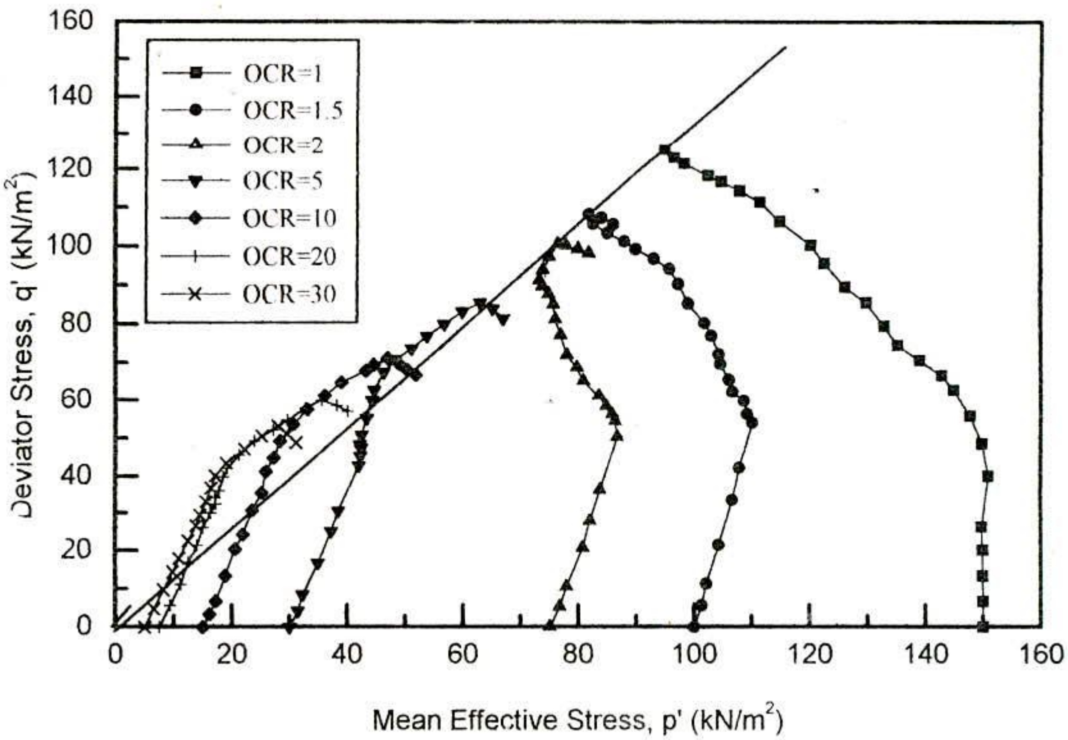


Fig. 5.6 Pore Pressure Change vs. Axial Strain Curves for Three Reconstituted Normally Consolidated Samples

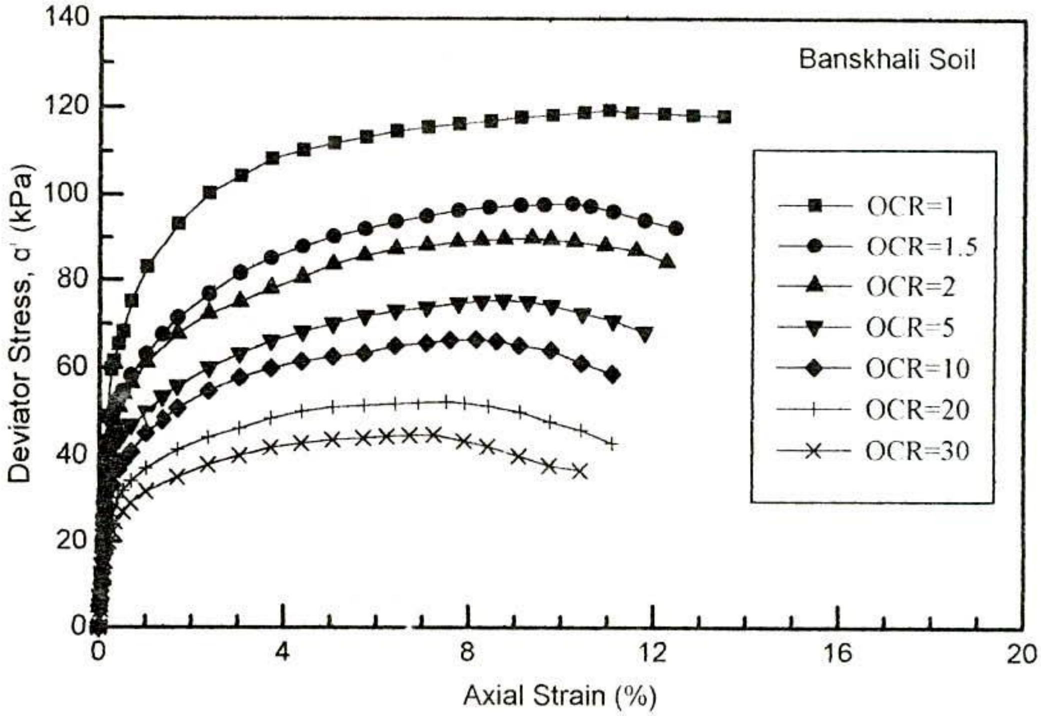


(a)

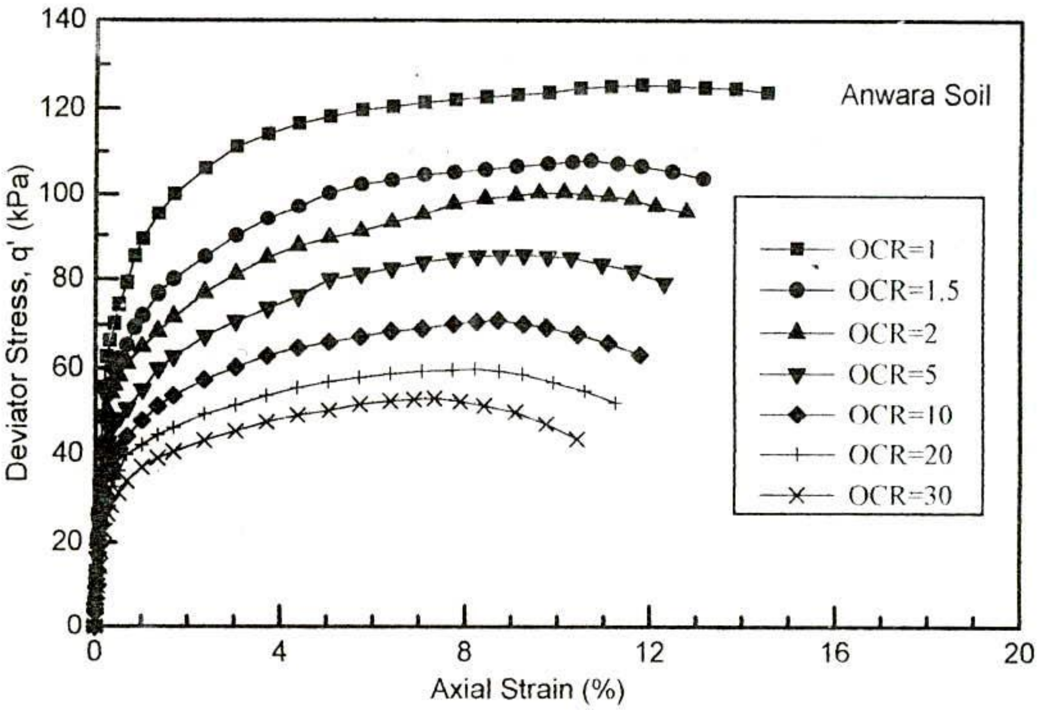


(b)

Fig. 5.7 Undrained Effective Stress Paths of Reconstituted Overconsolidated "Block" Samples: (a) Banskhali Soil, (b) Anwara Soil



(a)



(b)

Fig. 5.8 Undrained Stress-Strain Curves of Different OCR for Reconstituted Normally Consolidated and Overconsolidated "Block" Samples: (a) Banskhali Soil (b) Anwara Soil

Table 5.3 Undrained Shear Properties for Reconstituted Isotropic Normally Consolidated and Overconsolidated "Block" Samples of Banskhali Soil (PI = 10)

σ'_{vc} (kPa)	150	150	150	150	150	150	150
σ'_{vo} (kPa)	150	100	75	30	15	7.5	5
OCR	1	1.5	2	5	10	20	30
s_u (kPa)	59.6	49.0	45.0	37.8	31.0	26.2	22.5
ε_p (%)	11.0	10.22	9.35	8.72	8.14	7.5	7.24
A_p	0.82	0.43	0.33	-0.07	-0.13	-0.28	-0.34
s_u/σ'_{vo}	0.397	0.49	0.60	1.26	2.067	3.493	4.50
$(s_u/\sigma'_{vo})^{oc}/(s_u/\sigma'_{vo})^{nc}$	1.0	1.234	1.51	3.174	5.214	8.791	11.335
$s_u^{(oc)}/ s_u^{(nc)}$	1.0	0.82	0.755	0.634	0.52	0.44	0.378
E_i/σ'_{vo}	247.3	299.6	364.5	728.0	1255.0	1897.0	2416.6
E_{50}/σ'_{vo}	163.5	200.0	241.4	487.0	827.6	1278.5	1625.4
E_i/ s_u	622.3	611.5	607.5	578.0	559.0	543.0	537.0
E_{50}/ s_u	411.5	408.0	402.0	386.4	371.8	365.0	361.2

Table 5.4 Undrained Shear Properties for Reconstituted Isotropic Normally Consolidated and Overconsolidated "Block" Samples of Anwara Soil (PI = 16)

σ'_{vc} (kPa)	150	150	150	150	150	150	150
σ'_{vo} (kPa)	150	100	75	30	15	7.5	5
OCR	1	1.5	2	5	10	20	30
s_u (kPa)	62.7	54.0	50.25	42.75	35.55	30.0	26.6
ε_p (%)	11.8	10.7	10.12	9.25	8.7	8.2	7.32
A_p	0.79	0.38	0.28	-0.11	-0.19	-0.33	-0.40
s_u/σ'_{vo}	0.418	0.54	0.67	1.425	2.37	4.0	5.32
$(s_u/\sigma'_{vo})^{oc}/(s_u/\sigma'_{vo})^{nc}$	1.0	1.29	1.603	3.41	5.67	9.57	12.73
$s_u^{(oc)}/s_u^{(nc)}$	1.0	0.86	0.80	0.682	0.566	0.478	0.424
E_i/σ'_{vo}	262.3	333.5	409.0	835.4	1350.4	2218.0	2882.0
E_{50}/σ'_{vo}	174.3	222.5	271.3	557.0	894.7	1472.0	1931.0
E_i/s_u	627.5	617.6	610.4	586.0	569.8	554.5	541.7
E_{50}/s_u	417.0	412.0	405.0	391.0	377.5	368.0	363.0

Table 5.5 Undrained Shear Properties for Reconstituted Isotropic Normally Consolidated and Overconsolidated "Block" Samples of Chandanaish Soil (PI = 20)

σ'_{vc} (kPa)	150	150	150	150	150	150	150
σ'_{vo} (kPa)	150	100	75	30	15	7.5	5
OCR	1	1.5	2	5	10	20	30
s_u (kPa)	65.0	58.5	54.3	45.5	38.4	32.5	29.9
ε_p (%)	13.5	12.53	11.75	10.56	10.0	8.86	8.52
A_p	0.74	0.34	0.23	-0.14	-0.25	-0.37	-0.45
s_u/σ'_{vo}	0.433	0.585	0.724	1.517	2.56	4.33	5.98
$(s_u/\sigma'_{vo})^{oc}/(s_u/\sigma'_{vo})^{nc}$	1.0	1.35	1.67	3.5	5.9	10.0	13.8
$s_u^{(oc)}/s_u^{(nc)}$	1.0	0.90	0.836	0.70	0.59	0.50	0.46
E_i/σ'_{vo}	275.7	365.6	445.6	902.4	1478.0	2422.0	3277.0
E_{50}/σ'_{vo}	183.7	243.4	296.3	600.6	984.0	1625.0	2201.0
E_i/s_u	636	625	615	595	577	559	548
E_{50}/s_u	424	416	409	396	385	375	368

respectively. σ'_{vo} is the reduced consolidation pressure (i.e., preshear consolidation pressure) under which the sample was allowed to swell in triaxial cell. The strain at peak deviator stress (ε_p) varies between 7.24% and 13.5% for the three soils. It can be seen that, compared with normally consolidated samples, ε_p decreases with overconsolidation ratios 1.5 to 30 from about 7% to 34%, 9% to 38% and 7% to 37% for the samples from Banskhali, Anwara and Chandanaish, respectively. From Tables 5.3 to 5.5, it is also observed that for the same overconsolidation ratio, ε_p increased with the increase in plasticity index.

From Tables 5.3 to 5.5, it is exhibited that undrained shear strength of these samples varies with OCR. For the same preconsolidation pressure, the strength decreases with increase in OCR and from Fig. 5.8 it is observed that the undrained shear strength is reduced by swelling process. The amount of reduction of shear strength due to swelling depends on the extent to which the soil sample is allowed to swell. Thus for same maximum consolidation pressure, the undrained strength of normally consolidated sample is greater than that of overconsolidated sample since stress recession occurs at larger strain region. All these overconsolidated samples show clear peak strength. For these samples, failure stress has been considered at peak deviator stress. The undrained shear strength for these soils has been defined as half of the peak deviator stress.

The three overconsolidated coastal soils from Banskhali, Anwara and Chandanaish exhibited the undrained shear strength ratio (s_u/σ'_{vo}) ranging from 0.397 to 4.5, 0.418 to 5.32 and 0.433 to 5.98, respectively which are already presented in Tables 5.3 to 5.5. The variation of undrained shear strength ratio (s_u/σ'_{vo}) with OCR in semi-log scale and log-log scale for the three soils is also shown in Fig. 5.9 and Fig. 5.10. In the Fig. 5 9, 5.10, the three coastal soils have been compared with other clays (Ladd and Foott, 1974; Mahar and O'Neill, 1983; Ameen and Safiullah, 1986; Ansary, 1993). The three soils produced curves having trend similar to other clays and close to the upper bound of other clays. In Fig. 5.11, the data for the three overconsolidated samples have been plotted as s_u/σ'_{vo} versus OCR in log-log scale. In log-log scale, the variation of s_u/σ'_{vo} with OCR yielded a straight line for each "block" sample. The intercept of each line with the ordinate is the undrained shear strength

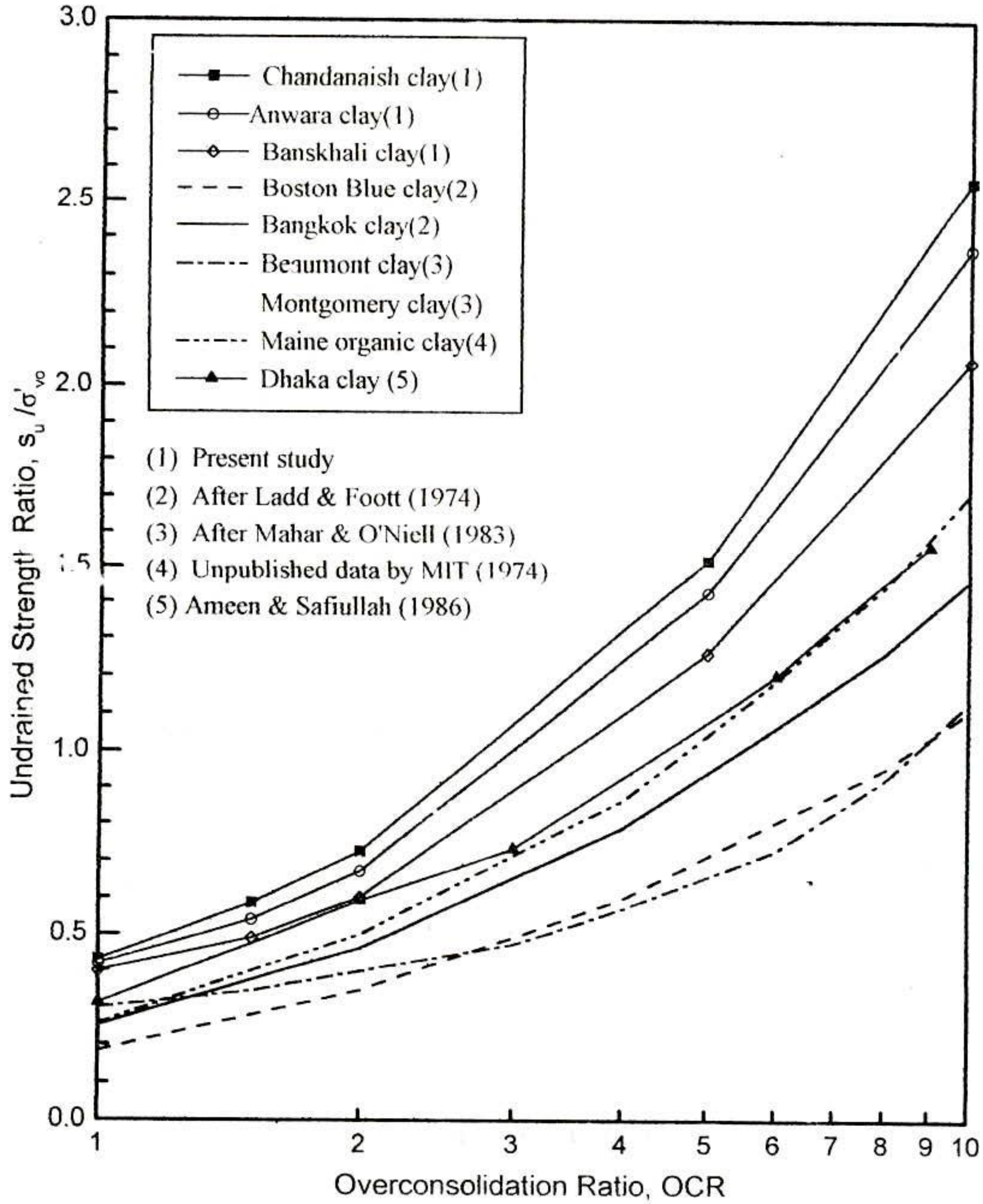


Fig. 5.9 Comparison of Undrained Strength Ratio vs. OCR Plots of the Coastal Soils Studied with other Six Different Types of Clays

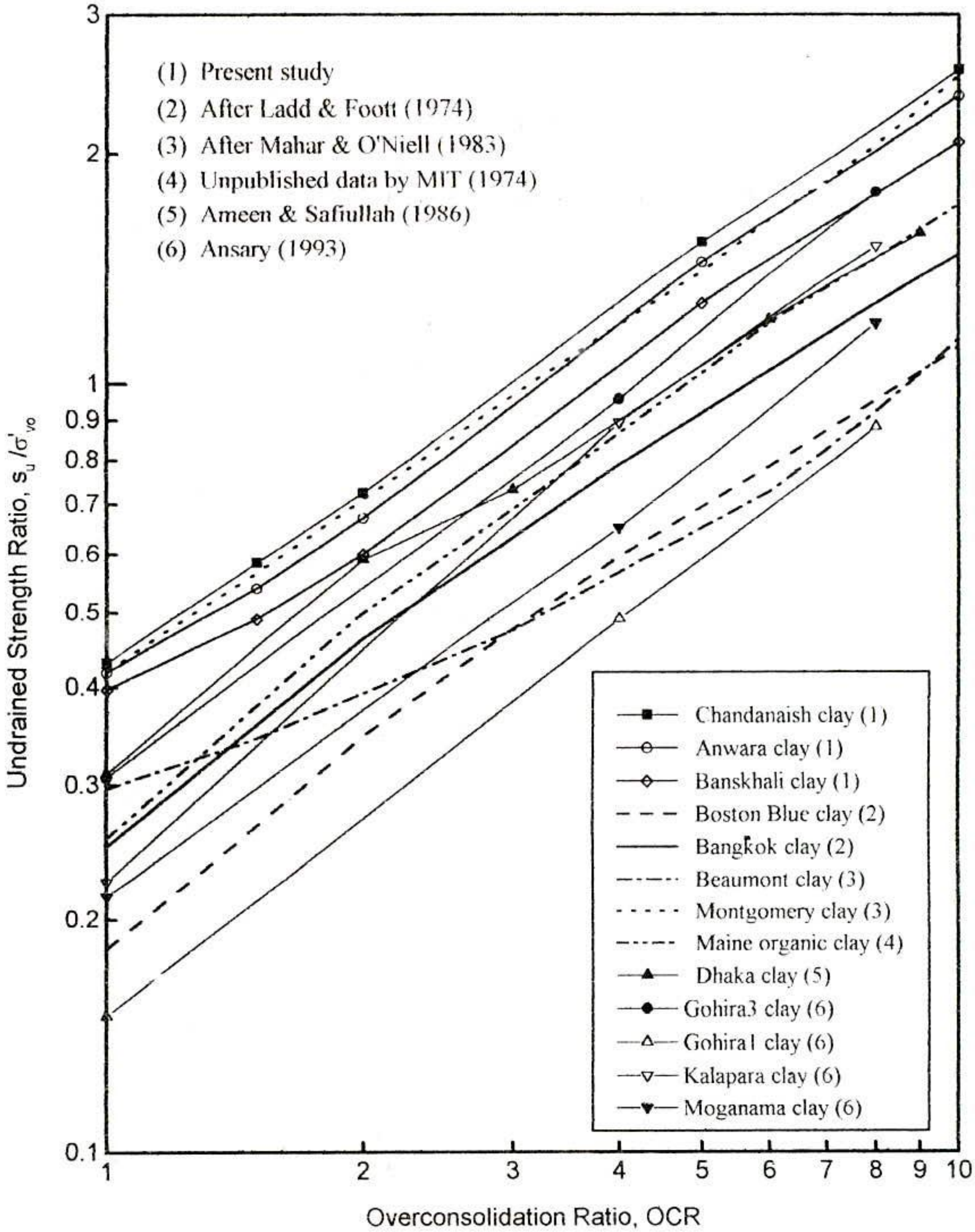


Fig. 5.10 Comparison of Undrained Strength Ratio vs. OCR Plots of the Three Samples of Coastal Soils Studied with Ten Various Types of Clays

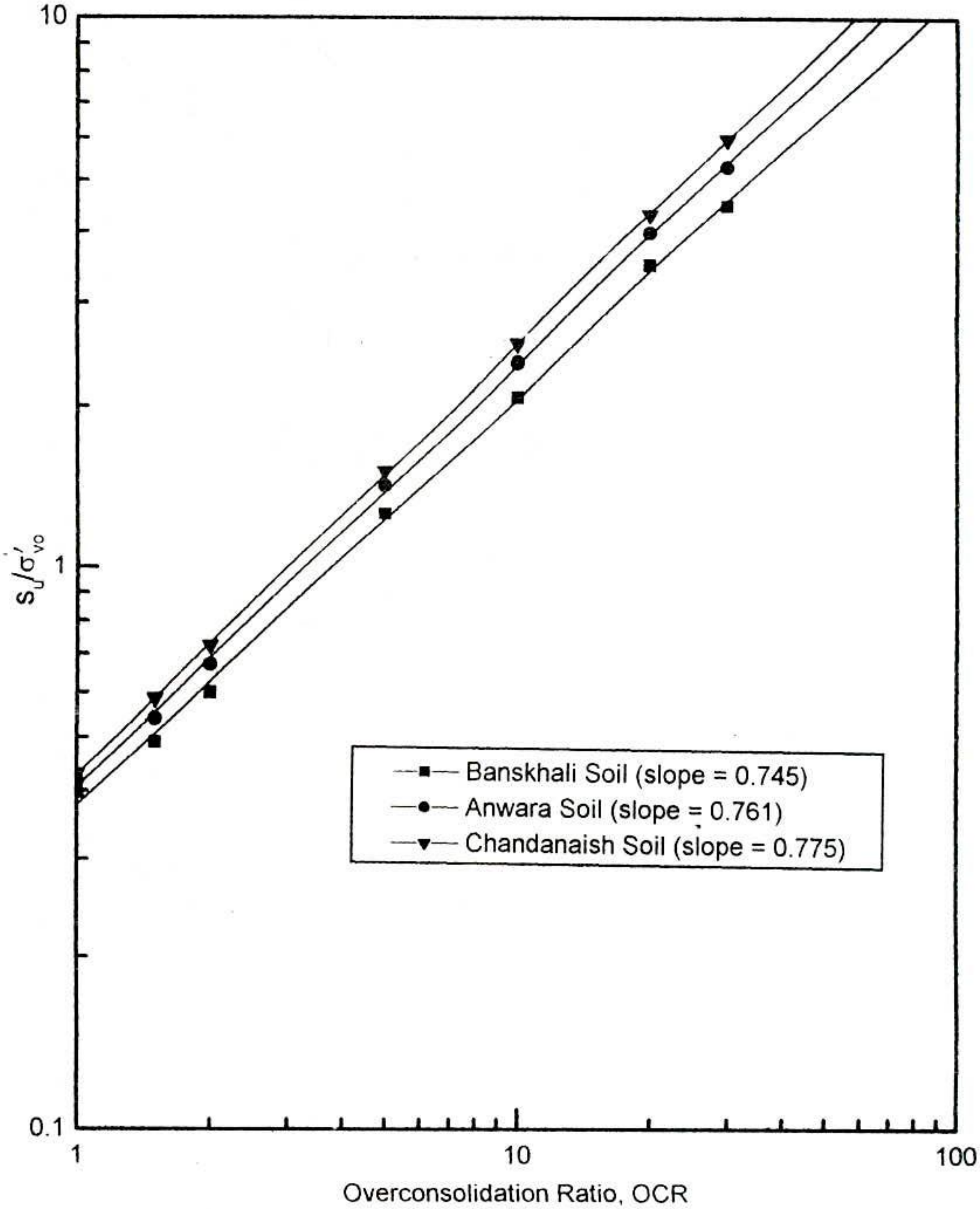


Fig. 5.11 Variation of Normalized Undrained Shear Strength (s_u/σ'_{vo}) with OCR

ratio at normally loaded state (OCR = 1) for that sample. At normally loaded state, the values of s_u/σ'_v obtained from curves are 0.397, 0.418 and 0.43 for the three samples from Banskhali, Anwara and Chandanaish respectively. Ameen and Safiullah (1986) and Kamaluddin (1999) reported that the values of s_u/σ'_{vc} for isotropic consolidation are 0.3 and 0.295 for soft Dhaka clay, respectively.

The slope of each line, $\Lambda^{(iso)}$, is the critical state pore pressure parameter for isotropic stress condition. $\Lambda^{(iso)}$ for the above three samples are equal to 0.745, 0.761 and 0.775, respectively. The average value of $\Lambda^{(iso)}$ is 0.76. This linear relationship indicates that the value of $\Lambda^{(iso)}$ is essentially constant with overconsolidation ratio for a soil. Ladd and Foott (1974), Mahar and O'Neill (1983) reported similar observations. Ladd et al. (1977) reported Λ values ranging from 0.75 to 0.85 from direct shear tests on five other clays. The values of Λ are shown in Table 5.6. Ameen and Safiullah (1986) reported that for Dhaka clay the value of $\Lambda^{(iso)}$ is 0.746 and Kamaluddin (1999) reported that for Dhaka clay the value of $\Lambda^{(iso)}$ is 0.734. Table 5.6 shows the difference between $\Lambda^{(iso)}$ experimentally determined from s_u/σ'_{vo} data and the approximate theoretical value of $\Lambda^{(iso)} = [1-C_s^{(iso)}/C_c^{(iso)}]$ determined from consolidation tests. It is postulated that this discrepancy occurs because of problems in properly determining the swelling index parameter (C_s). During most routine laboratory testing, little attention is given to defining a value of range of values for C_s. If appears that the swelling index is actually nonlinear on a plot of void ratio and log-effective stress and the value of C_s vary with different overconsolidation ratio, while the value of $\Lambda^{(iso)}$ determined from s_u/σ'_{vo} data is constant for any overconsolidation ratio.

Fig. 5.12 shows the variation of undrained shear strength ratio, $s_u^{(nc)}/\sigma'_{vc}$ of normally consolidated soils with plasticity index while Fig. 5.13 shows the variation of critical state pore pressure parameter, Λ with plasticity index. It can be seen that these plots are almost linear (R = 0.94 to 0.97). In Fig. 5.12 the data for other coastal soils have also been plotted for comparison with the results obtained from the present study. From Fig. 5.12 and 5.13, values of $s_u^{(nc)}/\sigma'_{vc}$ and Λ, respectively can be predicted for any soil of plasticity within 10 to 20% from the curves of present study. For practical purposes, the average value 0.76 of Λ may be used for the three soils, as this value is more accurate than the curves value (approximate value from consolidation test).

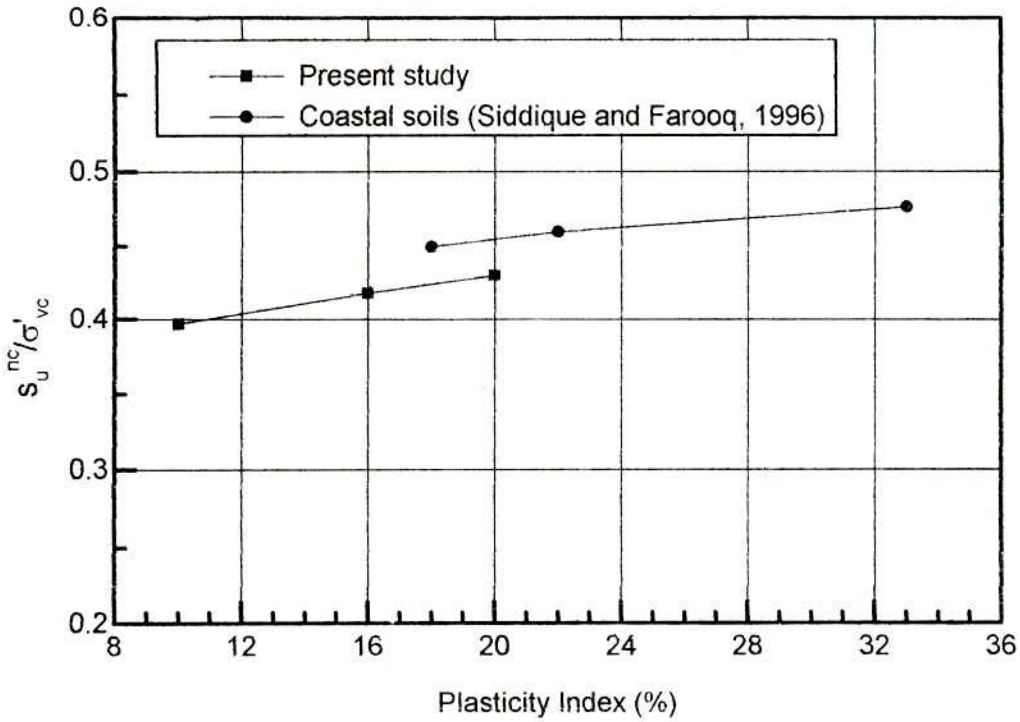


Fig. 5.12 Variation of Undrained Strength Ratio with Plasticity Index

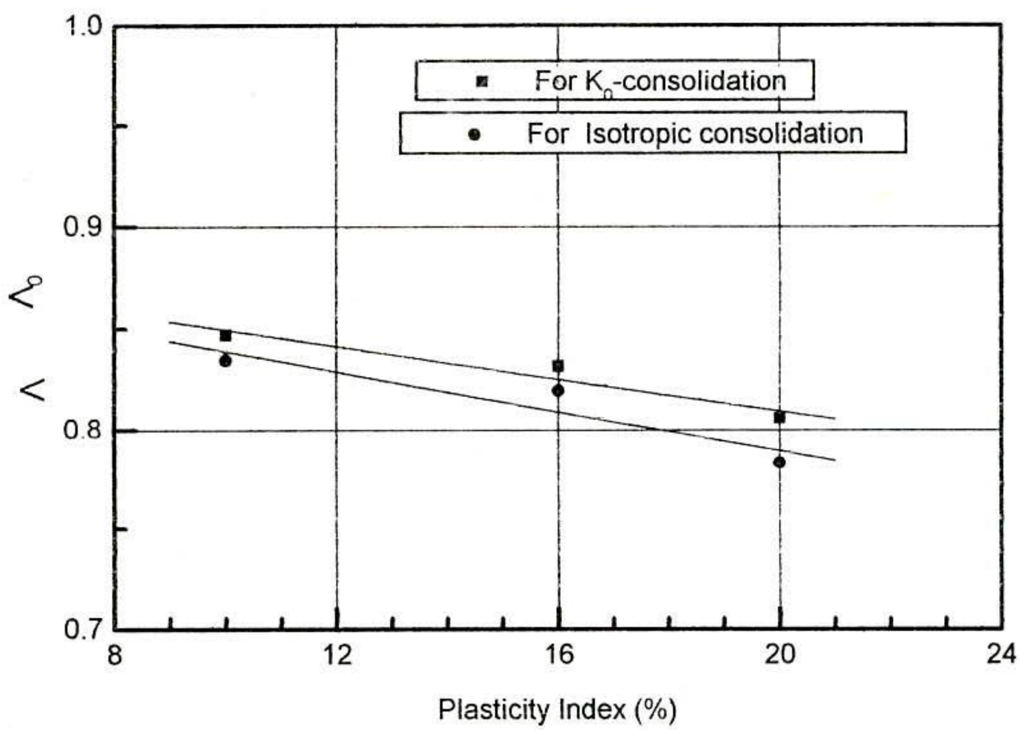


Fig. 5.13 Variation of Critical State Pore Pressure Parameter, Λ with Plasticity Index

Figs. 5.14 and 5.15 show the variation of normalized undrained strength ratio (α) with OCR and these curves are compared with the ranges provided by Ladd and Foott (1974) and also with the curves of Mahar and O'Neill (1983) and Ladd et al. (1977). In Fig. 5.14 normalized plots of four coastal soils investigated by Ansary, 1993 are also included for comparison with the present study. From Figs. 5.14 and 5.15, it is evident that almost all portion of the curves of the three reconstituted soils lie within the limits proposed by Ladd and Foott (1974). Ladd and Foott (1974), Mitachi and Kitago (1976), Mayne (1980), Mahar and O'Neill (1983), Ansary (1993) reported similar observations. The critical state pore pressure parameters ($\Lambda^{(iso)}$ and $\Lambda^{(Ko)}$) for isotropic and K_0 conditions have also been determined from consolidation and swelling indices as shown in Table 5.1. The values of $\Lambda^{(iso)}$ found from the relations $[1-C_s^{(iso)}/C_c^{(iso)}]$ become equal to 0.835, 0.82 and 0.783 for the samples from Banskhali, Anwara and Chandanaish, respectively. The values of $\Lambda^{(Ko)}$ found from the relations $[1-C_s^{(Ko)}/C_c^{(Ko)}]$ become equal to 0.847, 0.832, 0.806 for the samples from Banskhali, Anwara and Chandanaish, respectively. It shows that the values of $\Lambda^{(Ko)}$ obtained from e-logσ'_v plot give higher values than those of $\Lambda^{(iso)}$, but the difference is insignificant. It is also evident that the values of $\Lambda^{(iso)}$ from e-logσ'_v plot give higher values than that determined from experimental plot of log (s_u/σ'_{vo}) vs. log (OCR) for all the soils. Mitachi and Kitago (1976) also provided similar results but the values were close. Ameen and Safiullah (1986) and Kamaluddin (1999) reported that the value of $\Lambda^{(iso)}$ from e-logσ'_v plot gives higher value than that determined from experimental plot of log (s_u/σ'_{vo}) vs. log (OCR) for Dhaka clay. The predicted values of them are shown in Table 5.6.

The normalized parameters E_i/s_u, E_i/σ'_{vo}, E_{50}/s_u, E_{50}/σ'_{vo} are shown in Tables 5.3 to 5.5 for the samples from Banskhali, Anwara and Chandanaish. The initial tangent modulus (E_i) and the secant modulus (E_{50}) were predicted by the same procedure as described in Article 7.3.1. The ranges of E_i/s_u and E_i/σ'_{vo} are 537.0 to 636.0 and 247.3 to 3277.0 respectively (for OCR = 1.0 to 30) for three samples. The ranges of E_{50}/s_u, E_{50}/σ'_{vo} are 361.2 to 424.0 and 163.5 to 2201.0 respectively (for OCR = 1.0 to 30) for

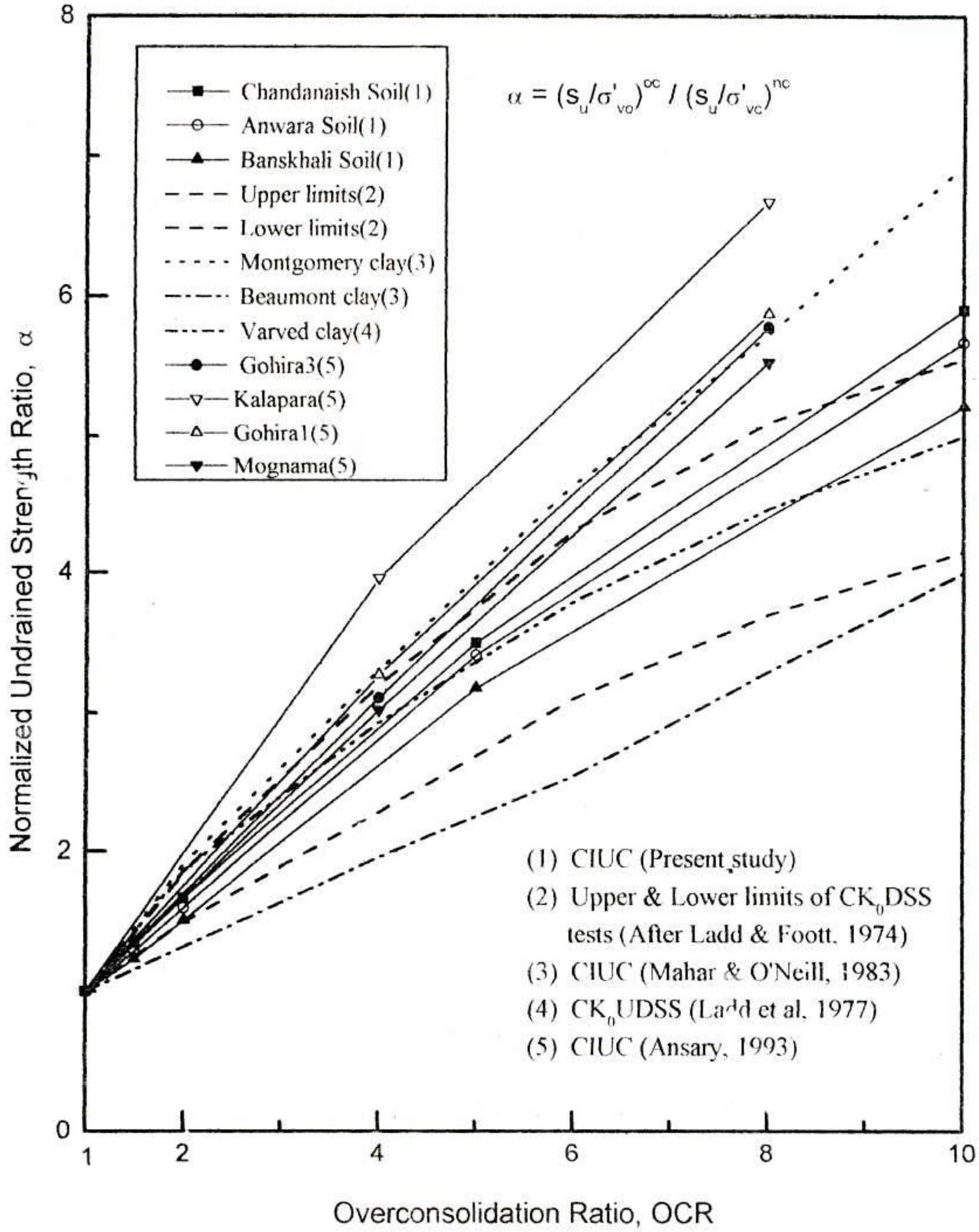


Fig.5.14 Comparison of Normalized Undrained Strength Ratio vs. OCR Plots of the Three Coastal Soils Studied with Other Clays

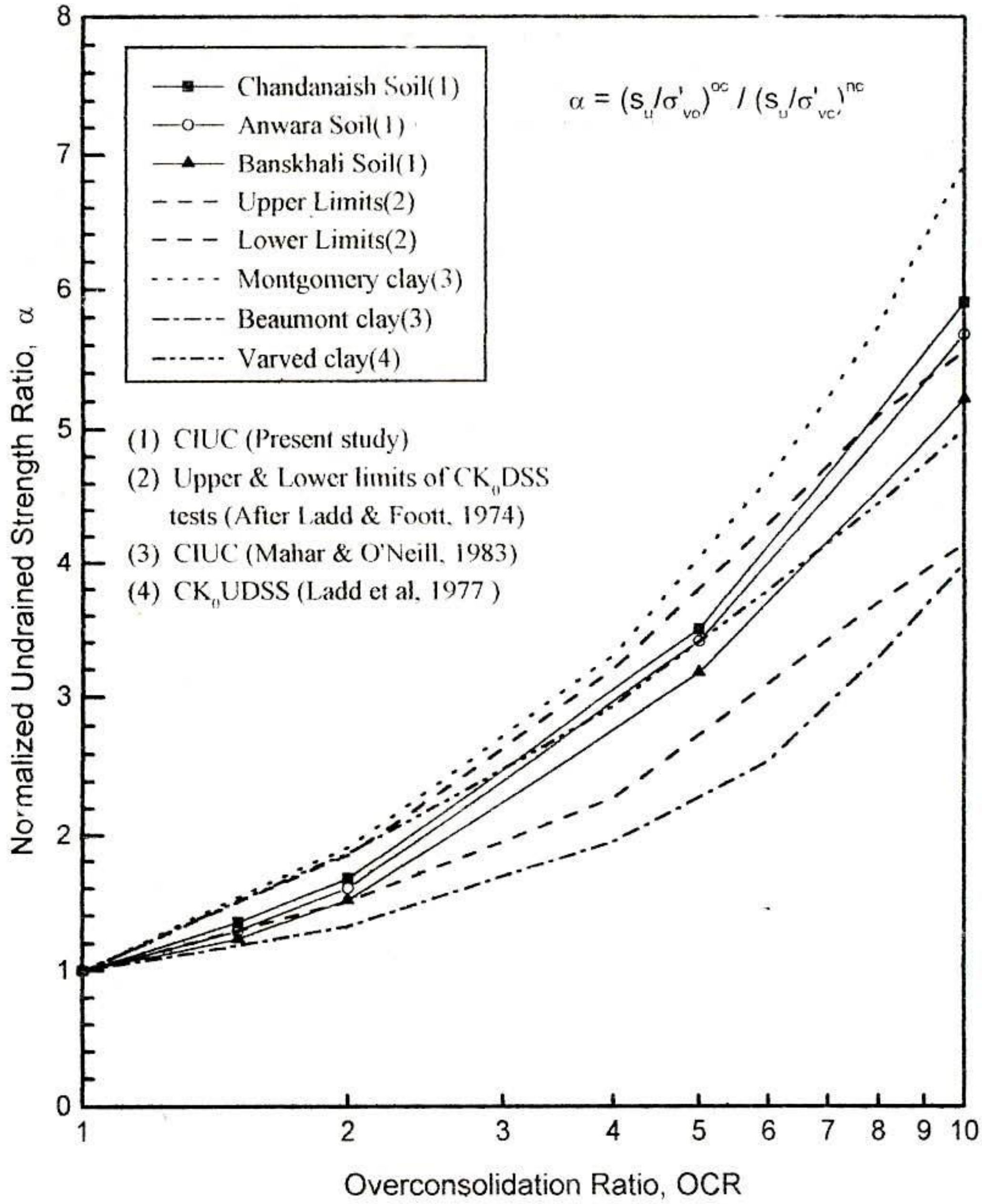


Fig. 5.15 Comparison of Normalized Undrained Strength Ratio vs. OCR Plots (semi-log scale) of the Three Coastal Soils Studied with Other Clays

Table 5.6 Critical State Pore Pressure Parameter, Λ

Location of Soil	PI %	Determined from $[1-C_s^{(iso)}/C_c^{(iso)}]$ (isotropic compression)	Determined from $[1-C_s^{(Ko)}/C_c^{(Ko)}]$ (K_0-compression)	Slope of log (s_u/σ'_{vo}) vs. log OCR (isotropic)	
Banskhali	10	0.835	0.847	0.745	
Anwara	16	0.820	0.832	0.761	0.76 (average)
Chandanaish	20	0.783	0.806	0.775	
Dhaka(1)	22	0.87	0.907	0.746	
Dhaka(2)	23	0.863	-	0.821	

(1) Ameen and Safiullah (1986)

(2) Kamaluddin (1999)

three samples. The variation of these normalized parameters as a function of OCR are presented in Figs. 5.16 to 5.19 in semi-log scale. From Figs. 5.16 and 5.19, it is concluded that the normalized parameters, E_i/σ'_{vo} and E_{50}/σ'_{vo}, increased with the increase of OCR. At higher OCR values, the same effect is highly pronounced. Yudhbir et al. (1975) had found similar trend for Weald clay, Rann of Kutch clay and Kanpur clay. Ladd (1964) and Varadarajan (1973) reported similar observation for other clays. It is also noted from Figs. 5.16 to 5.19 that the normalized parameters, E_i/s_u and E_{50}/s_u, decreased with the increase of OCR. The trend of variation of these curves for three samples from Banskhali, Anwara and Chandanaish is similar to the investigation made by Appolonia et al. (1971) on Bangkok clay (LL = 65, PI = 41), Maine organic clay (LL = 65 ± 10, PI = 33 ± 2) and Boston blue clay (LL = 41, PI = 22) and Kamaluddin (1999) on Dhaka clay (LL = 43, PI = 23).

5.4.3 Pore Pressure Characteristics

Fig. 5.20 presents the variation of excess pore pressure (Δu) with respect to axial strain (ε) for all the overconsolidated samples from Banskhali and Anwara, sheared from the isotropic stress state. Similar behaviour has also been shown by the sample from Chandanaish. Pore pressure in all samples decreases with increasing overconsolidation, the rate of reduction decreased with increasing values of OCR. For heavily overconsolidated samples (OCR > 4) the pore pressure is found to increase slightly in the initial phase of shear, thereafter pore pressure decreased continuously until failure is reached. However, for lightly overconsolidated samples, the pore pressure increased initially and then there was a gradual decrease until failure. The behaviour of the sample sheared from 100 kN/m^2 preshear consolidation is very similar to that of normally consolidated sample. This behaviour was observed similar for all the samples from Banskhali, Anwara and Chandanaish. The pore pressure parameter, A at peak deviator stress, A_p for each sample is shown in Tables 5.3 to 5.5.

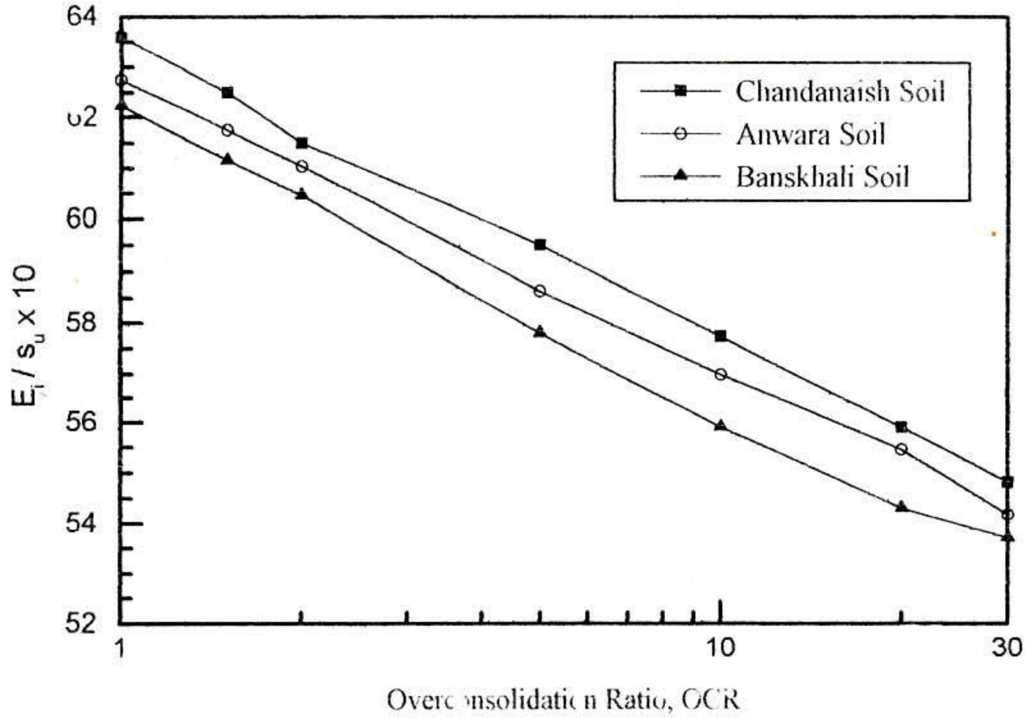


Fig. 5.16 Normalized Undrained Initial Tangent Modulus (E_i/s_u) vs. OCR on the Three Samples

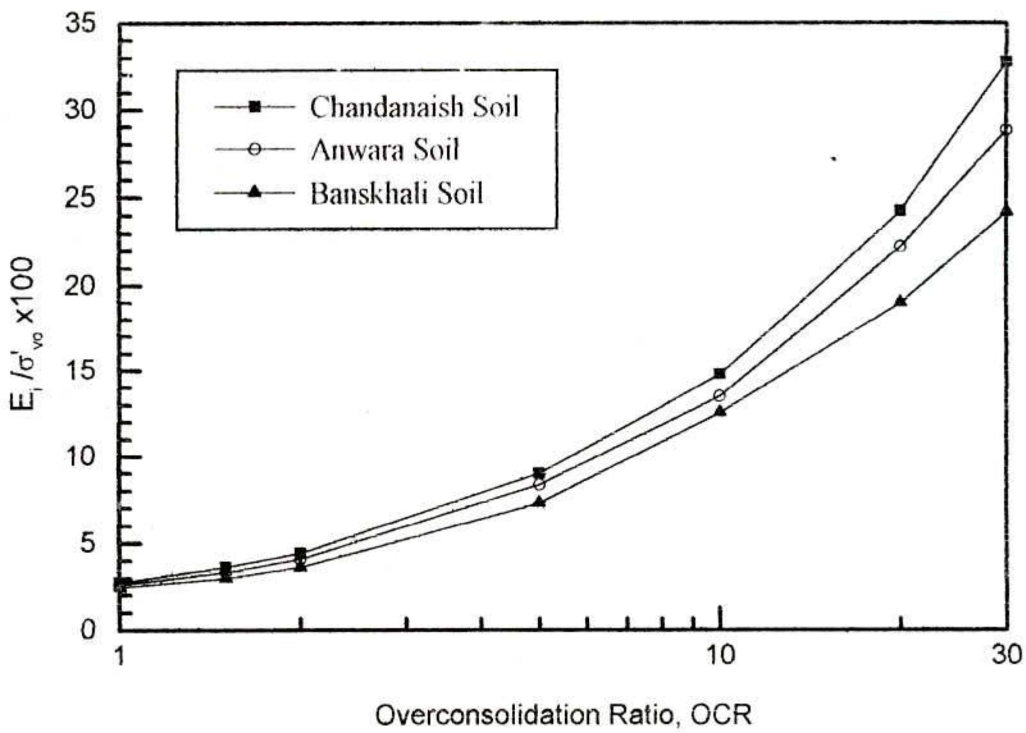


Fig. 5.17 Normalized Undrained Initial Tangent Modulus (E_i/σ'_{vo}) vs. OCR on the Three Samples

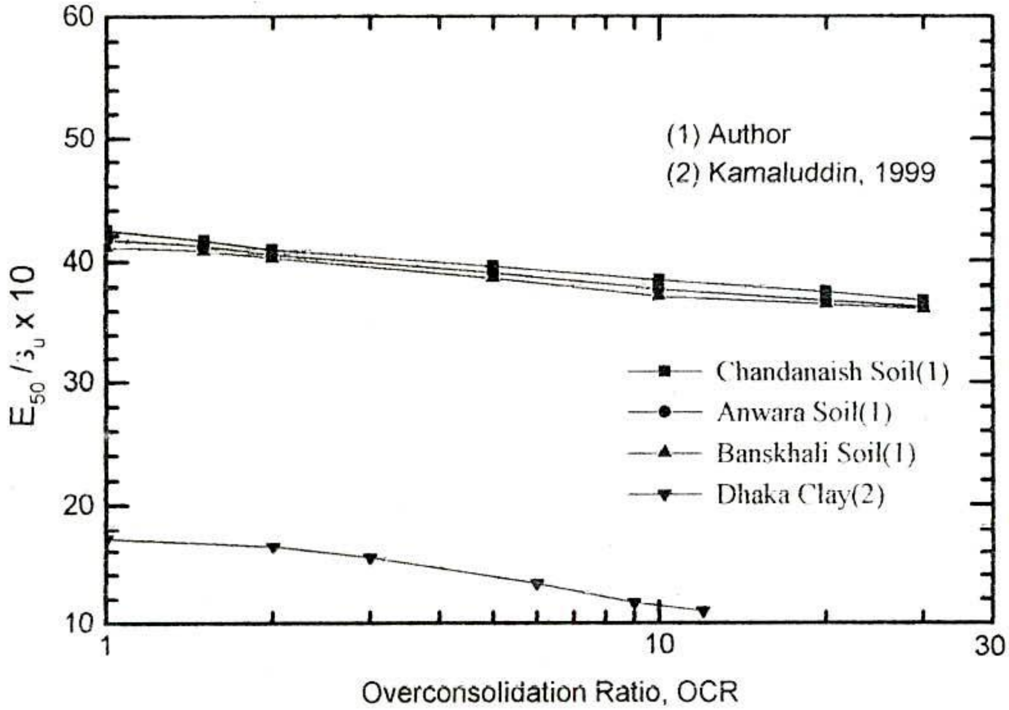


Fig. 5.18 Normalized Secant Modulus (E_{50}/s_u) vs. OCR Plots for Samples of Reconstituted Coastal Soils and Dhaka Clay

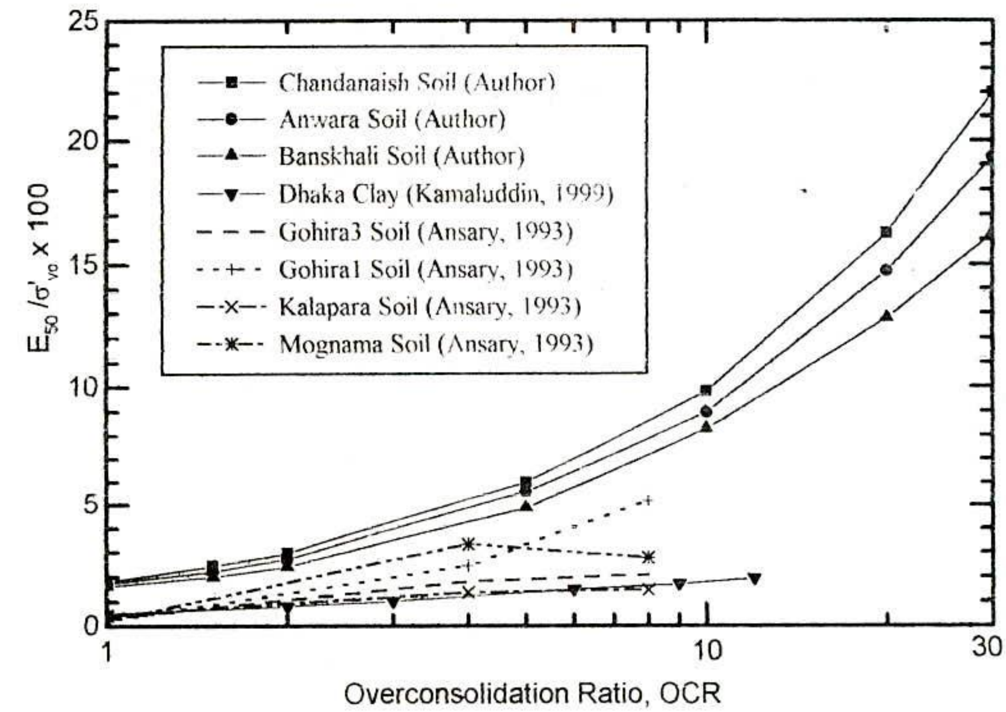
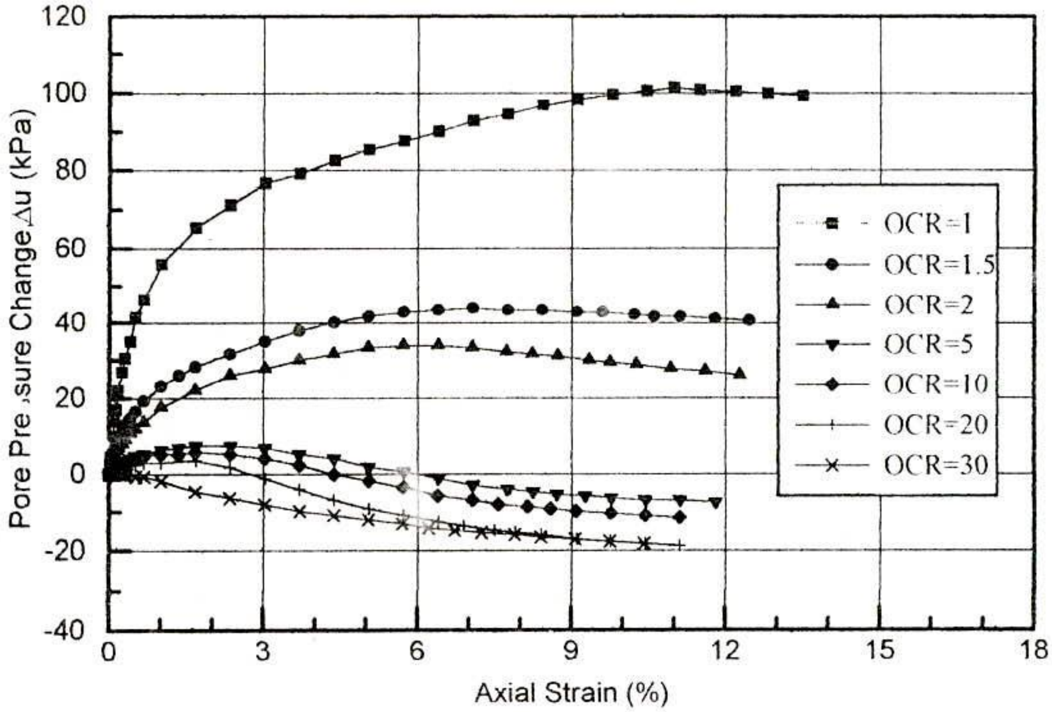
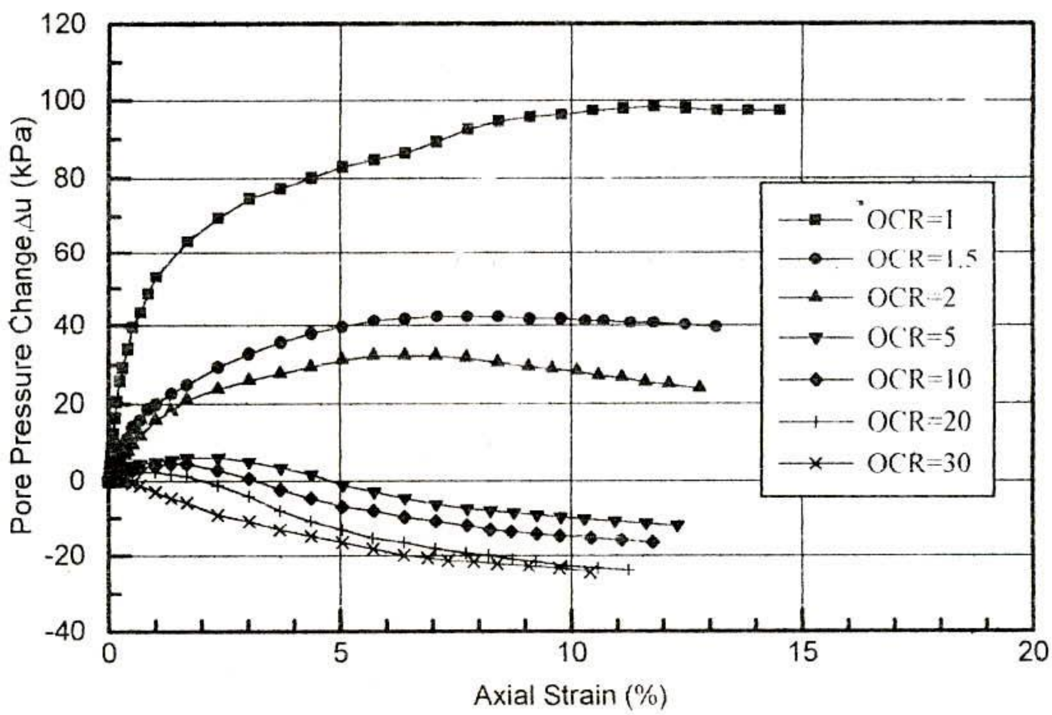


Fig. 5.19 Normalized Secant Modulus (E_{50}/σ'_{vo}) vs. OCR Plots for Samples of Reconstituted Three Coastal Soils and other Clays



(a)



(b)

Fig. 5.20 Effect of OCR on Pore Pressure Response with Axial Strain for CIU-Tests on (a) Banskhali Soil, (b) Anwara Soil

Fig. 5.21 shows the variation of the pore pressure parameter A at peak deviator stress, A_p with the overconsolidation ratio for the three reconstituted samples from Banskhali, Anwara and Chandanaish. These curves also compared with the typical curve given by Head (1986). From the Fig. 5.21, It might be concluded that the value of A_p decreases with the increase in the overconsolidation ratio. The observed values of A_p for the lightly overconsolidated sample (OCR < 4.0) remained positive and it was negative in the higher OCR for all samples from the three locations. Similar results also reported by Head (1986), Mayne and Stewart (1988).

5.5 Model for Prediction of Undrained Shear Strength of the Reconstituted Coastal Soils

5.5.1 Shear Strength Model for Normally Consolidated Soils

From the discussion of Art. 5.4.2, it was observed that the variation of undrained shear strength ratio (s_u/σ'_{vo}) with OCR in semi-log scale for the three soils is nonlinear, while in log-log scale they represents linear relationship. Similar graphs were also plotted by Mitachi and Kitago (1976), Mayne (1980) for other clays and those were straight lines in behaviour. From Fig. 5.11 it is noted that the intercept of each line is the undrained shear strength ratio at normally loaded state (OCR = 1) for that sample. At normally loaded state, the values at the intercepts are 0.397, 0.418 and 0.43 for the three samples from Banskhali, Anwara and Chandanaish respectively. These values are known as experimental values of undrained shear strength ratio (s_u/σ'_v) at isotropically normally loaded state. So these three experimentally determined values can be used to predict undrained shear strength of the respective soil at isotropically normally loaded state as the following equations.

$s_u^{(nc)} = 0.397\ \sigma'_v$, for reconstituted Banskhali soil (5.2a)

$s_u^{(nc)} = 0.418\ \sigma'_v$, for reconstituted Anwara soil (5.2b)

$s_u^{(nc)} = 0.43\ \sigma'_v$, for reconstituted Chandanaish soil (5.2c)

where σ'_v is the overburden pressure.

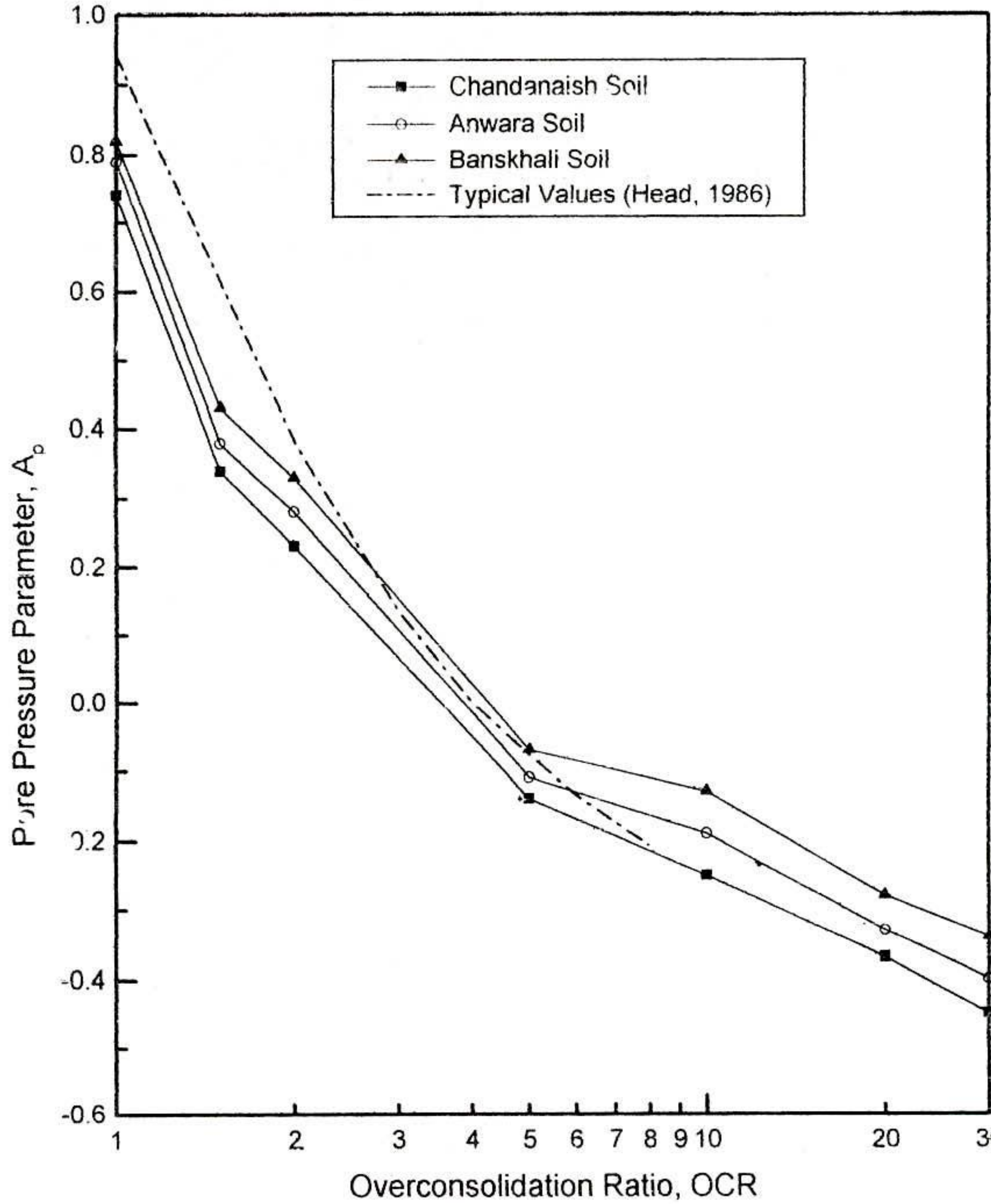


Fig. 5.21 Pore Pressure Parameter at Peak Deviator Stress vs. OCR Plots for Samples of Three Reconstituted Coastal Soils

5.5.2 Shear Strength Model for Overconsolidated Soils

Undrained shear strength of reconstituted coastal soils at overconsolidated state can be predicted by using curves shown in Fig. 5.15, where α versus OCR has been plotted in semi-log scale. Here α is the ratio of undrained shear strength ratio for overconsolidated state $[s_u^{(oc)}/\sigma'_{vo}]$ and normally consolidated state $[s_u^{(nc)}/\sigma'_v]$. The values of $s_u^{(nc)}/\sigma'_v$ for normally consolidated state have been found experimentally which are 0.397, 0.418 and 0.43 for the reconstituted soils from Banskhali, Anwara and Chandanaish, respectively. So from Fig. 5.14,

$\alpha = [s_u^{(oc)}/\sigma'_{vo}] / [s_u^{(nc)}/\sigma'_v]$

that is $s_u^{(oc)} = 0.397 \times \alpha \times \sigma'_{vo}$ for reconstituted Banskhali soil (5.3a)

that is $s_u^{(oc)} = 0.418 \times \alpha \times \sigma'_{vo}$ for reconstituted Anwara soil (5.3b)

that is $s_u^{(oc)} = 0.430 \times \alpha \times \sigma'_{vo}$ for reconstituted Chandanaish soil (5.3c)

Where σ'_{vo} is the in situ overburden pressure corresponding to isotropic stress condition. Simons (1960), Ladd et al. (1977) and Mahar and O'Neill (1983) used such curves to predict $s_u^{(oc)}$ for clays investigated by them.

For a particular value of OCR (where, OCR = σ'_v / σ'_{vo}), α can be found out from Fig. 5.15 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 (σ'_v) can be predicted from e – log σ'_{vc} curve (Fig. 5.1) if void ratio and overburden pressure of in situ soil are known. After locating the point of void ratio and the overburden pressure of in situ soil, in Fig. 5.1, 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 (σ'_v) of the soil.

The undrained shear strength of overconsolidated soils can also be predicted from the curves of Fig. 5.9 to Fig. 5.11. In these Figs 5.9 to 5.11, $s_u^{(oc)}/\sigma'_{vo}$ versus log OCR has been plotted for all the soils. For a particular OCR, The undrained shear strength of the three soils for isotropic stress condition can be found out from these curves. OCR can be predicted if void ratio and overburden pressure of the soils are known by using e - $\log\sigma'_v$ plot (Fig. 5.1) as stated above.

As discussed in literature review, Atkinson and Bransby (1978) 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 overconsolidated soils. The equation is as follows:

$$[\, s_u^{(oc)}/\, \sigma'_{vo} \,] = [\, s_u^{(nc)}/\, \sigma'_{vc} \,] x\, (OCR)^{\Lambda} \tag{5.4}$$

It has been shown in Article 5.4.1 that the experimental data of undrained shear strength ratio ($s_u^{(oc)}/\, \sigma'_{vo}$) and OCR when plotted in log-log scale produced straight lines for the three soils. This has been shown in Fig. 5.11. In this Figure, the equation of the lines for isotropic stress condition.is as follows:

$$[\, s_u^{(oc)}/\, \sigma'_{vo} \,] = [\, s_u^{(nc)}/\, \sigma'_{v} \,] x\, (OCR)^{\Lambda\,(iso)} \tag{5.5}$$

$\Lambda^{(iso)}$ is the slope of each line for overconsolidated samples shown in Fig. 5.11. The slope of the lines have been determined from the plot of Fig. 5.11 which are 0.745, 0.761 and 0.775 for the samples from Banskhali, Anwara and Chandanaish respectively. The intercepts of the lines at OCR = 1 are undrained shear strength ratio at normally consolidated state which are 0.397, 0.418 and 0.43 for the samples from Banskhali, Anwara and Chandanaish respectively. These straight lines can be used as model to determine undrained shear strength of the three soils. By replacing the values of $\Lambda^{(iso)}$ and $s_u^{(nc)}/\sigma'_{vc}$ for three soils, the equation 5.5 can be written as follows:

$$s_u^{(oc)} = 0.397 \text{ x } (\sigma'_{vc})^{0.745}\, (\sigma'_{vo})^{0.255} \quad \text{for Banskhali Soil} \tag{5.6a}$$

$$s_u^{(oc)} = 0.418 \text{ x } (\sigma'_{v})^{0.761}\, (\sigma'_{vo})^{0.239} \quad \text{for Anwara Soil} \tag{5.6b}$$

$$s_u^{(oc)} = 0.43 \text{ x } (\sigma'_{vc})^{0.775}\, (\sigma'_{vo})^{0.225} \quad \text{for Chandanaish Soil} \tag{5.6c}$$

Or in general, $s_u^{(oc)} = 0.415 \times (\sigma'_{vc})^{0.76} (\sigma'_{vo})^{0.24}$ for the three soils (5.6d)

For practical purposes, the model as in eqn. (5.6d) may be very helpful to predict $s_u^{(oc)}$, where 0.415 is the average value of the three values of $s_u^{(nc)}$ and 0.76 is the average value of Λ. σ'_v and σ'_{vo} are the past maximum pressure and overburden pressure, respectively. As discussed earlier, the past maximum pressure can be predicted from e – log σ'_{vc} curve (Fig. 5.1) if void ratio and overburden pressure are known.

For any soil of plasticity index within 10 to 20%, the equation (5.5) can also be applied to find out the undrained shear strength of overconsolidated soil if overburden pressure is known. The value 0.76 of $\Lambda^{(iso)}$ should be used in general eqn.for more accurate value of $s_u^{(oc)}$, as the value predicted from consolidation data is approximate.

Table 5.7 gives a list of model equations, discussed so far, useful for prediction of undrained shear strength of the three reconstituted coastal soils from Banskhali, Anwara and Chandanaish. After all present research suggests to use three models just above the last model or in general the last model of the Table 5.7 for predicting undrained shear strength of normally consolidated and overconsolidated soils.

Table 5.7 Models for Prediction of Undrained Shear Strength

Location of Soil	Stress History	Equation Derived from Plot/ Model	Equation for Isotropically Consolidated Soil
Banskhali	For NC soil	s_u vs. σ'_{vc} plot	$s_u^{(nc)} = 0.397\ \sigma'_{vc}$
Anwara	For NC soil	s_u vs. σ'_{vc} plot	$s_u^{(nc)} = 0.418\ \sigma'_{vc}$
Chandanaish	For NC soil	s_u vs. σ'_{vc} plot	$s_u^{(nc)} = 0.430\ \sigma'_{vc}$
Banskhali	For OC soil	α vs. OCR plot	$s_u^{(oc)} = 0.397 \times \alpha \times \sigma'_v$
Anwara	For OC soil	α vs. OCR plot	$s_u^{(oc)} = 0.418 \times \alpha \times \sigma'_v$
Chandanaish	For OC soil	α vs. OCR plot	$s_u^{(oc)} = 0.430 \times \alpha \times \sigma'_v$
Banskhali	For NC & OC	Mitachi & Kitago (1976)	$s_u^{(oc)} = 0.397 \times (\sigma'_v)^{0.745} (\sigma'_{vo})^{0.255}$
Anwara	For NC & OC	Mitachi & Kitago (1976)	$s_u^{(oc)} = 0.418 \times (\sigma'_v)^{0.761} (\sigma'_{vo})^{0.239}$
Chandanaish	For NC & OC	Mitachi & Kitago (1976)	$s_u^{(oc)} = 0.43 \times (\sigma'_v)^{0.775} (\sigma'_{vo})^{0.225}$
For 3 soils	For NC & OC	Mitachi & Kitago (1976)	$s_u^{(oc)} = 0.415 \times (\sigma'_v)^{0.76} (\sigma'_{vo})^{0.24}$ (General equation for three soils)

NC : Normally consolidated; OC : Overconsolidated

5.6 Void Index and Intrinsic Compression Lines

Figs. 5.1 and 5.2 show the K_0-consolidation and isotropic consolidation curves, respectively for the reconstituted soils of three locations of coastal region. These figures also show swelling curves. The compression and swelling curves plotted in Figs. 5.1 and 5.2 represent the intrinsic compression curves and intrinsic swelling curves for the three soils, since each soil was reconstituted at water content of 1.5 times of the liquid limit. It is useful to normalize these laboratory compression curves with respect to void ratio.

Referring to Burland's concept (1990) the intrinsic compression curves shown qualitatively in Fig. 2.9, has been normalized with respect to void ratio as shown in Fig. 2.11. Similarly, the intrinsic compression curves for the coastal region shown in Figs. 5.1and 5.2 have been normalized by plotting void index, I_v versus log σ'_v and shown in Figs. 5.22 and 5.23. Burland's curve are also shown in Figs. 5.22 and 5.23 for comparison. I_v is defined by the following equation in terms of the two intrinsic constants e^*_{100} and C_c^*.

$$I_v = \frac{e - e^*_{100}}{C_c^*} \tag{5.7a}$$

where $C_c^* = e^*_{100} - e^*_{1000}$

The intrinsic void ratios, e^*_{100} have been found as 0.7805, 0.812 and 0.832 for the three K_0-consolidated soils from Banskhali, Anwara and Chandanaish respectively, while 0.754, 0.773 and 0.797 for the three isotropic consolidated soils from Banskhali, Anwara and Chandanaish, respectively. The intrinsic void ratios (e^*_{1000}) have been found as 0.522, 0.52 and 0.518 for the three K_0-consolidated soils from Banskhali, Anwara and Chandanaish, respectively, while 0.489, 0.478 and 0.484 for the three isotropic consolidated soils from Banskhali, Anwara and Chandanaish, respectively. The intrinsic void ratio (e^*_{100}), intrinsic compressibility (C_c^*), void ratio at the liquid limit (e_L) and the equation of void index in both K_0 and isotropic compressive stress condition for the three locations of the coastal region are shown in Table 5.8. By using the equations given for the void index in Table 5.8, intrinsic compression lines for the three Coastal soils are plotted in Figs. 5.22 and 5.23 for K_0 and isotropic compression, respectively. In Fig. 5.22 the intrinsic compression lines of four other coastal soils

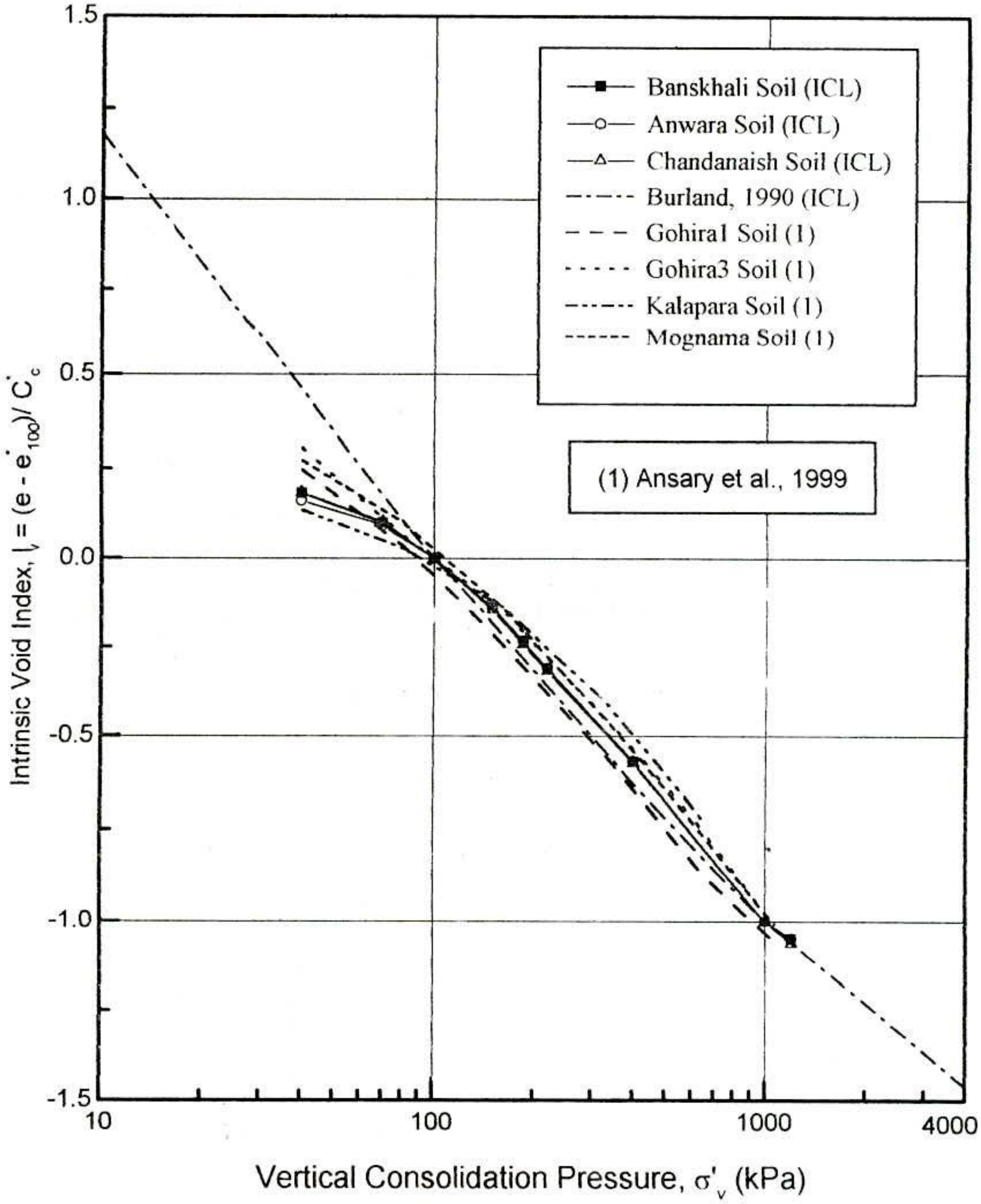


Fig. 5.22 Normalized Intrinsic K_0-Compression Curves Giving Intrinsic Compression Line (ICL)

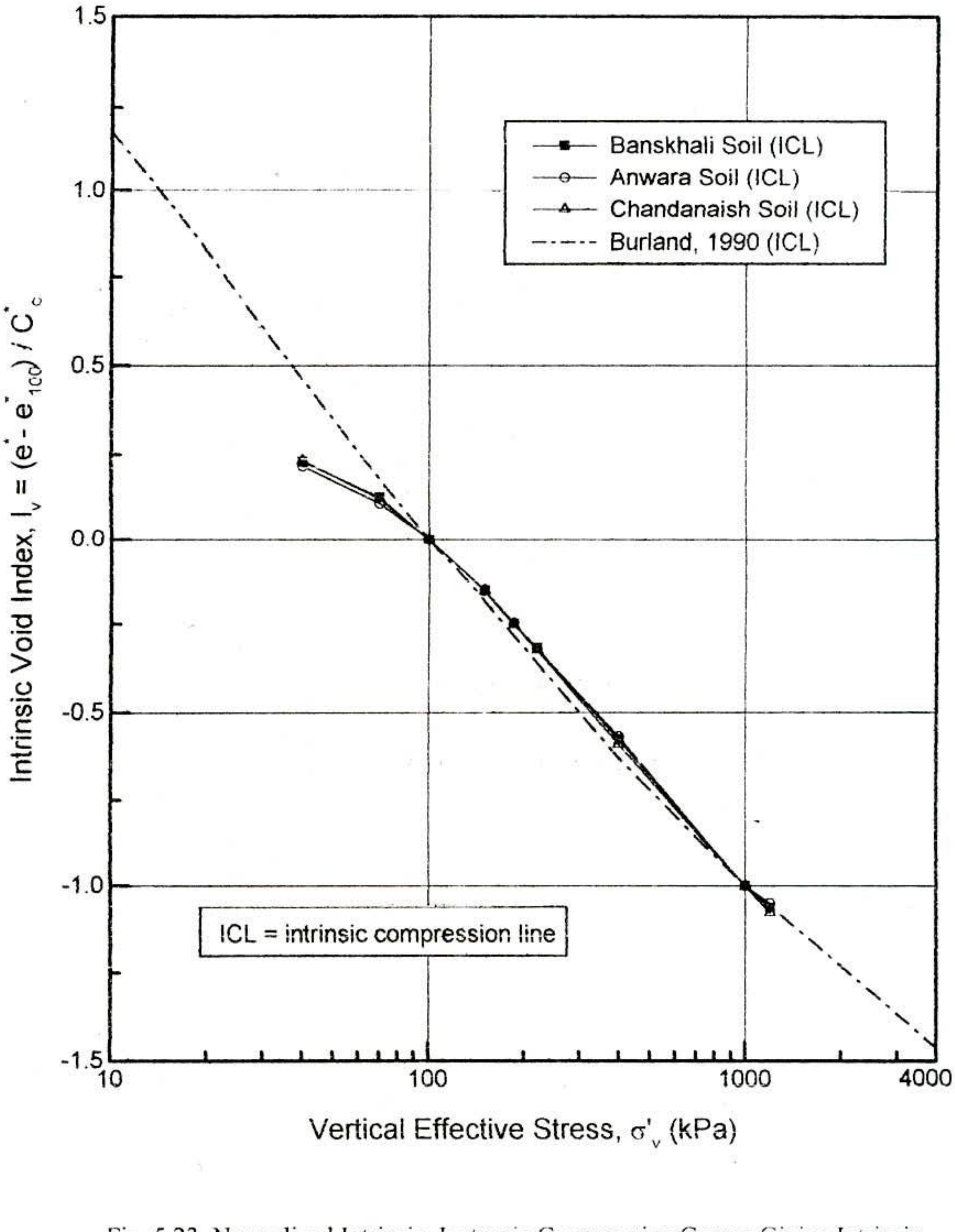


Fig. 5.23 Normalized Intrinsic Isotropic Compression Curves Giving Intrinsic Compression Line (ICL)

(Ansary, 1993) has been drawn for comparison. From the curves in Figs. 5.22 and 5.23, it is shown that the curves thus constructed almost coincide with the intrinsic compression line (ICL) drawn by the following equation (5.7b) furnished by Burland (1990) which represents ICL for most clays. So the following equation is also applicable to these three coastal soils.

$$I_V = 2.45 - 1.285x + 0.015x^3 \tag{5.7b}$$

where, $x = \log \sigma'_v$ in kN/m^2.

The intrinsic constants e^*_{100} and C^*_c may also be predicted from the following empirical equations (5.7c) and (5.7d) which was suggested by Burland (1990).

$$e^*_{100} = 0.109 + 0.679\, e_L - 0.089\, e_L^{\,2} + 0.016\, e_L^{\,3} \tag{5.7c}$$

and

$$C^*_c = 0.256\, e_L - 0.04 \tag{5.7d}$$

where e_L is the void ratio at liquid limit.

The values of intrinsic swelling index (C^*_s) have been found from Figs. 5.1 and 5.2 for K_0 and isotropic compression respectively of the three samples from Banskhali, Anwara and Chandanaish. These values are shown in Table 5.8. The intrinsic properties of the four other coastal soils are shown in Table 5.9 for comparison with the present study. It can be concluded that the variation of the values of intrinsic properties among two Tables 5.8 and 5.9 was not so deviated. From Fig. 5.22 and Fig. 5.23, it can be concluded that for both isotropic and K_0 consolidated soil Burland's (1990) equation can be used for constructing ICL of these three coastal soils.

Table 5.8 Intrinsic Properties of the Soils of Three Locations of Coastal Region

Intrinsic Parameters	Stress Condition (compression)	Location of the Soils		
		Banskhali	Anwara	Chandanaish
e_L	K_0	0.915	1.080	1.224
e^*_{100}	K_0	0.7805	0.812	0.832
C_c^*	K_0	0.256	0.288	0.309
C_s^*	K_0	0.039	0.048	0.060
Equation for I_v	K_0	(e-0.7805)/0.256	(e-0.812)/0.288	(e-0.832)/0.309
e_L	Isotropic	0.915	1.08	1.224
e^*_{100}	Isotropic	0.754	0.773	0.797
C_c^*	Isotropic	0.265	0.295	0.313
C_s^*	Isotropic	0.044	0.053	0.068
Equation for I_v	Isotropic	(e-0.754)/ 0.265	(e-0.773)/0.295	(e-0.797)/0.313

Table 5.9 Intrinsic Properties of the Soils of Four Locations of Coastal Region (after Ansary et al., 1999)

Soil	LL (%)	e_L	e^*_{100}	C_c^*	Eqn. for I_v
G1	59	1.65	1.02	0.405	(e-1.02)/0.405
G3	41	1.11	0.73	0.242	(e-0.73)/0.242
KA	45	1.19	0.80	0.295	(e-0.80)/0.295
	66	0.272	0.73	0.272	(e-0.73)/0.272

CHAPTER 6

CRITICAL STATE PARAMETERS AND STATE BOUNDARY SURFACE

6.1 Critical State Parameters for Isotropic Compression

Figs. 6.1 to 6.3 show the results of isotropic compression and swelling tests in triaxial cell for the three reconstituted samples from Banskhali, Anwara and Chandanaish respectively in (ln p′, v) space where p′ is the average effective stress $(\sigma_1'+\sigma_2'+\sigma_3')/3$ and v is the specific volume (i,e., 1+e). In the Figures the line AB is the isotropic virgin compression line (often called the isotropic normal consolidation line) and the line BC is the average line of swelling and recompression lines. The values of compression index, λ (slope of line AB in Figs. 6.1 - 6.3) were computed to be - 0.115, - 0.128 and - 0.136 and the values of swelling index, κ (slope of line BC) were computed to be - 0.019, - 0.023 and - 0.03 for the samples from Banskhali, Anwara and Chandanaish, respectively. From the Figs. 6.1 to 6.3, it was found that the values of N are 2.29, 2.37 and 2.45 for the samples from Banskhali, Anwara and Chandanaish respectively. The values of soil constants λ, N, κ and Λ for isotropic normal consolidation and swelling lines of the three samples from Banskhali, Anwara and Chandanaish are presented in Table 6.1.

It is evident that if the soil sample is loaded isotropically from A, the normal consolidation line (NCL) always follows the path AB and its state may be moved to the left of AB by unloading along a swelling line such as BC, but it is not possible to move the state of the soil to the right of AB. The line AB represents a boundary between possible states to the left and impossible states to the right as indicated in the Figs. 6.1 to 6.3. It can also be concluded that the values of λ, N, κ and v_k increased with the increase of plasticity for isotropically normally consolidated samples, while the values of Λ decreased with the increase of plasticity. Similar behaviour were observed by Hvorslev (1949) and Parry (1956).

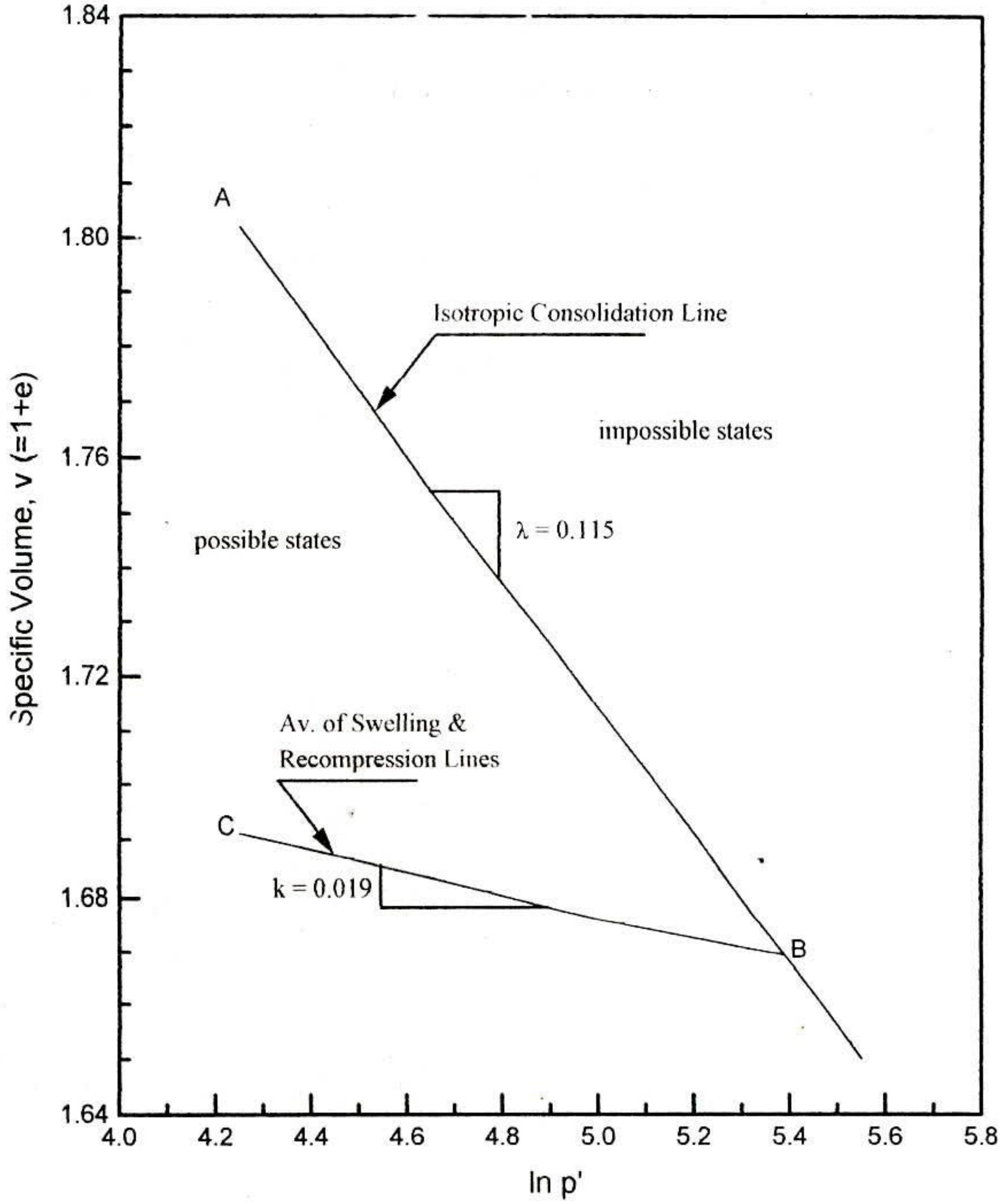


Fig. 6.1 Isotropic Consolidation and Swelling Curves of Reconstituted Banskhali Soil

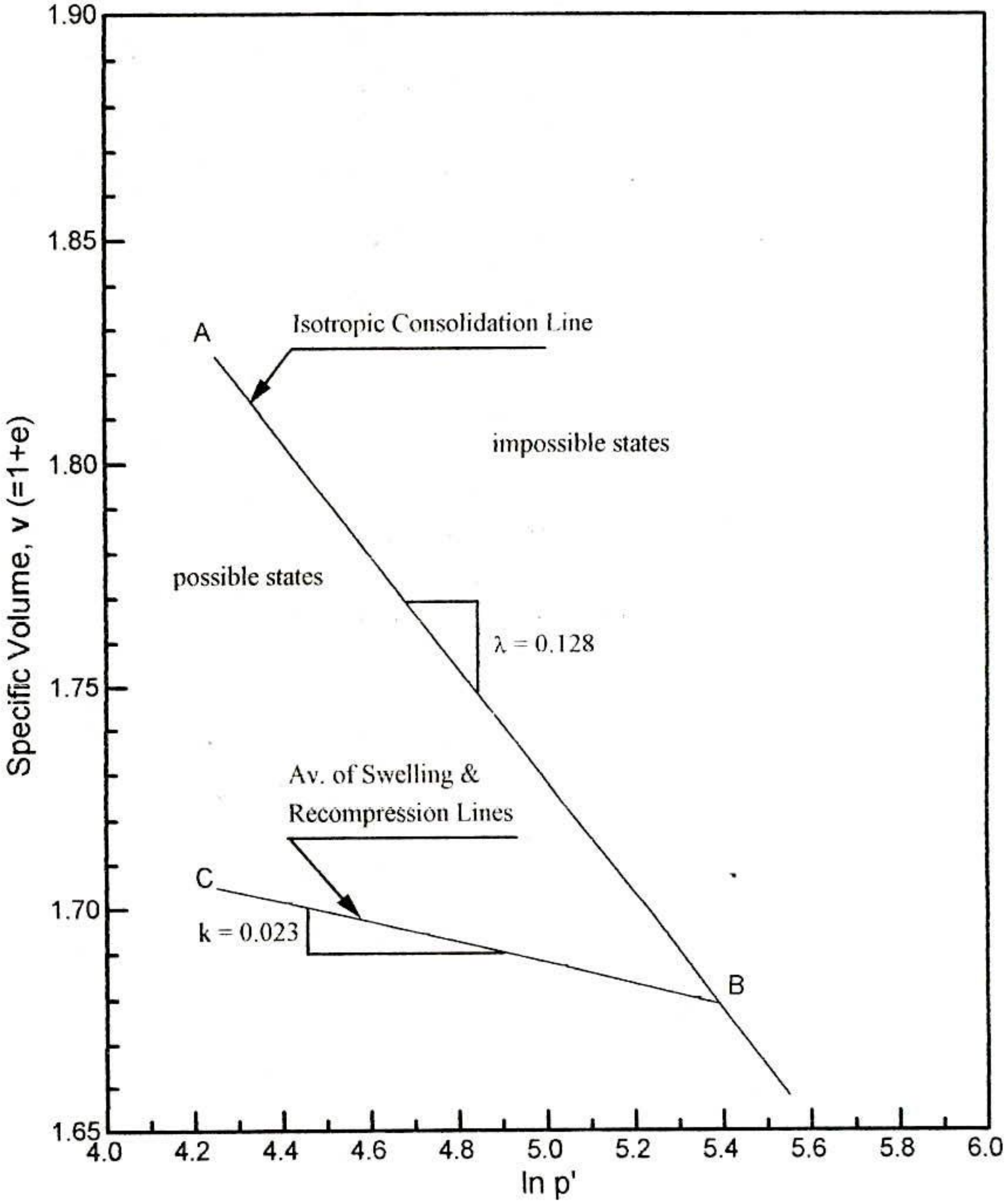


Fig. 6.2 Isotropic Consolidation and Swelling Curves of Reconstituted Anwara Soil

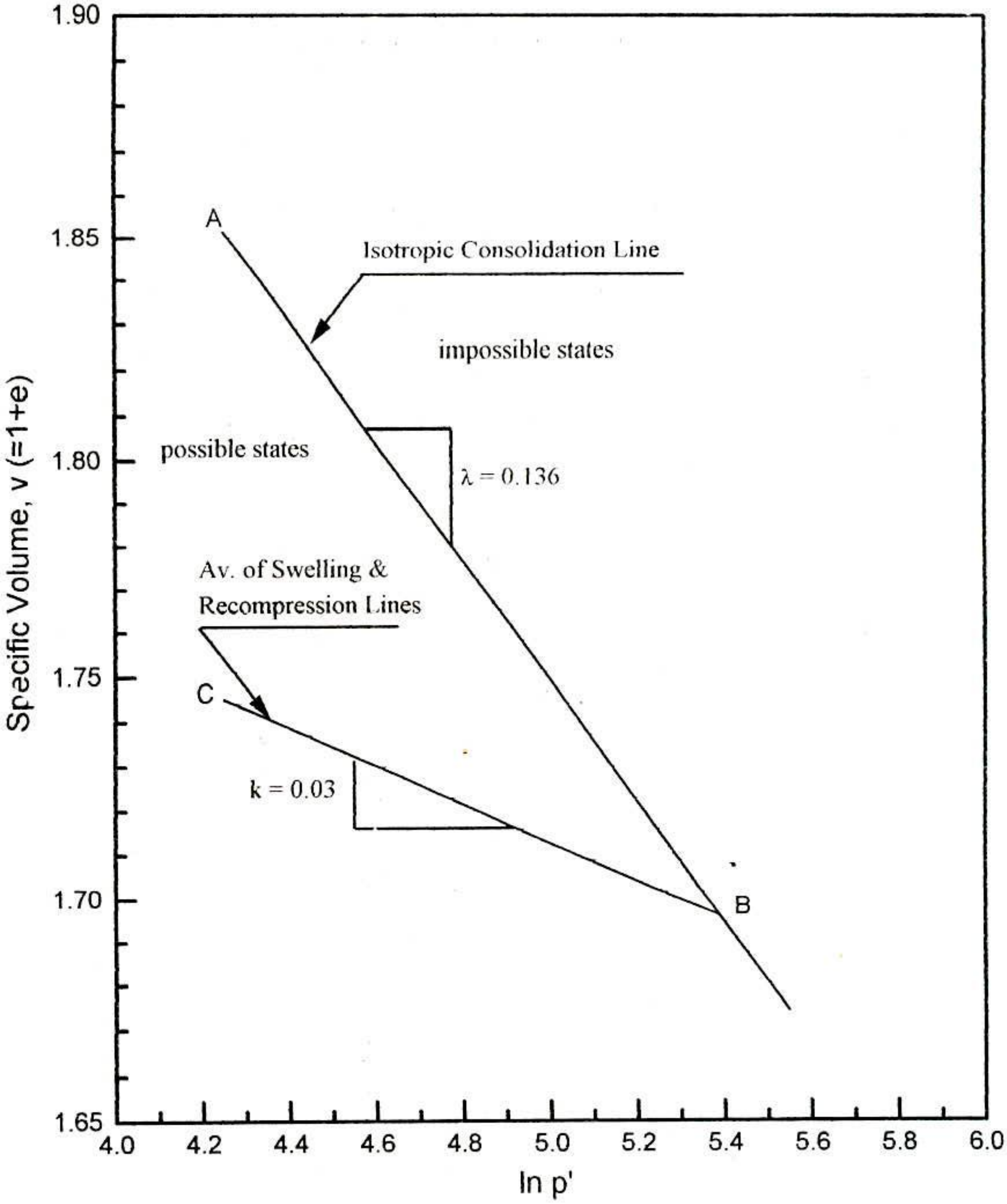


Fig. 6.3 Isotropic Consolidation and Swelling Curves of Reconstituted Chandanaish Soil

Table 6.1 Values of Critical State Soil Parameters under Isotropic Stress Condition

Parameters	Location of Soils		
	Banskhali	Anwara	Chandanaish
λ	0.115	0.128	0.136
κ	0.019	0.023	0.03
Λ	0.835	0.82	0.783
N	2.29	2.37	2.45
v_k	1.77	1.80	1.86

6.2 Critical State Parameters for K_0 Compression

Figs. 6.4 to 6.6 present the results of K_0-compression and swelling tests in triaxial cell for the three samples from Banskhali, Anwara and Chandanaish respectively in (ln p′, v) space. In the Figures line AB is the K_0 virgin compression line (often called the K_0 normal consolidation line) and the line BC is the average line of swelling and recompression line. From the Figs. 6.4 to 6.6, it is exhibited that the values of compression index, λ (slope of line AB) are 0.111, 0.125 and 0.134 and the values of swelling index, κ (slope of line BC) are 0.017, 0.021 and 0.026 for the samples from Banskhali, Anwara and Chandanaish, respectively. From the Figs. 6.4 to 6.6, it is found that the values of N_0 are 2.252, 2.345 and 2.402 for the samples from Banskhali, Anwara and Chandanaish, respectively. From the Figs. 6.4 to 6.6, it is found that the values of v_k are 1.786, 1.827 and 1.87 for the samples from Banskhali, Anwara and Chandanaish, respectively.

The values of soil constants λ, N_0, κ and Λ_0 for K_0 normal consolidation and swelling line of the three samples from Banskhali, Anwara and Chandanaish are presented in Table 6.2.

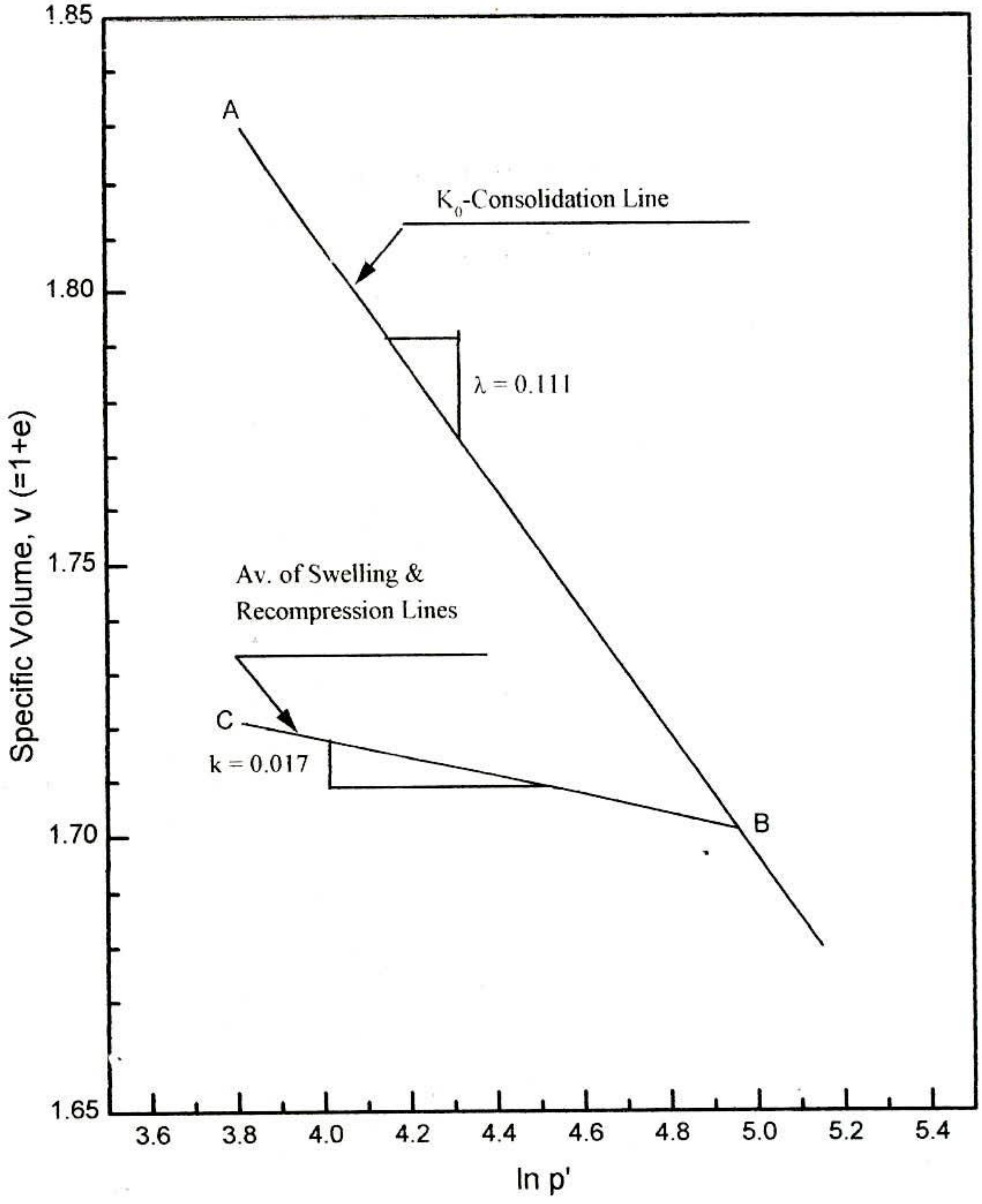


Fig. 6.4 K_0-Consolidation and Swelling Curves of Reconstituted Banskhali Soil

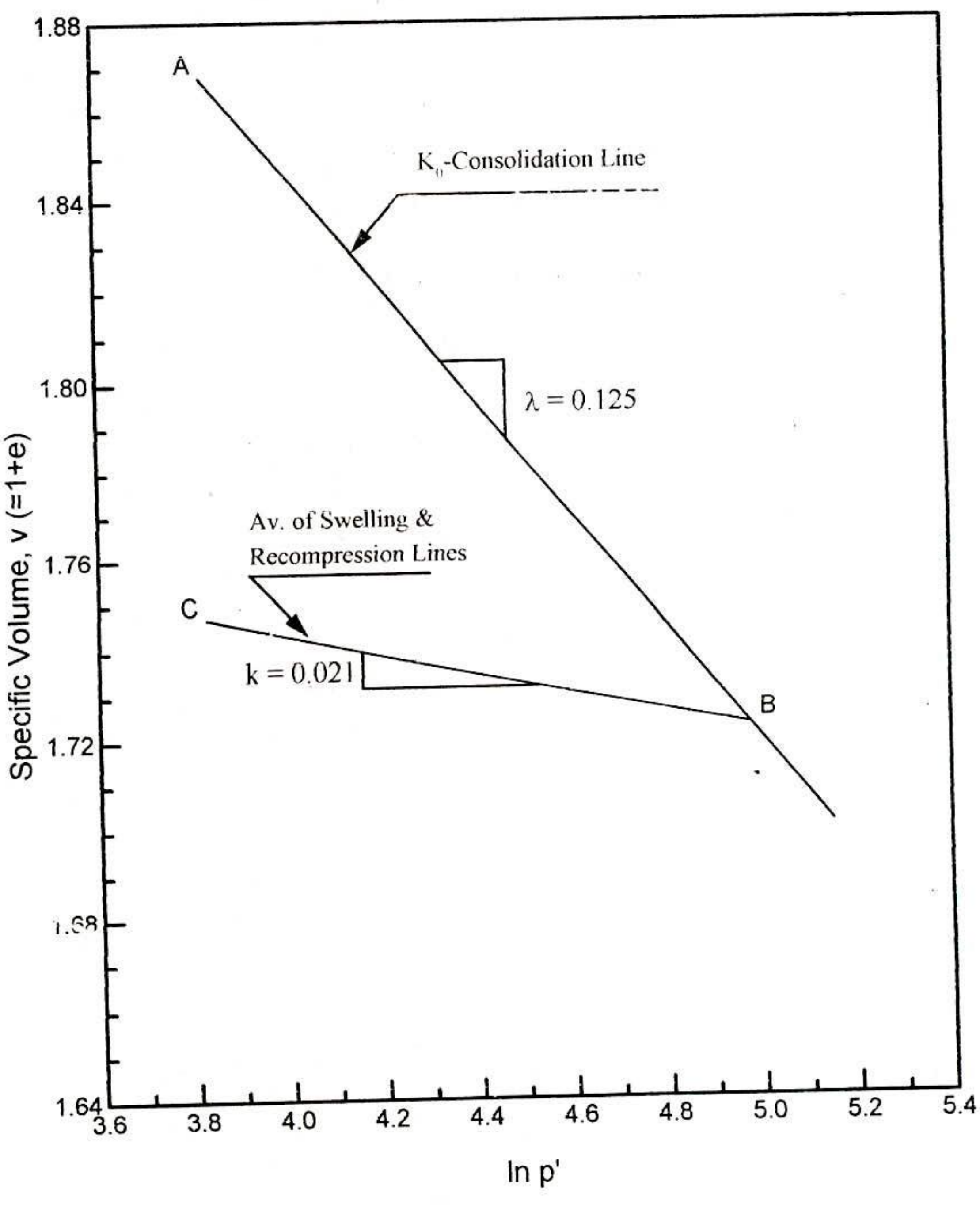


Fig. 6.5 K_0-Consolidation and Swelling Curves of Reconstituted Anwara Soil

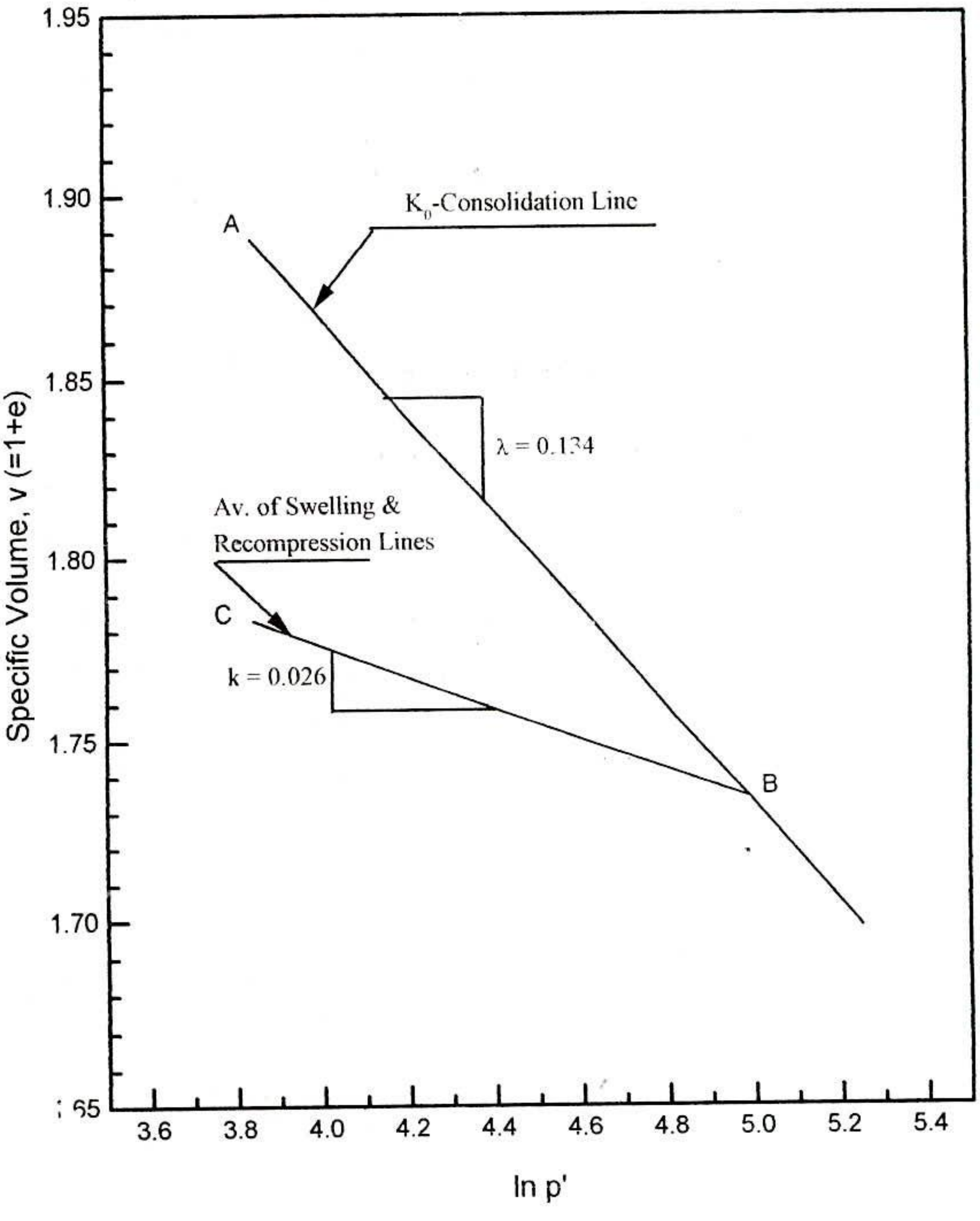


Fig. 6.6 K_0-Consolidation and Swelling Curves of Reconstituted Chandanaish Soil

Table 6.2 Values of Critical State Soil Parameters under K_0-Stress Condition

Parameters	Location of Soils		
	Banskhali	Anwara	Chandanaish
λ	0.111	0.125	0.134
κ	0.017	0.021	0.026
Λ_0	0.847	0.832	0.806
N_0	2.252	2.345	2.402
v_k	1.786	1.827	1.87

It is evident that if the soil sample is loaded one-dimensionally from A, the K_0 normal consolidation line (K_0 NCL) always will follow the path AB and its state may be moved to the left of AB by unloading along a swelling line such as BC, but it is not possible to move the state of the soil to the right of AB. The line AB represents a boundary between possible states to the left and impossible states to the right as indicated in the Figs. 6.4 to 6.6. It is also evident that K_0 normally consolidated samples behaved similar as in isotropically normally consolidated samples in terms of the values of λ, N_0, κ, v_k and Λ. From Tables 6.1 and 6.2 it is observed that the values of λ, κ, N are higher for isotropic stress condition while the values of v_k and Λ are higher for K_0 stress condition for three coastal soils but the differences are insignificant.

6.3 Critical State Lines of the Coastal Soils

Fig. 6.7 presents the critical state line (CSL) in (p', q') plot for three isotropically normally consolidated samples from Banskhali, Anwara and Chandanaish. Though the critical state line is a curved line in three dimensional (p',v, q') space as shown in Fig. 2.39, the projection of the CSL onto the q' : p' plane is a straight line and can be represented by the following equation

$$q' = M\,p' \tag{6.1}$$

where M is the gradient of the CSL. q' and p' are the values of deviator stress and mean effective stress respectively at the failure point. In the Fig. 6.7, the lines joining AB1, AB2 and AB3 are called the CSL of Banskhali, Anwara and Chandanaish, respectively. The values of M thus obtained are 1.34, 1.32 and 1.31 for the three samples from Banskhali, Anwara and Chandanaish, respectively. From the compression test, the effective angle of internal friction, ϕ' was found to be 33.21^0, 32.75^0 and 32.5^0 for isotropic consolidated samples from Banskhali, Anwara and Chandanaish, respectively.. From the results it is evident that the values of M and ϕ' increased with the decrease of plasticity for isotropically normally consolidated samples. Similar results were obtained by Hvorslev (1949) and Parry (1956).

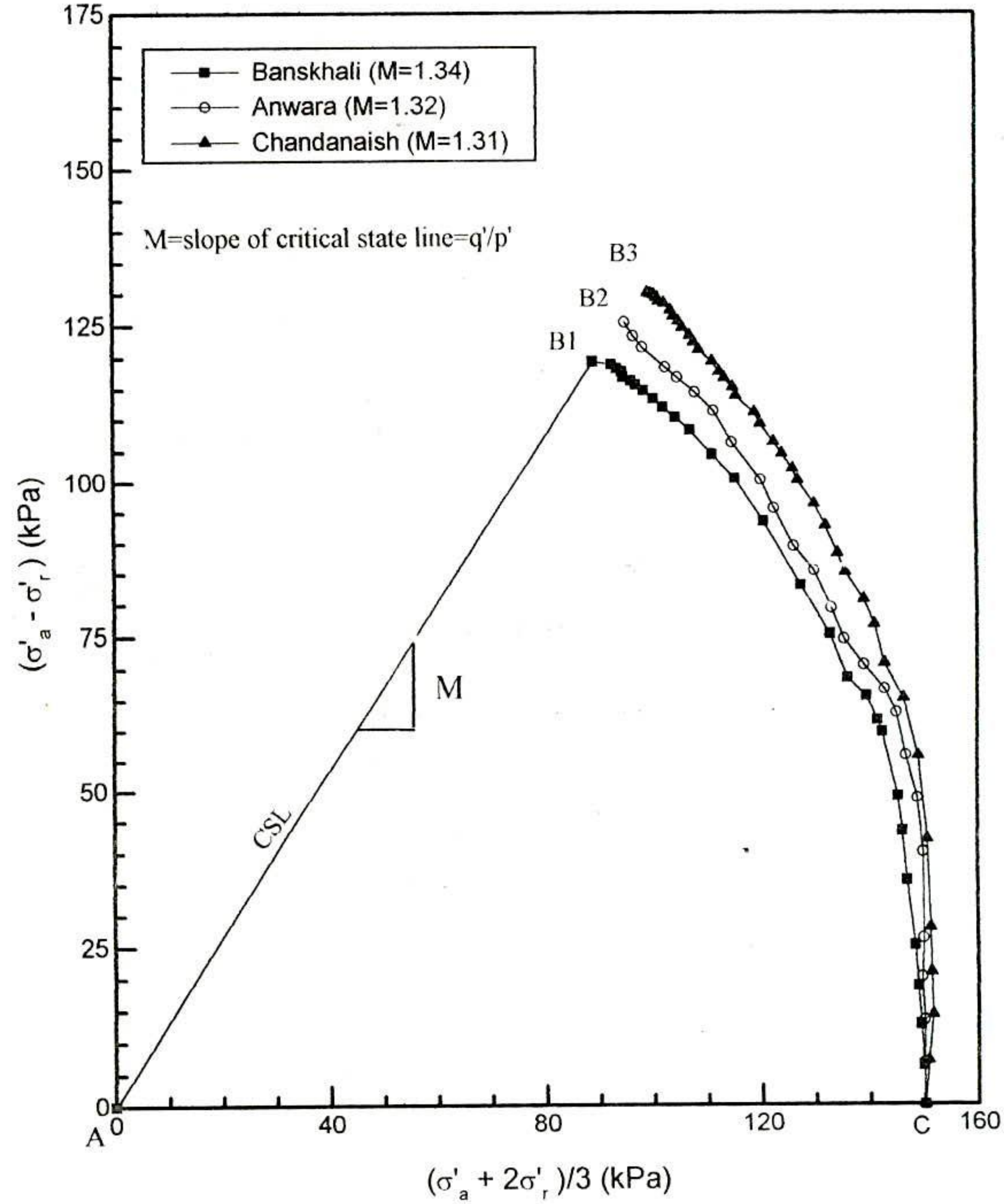


Fig. 6.7 Effective Stress Path for Three Reconstituted Isotropically Normally Consolidated Samples

6.4 State Boundary Surfaces of the Coastal Soils

To establish the state boundary surface of reconstituted soils, the samples of normally consolidated, lightly overconsolidated and heavily overconsolidated were sheared in triaxial cell under isotropic stress condition. The overconsolidation ratios were ranged from 1.5 to 30. The effective undrained stress paths in p′ - q′ space of the samples are shown in Figs. 6.8 to 6.10 for three soils. Since these undrained stress paths are sections of the state boundary surface by constant void ratio planes, it is possible to transform the 3-dimensional state boundary surface (p′ : e : q′ space) to a 2-dimensional curve by a suitable selection of stress parameters. The parameters selected were q'/p'_e and p'/p'_e, where the parameter p'_e is similar to that suggested by Hvorslev (1949).

$$p'_e = p' \exp[(e_0 - e)/\lambda] \tag{6.2}$$

where p′ and e_0 correspond to the isotropic stress and void ratio respectively on the isotropic consolidation line in an (e – log p′) plot. For undrained test, void ratio is constant throughout the shear test, i.e., $e_0 = e$. Then $p'_e = p'$. In these plots (Figs. 6.8 to 6.10) the critical state line is reduced to a single unique critical state point indicated by B. The normalized stress paths obtained from the undrained test on three samples are shown in Figs. 6.11 to 6.13 together with points representing the isotropic normal compression lines from Figs. 6.1 to 6.3 and the critical state lines for compression from Fig. 6.7. Together these define a smooth state boundary surface for each reconstituted sample of the three coastal soils as given by the line DB.

The framework provided by the intrinsic behaviour may be extended to include shearing behaviour. The critical state framework (Roscoe, Schofield and Wroth, 1958) unifies the work of Rendulic (1937) for normally consolidated soils and Hvorslev (1937) for overconsolidated soils. It is of the utmost importance to appreciate that this framework was developed from the results of tests on reconstituted soils from Banskhali, Anwara and Chandanaish.

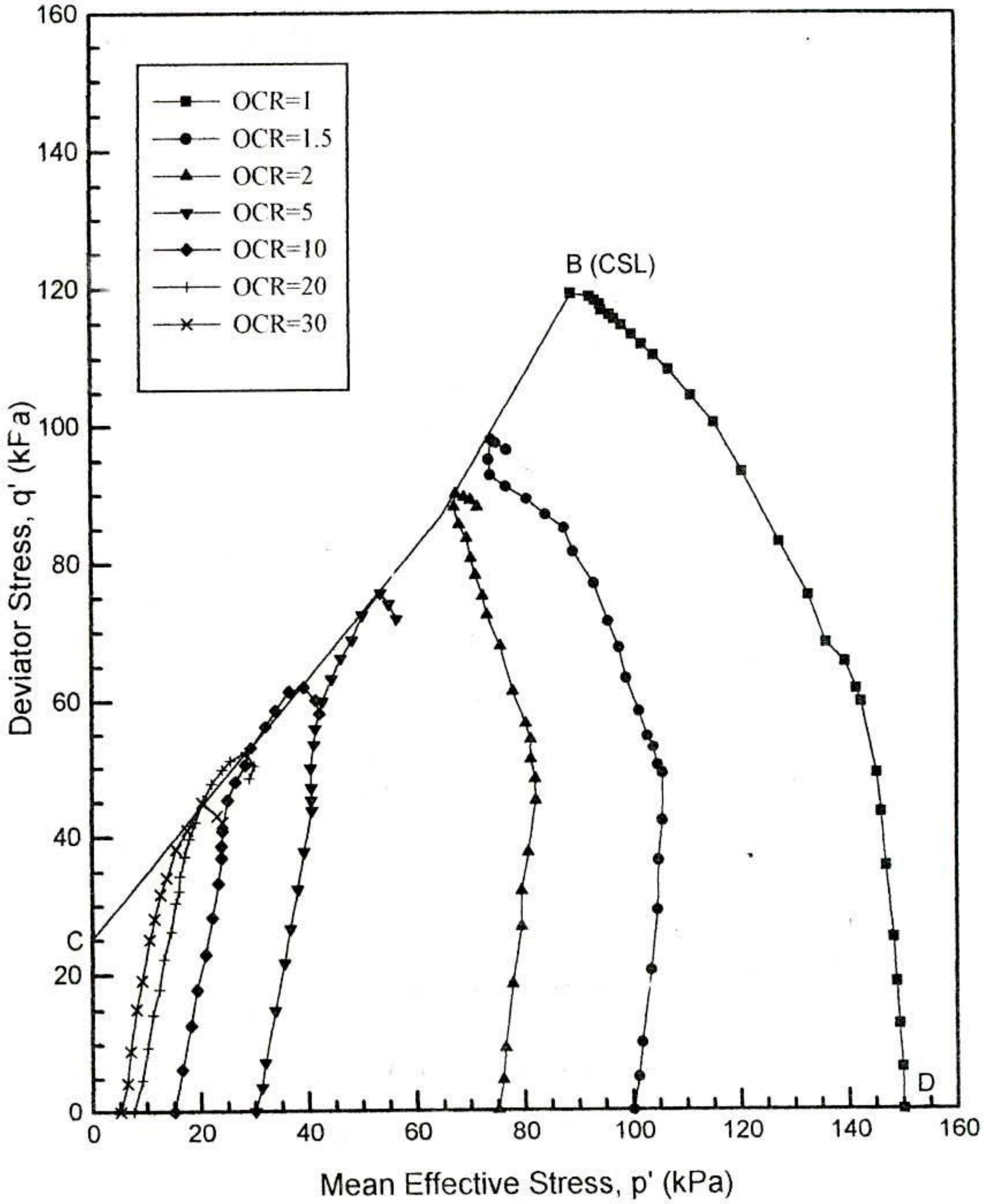


Fig. 6.8 Undrained Effective Stress Paths of Reconstituted Normally Consolidated and Overconsolidated Samples of Banskhali Soil

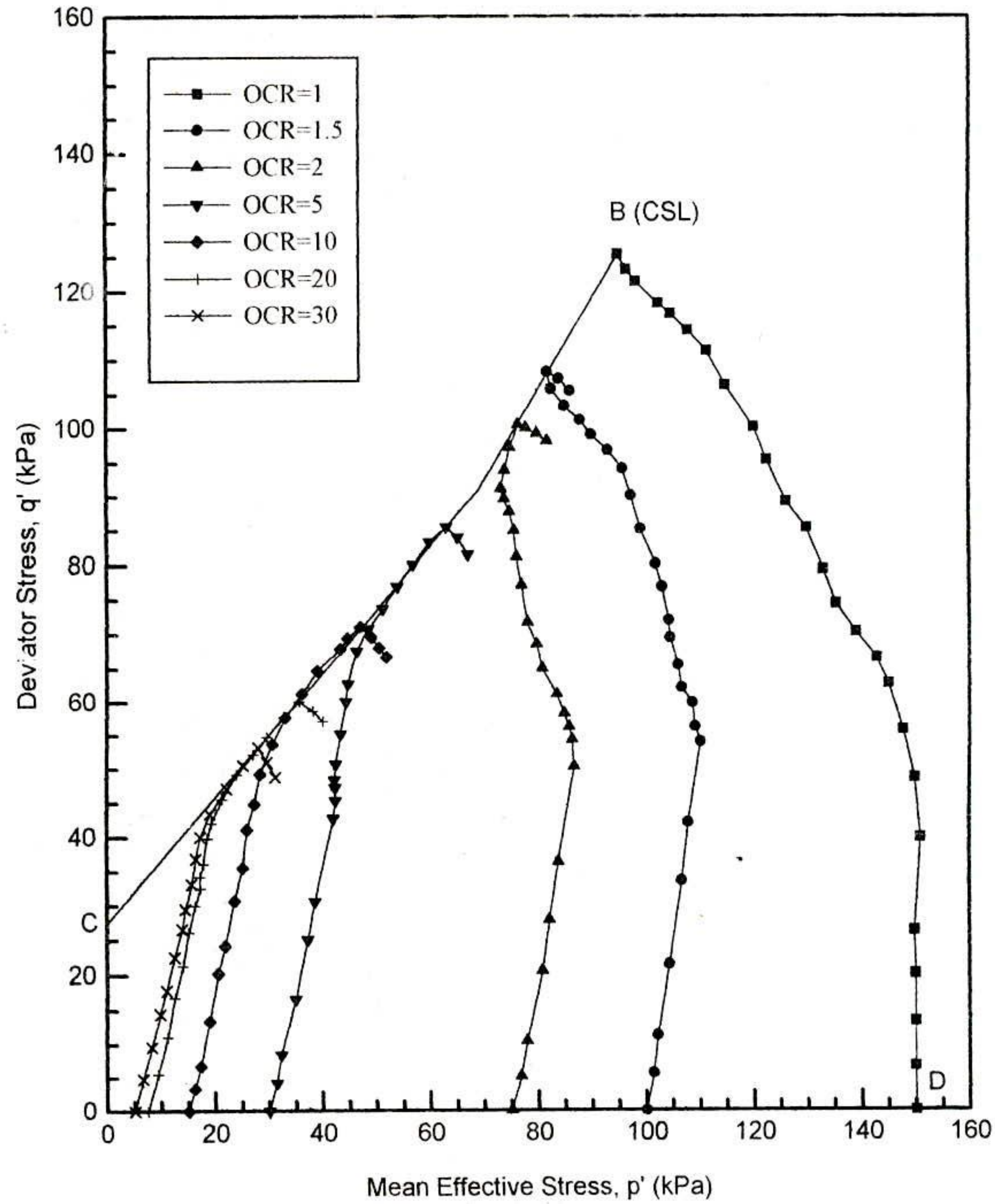


Fig. 6.9 Undrained Effective Stress Paths of Reconstituted Normally Consolidated and Overconsolidated Samples of Anwara Soil

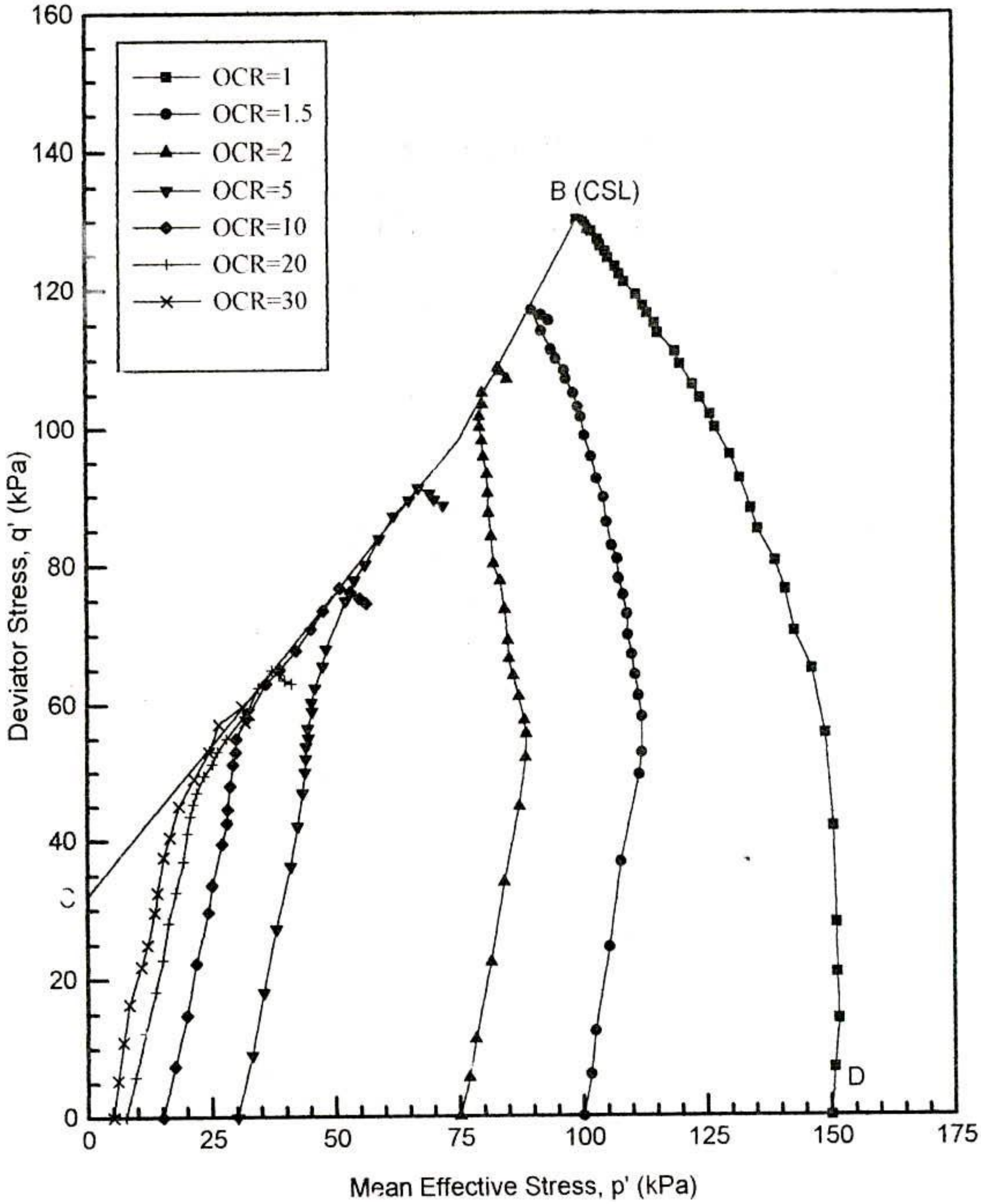


Fig. 6.10 Undrained Effective Stress Paths of Reconstituted Normally Consolidated and Overconsolidated Samples of Chandanaish Soil

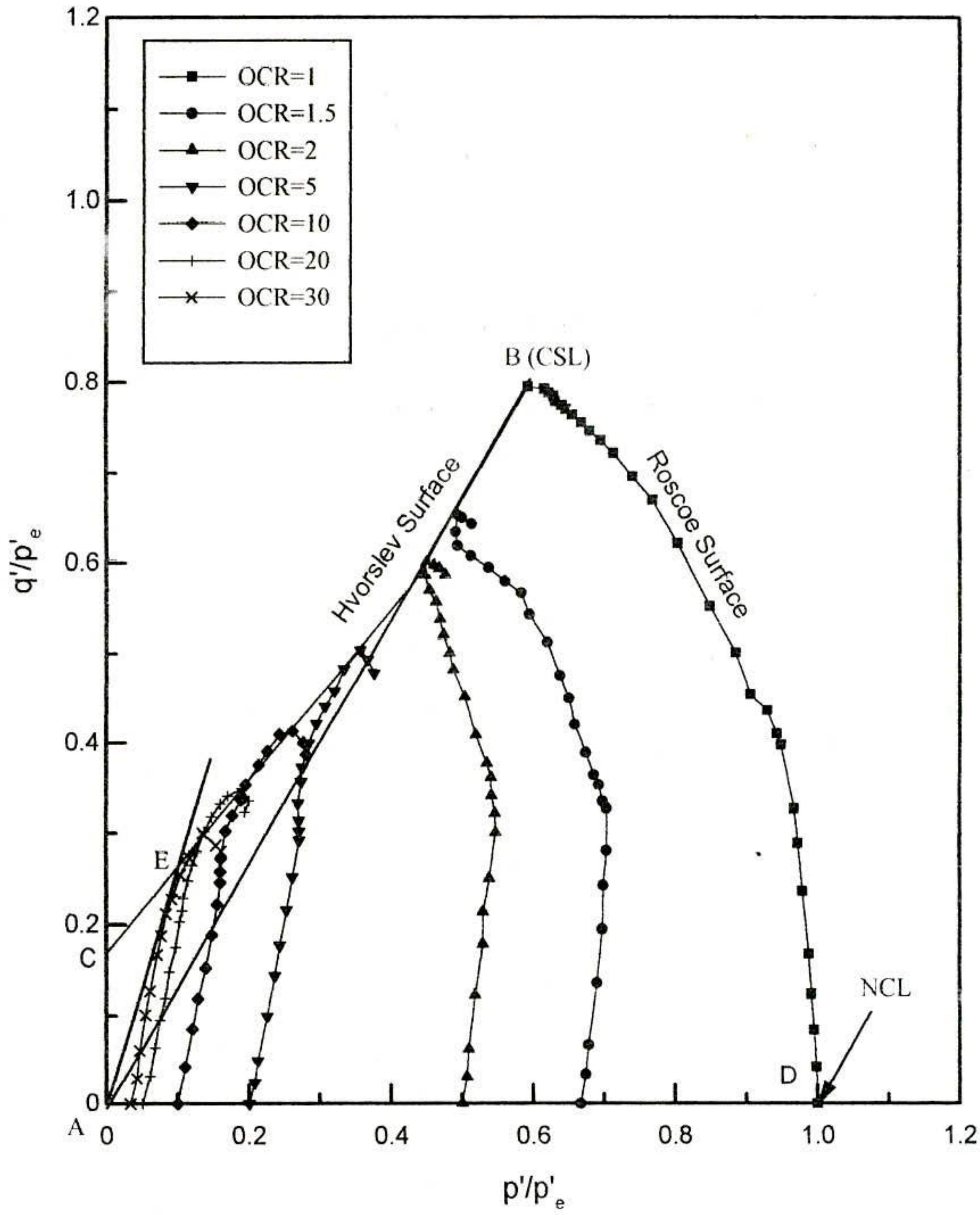


Fig. 6.11 Paths in $q'/p'_e : p'/p'_e$ Space for Undrained Tests on Overconsolidated Samples of Banskhali Soil

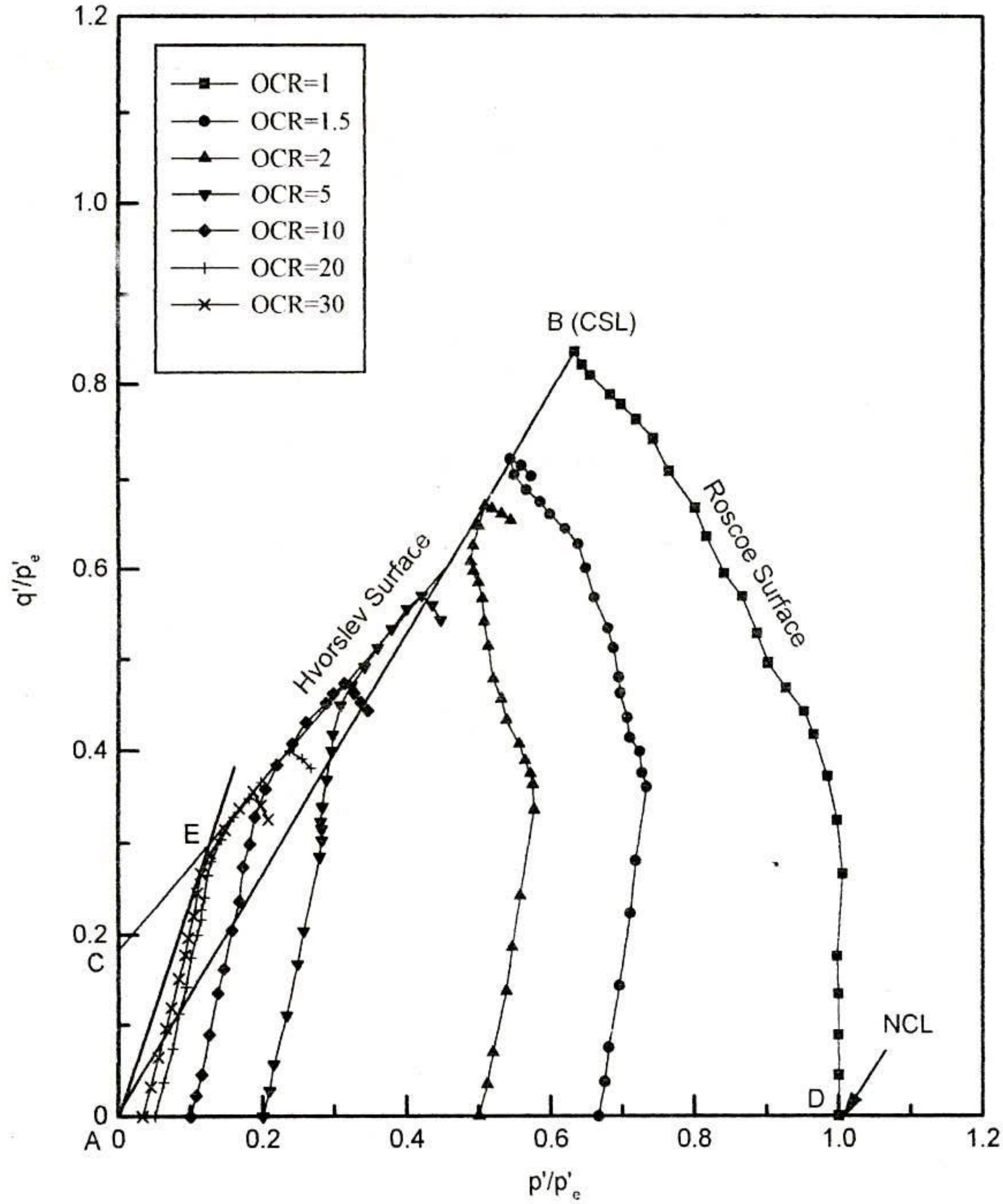


Fig. 6.12 Paths in q'/p'_e : p'/p'_e Space for Undrained Tests on Overconsolidated Samples of Anwara Soil

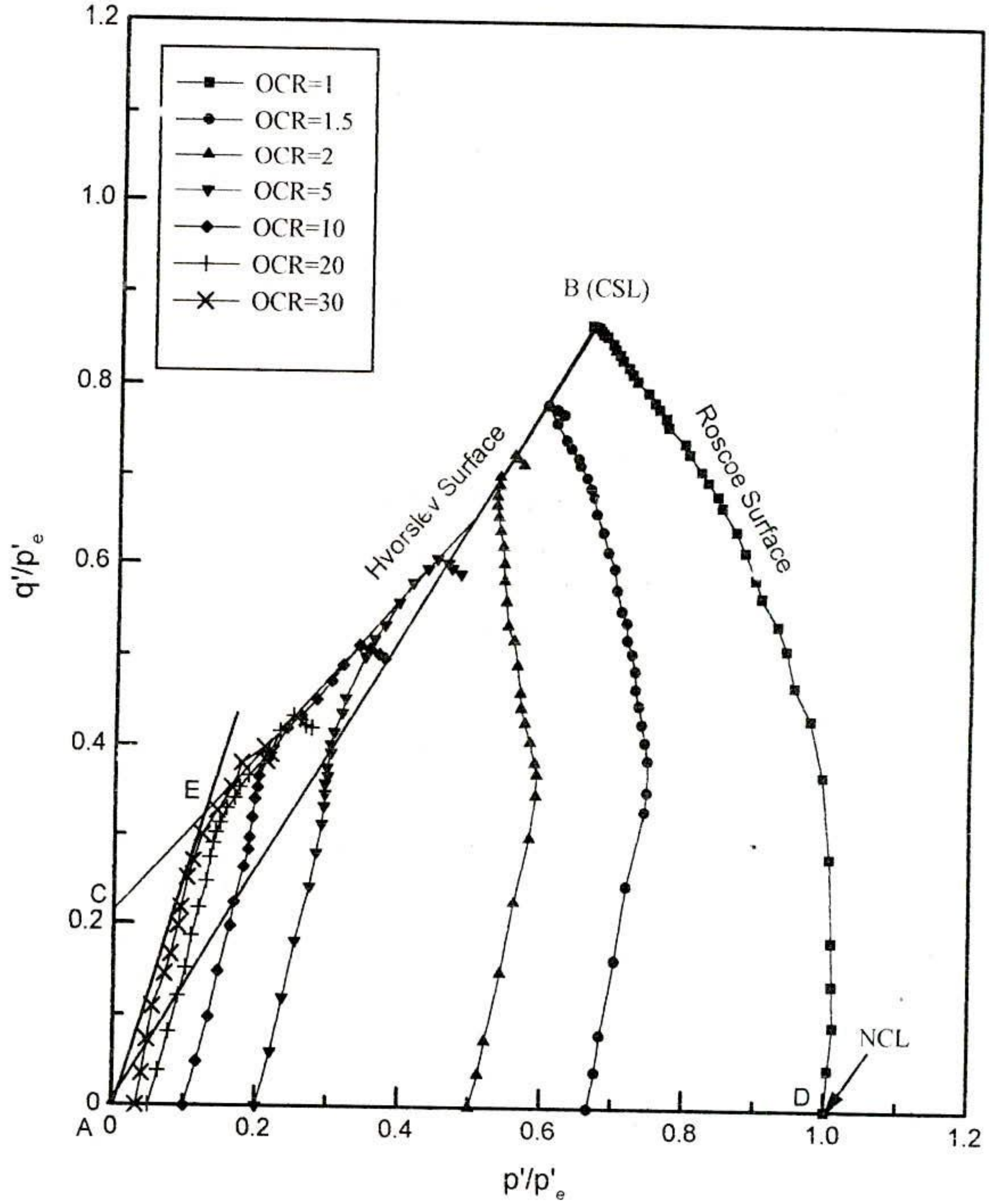


Fig.6.13 Paths in $q'/p'_e : p'/p'_e$ Space for Undrained Tests on Overconsolidated Samples of Chandanaish Soil

Figs. 6.11 to 6.13 show the failure envelope for the reconstituted soils in a normalized plot of q'/p'_e against p'/p'_e. The full line BC is the intrinsic Hvorslev failure surface, where failure is defined by the following normalized Mohr-Coulomb equation

$$\tau / p_e^* = x^* + (\sigma'/p_e^*) \tan\phi_e^* \quad (6.3)$$

where, x^* = the cohesive intercept of the intrinsic failure surface, which is given by the distance AC in Figs. 6.11 to 6.13,

p_e^* = equivalent pressure on the intrinsic compression curve corresponding to the void ratio of the natural soil at yield.

The line DB is the intrinsic Roscoe-Rendulic boundary surface for normally consolidated soils. The point B is the intrinsic critical state line (CSL) having an angle of intrinsic shearing resistance ϕ_{cs}^* and no cohesion. The location of the critical state line is defined by the important quantities p'/p_e^* and q'/p_e^*. The state defined by a point in (q', p', e) space, of a young reconstituted soil can not lie outside the lines DB and BC, which are therefore known as intrinsic state boundary surfaces. No tension cut-off line AE is also shown in Figs. 6.11 to 6.13. The slopes of Hvorslev surface, H for isotropic compression, were obtained as 1.053, 1.086 and 1.136 for reconstituted Banskhali, Anwara and Chandanaish soils, respectively.

The behaviour of the heavily overconsolidated samples (OCR = 10, 20 and 30) can be seen to be strongly dilatant, with the undrained effective stress paths travelling a long way up to the right before rupture.

6.5 Correlation of Critical State Parameters with Plasticity Index

To specify the some constitutive models such as Cam clay model (Schofield and Wroth, 1968), the following four basic soil parameters are required: λ, κ, M and N. In order to specify the behaviour of the models, three other values are also required to describe the current condition of the soils, namely, initial void ratio (or specific volume), current stress state and the K_0-values of the soils. The above critical state soil parameters of reconstituted samples were used to study whether it is possible to determine parameters specifying a constitutive soil model simply by using the plasticity index. In the following section the correlation of the listed five basic parameters with plasticity index has been established.

Figs. 6.14 and 6.15 show the compressibility index, λ and swelling index, κ data from the results of triaxial tests in isotropic and K_0 stress conditions, plotted against the plasticity index (PI). As shown in the Figs. 6.14 and 6.15, the compressibility of soils increases with increasing plasticity index. This trend has been repeatedly reported by various researchers. In their experiments, Kurata and Fujishita (1961) and Akio Nakase, et al. (1988) reported the existence of a linear relationship between PI and compressibility. Ogawa and Matsumoto (1978) found a similar relationship, deduced from a massive amount of oedometer test data related to the construction of port and harbor facilities in Japan. Critical state soil mechanics theory (Schofield and Wroth, 1968) also predicted this relationship, expressed as

$$\lambda = 0.00585\ PI \tag{6.4}$$

A similar relationship was found between the swelling index and the plasticity index. Similar behaviour was also observed by Hvorslev and Parry (After Schofield and Wroth, 1968) and Akio Nakase et al. (1988). Regression lines for these relationships obtained from the present data can be given as

for isotropic stress condition.

$$\lambda = 0.09405 + 0.00211\ PI \qquad 6.5(a)$$

$$\kappa = 0.00766 + 0.0011\ PI \qquad 6.5(b)$$

for K_0 stress condition.

$$\lambda = 0.088 + 0.0023\ PI \qquad 6.6(a)$$

$$\kappa = 0.00782 + 0.00088\ PI \qquad 6.6(b)$$

The values of the correlation coefficient (R) are high. R = 1.0 and 0.97 to 0.98 for the compressibility index and the swelling index respectively.

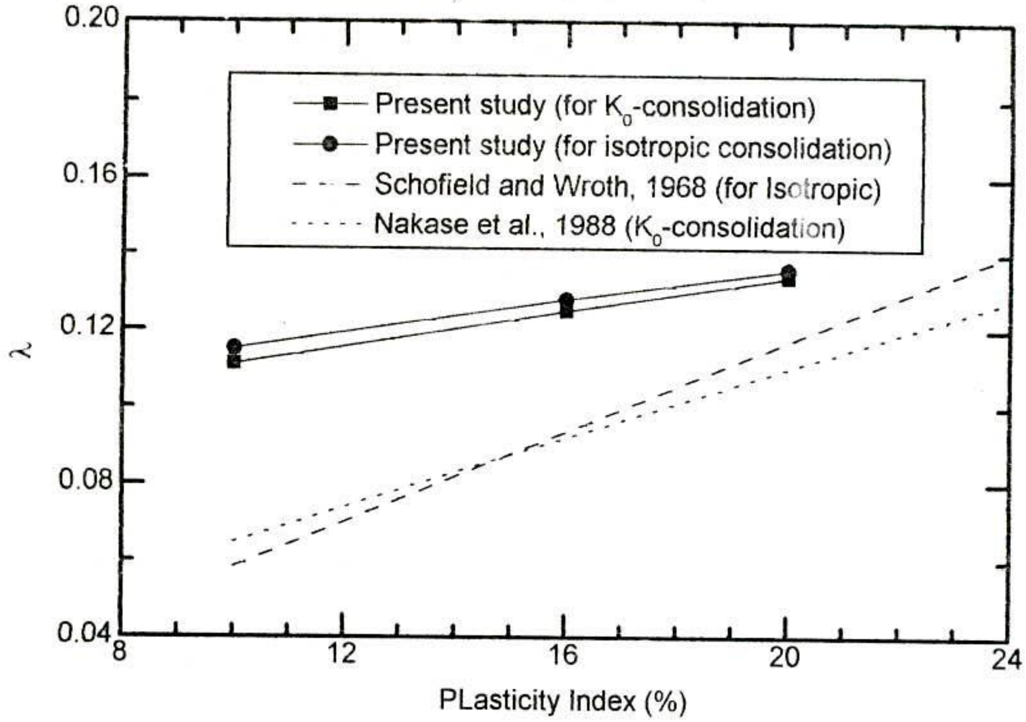


Fig. 6.14 Correlation between Compressibility Index and Plasticity Index

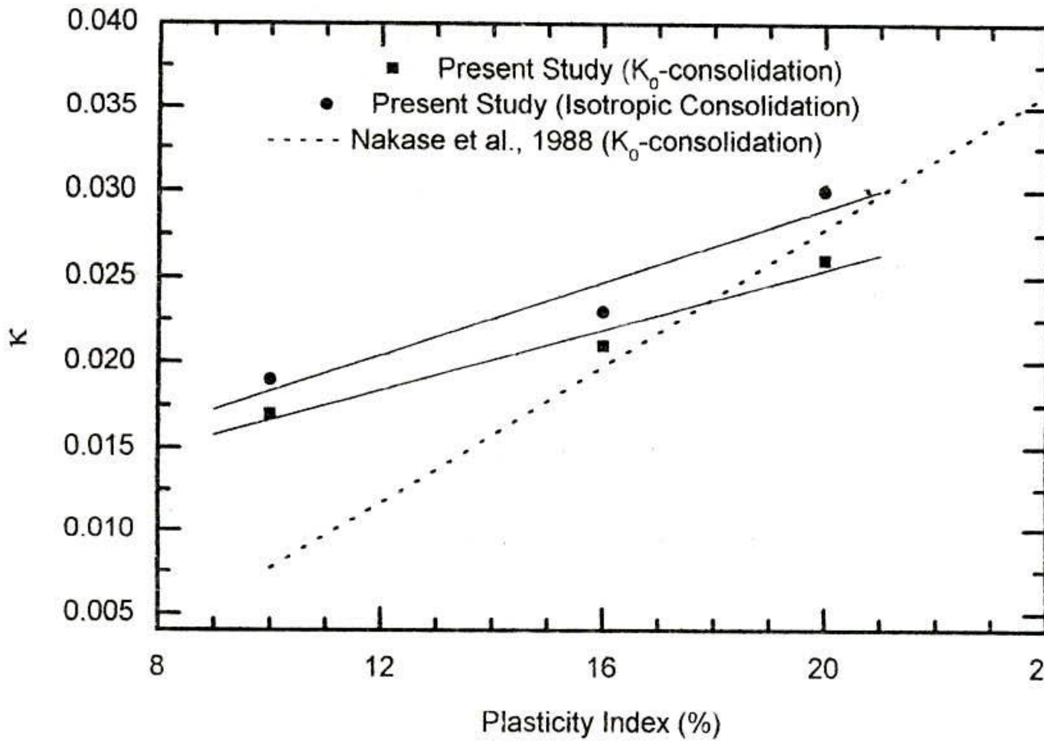


Fig. 6.15 Correlation between Swelling Index and Plasticity Index

Fig. 6.16 illustrates the relationship of $N^{(iso)}$ and N_0 [or $N^{(Ko)}$] (the values of specific volume at $p' = 1.0$ kPa on isotropically and K_0 normally consolidated line respectively) with PI. Fig. 6.17 illustrates the relationship of the critical state parameters, M^{ISO} with PI. It was found from the Figs. 6.16 and 6.17 that there exists a linear relationship of N^{ISO}, N^{KO} and M^{ISO} with PI. From Figs. 6.16 and 6.17 it is also evident that the values of N^{ISO} and N^{KO} increase with the increase of PI, while the values of M^{ISO} decrease with the increase of PI. Similar behaviour was reported by Schofield and Wroth, 1968) for isotropic stress condition. Akio Nakase et al. (1988) reported for K_0 stress condition that the values of N^{KO} increase with the increase of PI, while M^{ISO} is constant with PI. The resulting relationships for the present study are as follows:

for isotropic stress condition.

$$N = 2.128 + 0.0158\ PI \qquad 6.7(a)$$

$$M = 1.37\ (1 - 0.0022\ PI) \qquad 6.7(b)$$

for K_0 stress condition.

$$N_0 = 2.1024 + 0.015\ PI \qquad 6.8$$

The values of the coefficient of correlation (R) obtained are the remarkably high values of 0.99 to 1.0 for N^{ISO}, N^{KO} and M^{ISO}. Fig. 6.18 shows the relationship between the slope of Hvorslev surface, H and PI. It was found that the value of H decreases with the increase of PI. These correlations are summarized in Table 6.3.

6.6 Comparison of Experimental Stress-Strain and Stress Path with the Prediction Values from Two Critical State Models

In this section, the experimentally observed stress-strain and effective stress path are compared with those from Critical State Theories. Two theories have been employed and these are the Cam Clay Theory by Roscoe, Schofield and Thurairajah (1963); and Modified Cam Clay Theory by Roscoe and Burland (1968).

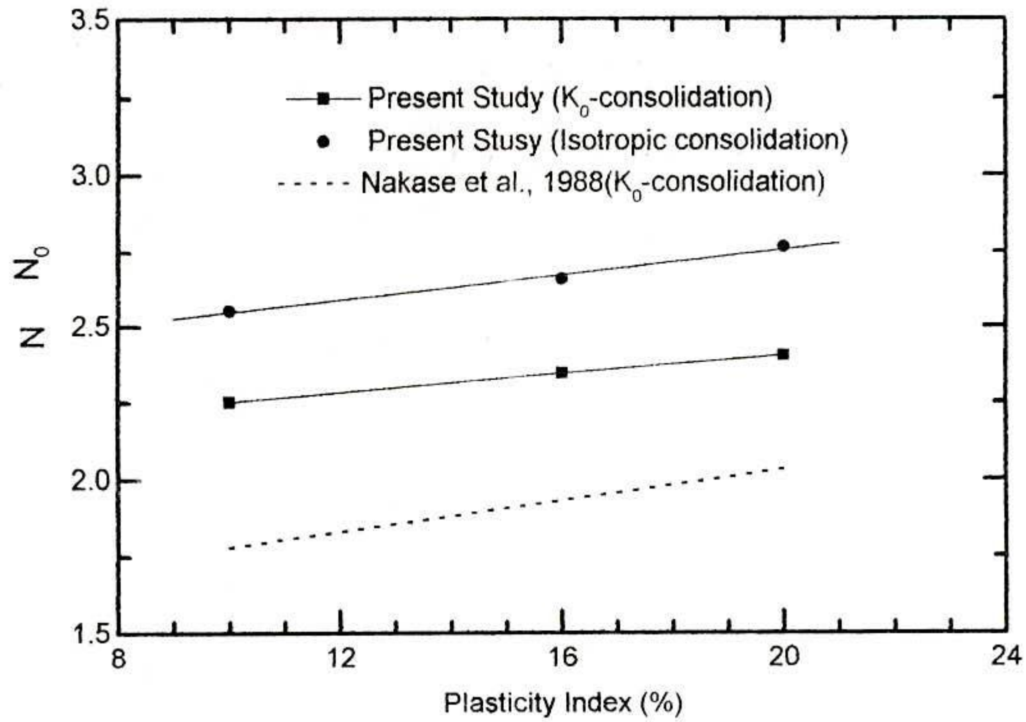


Fig. 6.16 Correlation between N-Value for Isotropic and K_0-consolidation and Plasticity Index

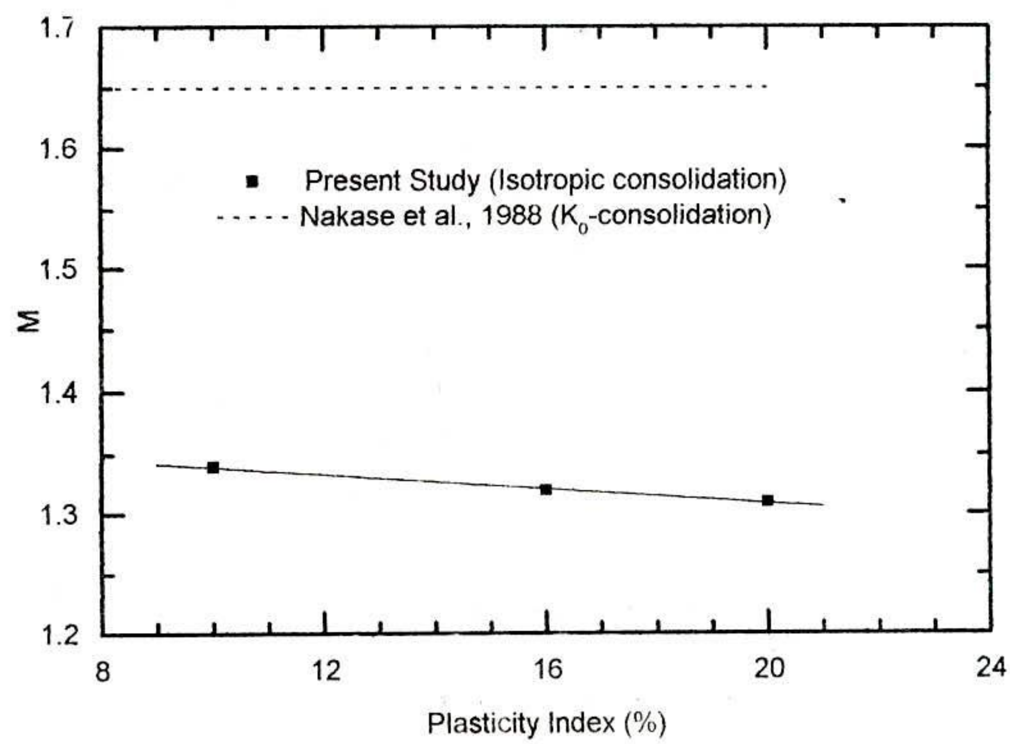


Fig. 6.17 Correlation between M-Value and Plasticity Index

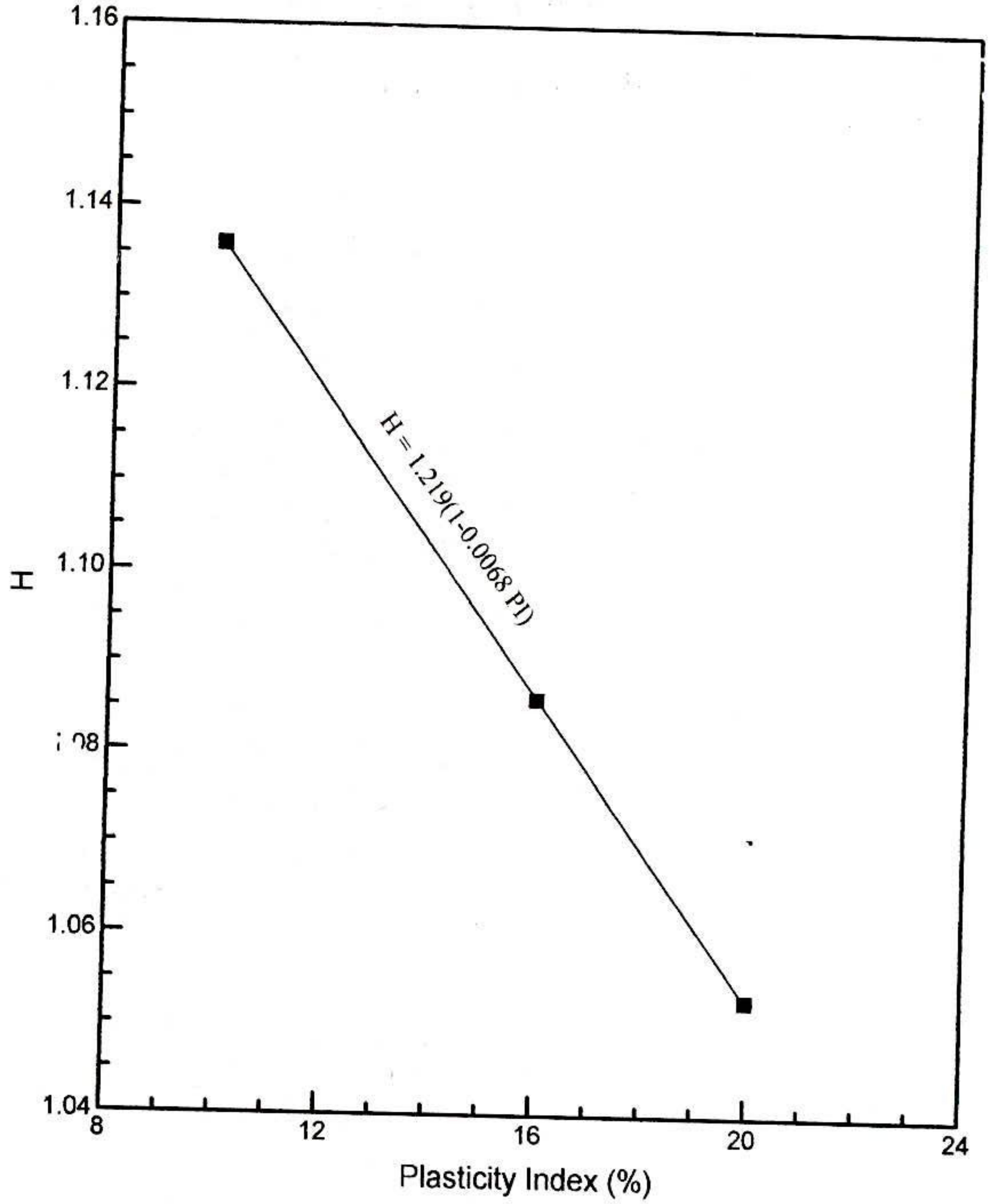


Fig. 6.18 Correlation between H-Value and Plasticity Index

Table 6.3 Constitutive Equations of Critical State Soil Parameters with Plasticity Index

Relationship of Parameters	Stress Condition	Coefficient of Correlation (R)
$\lambda = 0.094 + 0.0021$ PI	Isotropic consolidation	1.0
$\kappa = 0.00766 + 0.0011$ PI	Isotropic consolidation	0.97
$N = 2.128 + 0.0158$ PI	Isotropic consolidation	0.99
$M = 1.37(1-0.0022$ PI)	Isotropic consolidation	1.0
$H = 1.219(1-0.0068$ PI)	Isotropic consolidation	1.0
$\lambda = 0.088 + 0.0023$ PI	K_0-consolidation	1.0
$\kappa = 0.00782 + 0.00088$ PI	K_0-consolidation	0.98
$N_0 = 2.1024 + 0.015$ PI	K_0-consolidation	1.0

The fundamental soil parameters used in the Critical State Theories are λ, κ and M, in which λ = the slope of the isotropic normally consolidation line in v, ln p′ plot; κ = the slope of isotropic swelling line in v, ln p′ plot; and M = the slope of the critical state line in the q′, p′ plot. Previously it was found that the values of λ are 0.115, 0.128 and 0.136 for the reconstituted samples from Banskhali, Anwara and Chandanaish respectively, and the corresponding values of κ are 0.019, 0.023 and 0.03. Also, the critical state parameters, M, for the three soils have been found to be as 1.34, 1.32 and 1.31 respectively.

Two programmes were developed, one for Cam Clay model and another for Modified Cam Clay model. The values of λ, κ, M, Λ, v, w and preshear consolidation pressure as shown in the previous sections are used for the prediction of stress and strains using the two models. The programmes used are shown in "APPENDIX-I" and "APPENDIX-II". To develop the programs, equations (2.15d), (2.15e), (2.15g) were used for Cam Clay model and equations (2.18), (2.19a) and (2.20) were used for Modified Cam Clay model.

Figs. 6.19 to 6.21 show the comparative stress-strain plot of experimental data for Chandanaish soil with the predicted data using Cam Clay and Modified Cam Clay Models. From the Figs. 6.19 to 6.21, it is observed that Modified Cam Clay Model better represents the experimental curve than Cam Clay Model. Similar behaviours were found by Balasubramaniam and Chaudhry (1978) for Bangkok clay. At Low strain level (up to 4%) modified Cam Clay Model overestimates the values of deviator stress while at relatively large strain levels, modified Cam Clay Models slightly underestimates the values of deviator stress. Cam Clay Model highly underestimates the value of deviator stress at all strain levels. Figs. 6.22 to 6.24 show the typical effective stress path plots of experimental data for three soils with the predicted data using Cam Clay and Modified Cam Clay Models. From the Figs. 6.22 to 6.24, it has

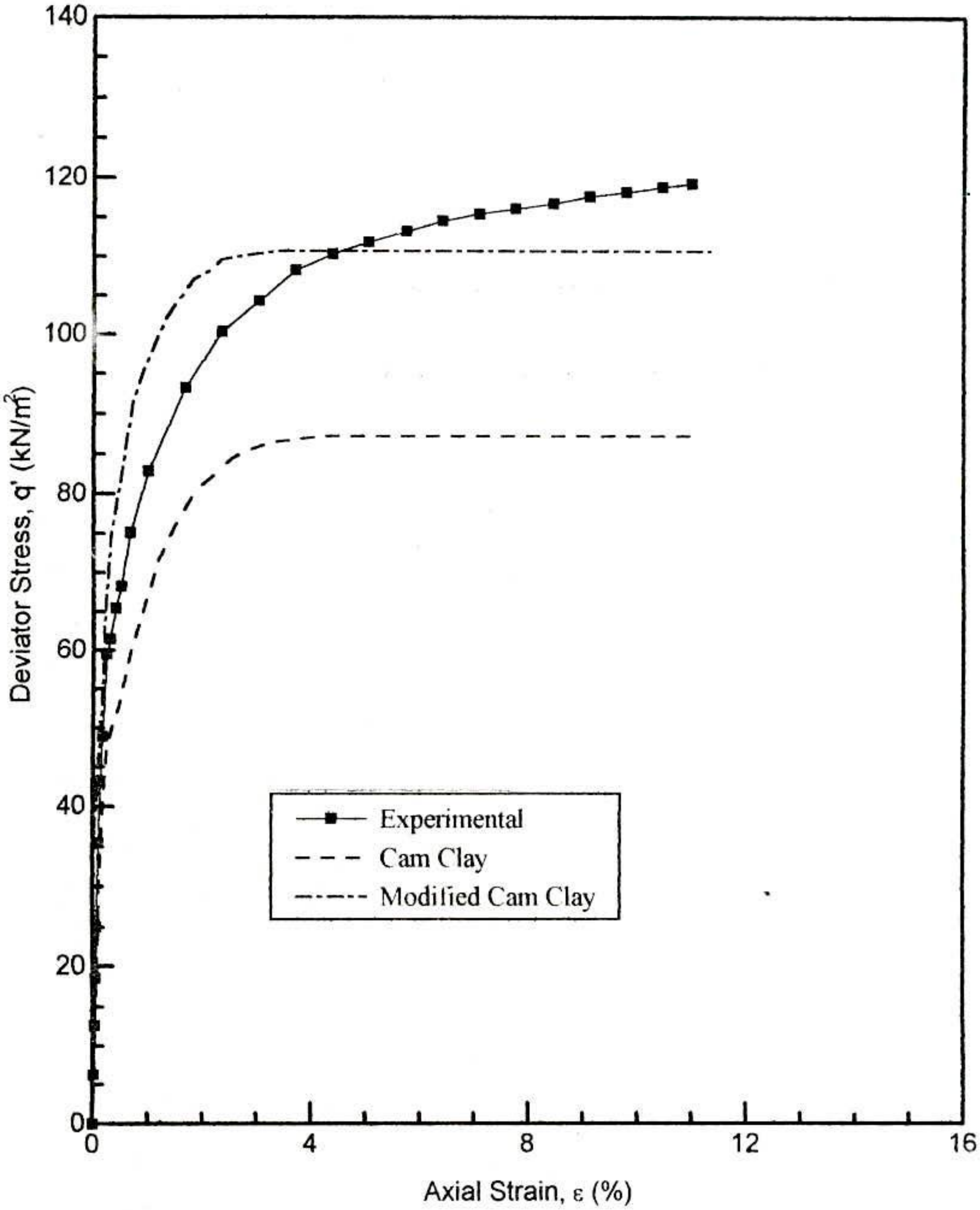


Fig. 6.19 Comparison of Observed & Predicted Stress-Strain Curves for Banskhali Soil

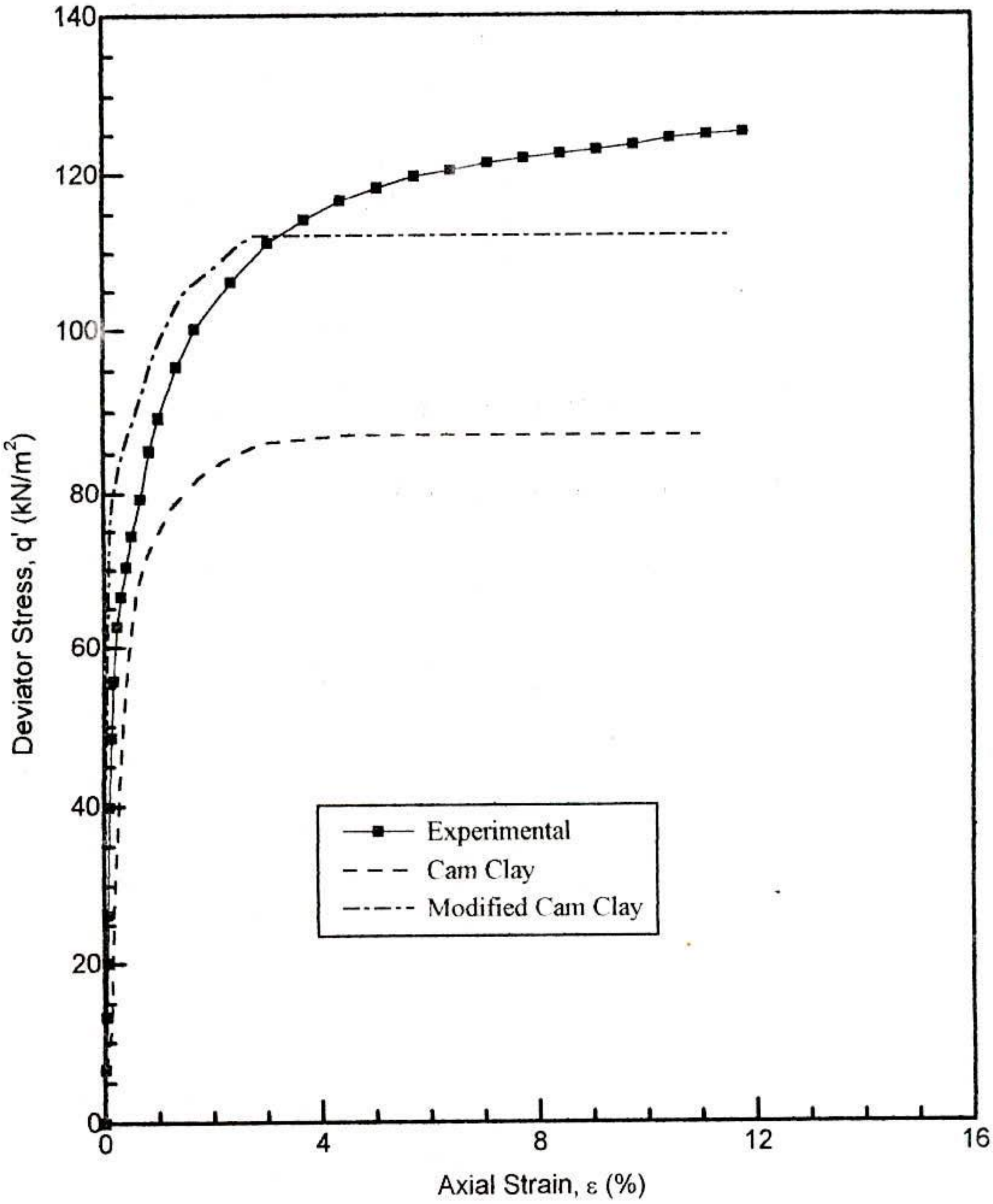


Fig. 6.20 Comparison of Observed & Predicted Stress-Strain Curves for Anwara soil

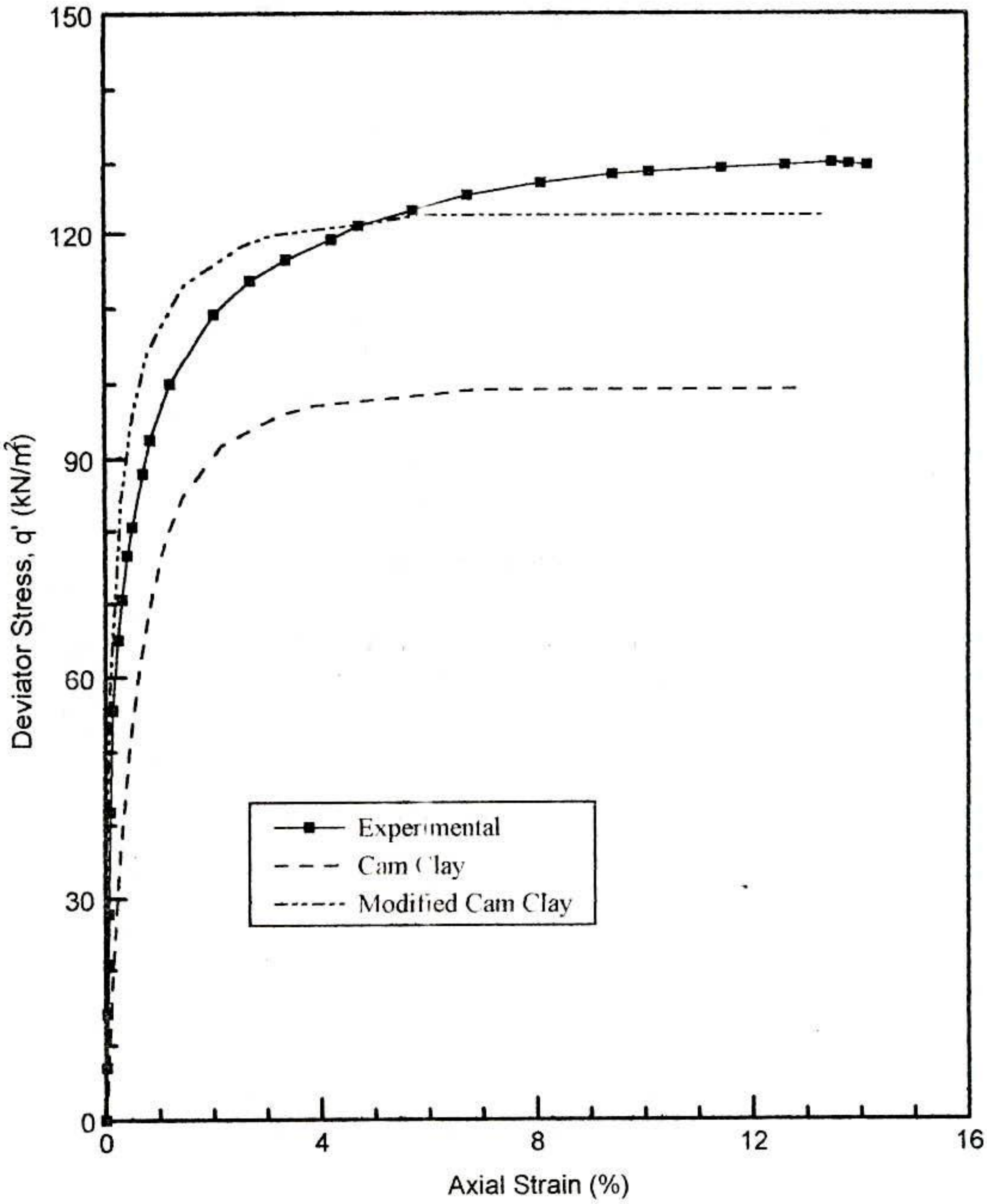


Fig. 6.21 Comparison of Observed & Predicted Stress-Strain Curves for Chandanaish soil

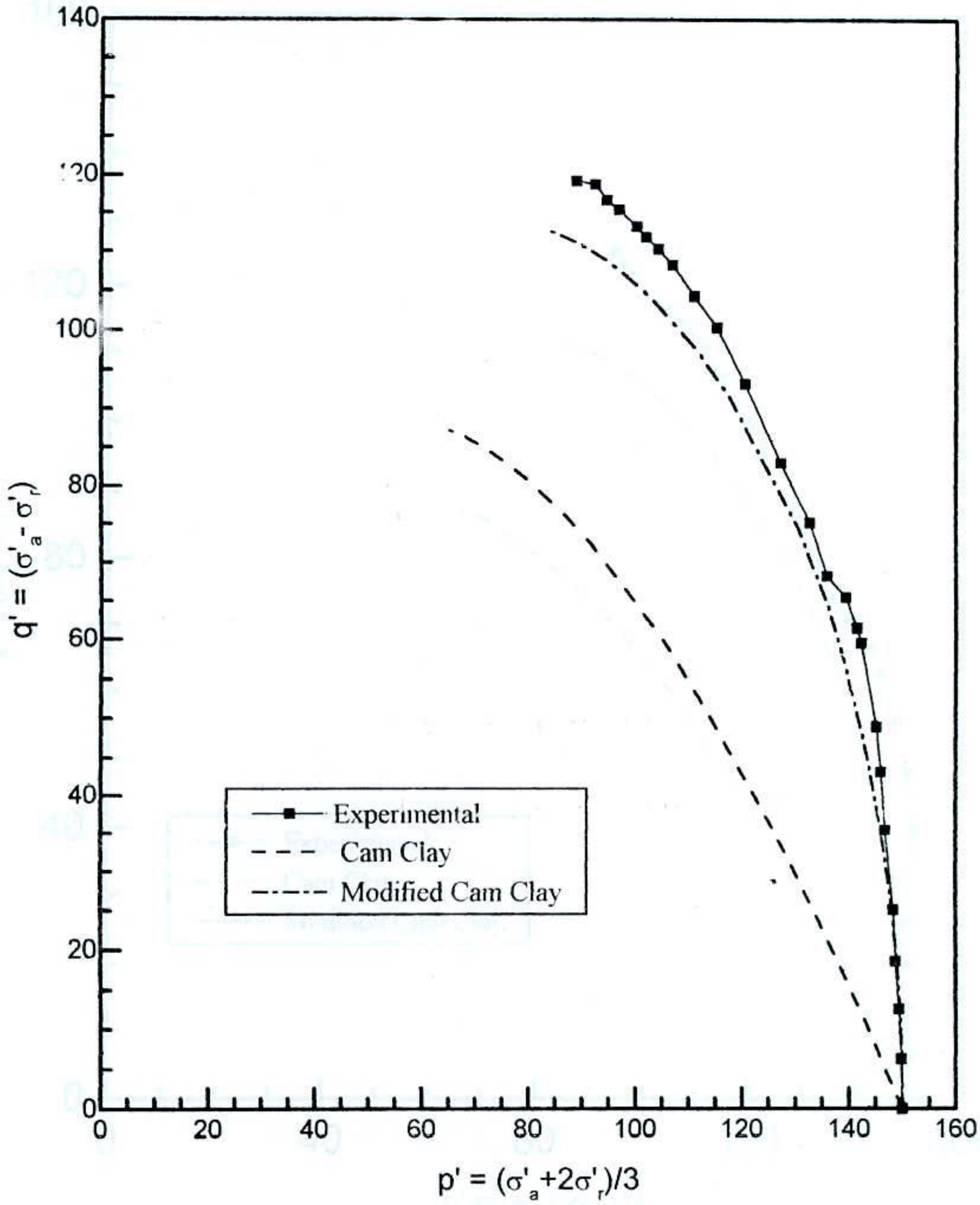


Fig.6.22 Comparison of Observed and predicted Effective Stress Paths for Banskhali Soil

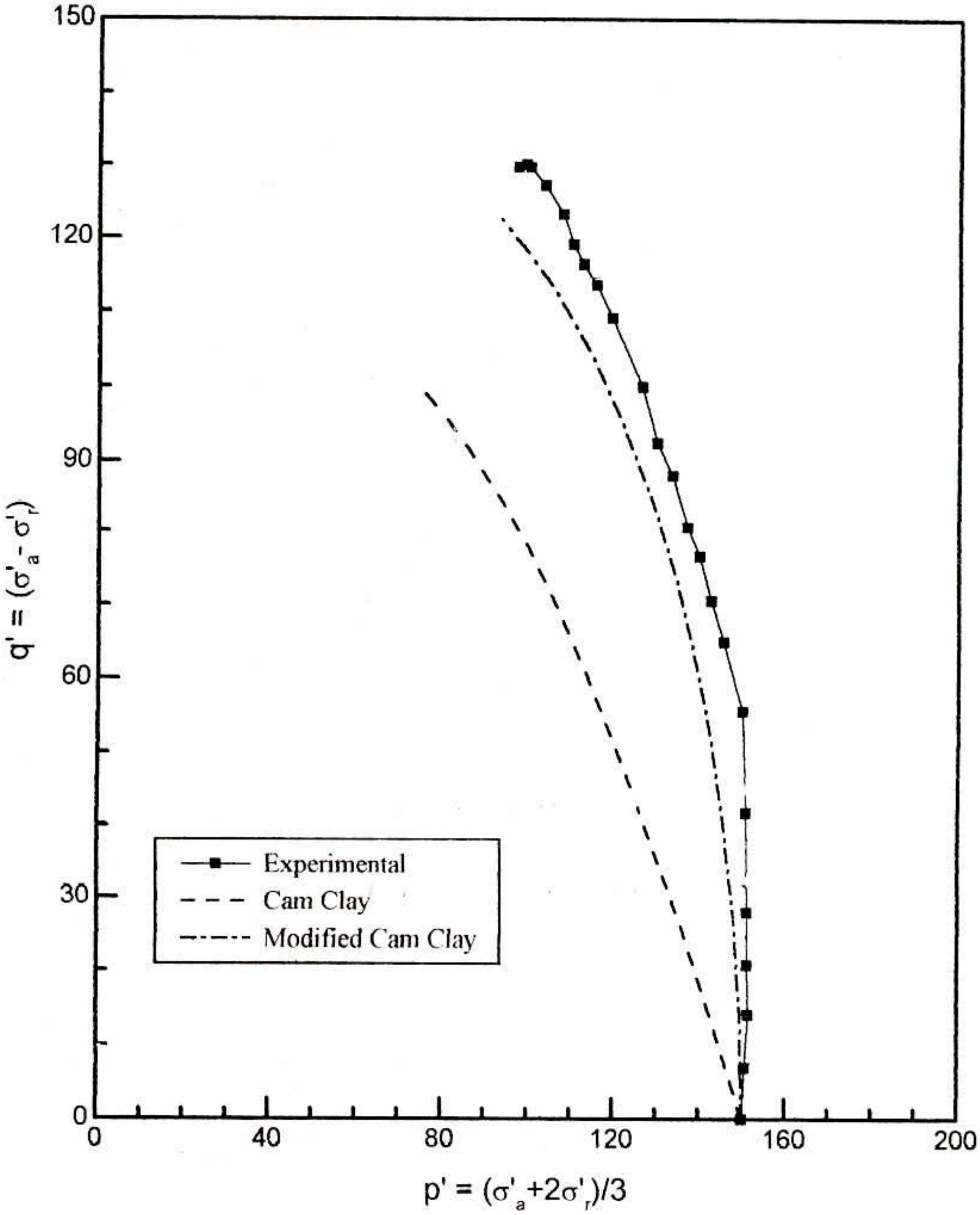


Fig. 6.24 Comparison of Observed and Predicted Effective Stress Paths for Chandanaish Soil

been observed that effective stress path from Modified Cam Clay Model is very close to the experimental curve than Cam Clay Model. Similar results were found by Balasubramaniam and Chaudhry (1978) for Bangkok clay. From the Fig. 6.22 to 6.24 it can be concluded that the experimental stress paths are in much better agreement with that predicted using Modified Cam Clay Model than that predicted using Cam Clay Model.

CHAPTER 7

BEHAVIOUR OF "IN SITU" AND "PERFECT" SAMPLES AND EFFECTS OF "PERFECT" SAMPLING DISTURBANCE

7.1 General

A basic idea of the present research is that the behaviour of "in situ" samples will be used as a reference to assess the effects of sampling disturbances and a "in situ" sample consolidated to the required stress state and tested without subjecting it to stress or mechanical disturbances can be regarded as an undisturbed sample. The findings of the laboratory investigations on "in situ" and "perfect" samples of three reconstituted soils collected from the coastal belt of Chittagong are reported in this chapter. The stress-strain-strength, stiffness and pore-pressure characteristics of the reconstituted coastal soil samples are presented and discussed in the following sections of this chapter. The behaviour of "perfect" samples might be examined quantitatively within a reference to the behaviour of "in situ" samples. In this chapter attention should also be directed towards the suitability of various reconsolidation techniques to minimize perfect sampling disturbance effects.

7.2 Behaviour of "In Situ" Samples

To determine the reference "in situ" behaviour of the reconstituted soil samples, undrained triaxial compression tests on "in situ" samples were performed. The undrained shear characteristics of the "in situ" samples are discussed in the following articles.

7.2.1 Effective Stress Paths

For the sample tested with pore pressure measurement, it was possible to construct the effective stress path during undrained condition. Fig. 7.1 shows the effective stress paths in p'- q' [$p' = (\sigma'_a + 2\sigma'_r)/3$, $q' = (\sigma'_a - \sigma'_r)$] space for undrained triaxial compression tests on three "in situ" samples BI, AI and CI from Banskhali, Anwara and Chandanaish, respectively. It can be seen from Fig. 7.1 that for the "in situ" samples, initially p′ increases with the increase in q′ and then it decreases with further increase in q′ and at the end of each test, again p′ increases with the increase in q′ as failure approaches. The effective stress paths for the "in situ" samples are typically similar to those of normally consolidated clays. Similar effective stress paths have also been found for other reconstituted normally consolidated "in situ" clay samples (Skempton and Sowa, 1963; Ladd and Lambe, 1963; Atkinson and Kubba, 1981; Hight, Gens and Jardine, 1985; Siddique and Farooq, 1996; Siddique and Sarker, 1998; Rahman, 2000).

7.2.2 Stress-Strain and Stiffness Properties

A comparison of deviator stress (q′) versus axial strain (ε) plots for the "in situ" samples of the three coastal soils is presented in Fig. 7.2. The following features can be noted from the stress-strain curves as shown in Fig. 7.2.

- The peak undrained strength is mobilized at relatively large axial strain.
- The strength mobilized at ultimate strain is slightly lower than that mobilized at peak. The soils, therefore, do not show any undrained brittleness or strain hardening behaviour when sheared in compression.
- Axial strain at peak deviator stress is increased with the increase of plasticity.
- The stress-strain relationships are nonlinear.

The undrained shear strength (s_u), axial strain at peak deviator stress (ε_p), initial tangent modulus (E_i), secant modulus at half the peak deviator stress (E_{50}) and secant stiffness (E_u) at small strain levels (upto 1%), have been determined from the stress-strain curves of Fig. 7.2 for the three soils. The initial tangent modullii (E_i) have been determined from the following hyperbolic expression for stress-strain curves proposed by Kondner and Zelasko (1963).

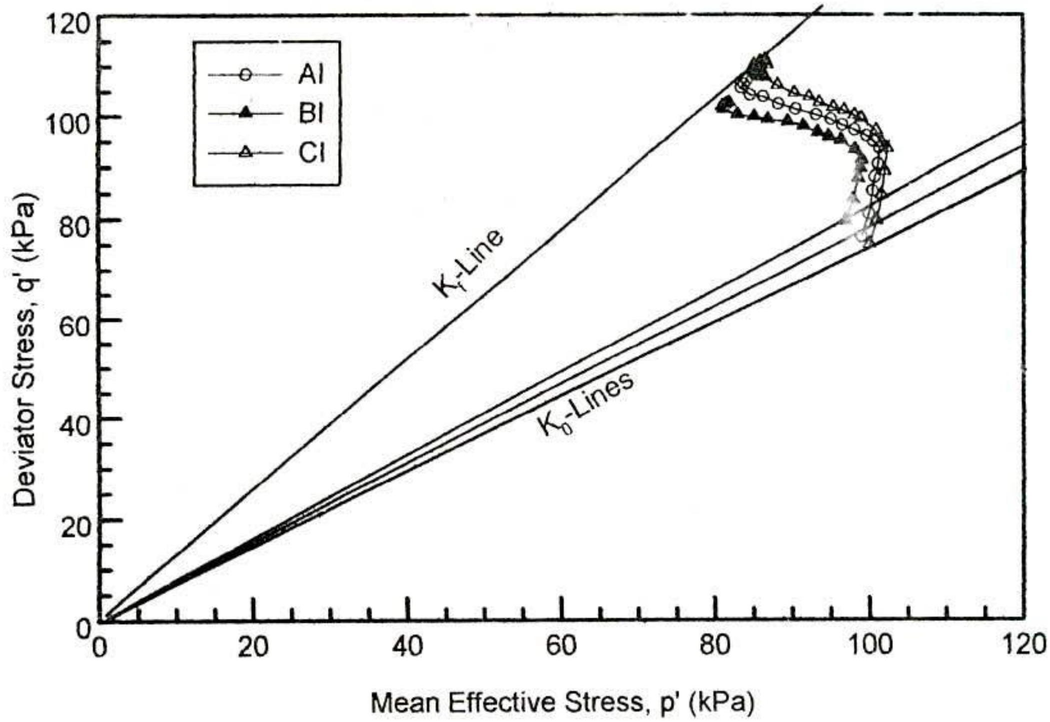


Fig. 7.1 Comparison of Effective Stress Paths of "In Situ" Samples for the Three Soils

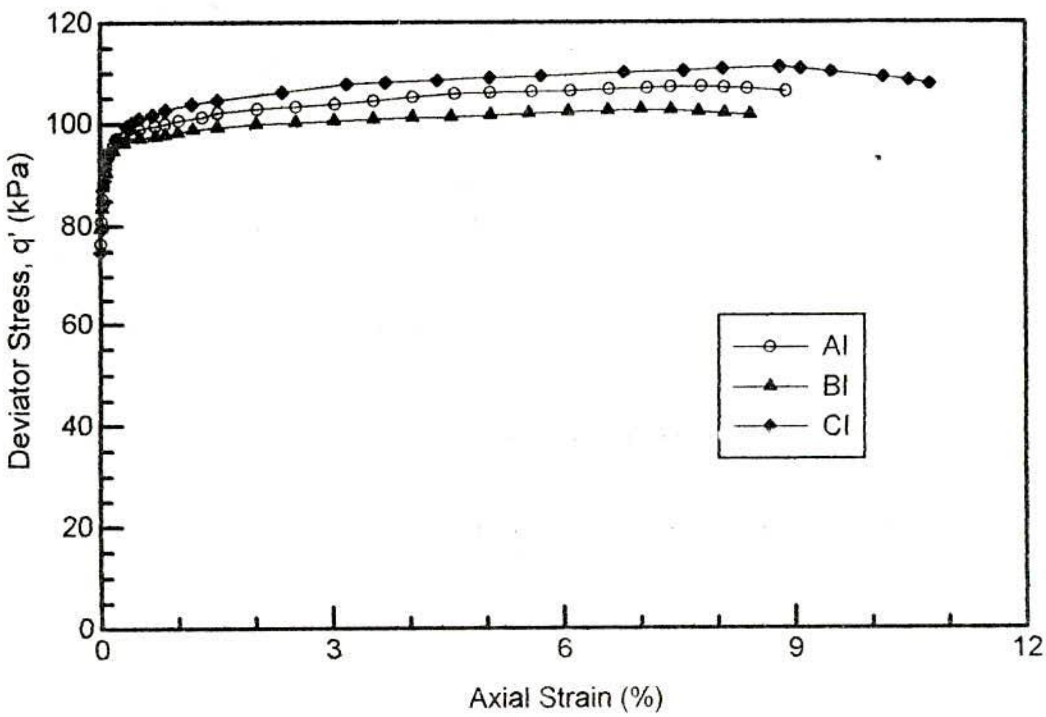


Fig. 7.2 Deviator Stress Versus Axial Strain Plots for "In Situ" Samples

$$(\sigma'_a - \sigma'_r) = \varepsilon / (a + b\,\varepsilon) \tag{7.1}$$

where, $(\sigma'_a - \sigma'_r)$ is the deviator stress in the triaxial test, ε is the axial strain. a and b are constants for one stress-strain curve, but dependent on the soil type, relative density, and confining pressure. Transformation of the hyperbolic expression into

$$\varepsilon / (\sigma'_a - \sigma'_r) = a + b\,\varepsilon \tag{7.2}$$

as shown in Fig. 7.3 for Anwara sample, facilitates the determination of the values of a and b. From Fig. 7.3, intercept with the ordinate, a, is the reciprocal value of the initial tangent modulus and slope b is the reciprocal value of the ultimate deviator stress, which the stress-strain curve approaches at large values of strain. Undrained shear strength is equal to half of the peak deviator stress. The predicted values of s_u, ε_p, E_i and E_{50} are listed in Table 7.1. It can be seen from Table 7.1 that undrained strength for the coastal soils varied between 51.0 kN/m^2 and 55.5 kN/m^2, being maximum for the most plastic sample CI (PI = 20) and minimum for the least plastic sample BI (PI = 10). The values of ε_p, E_i and E_{50} vary from 7% to 8.8%, 24570 kN/m^2 to 27600 kN/m^2 and 18720 kN/m^2 to 22050 kN/m^2, respectively and it is observed that the values of them increase with the increase of plasticity. In Fig. 7.4, secant stiffness (E_u) at small strain levels (up to 1%) have been plotted for the three "in situ" samples BI, AI and CI. It can be seen from Fig. 7.4 that, in general, secant stiffness decreases with increasing levels of strains. Similar trend has also been reported for "in situ" normally consolidated reconstituted clays (Hight et al., 1985; Hajj, 1990; Hopper, 1992; Siddique et al., 1999; Rahman, 2000).

Table 7.1 Undrained Shear Characteristics of "In Situ" Samples

Sample Designation	s_u (kN/m^2)	ε_p (%)	E_i (kN/m^2)	E_{50} (kN/m^2)	A_p	E_u at 1%
BI	51.0	7.0	24570	18720	0.76	5400
AI	53.6	7.8	26280	19845	0.74	6125
CI	55.5	8.8	27600	22050	0.71	6590

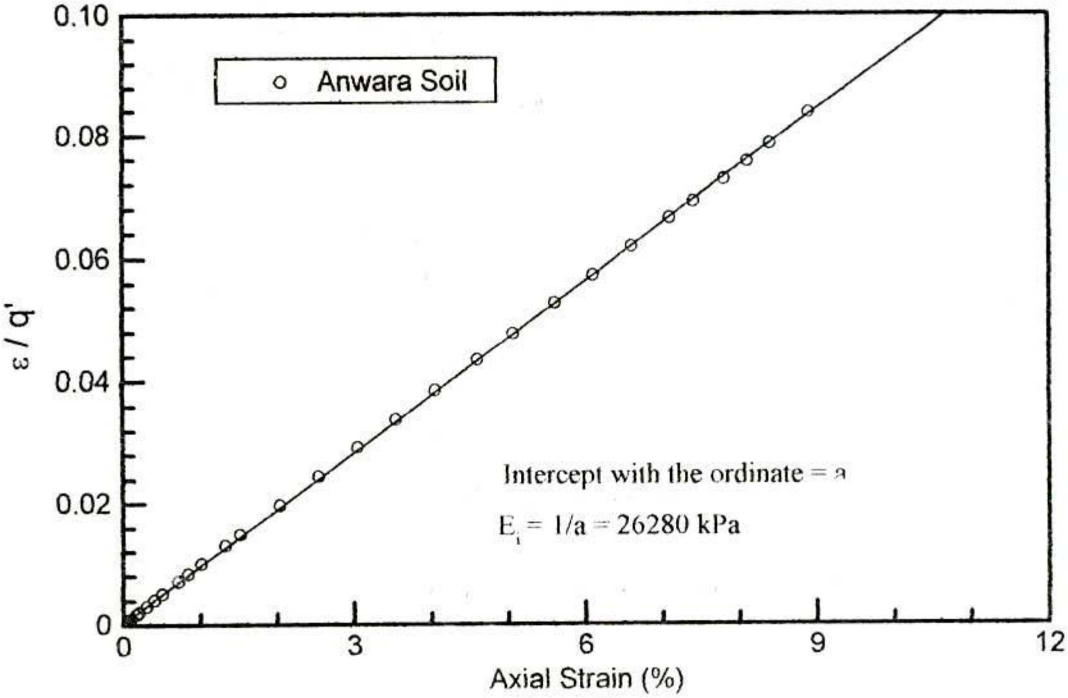


Fig. 7.3 Typical Transformed Hyperbolic Representation of Stress-Strain Relationship for "In Situ" Sample

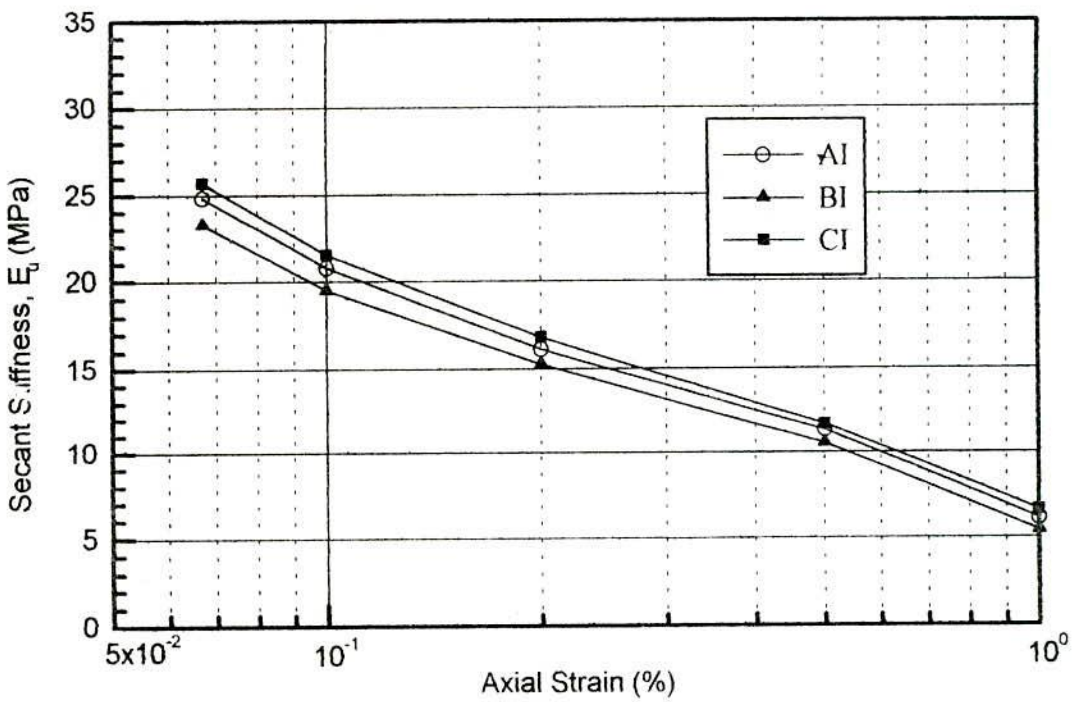


Fig. 7.4 Secant Stiffness vs. Axial Strain Plots for "In Situ" Samples of the Three Soils

7.2.3 Pore Pressure Responses

The response of pore pressure was observed throughout the whole shearing stage for each "in situ" sample. Fig. 7.5 shows the variation in Skempton's pore pressure parameter, A with axial strain for the three "in situ" samples. It can be seen from Fig. 7.5 that during undrained shearing, the pore pressure parameter A increases rapidly with the increase in deviator stress for strain levels of about 2.0% and then increases slowly. After a certain strain level the pore pressure parameter decreases slowly before peak deviator stress. Skempton's pore pressure parameter A at peak deviator stress (A_p) of three coastal soils are also shown in Table 7.1. It can be seen from Table 7.1 that the values of A_p decrease with the increase of plasticity.

7.3 Behaviour of "Perfect" Samples

The purpose of this article is to examine the fundamental behaviour of "perfect" samples of the three reconstituted coastal soils and to assess the effects of stress relief or "perfect" sampling disturbance. The effects of "perfect" sampling might be examined qualitatively and quantitatively within the reference to the behaviour of "in situ" samples. To study the undrained shear behaviour of "perfect" samples, "perfect" samples have been modeled on three coastal soils in the laboratory by undrained release of the total stresses from the in situ stress state. The "perfect" samples were sheared in compression under undrained condition up to failure. The effects of "perfect" sampling on undrained shear characteristics are discussed in the following sections.

7.3.1 Changes in Effective Stress Paths and Mean Effective Stress

Fig. 7.6 shows the effective stress paths in p'-q' [$p' = (\sigma'_a+2\sigma'_r)/3$, $q' = (\sigma'_a - \sigma'_r)$] space for undrained triaxial compression tests on "perfect" samples for the three coastal soils (which simulated total stress release). The effective stress paths of the "in situ" samples are also shown in Fig. 7.6 for comparison. It can be seen from Fig. 7.6 that for the "in situ" samples, initially mean effective stress (p') slightly increases with the increase in deviator stress (q') and then it decreases with further increase in q' and as failure

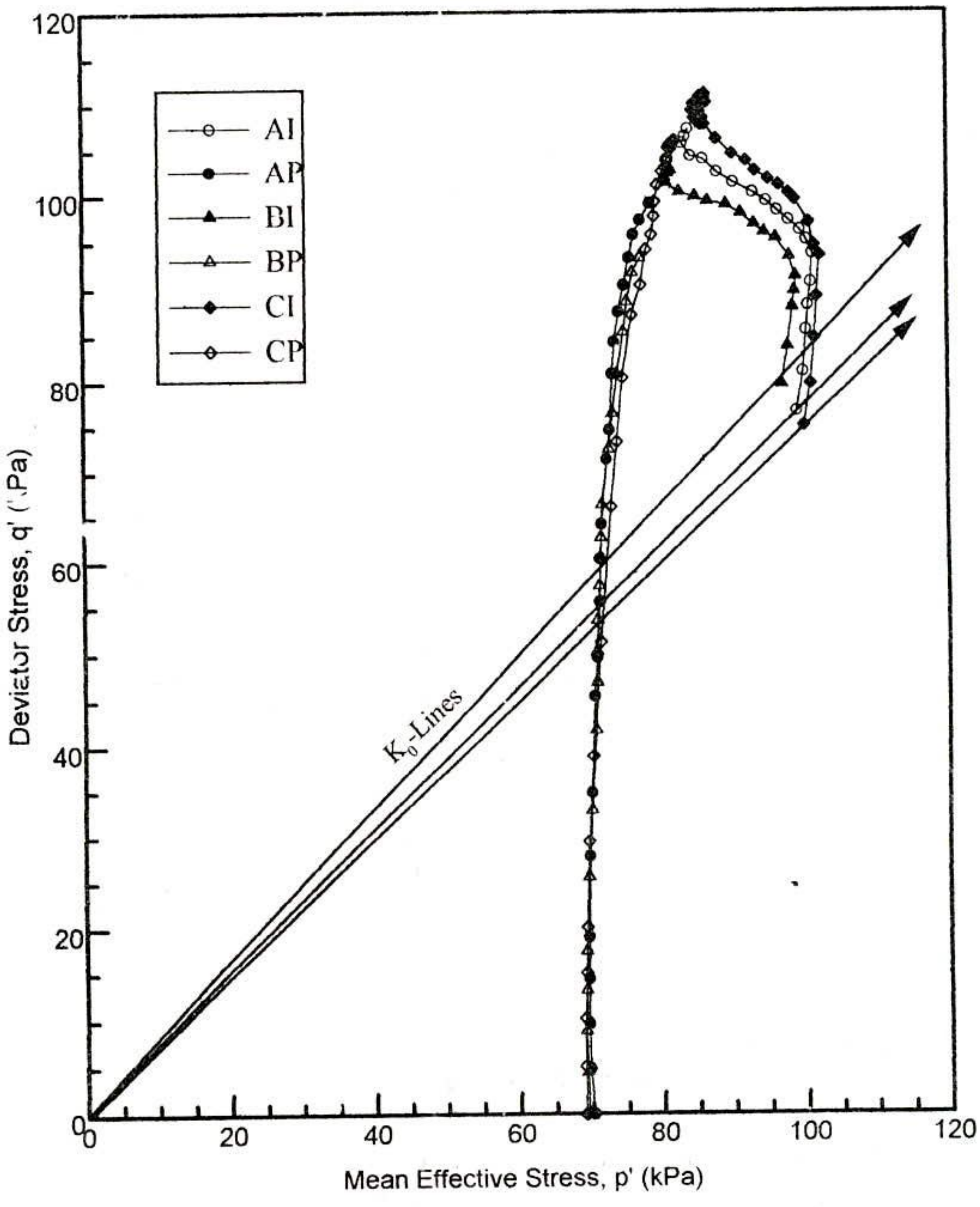


Fig. 7.6 Comparison of Effective Stress Paths of "In Situ" and "Perfect" Samples for the Three Reconstituted Soils

approaches p' increases with the incr·ase in q'. For the "perfect" samples, however, p' remains almost constant with the increase in q' during the most part of undrained shearing and as failure approaches p', however, slightly increases with the increase in q'. "Perfect" sampling, therefore, produced appreciably different effective stress paths. The effective stress paths for the "in situ" samples are typically similar to those of normally consolidated clays. However, although the "perfect" samples have been prepared from the normally consolidated "in situ" samples they adopt stress paths similar to those for overconsolidated samples. Significant difference in the effective stress paths between the "in situ" and "perfect" samples has also been reported for the regional clays of Bangladesh (Siddique and Farooq, 1996; Siddique and Sarker, 1998; Rahman, 2000). Similar results have also been reported by several investigators (Skempton and Sowa, 1963; Ladd and Lambe, 1963; Atkinson and Kubba, 1981; Hight et al., 1985).

Another significant effect of "perfect" sampling is the reduction of mean effective stress, p' which is also evident from Fig. 7.6. Due to "perfect" sampling the mean effective stresses of the "in situ" samples of Banskhali, Anwara and Chandanaish reduced by 34.5%, 32.7% and 31% respectively. It is also evident that the reductions in p' due to "perfect" sampling disturbance increases with decreasing plasticity of the soils.

7.3.2 Changes in Stress-Strain and Stiffness Properties

A comparison of deviator stress (q') versus axial strain (ε) plots for the "in situ" and "perfect" samples is presented in Fig. 7.7. From the stress-strain data the undrained strength (s_u), axial strain at peak deviator stress (ε_p), initial tangent modulus (E_i), secant modulus at half the peak deviator stress (E_{50}) and secant stiffness (E_u) at small strain levels have been determined for both the "in situ" and "perfect" samples. A comparison of the undrained soil parameters of the "in situ" and "perfect" samples is presented in Table 7.2. The test results are not corrected with respect to water content. It can be seen from Table 7.2 that because of the relief of total stress, the values of s_u of the "perfect" samples reduced. Reduction of strength might be resulted from less mean effective stress in "perfect" samples due to disturbance caused by stress relief. The values of s_u of the "perfect" samples BP, AP and CP reduced by about 11.2%, 7.6% and 5.4%,

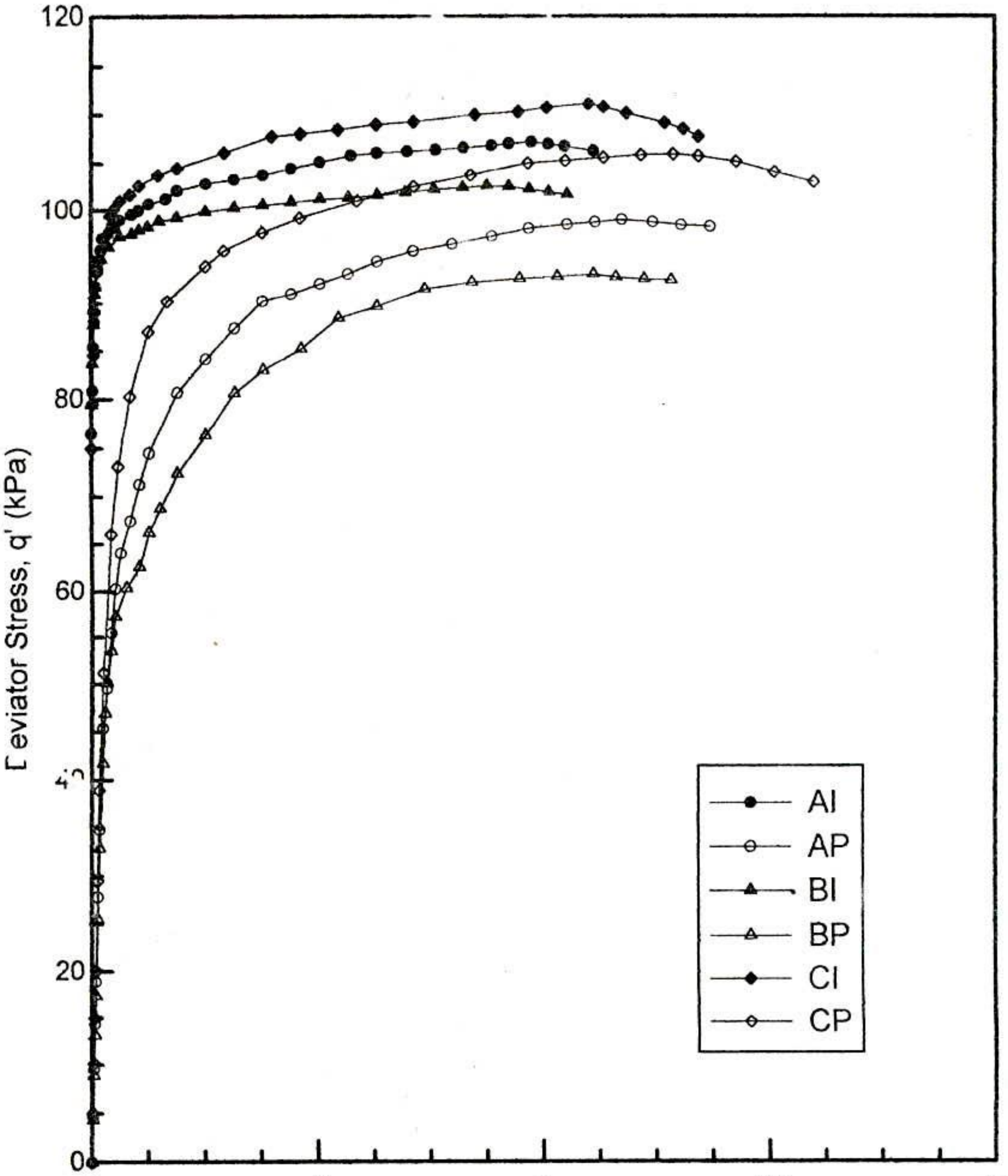


Fig. 7.7 Deviator Stress vs. Axial Strain Plots for "In Situ" and "Perfect" Samples

Table 7.2 Comparison of Undrained Shear Properties of "In Situ" and "Perfect" Samples of the Three Coastal Soils

Undrained Shear Parameters	Samples					
	BI	BP	AI	AP	CI	CP
s_u (kN/m^2)	51.0	45.3	53.55	49.5	55.5	52.5
ε_p (%)	7.0	8.9	7.8	9.4	8.8	10.3
E_i (kN/m^2)	24570	22410	26280	24406	27600	26120
E_{50} (kN/m^2)	18720	17162	19845	18486	22050	20844
A_p	0.76	0.41	0.74	0.37	0.71	0.31
A_u	-	0.132	-	0.125	-	0.14

respectively. Values of axial strain at peak deviator stress (ε_p), however, increased due to disturbance caused by stress relief. Values of ε_p increased by 27.1%, 20.5% and 17% for the "perfect" samples BP, AP and CP, respectively. Decrease in undrained strength due to stress relief has been found for other normally consolidated clays by a number of researchers (Skempton and Sowa, 1963; Noorany and Seed, 1965; Davis and Poulos, 1967; Ladd and Varallyay, 1965; Kirkpatrick and Khan, 1984; Hight et al, 1985; Siddique and Sarker, 1998; Siddique and Farooq (1996); Rahman (2000). Ladd and Varallyay (1965), Kirkpatrick and Khan (1984), Siddique and Sarker (1998), Siddique and Farooq (1996), Rahman (2000) also observed considerable increase in ε_p for normally consolidated Boston Blue clay (LL = 33, PI = 15), Kaolin (PI = 30) and Illite (PI = 40), reconstituted soft Dhaka clay (LL = 45, PI = 23), two reconstituted Chittagong coastal soils (LL = 44 and PI = 18; LL = 57 and PI = 33) and reconstituted firm Dhaka clay (LL = 46, PI = 26), respectively. From present study, it is also evident that the degree of reduction in undrained strength and increase in axial strain at peak deviator stress increases with decreasing plasticity of the soils. This might be due to increase of disturbance with the decrease of plasticity. Kirkpatrick and Khan (1984) and, Siddique and Farooq (1996) also found larger reduction in undrained strength in less plastic soils than in more plastic soils due to "perfect" sampling. A noticeable behaviour observed in these coastal soils, is that the values of ε_p for these samples (both "in situ" and "perfect") are considerably large. Similar results were also reported by Siddique and Farooq (1996) for other coastal soils of Bangladesh.

Table 7.2 also shows that because of disturbance due to "perfect' sampling, both the initial tangent modulus (E_i) and secant modulus at half the peak deviator stress (E_{50}) decreased. Compared with the "in situ" samples, the values of E_i of the "perfect" samples from Banskhali, Anwara and Chandanaish decreased by 8.8%, 7.13% and 5.36%, respectively while the values of E_{50} of the respective samples decreased by 8.32%, 6.85% and 5.47%, respectively. These results contrast with those reported by Siddique and Sarker (1998), Rahman (2000), Siddique and Farooq (1996), for normally consolidated reconstituted samples of soft Dhaka clay (LL = 45, PI = 23), firm Dhaka clay (LL = 47, PI = 26) and two reconstituted coastal soils (LL = 44 and PI = 18; LL =

57 and PI = 33) of Bangladesh. Atkinson and Kubba (1981), and Kirkpatrick and Khan (1984), however, found reduction in stiffness because of "perfect" sampling disturbance.

Plottings of secant stiffnesses (E_u) at small strain levels (up to 1%) for "in situ" and "perfect" samples are shown in Fig. 7.8. It can be seen from Fig. 7.8 that, in general, secant stiffnesses of the "in situ" and "perfect" samples reduced with the increase in axial strain. It can also be seen from Fig. 7.8 that in each soil, secant stiffnesses (at all strain levels) of the "perfect" sample are lower than those for the "in situ" sample. For comparison, the values of secant stiffnesses (at 0.1% axial strain) of the "perfect" samples from Banskhali, Anwara and Chandanaish decreased by about 11.63%, 11.14% and 10.93%, respectively.

7.3.3 Changes in Pore Pressure Responses

Fig. 7.9 shows the comparison of Skempton's pore pressure parameter A with axial strain between the "in situ" and "perfect" samples of reconstituted soils. It can be observed from Fig. 7.9 that, compared with the "in situ" samples the values of Skempton's pore pressure parameter A for the "perfect" samples are considerably less. From Fig. 7.9, it appears that for the "perfect" samples at low strains the pore pressure parameter A increases rapidly with the increase in deviator stress. The pore pressure, however, typically start to decrease before the peak deviator stress has reached resulting in considerably lower values of Skempton's pore pressure parameter A at peak deviator stress (A_p) than those for the "in situ" sample. The "perfect" samples despite being prepared from normally consolidated "in situ" sample thus showed a pore pressure response which is more typical of overconsolidated clay. The values of Skempton's pore pressure parameter A at peak deviator stress, A_p were also determined and have been shown in Table 7.2. It can be seen from Table 7.2 that for each soil, the values of A_p of the "perfect" sample are considerably less than that for the "in situ" sample. The values of A_p reduced by 46.1%, 50% and 56.3% for "perfect" samples BP, AP and CP, respectively because of disturbance due to stress relief. These results also

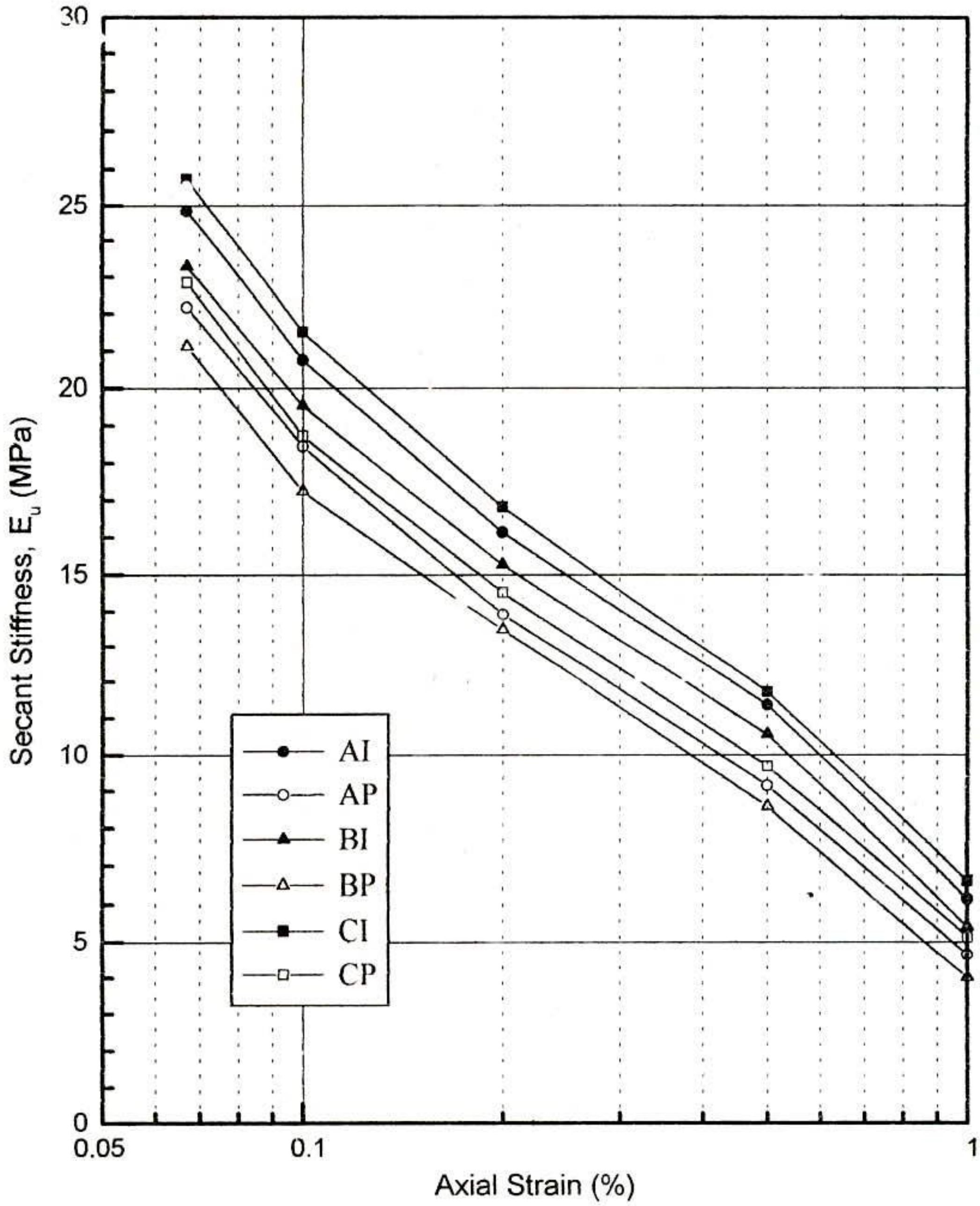


Fig. 7.8 Secant Stiffness vs. Axial Strain Plots on "In Situ" and "Perfect" Samples for the Three Soils

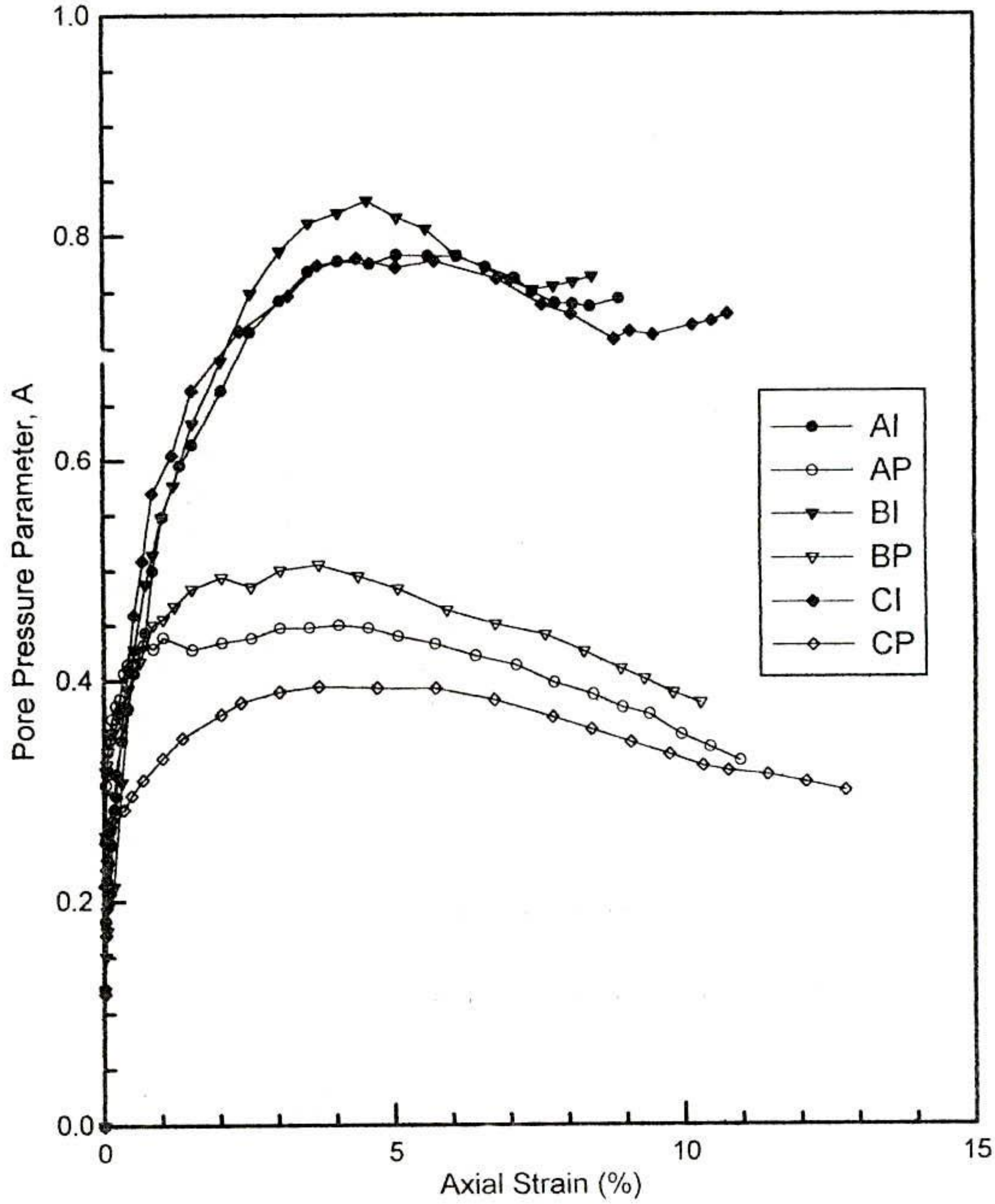


Fig. 7.9 Comparison of Pore Pressure Parameter, A vs. Axial Strain Plots for "In Situ" and "Perfect" Samples

indicate that the reduction in the values of A_p increased with increasing plasticity of the soils. Similar findings were also reported by Siddique and Farooq (1996) for other coastal soils. Significant reduction in A_p due to stress relief has also been reported by a number of investigators (Skempton and Sowa, 1963; Seed et al., 1964; Noorany and Seed, 1965; Ladd and Varallyay, 1965; Kirkpatrick and Khan, 1984; Siddique and Sarker, 1998; Siddique and Farooq, 1996; Rahman, 2000). Much of these differences can be attributed to the nonlinearity of the relationship between pore pressure change and deviator stress change, but they are also caused, in part, by the nonreversibility of pore pressure change caused by the release of the deviator stress from an "in situ" sample (Noorany and Seed, 1965).

The values of Skempton's pore pressure parameter for the undrained release of shear stress, A_u and the isotropic effective stress, σ'_{ps} of the "perfect" sample have also been determined using Equations (2.5b) and (2.5a), respectively. The values of A_u for the "perfect" samples from Banskhali, Anwara and Chandanaish are 0.132, 0.125 and 0.14, respectively and the respective values of σ'_{ps} of the "perfect" samples are 81.0 kN/m^2, 83.06 kN/m^2 and 85.5 kN/m^2, respectively. The ratio of σ'_{ps} / σ'_{vc} are 0.54, 0.554 and 0.57 for BP, AP and CP, respectively. The values of A_u and σ'_{ps} / σ'_{vc} for a number of clays investigated have already been presented in Table 2.8. Rahman (2000) reported that the values of σ'_{ps} and A_u for firm Dhaka clay are 88.7 kPa and 0.182, respectively. Rahman (2000) also reported that the value of σ'_{ps} / σ'_{vc} for Dhaka clay is 0.59. The values of A_u and σ'_{ps} / σ'_{vc} of BP, AP and CP also compare reasonably with those reported by a number of investigators for different clays (Ladd and Lambe, 1963; Skempton and Sowa, 1963; Seed et al., 1964; Noorany and Seed, 1965; Nelson et al. 1971; Siddique and Farooq, 1996; Siddique and Sarker, 1998).

From the aforementioned effects of "perfect" sampling disturbances in unaged Chittagong coastal soils, it is evident that substantial changes in the effective stress path and undrained soil parameters between the "in situ" and "perfect" samples occurred. Therefore, appropriate technique should be adopted to minimize the "perfect" sampling effects in these coastal soils. Although the effect of reconsolidation of "perfect" samples in order to recover the in situ behaviour for these unaged samples was not investigated, this suggested the need to minimize the "perfect" sampling disturbance by reconsolidating before being sheared. The effects of reconsolidation of "perfect" samples to different stress levels are discussed latter.

7.4 Characteristics of Reconsolidated "Perfect" Sample

The undrained stress-strain-strength, stiffness and pore pressure response of "perfect" samples due to reconsolidation have been investigated and presented in the following subsections. All the "perfect" samples were reconsolidated isotropically and anisotropically before being sheared in compression in order to select the appropriate reconsolidation technique to recover the "in situ" behaviour and properties. The reconsolidation techniques mentioned in Article 4.7 were used for the " perfect" samples:

7.4.1 Normalized Effective Stress Paths

Figs. 7.10 to 7.12 show the comparison of normalized effective stress paths of "in situ" and reconsolidated "perfect" samples from Banskhali, Anwara and Chandanaish respectively. Comparing the effective stress paths of the three reconsolidated "perfect" samples with the respective three "in situ" samples in Figs. 7.10 to 7.12, it can be seen that for each sample the effective stress path of the "perfect" sample reconsolidated under K_0-condition to vertical effective stress equal to in situ vertical effective stress, σ'_{vc}, i.e., Bjerrum (1973) procedure, compares most favourably with the "in situ" sample.

7.4.2 Stress-Strain and Stiffness Properties

The normalized deviator stress, q'/σ'_{vo} versus axial strain plots for the reconsolidated "perfect" samples from Banskhali and Anwara are presented in Figs 7.13 and 7.14 respectively. In Figs. 7.13 and 7.14, the corresponding plots for the "in situ" samples are also shown for comparison with the reconsolidated samples. It can be seen from Figs. 7.13 and 7.14 that deviator stress versus axial strain plots of the "perfect" samples reconsolidated using Bjerrum (1973) procedure (i.e., K_0- consolidation to vertical

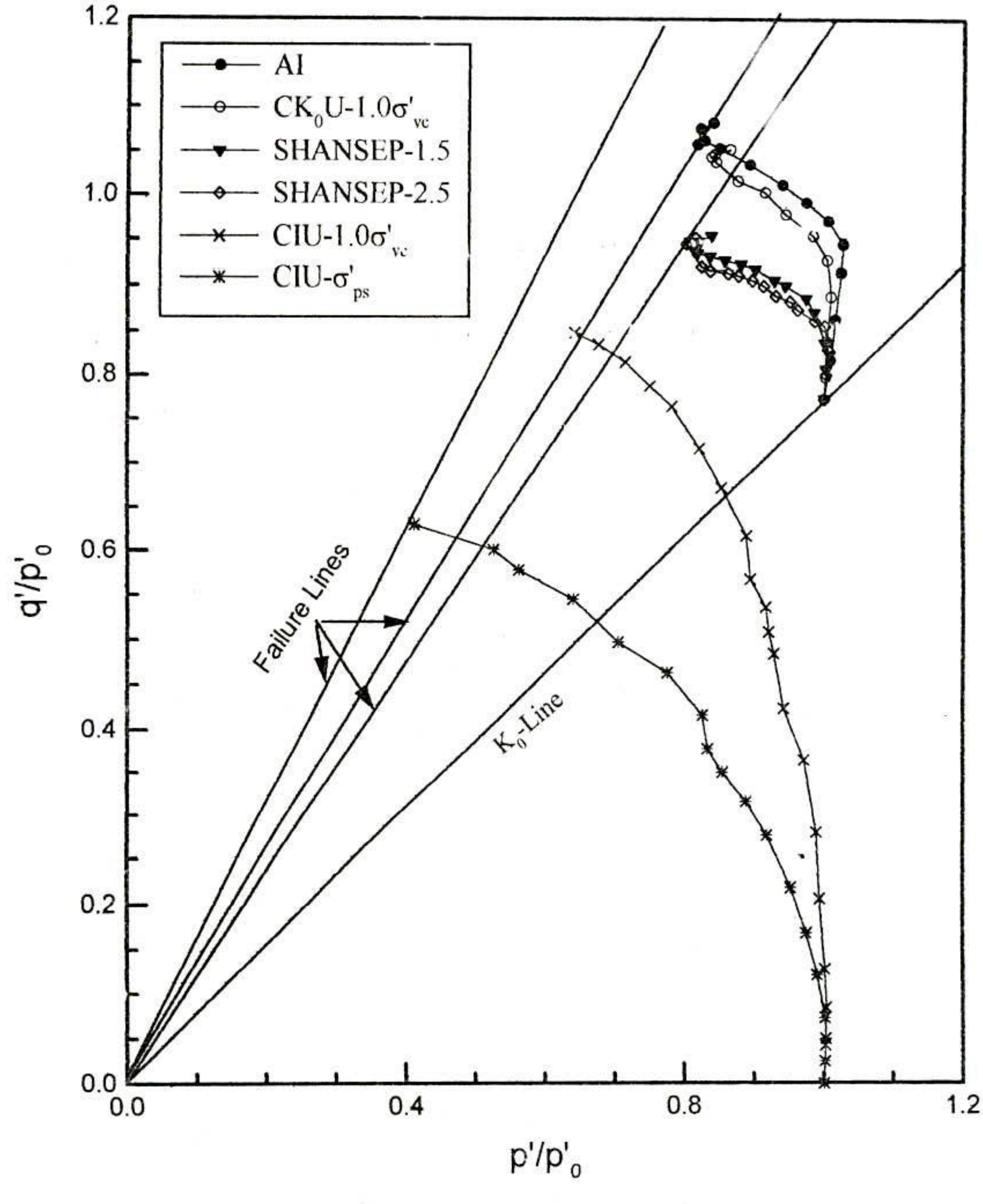


Fig. 7.10 Comparison of Normalized Effective Stress Paths of "In Situ" and Reconsolidated "Perfect" Samples from Banskhali

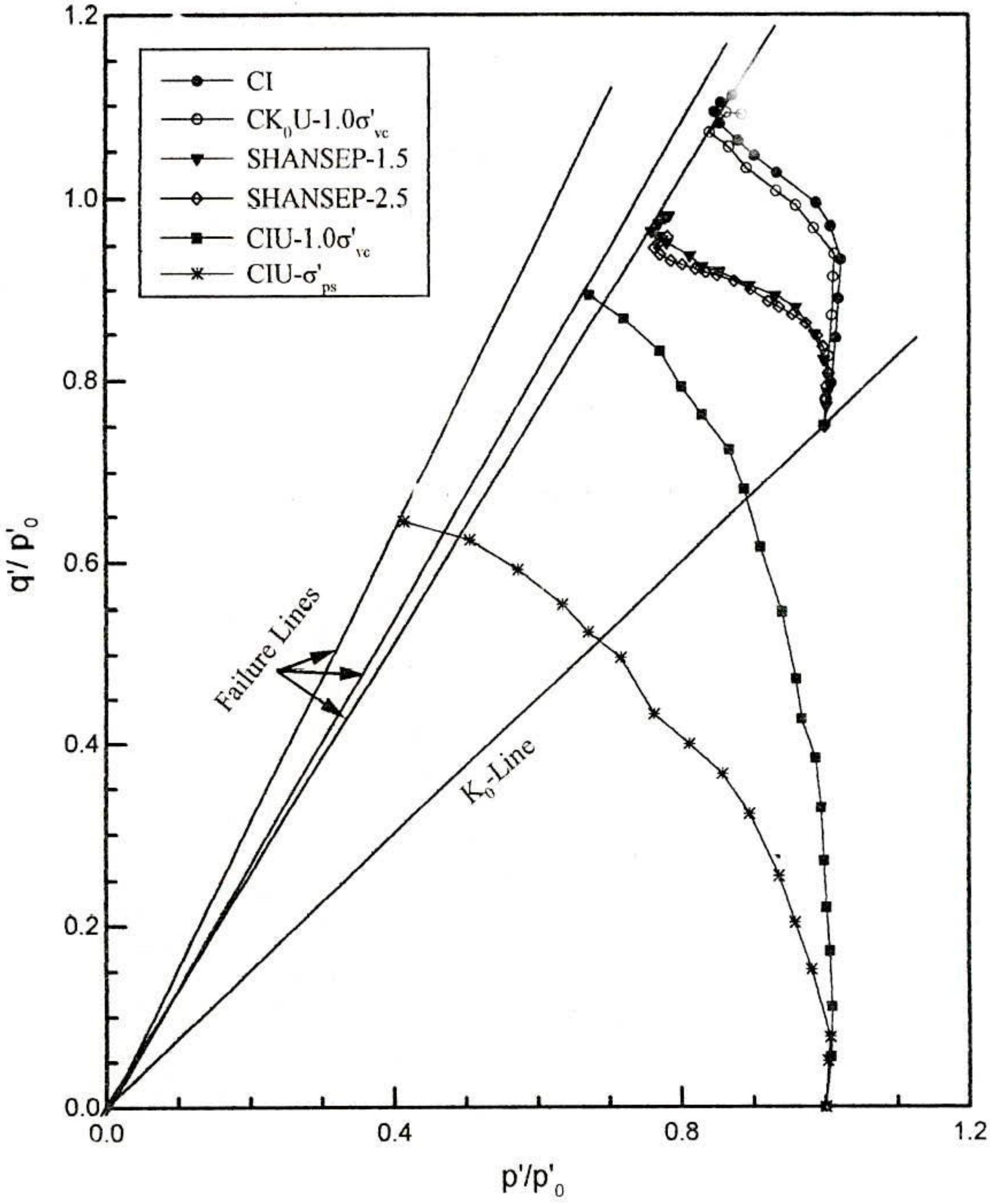


Fig. 7.11 Comparison Normalized Effective Stress Paths of "In Situ" and Reconsolidated "Perfect" Samples from Anwara

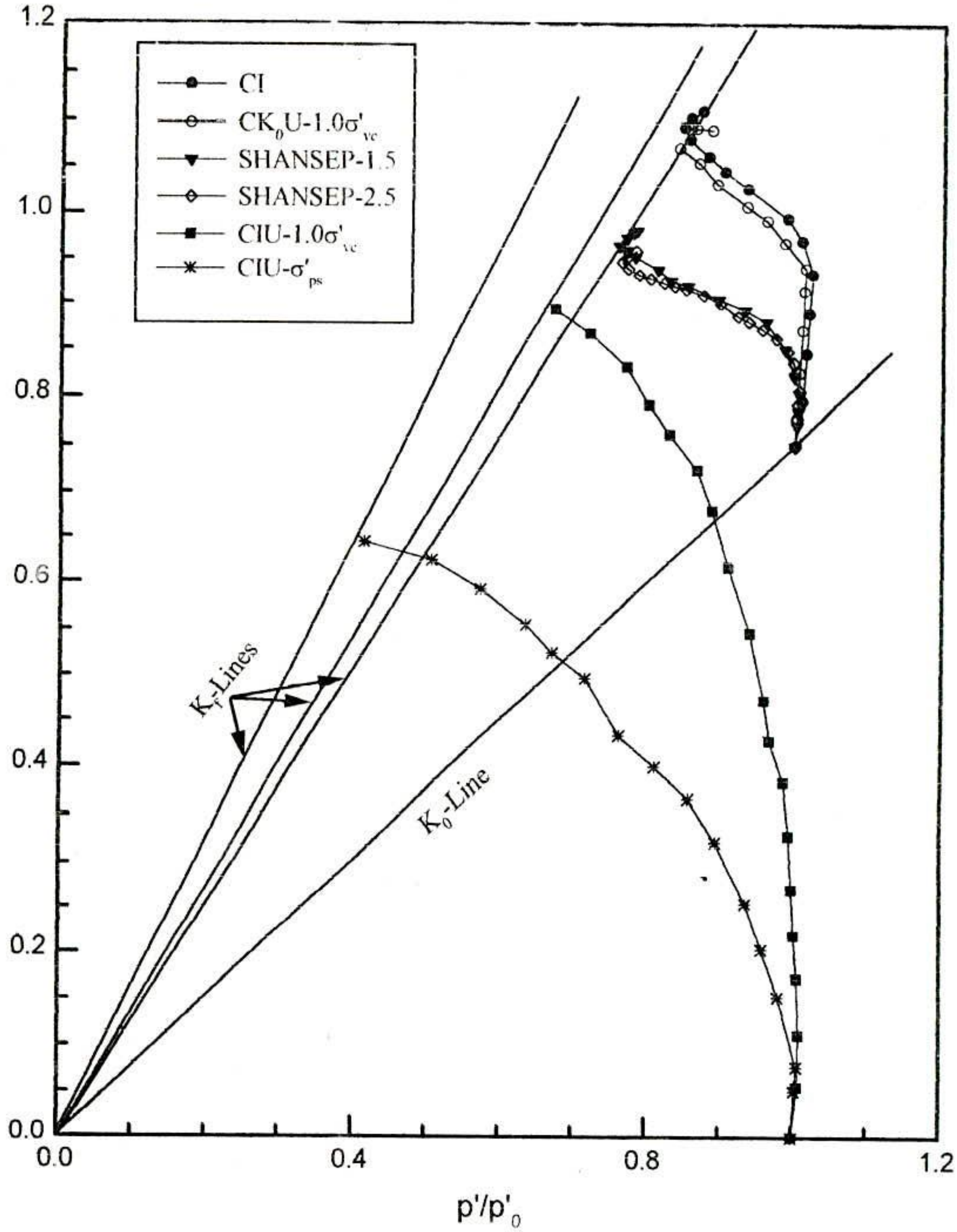


Fig. 7.12 Comparison Normalized Effective Stress Paths of "In Situ" and Reconsolidated "Perfect" Samples from Chandanaish

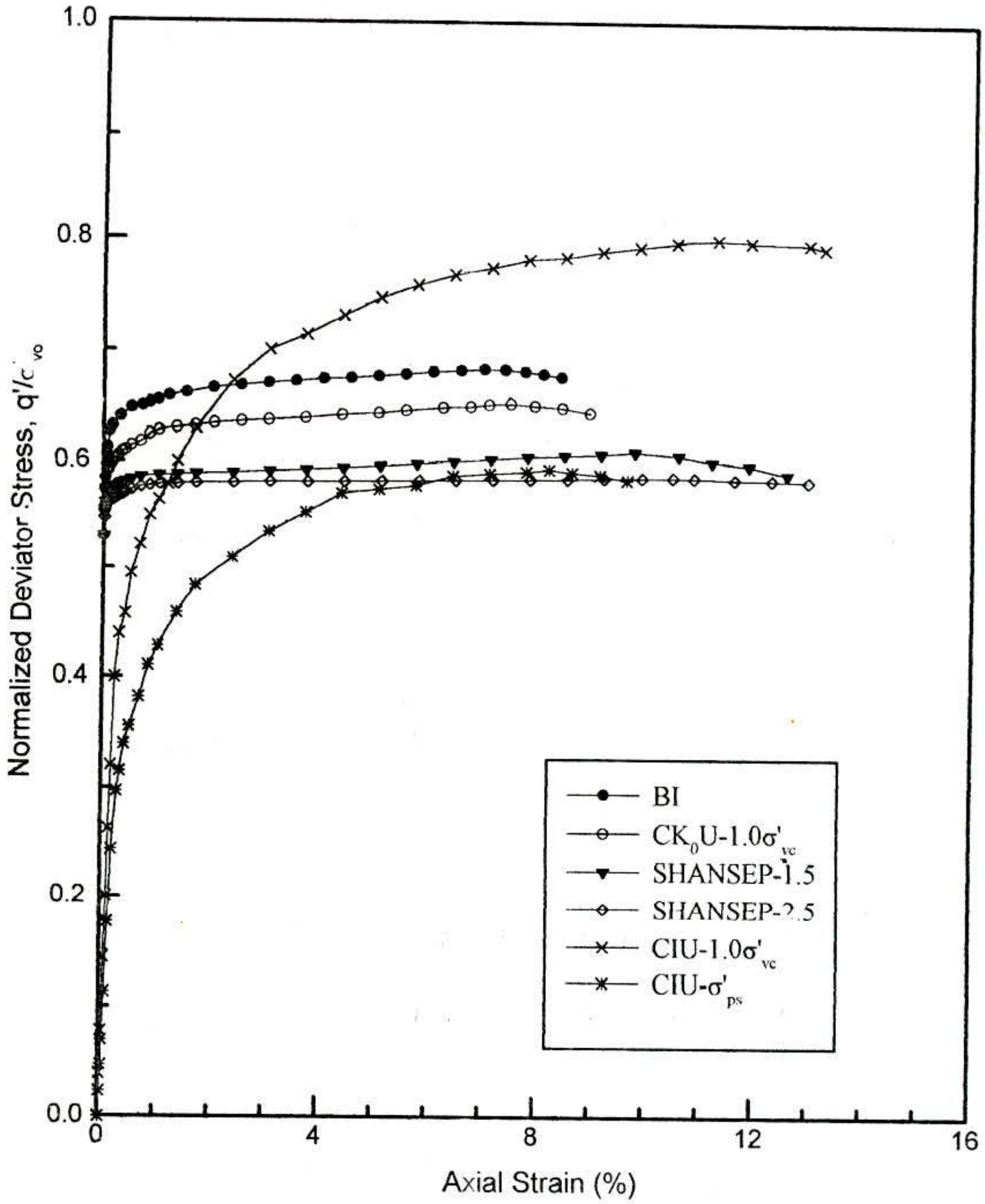


Fig. 7.13 Deviator Stress vs. Axial Strain Plots for "In Situ" and Reconsolidated "Perfect" Samples from Banskhali

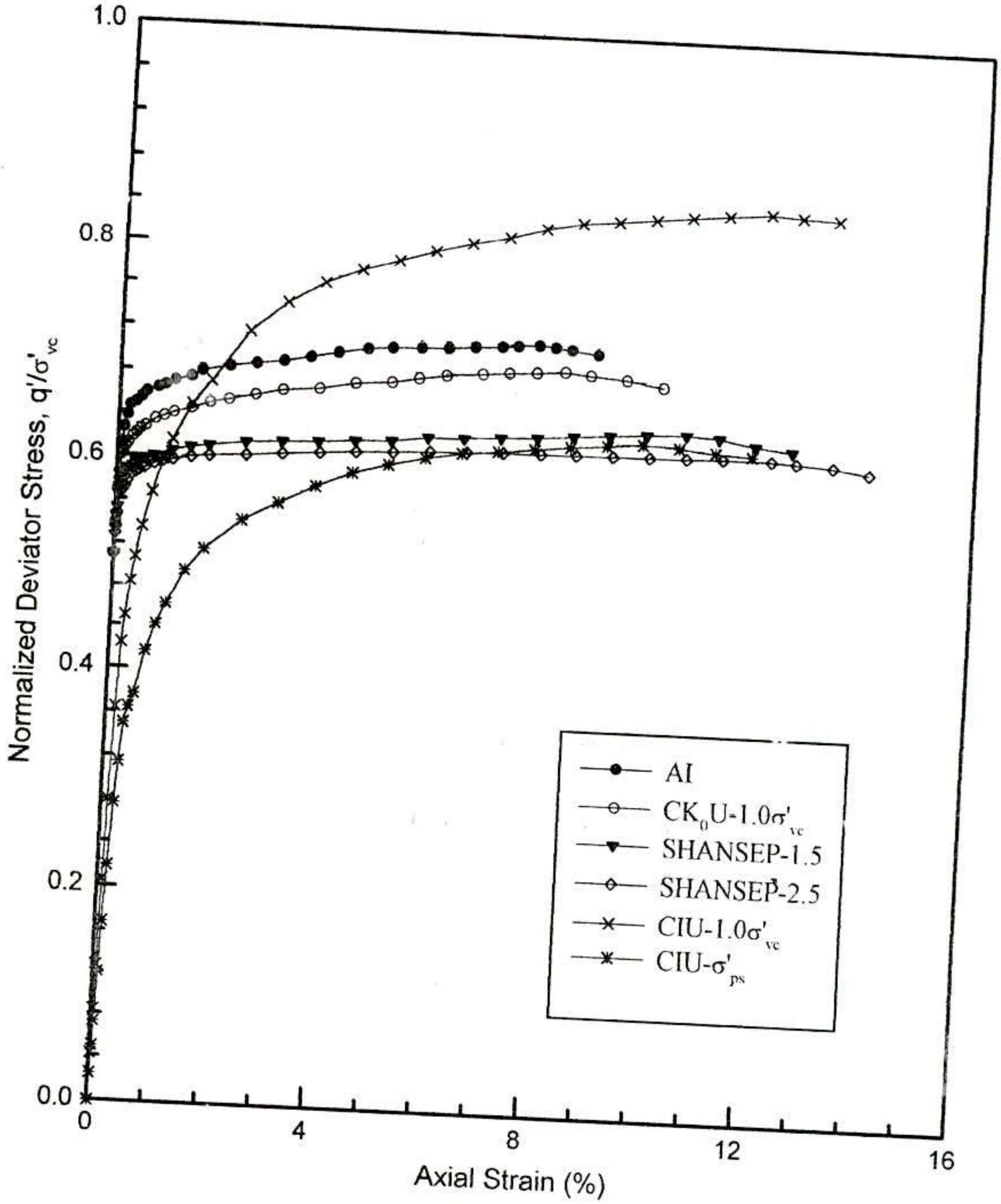


Fig. 7.14 Deviator Stress vs. Axial Strain Plots for "In Situ" and Reconsolidated "Perfect" Samples from Anwara

effective stress equal to in situ vertical effective stress, σ'_{vc}) produced the best agreement with the "in situ" samples of Banskhali and Anwara. Similar results were also obtained for the samples of another soil from Chandanaish.

Undrained shear strength (s_u), initial tangent stiffness (E_i), secant modulus at half the deviator stress (E_{50}) and axial strain at peak deviator stress (ε_p), have been determined from the stress-strain data for the "in situ" and reconsolidated "perfect" samples of the three soils. A comparison of normalized undrained shear parameters between the "in situ" and reconsolidated "perfect" samples is presented in Table 7.3.

It is observed from Table 7.3 that isotropic reconsolidation using a pressure equal to the vertical effective "in situ" stress, σ'_{vc} has the effect of overestimation of "in situ" strength ratios (s_u/σ'_{vo}) and ε_p for all the soils. Values of s_u/σ'_{vo} and ε_p have been overestimated up to 20.8% and 61.4% respectively. Similar results were also reported by Ladd (1965), Crooks and Graham (1976), Mitachi and Kitago (1979), Nakase and Kamei (1983), Kirkpatrick and Khan (1984), Mayne (1985), Wijeyakulasuriya (1986), Graham et al. (1987), Siddique and Farooq (1996), Siddique and Sarker (1998), Rahman (2000). Compared with the "in situ" samples, the values of stiffness ratios, E_i/σ'_{vo} and E_{50}/σ'_{vo} increased significantly for the "perfect" samples reconsolidated using isotropic pressure equal to σ'_{vc}. Values of E_i/σ'_{vo} and E_{50}/σ'_{vo} increased up to 60.3% and 42.6%, respectively due to reconsolidation using an isotropic pressure equal to σ'_{vc}. Kirkpatrick and Khan (1984), Siddique and Sarker (1998), Siddique and Farooq (1996) also found increased stiffness of "perfect" samples reconsolidated using an isotropic pressure equal to σ'_{vc}. The higher values of s_u, E_i and E_{50} probably resulted from less water content in samples under isotropic reconsolidation to σ'_{vc} caused by the higher mean normal consolidation pressure.

Isotropic reconsolidation using a pressure equal to σ'_{ps} underestimated the values of s_u/σ'_{vo}, E_i/σ'_{vo} and E_{50}/σ'_{vo} for the three soils. Values of s_u/σ'_{vo}, E_i/σ'_{vo} and E_{50}/σ'_{vo} reduced up to 12.6%, 17.4% and 21.8%, respectively. The values of ε_p were, however, overestimated up to 25.6% for the soils. Similar results have also been reported by

Table 7.3 Comparison of Undrained Shear Characteristics of "In Situ" and Reconsolidated "Perfect" Samples of the Three Soils

Soil	Test type	s_u/σ'_{vo}	ε_p (%)	E_i/σ'_{vo}	E_{50}/σ'_{vo}	A_p
Banskhali Soil	CIU-1.0σ'_{vc}	0.40	11.3	230.0	175.0	0.81
	CIU-σ'_{ps}	0.297	8.2	139.0	102.5	0.56
	CK_0U-1.0σ'_{vc}	0.336	7.5	156.1	116.3	0.74
	SHANSEP-1.5	0.305	9.8	123.8	105.2	1.31
	SHANSEP-2.5	0.293	10.5	103.1	84.5	1.40
	BI	0.34	7.0	163.8	124.8	0.76
Anwara Soil	CIU-1.0σ'_{vc}	0.424	12.0	250.0	188.7	0.79
	CIU-σ'_{ps}	0.315	9.8	146.3	110.3	0.58
	CK_0U-1.0σ'_{vc}	0.346	8.3	170.7	128.3	0.72
	SHANSEP-1.5	0.32	10.6	132.0	112.4	1.20
	SHANSEP-2.5	0.31	12.2	109.9	89.7	1.32
	AI	0.357	7.8	175.2	132.3	0.74
Chandanaish Soil	CIU-1.0σ'_{vc}	0.447	13.2	295.0	203.0	0.76
	CIU-σ'_{ps}	0.322	10.14	152.0	115.0	0.535
	CK_0U-1.0σ'_{vc}	0.364	9.3	173.0	139.0	0.73
	SHANSEP-1.5	0.327	11.23	140.0	120.0	1.27
	SHANSEP-2.5	0.32	11.63	112.0	96.0	1.38
	CI	0.37	8.8	184.0	147.0	0.71

Kirkpatrick and Khan (1984), Graham et al. (1987) and, Graham and Lau (1988). The lower values of s_u, E_i and E_{50} probably resulted from more water content in samples under isotropic reconsolidation to σ'_{ps} caused by the lesser mean normal consolidation pressure than that of "in situ" sample.

It can be seen from Table 7.3 that the strength ratios and stiffness ratios of the "perfect" samples reconsolidated using Bjerrum (1973) procedure (i.e., consolidation under K_0-condition to vertical effective stress equal to in situ vertical effective stress, σ'_{vc}) and SHANSEP (Ladd and Foott, 1974) procedures (i.e., K_0-consolidation to vertical effective stress equal to 1.5 times and 2.5 times the in situ vertical effective stress, σ'_{vc}) are less than those for the "in situ" samples. The undrained strength ratio (s_u/σ'_{vo}) for the "perfect" samples from Banskhali reconsolidated using Bjerrum (1973), SHANSEP-1.5 and SHANSEP-2.5 procedures reduced by 1.2%, 10.3% and 13.8%, respectively. The respective reductions for the samples from Anwara are 3.1%, 10.4% and 13.2%, respectively while those for the samples from Chandanaish are 1.6%, 11.6% and 13.5%, respectively. The undrained stiffness ratio (E_i/σ'_{vo}) reduced by 4.7%, 24.4% and 37.1% for "perfect" samples from Banskhali reconsolidated using Bjerrum (1973), SHANSEP-1.5σ'_{vc} and SHANSEP-2.5σ'_{vc} procedures, respectively. The respective reductions for the samples from Anwara are 2.6%, 24.7% and 37.3%. The stiffness ratio (E_i/σ'_{vo}) reduced by 6%, 23.9% and 39.1% for "perfect" samples from Chandanaish reconsolidated using Bjerrum, SHANSEP-1.5 and SHANSEP-2.5 procedures, respectively. The undrained stiffness ratio (E_{50}/σ'_{vo}) reduced for samples from Banskhali by 6.8 %, 15.7% and 32.3% ; for samples from Anwara by 3%, 15% and 32.2%; and, for samples from Chandanaish by 5.4%, 18.4% and 34.7% due to reconsolidation using the above Bjerrum, SHANSEP-1.5 and SHANSEP-2.5 procedures, respectively.

The values of ε_p of all the "perfect" samples reconsolidated using above Bjerrum (1973), SHANSEP-1.5 and SHANSEP-2.5 procedures, however, increased. The values of ε_p increased for samples from Banskhali by 7.15%, 40% and 50%; for samples from Anwara by 6.4%, 35.9% and 56.4%; and for samples from Chandanaish by 5.7%, 27.3% and 31.8% due to reconsolidation using the Bjerrum, SHANSEP-1.5 and SHANSEP-2.5 procedures, respectively.

From the above comparisons of the values of s_u/σ'_{vo}, E_i/σ'_{vo}, E_{50}/σ'_{vo} and ε_p between the "in situ" and reconsolidated "perfect" samples, it is evident that for each soil, despite all the anisotropic reconsolidation procedures provided a lower bound strength and stiffness, and an upper bound axial strain at peak deviator stress, the values of s_u/σ'_{vo}, E_i/σ'_{vo}, E_{50}/σ'_{vo} and ε_p of the "perfect" sample reconsolidated using Bjerrum procedure (i.e., consolidation under K_0-condition to vertical effective stress equal to in situ vertical effective stress, σ'_{vc}) compared more closely with the "in situ" sample in terms of strength, deformation and stiffness properties than the "perfect" samples reconsolidated using SHANSEP-1.5 and SHANSEP-2.5 procedures. Kirkpatrick and Khan (1984), Graham et al. (1987) and Graham and Lau (1988), Rahman (2000) also found that K_0-reconsolidation of the "perfect" sample to σ'_{vc} produced the best agreement between the "perfect" and "in situ" samples in terms of the strength, deformation and stiffness. Siddique and Sarker (1998) reported that reconsolidation of "perfect" specimens using SHANSEP procedures could not restore the characteristics of the "in situ" specimen for normally consolidated Dhaka clay. Siddique and Farooq (1996) found that K_0-reconsolidation of "perfect" sample to SHANSEP-1.5 produced the best agreement for two coastal soils. Siddique and Sarker (1998) and Siddique and Farooq (1996), however, did not adopt the reconsolidation of "perfect" samples using Bjerrum procedure.

In recommending the CK_0U test consolidating to in situ stresses as a means of predicting in situ behaviour from samples it is recognized that difficulties can arise in testing. Firstly, although the in situ vertical stress may readily be found, K_0 may not always be known. For the soils examined here, however, it may be possible to determine the value of K_0 by K_0-consolidometer (if available in the laboratory) or may be assumed 0.5 for K_0 for quick test as typical values of normally consolidated clays are 0.4 to 0.7. Though this value is approximate value but this must give the better result in finding undrained soil parameters than isotropic consolidation. A second problem arises in the practical sense of applying an anisotropic consolidation pressure. The use of an incremental system or consolidations, employing small steps of deviator component of stress is both troublesome and time consuming. Comparison between incremental stress and two steps method of consolidation, two steps method is relatively simple and of shorter duration, where the total hydrostatic component is applied, followed by the total deviatoric component allowing consolidation. Kirkpatrick and Khan (1984) found that the final results are almost same for both incremental methods and two step methods to apply the preshear consolidation pressure.

7.4.3 Pore Pressure Responses

Fig. 7.15 shows typical plottings of the variation of Skempton's pore pressure parameter, A with axial strain for "in situ" and reconsolidated "perfect" samples from Banskhali. It can be seen from Fig. 7.15 that the pore pressure response (as evaluated in terms of Skempton's pore pressure parameter, A) of the "perfect" sample reconsolidated using Bjerrum (1973) procedure (i.e., consolidation under K_0-condition to vertical effective stress equal to in situ vertical effective stress, (σ'_{vc}) produced the best agreement with the "in situ" sample than the "perfect" samples reconsolidated using other techniques. Skempton's pore pressure parameter, A at peak deviator stress (A_P) has been determined from stress-strain and pore pressure data. Similar results were found for other soils (Rahman, 2000). The values of A_p for all samples of the three soils are listed in Table 7.3. The following points can be noted from Table 7.3:

- Isotropic reconsolidation with pressure equal to σ'_{vc}, overestimated (up to 7.0%) the values of A_P.
- Isotropic reconsolidation with pressure equal to σ'_{ps}, underestimated (up to 26.3%) the values of A_P.
- Anisotropic reconsolidation using Bjerrum (1973) procedure slightly underestimated or overestimated (up to 3%) the values of A_P.
- Anisotropic reconsolidation using Bjerrum (1973) procedure slightly underestimated or overestimated (up to 3%) the values of A_P.
- Anisotropic reconsolidation using SHANSEP-1.5σ'_{vc} and SHANSEP-2.5σ'_{vc} procedures grossly overestimated (up to 94.4%) the values of A_P.

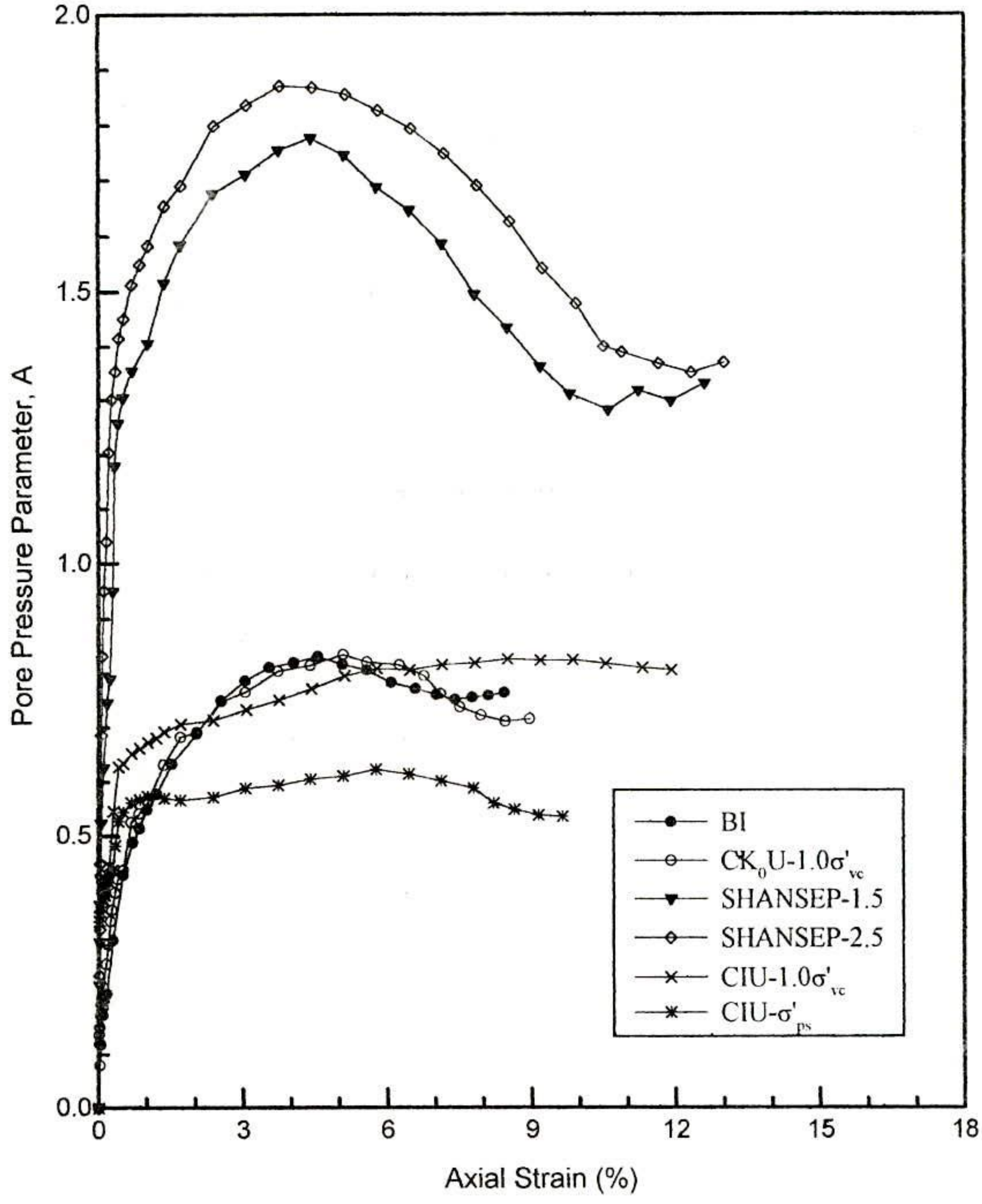


Fig. 7.15 Pore Pressure Parameter, A vs. Axial Strain Plots for "In Situ" and Reconsolidated "Perfect" Samples from Banskhali

It is generally known that the pore pressure generated during undrained shear is not a unique property of the soil, but it depends on the applied stress increments. The pore pressure can be split into two components, namely the shear induced pore pressure and the pore pressure due to the increase in the applied mean normal stress. Thus, in case of normally consolidated samples under "in situ" stress conditions, the total amount of excess pore pressure is reduced when it was compared with that of isotropically consolidated samples, since a large portion of the stress difference was applied prior to the undrained shear due to anisotropic consolidation. Several investigators (Ladd, 1965; Crooks and Graham, 1976; Kirkpatrick and Khan, 1984; Graham et al., 1987; Graham and Lau, 1988; Siddique and Farooq, 1996) have shown that the isotropic reconsolidation resulted in higher values of A_p than those obtained from "in situ" samples.

From the above comparisons, it is evident that the values of A_p of the "perfect" samples reconsolidated using Bjerrum (1973) procedure compares most favourably with the "in situ" sample than the "perfect" samples reconsolidated using other techniques. Kirkpatrick and Khan (1984), Graham et al. (1987) and, Graham and Lau (1988) also found that K_0-reconsolidation of the "perfect" samples to σ'_{vc} produced the best agreement between the "perfect" and "in situ" samples in terms of the values of A_p.

CHAPTER 8

BEHAVIOUR OF RECONSTITUTED "TUBE" SAMPLES AND EFFECTS OF "TUBE" SAMPLING DISTURBANCE

8.1 General

This chapter presents the experimental results of the influence of tube sampling disturbances on undrained shear properties of reconstituted samples of the three coastal soils. In order to investigate the effect of tube sampling on undrained shear behaviour, triaxial compression tests were carried out on samples collected from reconstituted soils with sampling tubes of varying thicknesses (t), i.e., of varying area ratio (AR). Attempt has been made to assess the effects of sampler characteristics, namely area ratio and external diameter to thickness (D_e/t) ratio of sampler, on the measured undrained soil parameters. The investigation also examines the suitability of different reconsolidation procedures, both isotropic and anisotropic, in order to minimize the effects of tube sampling disturbance in the coastal soils. Initial effective stress, undrained stress-strain-strength, stiffness and pore pressure characteristics of "tube" samples of the three reconstituted soils were determined from unconsolidated undrained triaxial compression tests. The experimental results are presented and discussed in the following sections. Comparisons of "tube" behaviour with the "in situ" behaviour are also presented.

8.2 Effects of Tube Sampling Disturbance

8.2.1 Changes in Initial Effective Stress

In order to determine initial effective stress, σ'_i of a "tube" sample, relatively high cell pressure was applied on sample under undrained condition and a steady pore pressure generated within the sample was recorded. Initial effective stress has been calculated by subtracting pore water pressure from all-round cell pressure (Skempton, 1961; Baldi et al., 1988). The initial effective stress of the "tube" samples of each location was compared with the isotropic effective stress, σ'_{ps} in a "perfect" sample of the respective location. The isotropic effective stress in a "perfect" saturated sample of each type of sample which had in-situ vertical and horizontal effective stresses of σ'_v and $K_0\,\sigma'_v$, respectively, has been determined by the following expression (Ladd and Lambe, 1963; Ladd and Varallyay, 1965):

$$\sigma'_{ps} = \sigma'_v\left[K_0 + A_u\left(1 - K_0\right)\right] \tag{8.1}$$

where K_0 is the coefficient of earth pressure at rest and A_u is the pore pressure parameter for the undrained release of the in-situ shear stress existed at the K_0-conditions. The parameter A_u for a saturated clay (i.e., Skempton's B parameter is equal to unity) is given by the follwing expression:

$$A_u = \frac{\Delta u - \Delta\sigma_h}{\Delta\sigma_v - \Delta\sigma_h} \tag{8.2}$$

where, Δu is the pore pressure change; and $\Delta\sigma_v$ and $\Delta\sigma_h$ are the changes of vertical and horizontal total stresses. The values of A_u and σ'_{ps} of the samples BP, AP and CP are shown in Table 8.1.

Table 8.2 shows comparison of initial effective stress of "tube" samples with isotropic effective stress of the respective "perfect" sample. Table 8.2 shows that compared with isotropic effective stress, σ'_{ps} of "perfect" samples, σ'_i of "tube" samples reduced significantly because of disturbance caused by penetration of tubes. It can be seen from Table 8.2 that the values of σ'_i reduced by 14.2% to 36.3%, 12.95% to 32.94% and 10.53% to 25.73% for samples of Banskhali, Anwara and Chandanaish, respectively. Reduction in effective stress due to tube sampling disturbance has also been reported for the regional clays of Bangladesh (Siddique and Sarker, 1995; Siddique et al., 2000). Siddique and Sarker (1995) reported a reduction in initial effective stress between 19% to 42% in reconstituted normally consolidated Dhaka clay. Siddique et al. (2000) also reported that initial effective stress reduced between 8.6% to 33.7%, 7.3% to 30% and 5.4% to 22.4% in reconstituted normally consolidated samples of three coastal soils due to tube sampling disturbance. The results of the present investigation, however, indicated that the less plastic coastal soils used in this study suffered higher reduction in σ'_i than the more plastic coastal soils reported by Siddique and Farooq (1998). Significant reduction in initial mean

Table 8.1 Values of K_0, A_u, σ'_{ps} and s_{up} of "Perfect" Samples of the Coastal Soils

Sample	σ'_{vc} (kN/m^2)	K_0	A_u	σ'_{ps} (kN/m^2)	s_{up} (kN/m^2)
BP	150	0.47	0.132	81.0	46.57
AP	150	0.49	0.125	83.06	49.50
CP	150	0.50	0.140	85.5	53.00

Table 8.2 Comparison of Initial Effective Stress of "Tube" Samples with Isotropic Effective Stress of "Perfect" Samples of the Three Coastal Soils

Sample designation	Initial effective stress, σ'_i (kN/m^2)	% Reduction in σ'_i compared with σ'_{ps}
BT	69.5	14.2
BM	64.2	20.7
BH	51.6	36.3
AT	72.3	13.0
AM	68.2	17.9
AH	55.7	32.9
CT	76.5	10.5
CM	73.1	14.5
CH	63.5	25.7

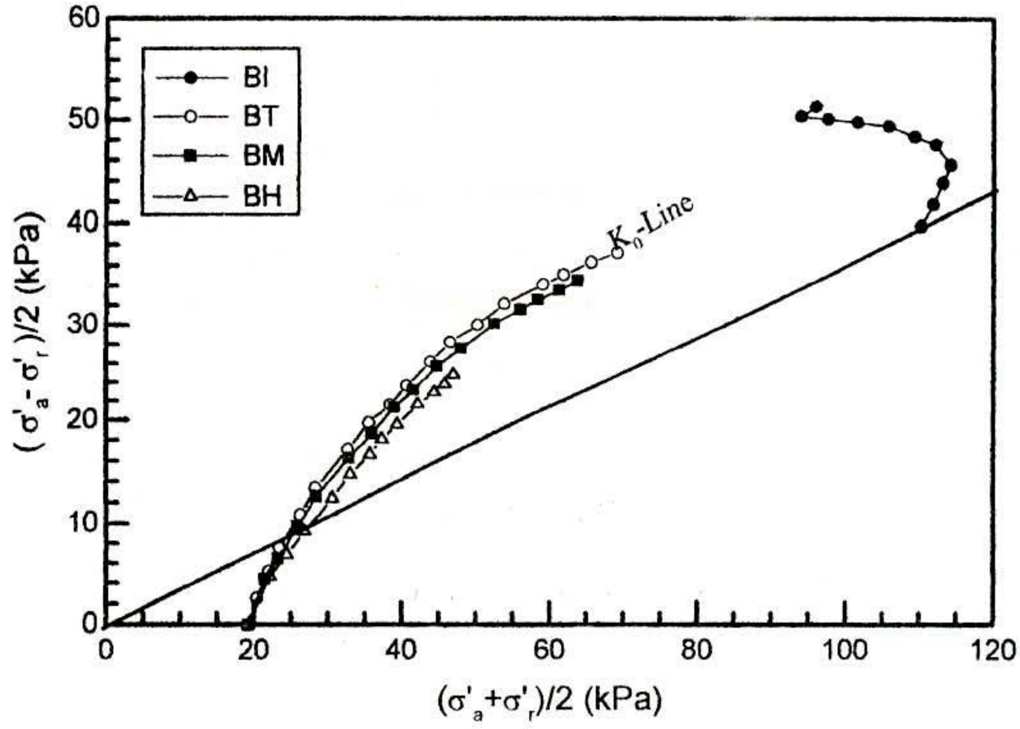
effective stress due to application of tube sampling strains (Baligh, 1985) has also been reported for other reconstituted normally consolidated soils (Baligh et al., 1987; Hajj, 1990; Clayton et al., 1992; Hird and Hajj, 1995; Siddique and Clayton, 1995; Siddique et al., 1999).

8.2.2 Changes in Stress Paths

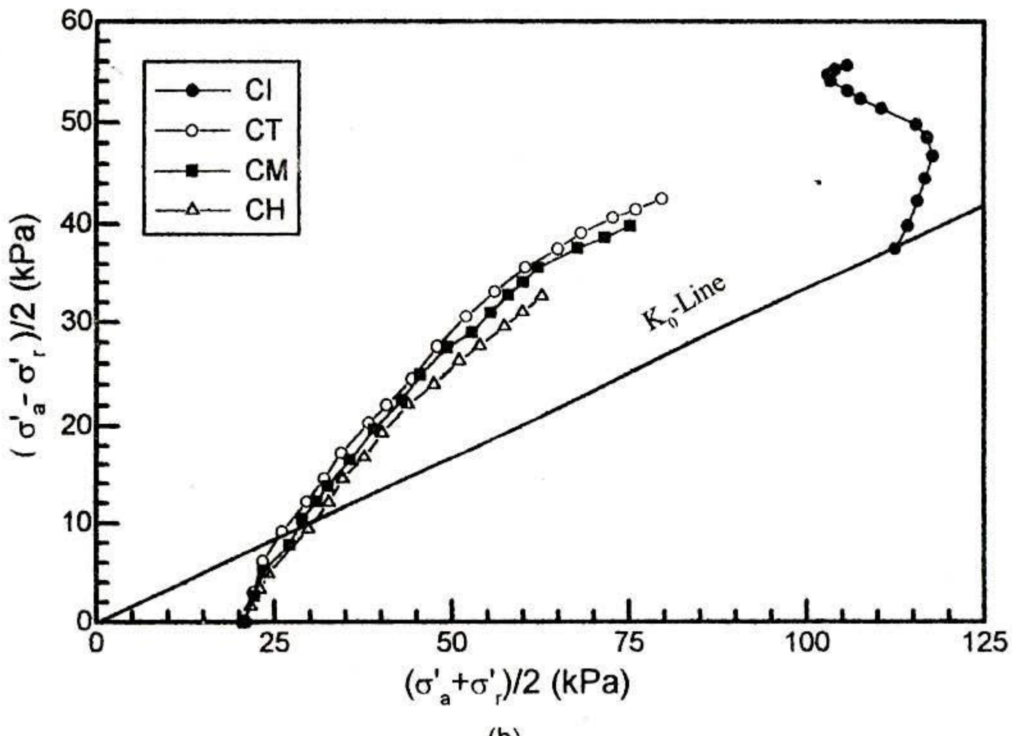
Fig. 8.1 presents the effective stress paths in s'-t' [$s' = (\sigma'_a + \sigma'_r)/2$, $t' = (\sigma'_a - \sigma'_r)/2$] space for "tube" samples for the two soils from Banskhali and Chandanaish. Fig. 8.1 also shows the stress path of the corresponding "in situ" samples to compare with the "tube" samples. It can be seen from Fig. 8.1 that the stress paths of "in situ" samples are markedly different from that of "tube" samples of both the soils. The "tube" samples adopt stress paths similar to those for overconsolidated samples. Marked difference in the effective stress paths between normally consolidated "in situ" and "tube" samples of three Bangladesh coastal soils has also been reported by Siddique and Farooq (1998) and Siddique et al. (2000). The present results therefore compares favourably with those of other Bangladesh coastal soils. Difference in the effective stress paths between the "in situ" and "tube" samples were also reported for other reconstituted soils (Atkinson and Kubba, 1981; Hight et al., 1985; Siddique and Sarker, 1995; Siddique and Rahman, 2000).

8.2.3 Changes in Strength, Deformation and Stiffness Properties

The values of undrained shear strength (s_u), axial strain at peak deviator stress (ε_p), initial tangent modulus (E_i), secant modulus at half the peak deviator stress (E_{50}) and secant stiffness (E_u) at small strain levels of the "tube" samples have been determined from the stress-strain data of all the "tube" and "in situ" samples. Fig. 8.2 shows the typical stress-strain curves for "in situ" and "tube" samples from Anwara. Table 8.3 presents comparisons of the undrained shear properties between the "in situ" and "tube" samples of Banskhali, Anwara and Chandanaish soils, respectively. It can be seen from Table 8.3 that compared with the "in situ" samples, the values of s_u, E_i and E_{50} reduced for the three soils. The values of ε_p, however, increased significantly due to disturbance caused by penetration of sampler. Similar effects have also been reported for normally consolidated clays by Okumura (1971), Siddique and Sarker (1995), Siddique et al. (2000), Siddique and Rahman (2000), for Honmoku Marine clay, soft Dhaka clay, Coastal soils and firm Dhaka clay, respectively.



(a)



(b)

Fig. 8.1 Comparison of Effective Stress Paths for "In Situ" and "Tube" Samples from (a) Banskhali and (b) Chandanaish

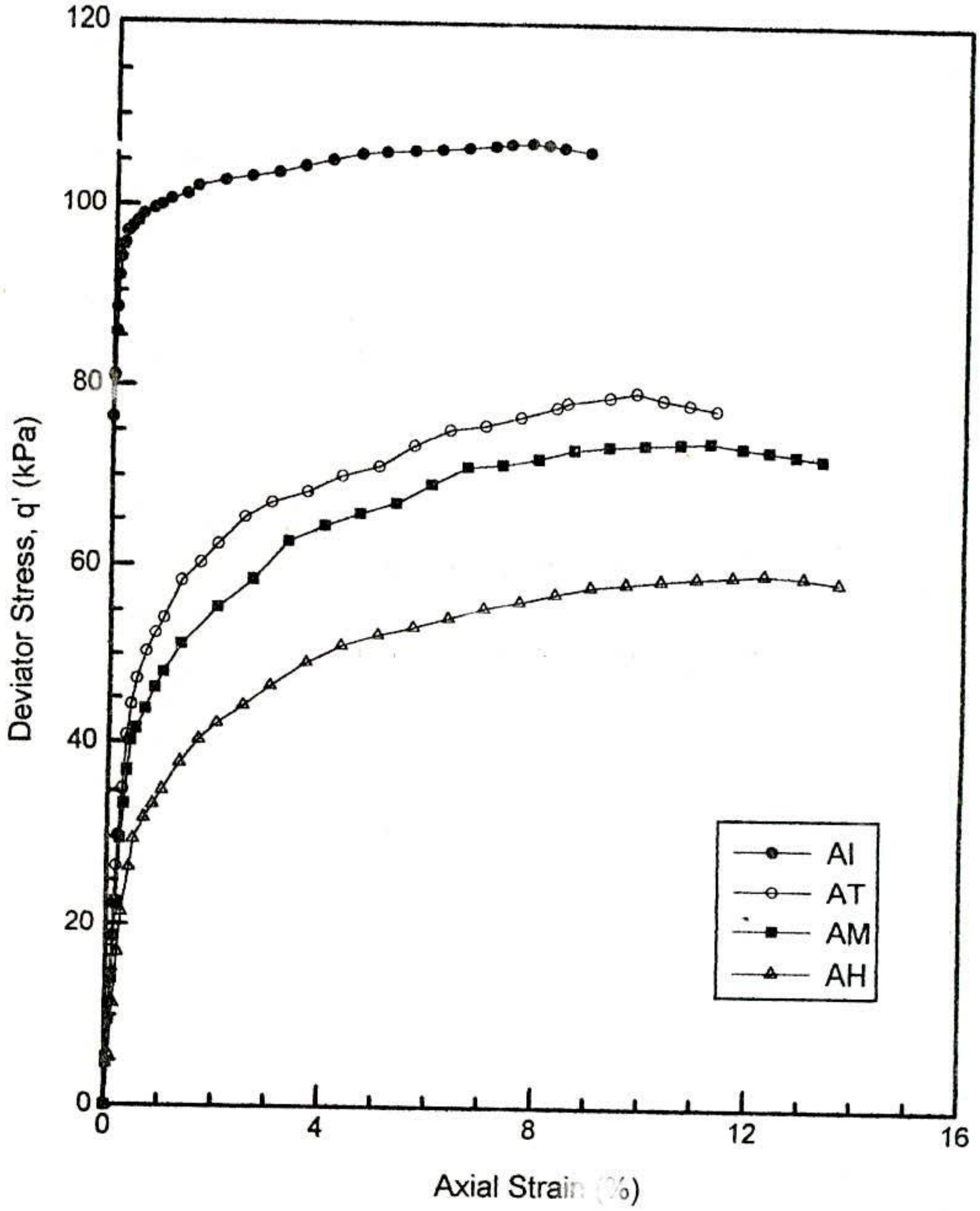


Fig. 8.2 Comparison of Deviator Stress vs. Axial Strain for "In Situ" and "Tube" Samples from Anwara

Table 8.3 Comparison of Undrained Shear Properties of Samples of the Three Coastal Soils

Sample Designation	s_u (kN/m^2)	ε_p (%)	E_i (kN/m^2)	E_{50} (kN/m^2)	A_p
BI	51.0	7.0	24570	18720	0.76
BT	37.2	9.45	15390	11010	-0.135
BM	34.5	10.72	13170	9180	-0.151
BH	24.8	11.5	6851	5480	-0.215
AI	53.6	7.8	26280	19845	0.74
AT	39.8	9.83	16950	12255	-0.12
AM	37.1	11.22	14250	10425	-0.137
AH	29.7	12.3	8460	6150	-0.176
CI	55.5	8.8	27600	22050	0.71
CT	42.5	10.67	18250	14046	-0.102
CM	39.8	11.5	15580	11840	-0.116
CH	32.6	13.0	9650	7233	-0.144

The results of the present investigation confirm and validate the findings of the previous study on other coastal soils. The present results also show that the generalized stress-strain and stiffness behaviour of the less plastic coastal soils used in this study is similar to more plastic coastal soils.

Small strain stiffness behaviour of the "in situ" and 'tube" samples has also been investigated. Plottings of secant stiffnesses (E_u) at small strain levels (up to 1%) for "in situ" and "tube" samples of a typical sample from Anwara are shown in Fig. 8.3. It can be seen from Fig. 8.3 that, in general, secant stiffnesses of the "in situ" and "tube" samples reduced with the increase in axial strain. It can also be seen from Fig. 8.3 that in each soil, secant stiffnesses (at all small strain levels) of the "tube" samples are considerably lower than those for the "in situ" sample. For comparison, the values of secant stiffnesses (at 0.1% axial strain) of the "tube" samples AT, AM and AH from Anwara decreased by 26.54%, 39.1% and 64.45%, respectively. Similar results have been exhibited by the samples from Banskhali and Chandanaish. The values of secant stiffnesses (at 0.1% axial strain) of the "tube" samples BT, BM and BH from Banskhali decreased by 31.25%, 42.47% and 70.1%, respectively. The values of secant stiffnesses (at 0.1% axial strain) of the "tube" samples CT, CM and CH from Chandanaish decreased by 22.93%, 36.86% and 58.63%, respectively. Compared with the "in situ" sample, significant reduction in stiffness for soft normally consolidated Dhaka clay (Siddique and Sarker, 1995), firm normally consolidated Dhaka clay (Siddique and Rahman, 2000) and soft normally consolidated Coastal soils (Siddique et al., 2000) have also been reported. Considerable reduction in stiffness for reconstituted and intact clays due to application of tube sampling strains (Baligh et al., 1985) have also been reported (Baligh et al., 1987; Lacasse and Berre, 1988; Hajj, 1990; Hopper, 1992; Hight and Georgiannou, 1995; Siddique and Clayton, 1995; Siddique et al., 1999; Siddique and Rahman, 2000)

Small strain stiffness behaviour of "in situ" and "tube" samples of coastal soils has been found to be similar to other normally consolidated reconstituted and intact soils (Hight and Georgiannou, 1995; Siddique et al., 1999).

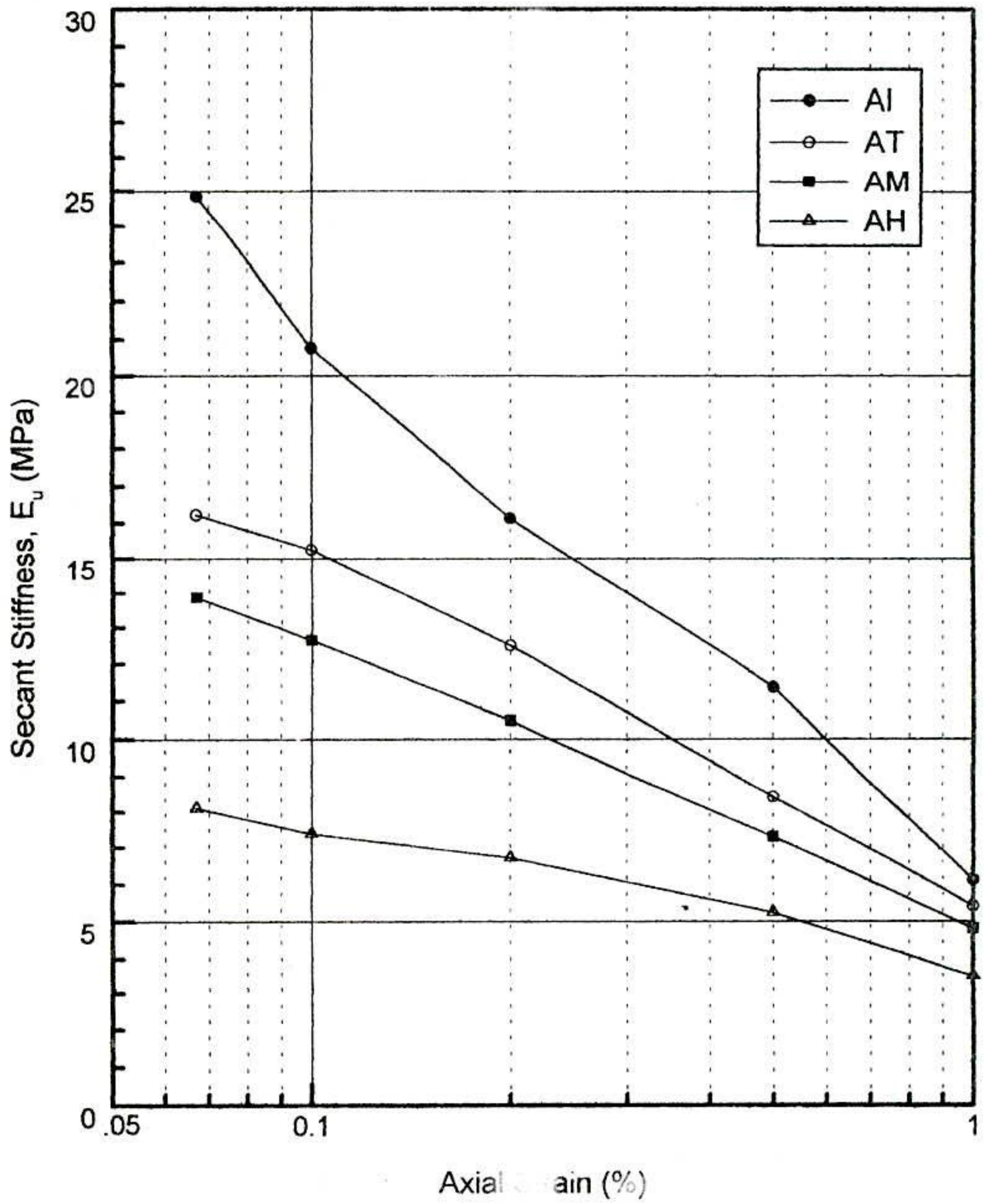


Fig. 8.3 Secant Stiffness vs. Axial Strain Plots on "In Situ" and "Tube" Samples from Anwara

8.2.4 Changes in Pore Pressure Responses

A typical comparison of the changes in pore pressure with axial strain during undrained shearing between "tube" and "in situ" samples of Banskhali soil is shown in Fig. 8.4. It can be seen from Fig. 8.4 that compared with the "in situ" sample, the pore pressure responses of the "tube" samples are considerably less. For the "tube" samples, the pore pressure responses are slightly negative at peak deviator stress. It is also evident that the pore pressure responses of the "tube" samples are similar to those of overconsolidated clays. Similar pore pressure responses in reconstituted soft normally consolidated "in situ" and "tube" samples of soft Dhaka clay (Sarker, 1994), Coastal soils (Farooq, 1995) and firm Dhaka clay (Siddique and Rahman, 2000) were reported.

Skempton's pore pressure parameters A at peak deviator stress (A_p) of the "tube" and "in situ" samples were determined which have already been presented in Table 8.3. It can be seen from Table 8.3 that compared with the "in situ" samples, the values of A_p of the "tube" samples reduced considerably. The values of A_p of the "tube" samples varied between -0.102 to -0.215. These results compared favourably with those of more plastic reconstituted samples of three coastal soils (Siddique et al., 2000). Considerable reductions in A_p-values due to tube sampling have also been reported by Siddique (1990) for normally consolidated reconstituted soft London clay (LL = 69, PI = 45). Siddique and Sarker (1995), Siddique et al., (2000) and Siddique and Rahman (2000) also found that the values of A_p decreased significantly for the "tube" samples in reconstituted soft Dhaka clay (LL = 45, PI = 23), reconstituted coastal soils and reconstituted firm Dhaka clay, respectively.

8.3 Assessment of Effect of Sampler Geometry on Undrained Soil Parameters of Reconstituted Coastal Soils

Initial effective stress, undrained stress-strain-strength stiffness and pore pressure parameters of "tube" and "in situ" samples were determined from undrained triaxial compression tests. The experimentally measured soil parameters of the "in situ" and "tube" samples have been presented in the previous section. The effect of sampler geometry on disturbance have been assessed by comparing the soil parameters of the "tube" samples with those of the "in situ" samples retrieved with samplers of different area ratio and external diameter (D_e) to thickness (t) ratio.

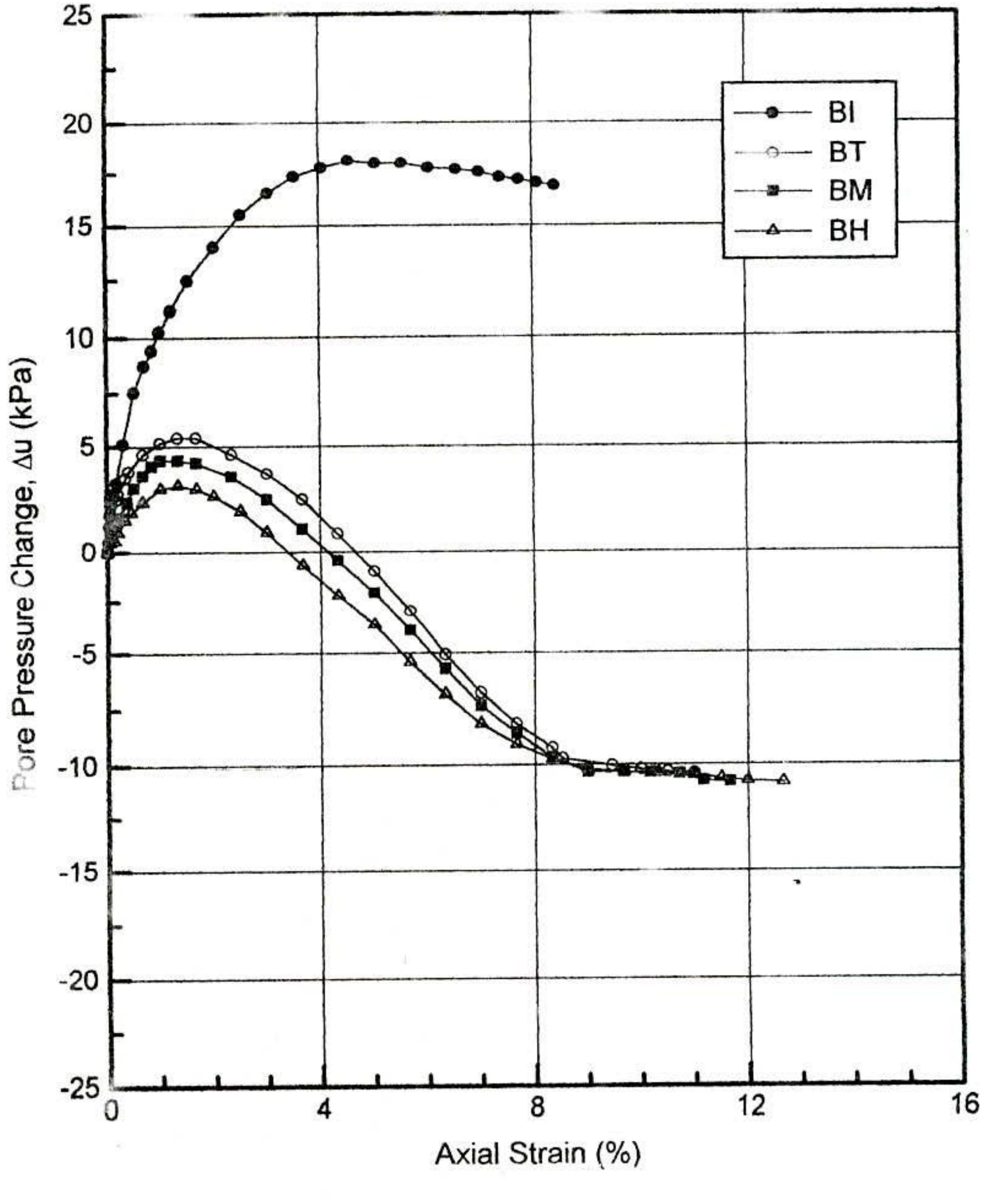


Fig. 8.4 Comparison of Pore Pressure Response Between "In Situ" and "Tube" Samples from Banskhali

8.3.1 Effect of Area Ratio and D_e/t Ratio

Fig. 8.5 shows typical comparisons of the changes in undrained shear strength (s_u), axial strain at peak deviator stress (ε_p), initial tangent modulus (E_i) and secant modulus at half the peak deviator stress (E_{50}) due to change in area ratio of the samplers used to retrieve samples of Chandanaish soil. It can be seen from Fig. 8.5 that increasing area ratio (or reducing D_e/t ratio) caused increasing reductions in s_u, E_i and E_{50}. Increasing area ratio (or decreasing D_e/t ratio) of sampler, however, caused an increase in ε_p. Similar results were observed in case of samples from Banskhali and Anwara as shown in Table 8.4. Compared with the "in situ" sample, the following effects on the measured soil parameters have been observed due to increasing area ratio (or decreasing D_e/t ratio) of samplers:

(1) Values of s_u decreased from 27% to 51.5%, 25.8% to 44.5% and 23.5% to 41.4% in samples from Banskhali, Anwara and Chandanaish, respectively due to increase in area ratio from 16.4 to 73.1% (or decrease in D_e/t ratio from 27.3 to 8.3).

(2) Values of ε_p increased from 35% to 64.3%, 26% to 58% and 21.3% to 47.7% in samples from Banskhali, Anwara and Chandanaish, respectively due to increasing area ratio (or decreasing D_e/t ratio).

(3) Values of E_i decreased by 37.4% to 72%, 35.5% to 67.8% and 33.7% to 65.2% in samples from Banskhali, Anwara and Chandanaish, respectively due about 4.5 times increase in area ratio (or about 70% reduction in D_e/t ratio).

(4) Values of E_{50} decreased by 41.2% to 70.7%, 38.2% to 69% and 36.8% to 67.2% in samples from Banskhali, Anwara and Chandanaish, respectively due about 4.5 times increase in area ratio (or about 70% reduction in D_e/t ratio).

Compared with the A_p values of the "in situ" samples (Table 8.4), it has been found that the pore pressure responses of the samples collected with varying area ratio are considerably less, resulting in significantly lower values of A_p. Table 8.2 indicates that initial effective stress reduced up to maximum 36.3%, 32.9% and 25.7% due to increase in area ratio from 16.4% to 73.1% for the samples from Banskhali, Anwara and Chandanaish, respectively.

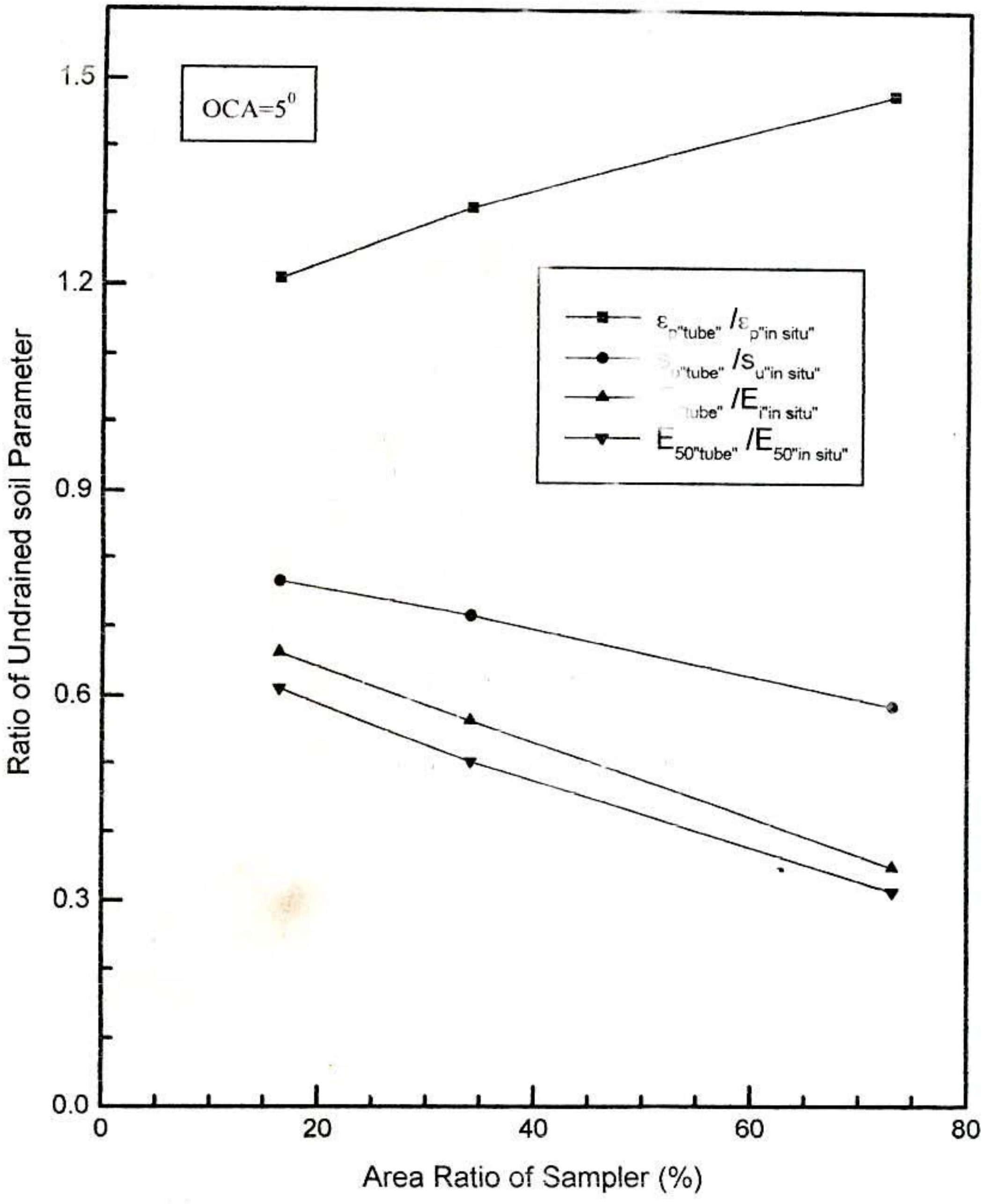


Fig. 8.5 Influence of Area Ratio of Sampler on Undrained Soil Parameters for Samples from Chandanaish

Table 8.4 Influence of Increasing Area Ratio (or Decreasing D_e/t Ratio) of Sampler on Undrained Shear Properties of Samples of the Three Coastal Soils

Sample designation	Area ratio (%)	D_e/t ratio	*Ratio of			
			s_u	ε_p	E_i	E_{50}
BT	16.4	27.33	0.73	1.35	0.626	0.588
BM	34.1	14.67	0.676	1.53	0.536	0.490
BH	73.1	8.33	0.485	1.64	0.280	0.293
AT	16.4	27.33	0.742	1.26	0.645	0.618
AM	34.1	14.67	0.692	1.44	0.542	0.525
AH	73.1	8.33	0.555	1.58	0.322	0.310
CT	16.4	27.33	0.765	1.21	0.663	0.632
CM	34.1	14.67	0.716	1.31	0.565	0.537
CH	73.1	8.33	0.586	1.48	0.348	0.328

* Ratios are compared with the values of "in situ" samples.

Siddique and Sarker (1996) and Siddique et al. (2000) investigated the effect of area ratio on soil parameters of reconstituted normally consolidated soft Dhaka clay and Chittagong coastal soils, respectively. Siddique and Sarker (1996) reported reduction in σ'_i (up to 42%), s_u (up to 35%) and E_i (up to 49%) and an increase in ε_p (up to 81%) in reconstituted normally consolidated soft Dhaka clay due to increase in area ratio of samplers from 10.8% to 55.2%. Due to increase in area ratio, Siddique et al. (2000) also reported that the values of σ'_i, s_u and E_i reduced significantly while the value of ε_p increased of reconstituted normally consolidated soft samples of three Chittagong coastal soils

The results of the previous and present investigations on coastal soils clearly demonstrate that in order to minimize disturbance due to sampling in coastal clays of low to high plasticity, area ratio of sampler should be kept as low as possible. Practically, the area ratio should not exceed 10%.

8.4 Quantitative Assessment of Sample Disturbance

There is no way of obtaining a soil sample so as to maintain exactly the in situ conditions. This is because its removal involves a change in the in situ state of stress and usually some disturbance due to sampling and handling. So, degree of disturbance can be assessed by investigating the behaviour of the least disturbed sample which is usually a laboratory simulated "perfect" sample. Because of additional disturbances other than that occurred due to total stress release, the residual or initial effective stress of "tube" (disturbed) sample, σ'_i is usually less than the isotropic effective stress, σ'_{ps} of a "perfect" sample. Based on the values of σ'_{ps} of "perfect" sample and initial effective stress, σ'_i for "tube" sample, degree of disturbance (D_d) has been calculated using the following equation proposed by Okumura (1971) and Nelson et al. (1971):

$$D_d = 1 - \frac{\sigma'_i}{\sigma'_{ps}} \tag{8.3}$$

The values of D_d of the "tube" samples have been presented in Table 8.5. Table 8.5 shows that the values of D_d of the "tube" samples varied from 0.14 to 0.36, 0.13 to 0.33 and 0.06 to 0.22 for the samples from Banskhali, Anwara and Chandanaish, respectively.

Table 8.5 Values of Degree of Disturbance for "Tube" Samplers of Different Wall Thickness (t)

Sample Designation	BT	BM	BH	AT	AM	AH	CT	CM	CH
Wall Thickness, t in mm	1.5	3	6	1.5	3	6	1.5	3	6
Degree of Disturbance, $D_d = 1-(\sigma'_i/\sigma'_{ps})$	0.14	0.21	0.36	0.13	0.18	0.33	0.06	0.10	0.22

Figs. 8.6, 8.7 and 8.8 show the plots of degree of disturbance versus ε_p, E_i/σ'_{vo} and E_{50}/σ'_{vo}, respectively for "tube" samples of the three coastal soils. Fig. 8.6 shows that for each soil, there is a trend of increase in the values of ε_p with increasing degree of disturbance. Figs. 8.7 and 8.8, however, show that the stiffness ratios (E_i/σ'_{vo} and E_{50}/σ'_{vo}) reduced sharply with the increase in the degree of disturbance. Similar behaviour as shown in Figs. 8.6, 8.7 and 8.8 has also been found by Okumura (1971) for Honmoku Marine Clay (LL = 93, PI = 51); and Siddique et al. (2000) for three coastal soils (LL = 43 to 57, PI = 18 to 33) of Bangladesh. Fig. 8.9 shows the plot of disturbed strength ratio (s_{ut}/s_{up}) versus degree of disturbance for nine "tube" samples of three coastal soils. s_{ut} and s_{up} are the undrained shear strength of the "tube" and the corresponding "perfect" sample, respectively. The values of s_{up} of the samples of Banskhali, Anwara and Chandanaish (i.e., samples BP, AP and CP, respectively) have been found to be 46.57 kN/m^2, 49.5 kN/m^2 and 53.0 kN/m^2, respectively as shown in Table 8.1. It can be seen from Fig. 8.9 that the disturbed strength ratio (s_{ut}/s_{up}) reduces with increase in degree of disturbance. The quantitative values of the degree of disturbance have also been found to be dependent upon the design parameters of the samplers used for sampling the clays. Degree of disturbance has been plotted against area ratio and D_e/t ratio as shown in Figs. 8.10 and 8.11, respectively. It can be seen from Fig. 8.10 that the values of D_d increased with the increasing values of area ratio. While the values of D_d, however, increased with decreasing values of D_e/t ratio of sampler as shown in Fig. 8.11. Degree of disturbance increased up to 2.6, 2.5 and 3.9 times for increase in area ratio (or decrease in D_e/t ratio between 27.3 and 8.3) between 16.4% and 73.1% for the samples from Banskhali, Anwara and Chandanaish, respectively. Increase in the degree of disturbance increased due to increase in area ratio (or decrease in D_e/t ratio) was also reported for reconstituted soft samples of Dhaka clay (Siddique and Sarker, 1996), soft samples of coastal soils (Siddique and Farooq, 1998; Siddique et al., 2000) and firm samples of Dhaka clay (Siddique and Rahman, 2000).

Marked increase in the degree of disturbance (measured in terms of tube sampling strains) with decreasing D_e/t ratio of sampler has also been analytically predicted (Baligh et al., 1987; Baligh, 1985). Clayton et al. (1998) from numerical finite element analyses also predicted considerable increase in the degree of disturbance with increasing area ratio (or decreasing D_e/t ratio).

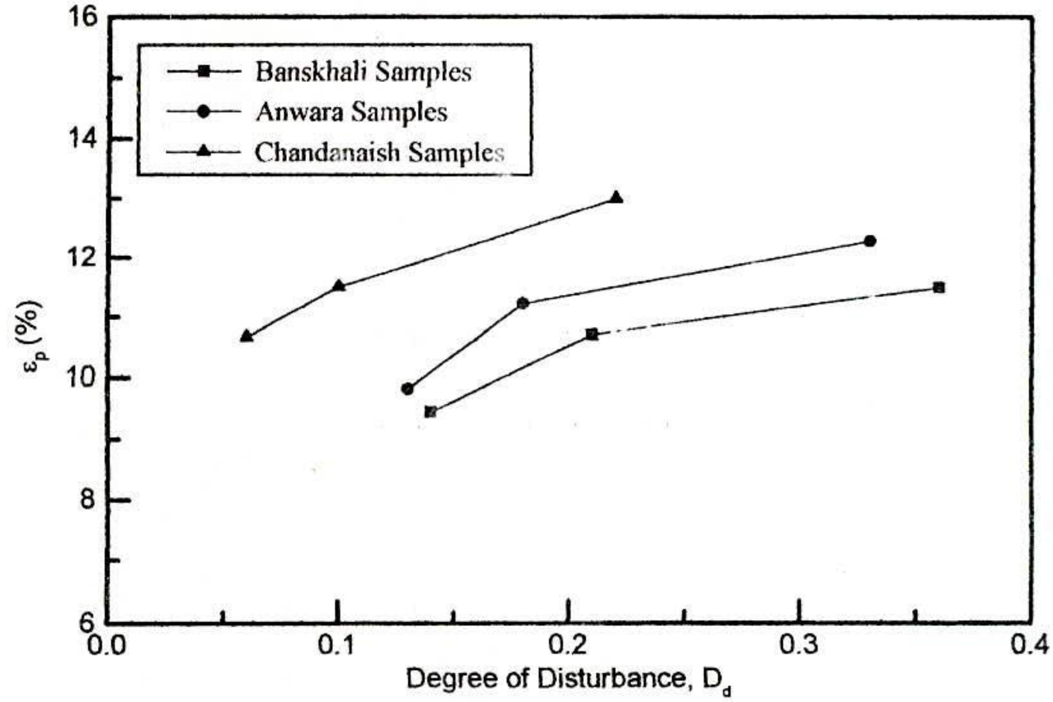


Fig. 8.6 Axial Strain at Peak Deviator Stress vs. Degree of Disturbance Plot for Samples of the Three Coastal Soils

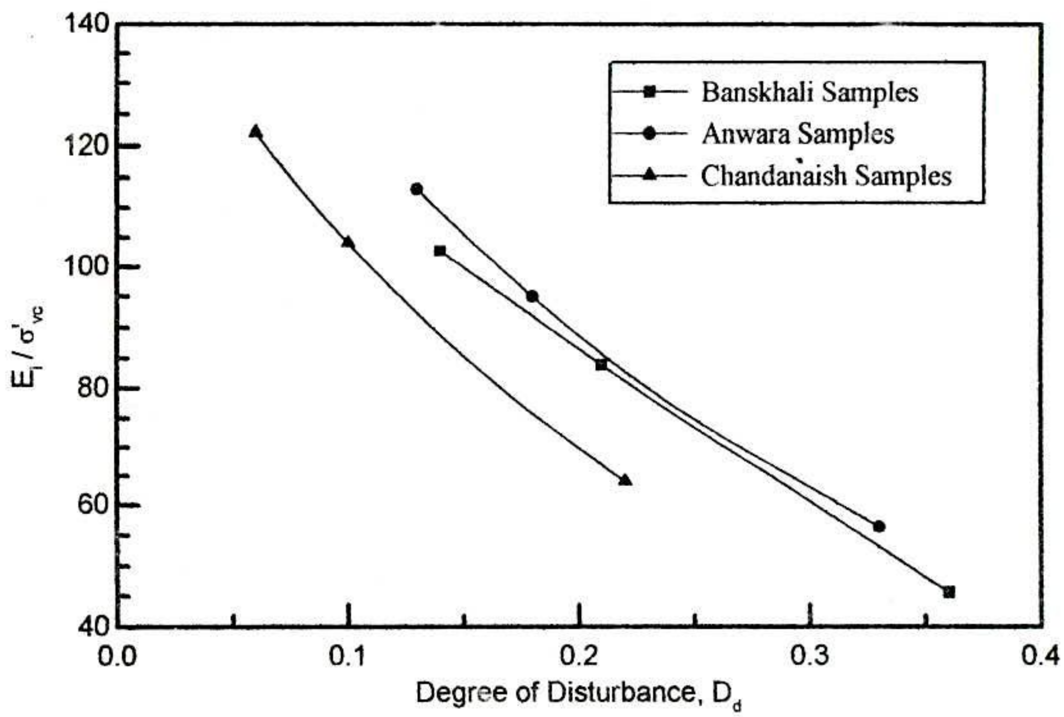


Fig. 8.7 Stiffness Ratio vs. Degree of Disturbance Plot for Samples of the Three Coastal Soils

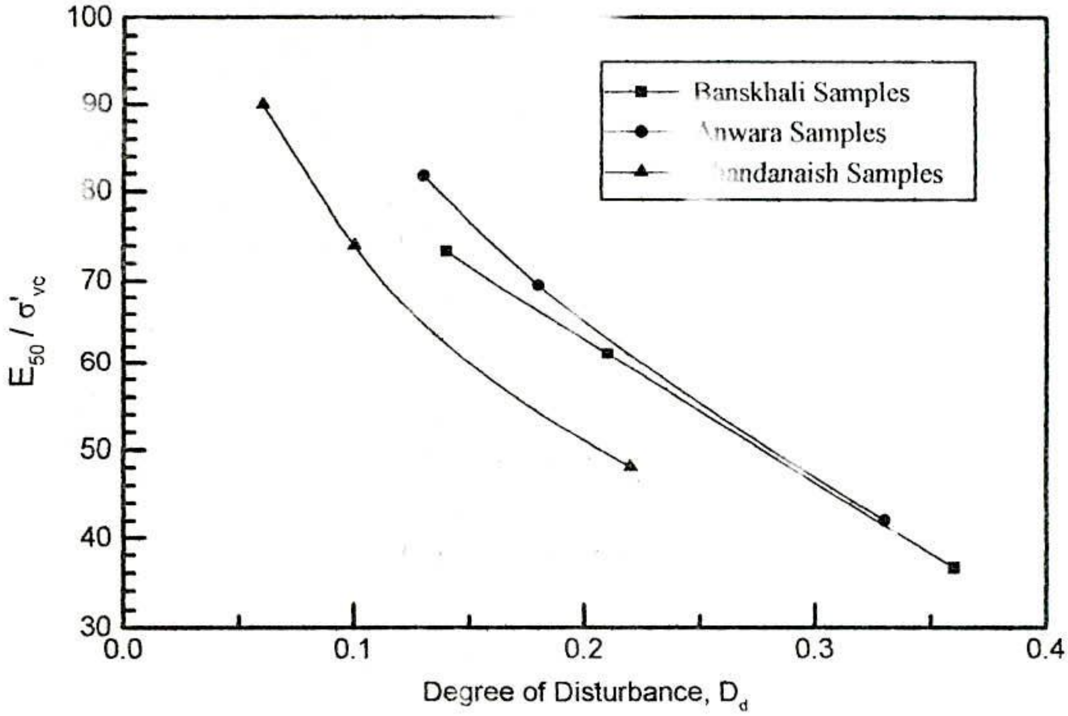


Fig. 8.8 Stiffness Ratio vs. Degree of Disturbance Plot for Samples of the Three Coastal Soils

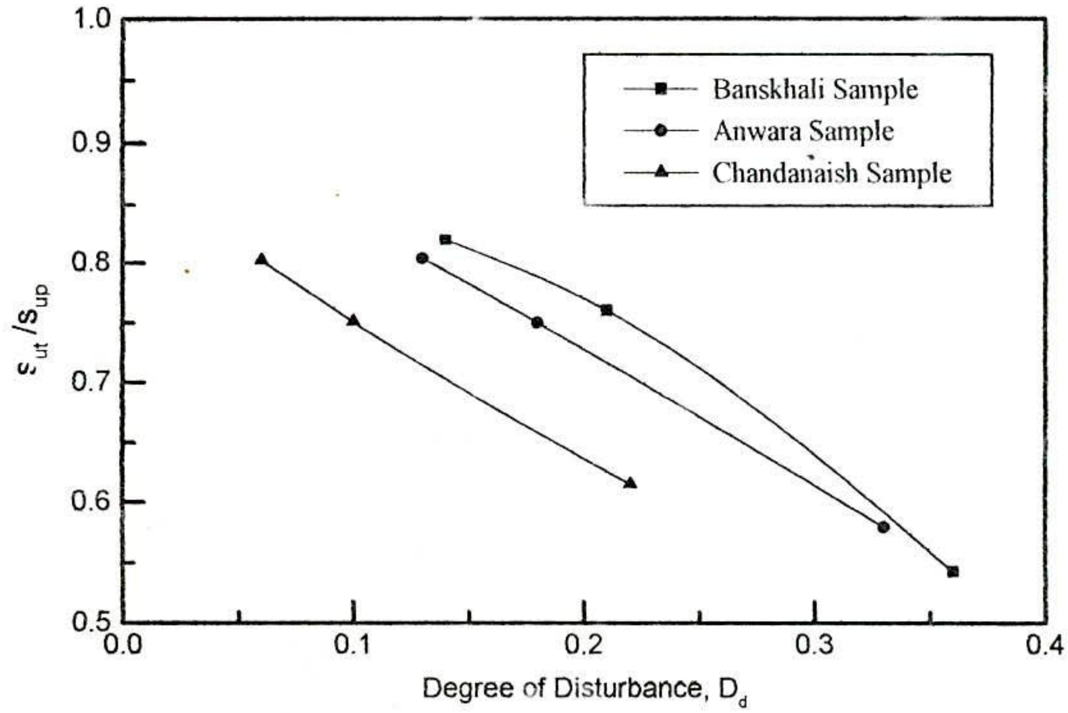


Fig. 8.9 Disturbed Strength Ratio vs. Degree of Disturbance Plot for Samples of the Three Coastal Soil

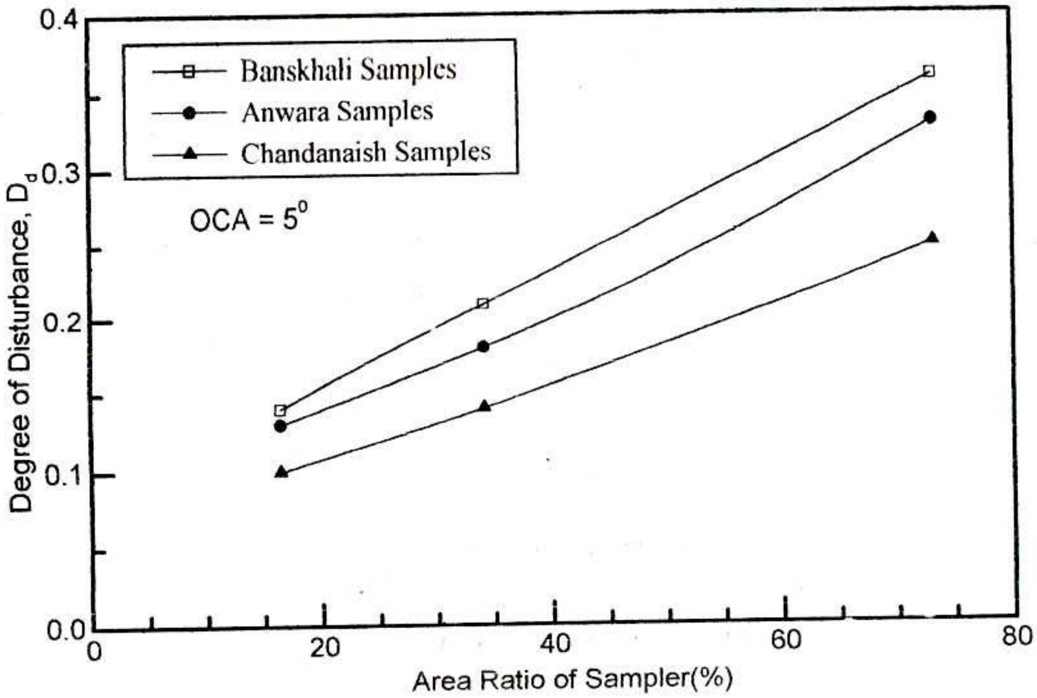


Fig. 8.10 Variation of Degree of Disturbance with Area Ratio of Sampler for Samples of the Three Coastal Soils

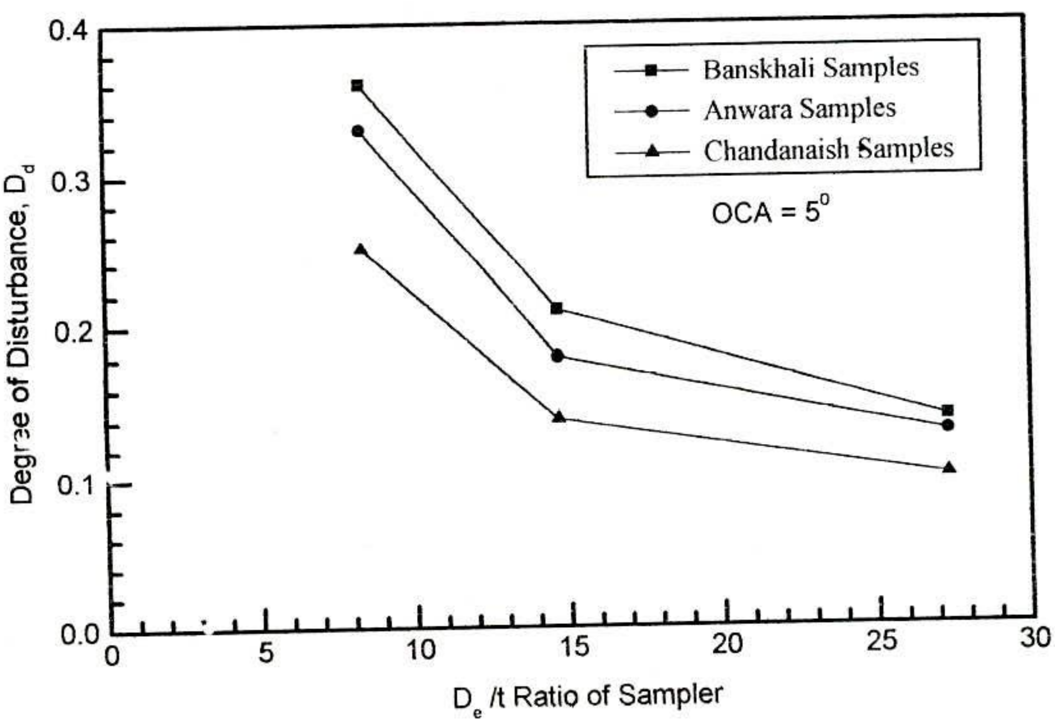


Fig. 8.11 Variation of Degree of Disturbance with D_e /t Ratio of Sampler for Samples of the Three Coastal Soils

8.5 Correction of Unconsolidated Undrained Shear Strength

It has been observed from the present study that because of sample disturbance undrained strengths of "tube" samples are always less than those of "in situ" samples. So, from practical point of view, although designs based on strength of soils obtained from laboratory tests of tube samples are on the safe side but it would lead to uneconomic and overdesign of structures. For optimum and economic design, undrained shear strength obtained from laboratory test on tube samples, therefore, should be corrected before being used in the design. In the present research, a correction procedure for estimating undrained shear strength of the coastal soils studied has been proposed. The relationships shown in Fig. 8.9, in conjunction with Figs. 8.10 and 8.11, can be used as for correcting the undrained shear strength of samples retrieved from the coastal region studied using sampling tubes of varying area ratio and D_e/t ratio. The proposed method of correcting the undrained shear strength for sample disturbance involves the following four steps:

(1) For a sample of given plasticity index (PI) and area ratio or D_e/t ratio of the sampler used for sampling, the degree of disturbance (D_d) is estimated from the curves shown in Fig. 8.10 or Fig. 8.11.

(2) From the value of D_d, the s_{ut}/s_{up} ratio of the tube sample is determined using the correction curves shown in Fig. 8.9

(3) Unconsolidated undrained (UU) triaxial compression test is to be performed on the sample to find its disturbed undrained shear strength, i.e., s_{ut}

(4) Dividing the undrained shear strength of the sample by the s_{ut}/s_{up} ratio of the sample, the perfectly undisturbed strength, i.e., s_{up} of the sample is obtained.

As an example, from Fig. 8.10 the value of D_d of a sample (PI = 16) obtained using a sampler tube of area ratio of 40% is approximately 0.2. From Fig. 8.9, the s_{ut}/s_{up} ratio is approximately 0.72 when D_d = 0.2. If the laboratory measured undrained shear strength of the sample is 40 kN/m^2, then the corrected undrained strength will be 55.5 kN/m^2. Correction of unconsolidated undrained strengths by use of the above procedure should be considered as an approximate engineering approach.

It is to be noted that the relationships shown in Figs. 8.9 to 8.11, however, can be used for coastal soil samples of plasticity index varying from 10 to 20 and samples retrieved with sampler tubes of different area ratio or D_e/t ratio but fixed OCA and ICR values of 5° and 0%, respectively. Further investigations may be carried out to establish correction curves for coastal soils of high plasticity and for samples retrieved with sampler tubes having various OCA and ICR values.

8.6 Effect of Soil Type on Disturbance

Disturbance due to tube sampling has been found to depend on the plasticity of the samples of the three coastal soils. From the data of Table 8.2, it can be seen that depending on soil type, the highest reduction in initial effective stress, σ'_i occurred in the least plastic Banskhali samples (PI = 10) whereas the minimum reduction in σ'_i occurred in the most plastic Chandanaish samples (PI = 20). Among the samples of the coastal soils, the least plastic (PI = 10) samples from Banskhali produced higher degree of disturbance than the most plastic (PI = 20) samples from Chandanaish. These results agreed well with those reported by Siddique et al. (2000) for reconstituted samples of three coastal soils. Compared with less plastic (PI = 20 ± 2.5) reconstituted Boston Blue clay (Baligh et al., 1987), Siddique (1990) also reported lesser degree of disturbance in highly plastic (PI = 45) reconstituted London clay due to application of tube sampling strains.

Fig. 8.12 also shows the variation of degree of disturbance with plasticity index for different D_e/t ratios. From Fig. 8.12 it is observed that for a particular D_e/t ratio, degree of disturbance increases with the decrease of plasticity index.

8.7 Influence of Reconsolidation of "Tube" Samples

Isotropic and anisotropic reconsolidation procedures were adopted in order to assess the suitability of different reconsolidation procedure to minimize the sampling disturbance effects in samples of the reconstituted coastal soils. Reconsolidation was carried out on samples from Banskhali, Anwara and Chandanaish retrieved using the sampler of area

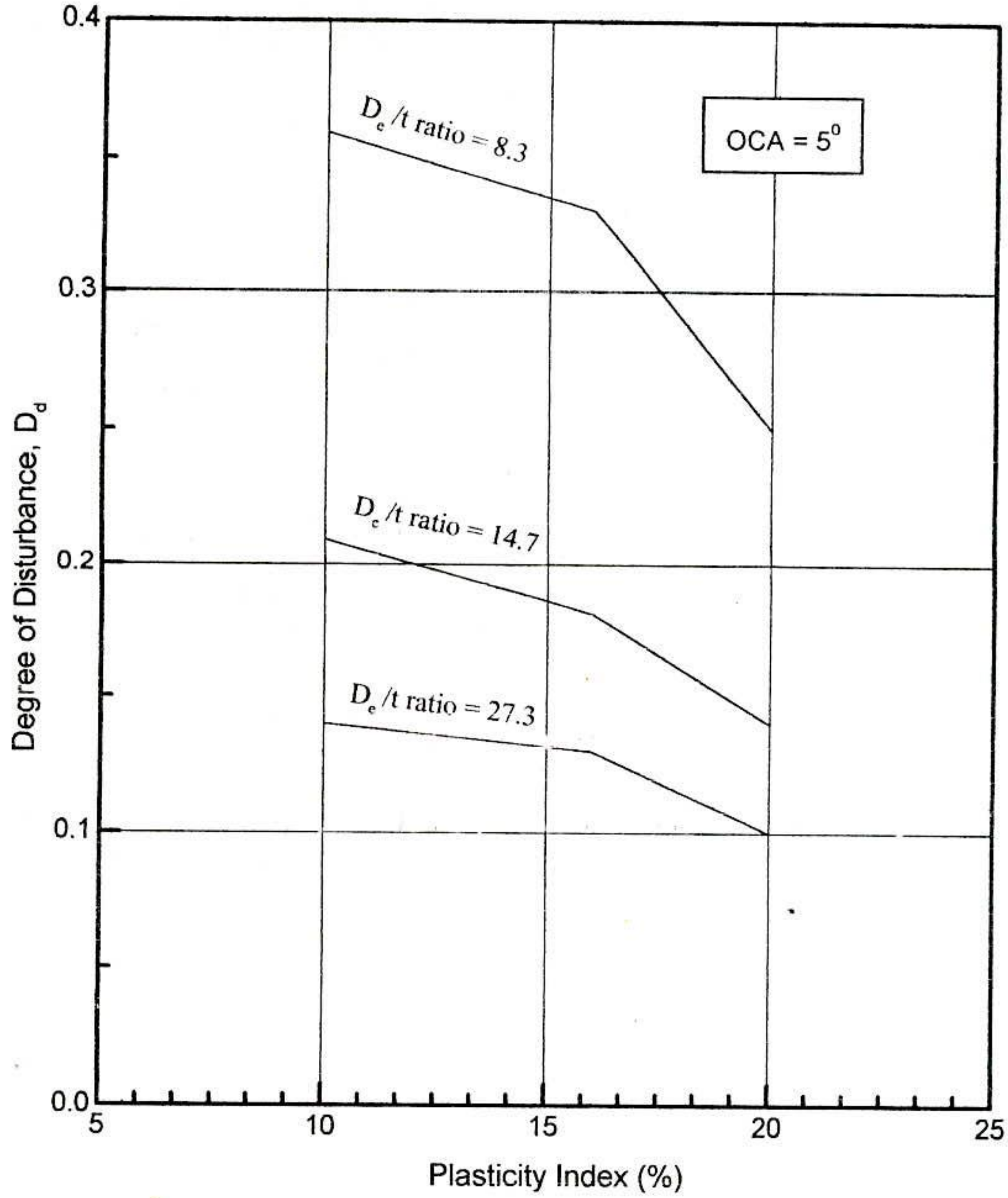


Fig. 8.12 Degree of Disturbance vs. Plasticity index Plots with Different D_e /t Ratio for Three Coastal Samples

ratio = 34.1% (t = 3 mm) and OCA = 5°. The reconsolidation techniques mentioned in Article 4.7 were used for the " tube" samples. The undrained soil parameters of the reconsolidated "tube" samples have been determined from stress-strain and pore pressure data.

8.7.1 Normalized Effective Stress Paths

The normalized effective stress paths of the reconsolidated "tube" samples from Banskhali, Anwara and Chandanaish are presented in Figs. 8.13 to 8.15, respectively. In Figs. 8.13 to 8.15, the corresponding plots for the "in situ" samples are shown for comparison with the reconsolidated samples. From these figures it can be observed that the effective stress path of the "tube" sample reconsolidated under K_0-condition to vertical effective stress equal to in situ vertical effective stress, σ'_{vc}, i.e., Bjerrum procedure, compares better with the "in situ" samples of the three soils.

8.7.2 Stress-Strain-Strength and Stiffness Properties

Fig. 8.16 shows a typical comparison of normalized deviator stress as a function of axial strain of the reconsolidated "tube" samples with the "in situ" sample from Banskhali. It can be seen from Fig. 8.16 that deviator stress versus axial strain plot of the "tube" sample reconsolidated using Bjerrum (1973) procedure produced the better overall estimate with the "in situ" sample. Similar results were also found for samples of the other soils from Anwara and Chandanaish.

Undrained shear strength (s_u), initial tangent stiffness (E_i), secant modulus at half the deviator stress (E_{50}) and axial strain at peak deviator stress (ε_p), have been determined from the stress-strain data for the "in situ" and reconsolidated "tube" samples of the three soils. A comparison of normalized undrained shear parameters between the "in situ" and reconsolidated "tube" samples is presented in Table 8.6.

It can be seen from Table 8.6 that isotropic reconsolidation, using in situ vertical effective stress equal to σ'_{vc} (i.e., 150 kN/m^2), has the effect of grossly overestimation of "in situ" strength ratio (s_u/σ'_{vo}) and ε_p for all the soils. Values of s_u/σ'_{vo} and ε_p have been

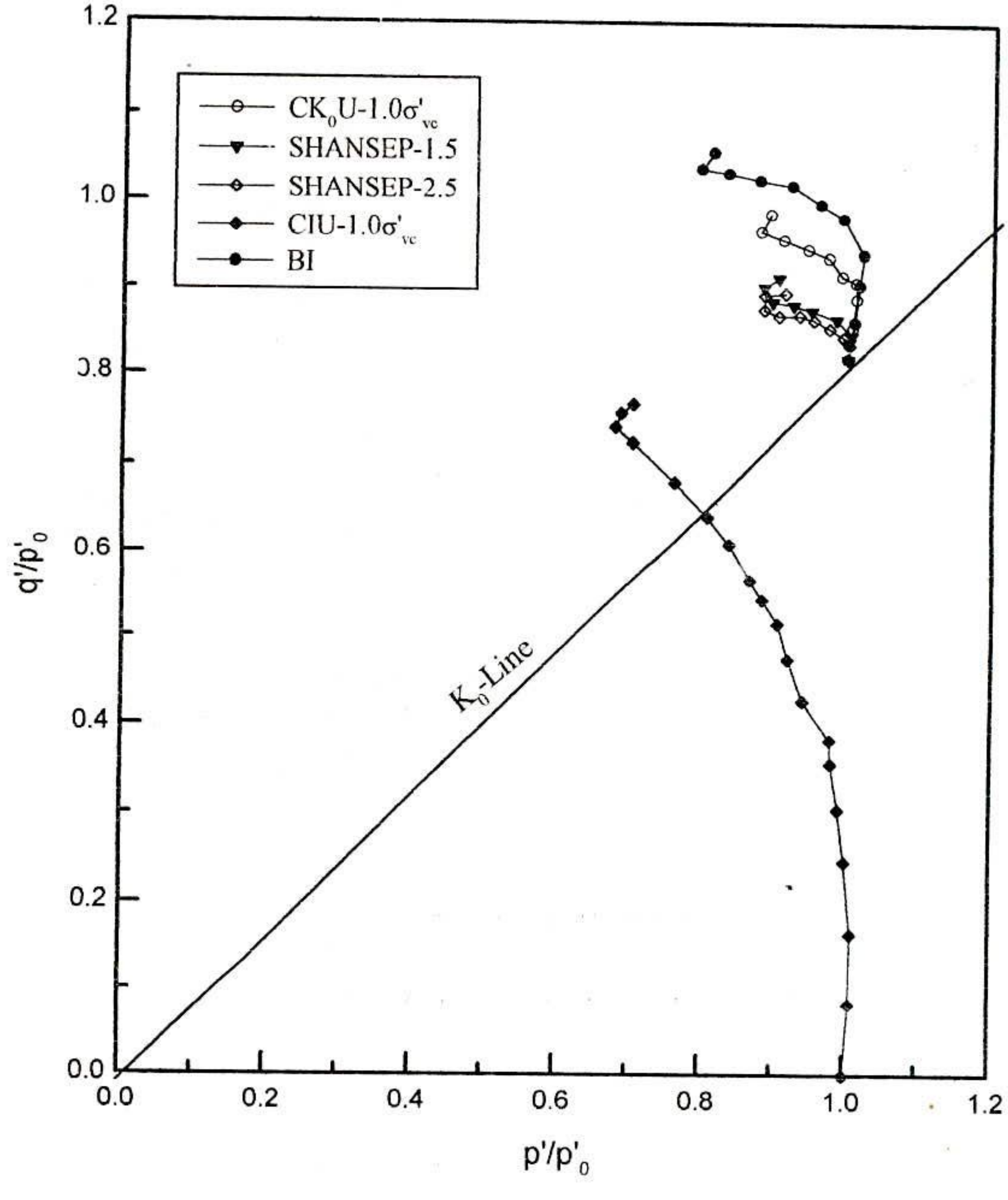


Fig. 8.13 Comparison of Normalized Effective Stress Paths of "In Situ" and reconsolidated "Tube" Samples from Banskhali

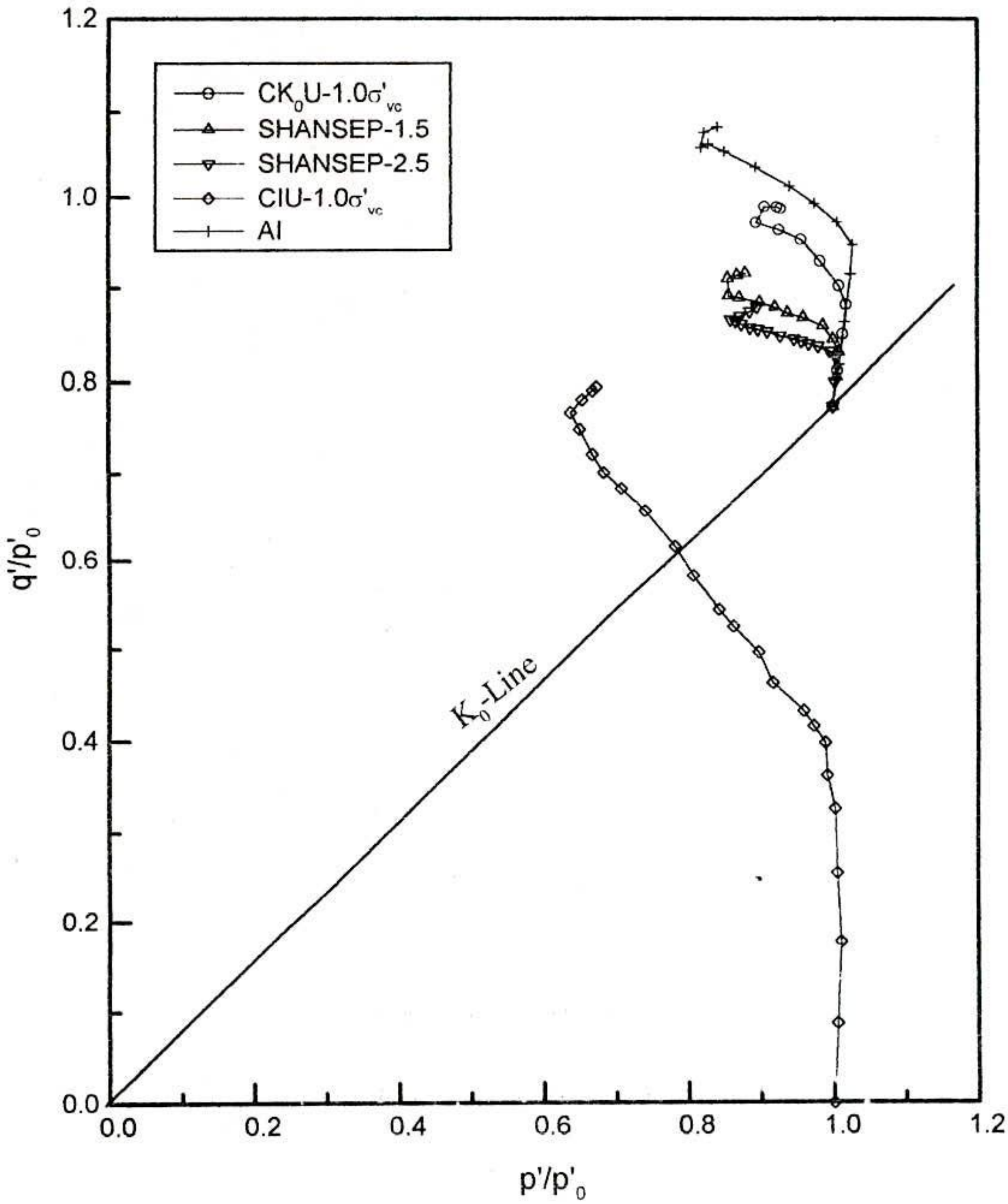


Fig. 8.14 Comparison of Normalized Effective Stress Paths of "In Situ" and Reconsolidated "Tube" Samples from Anwara

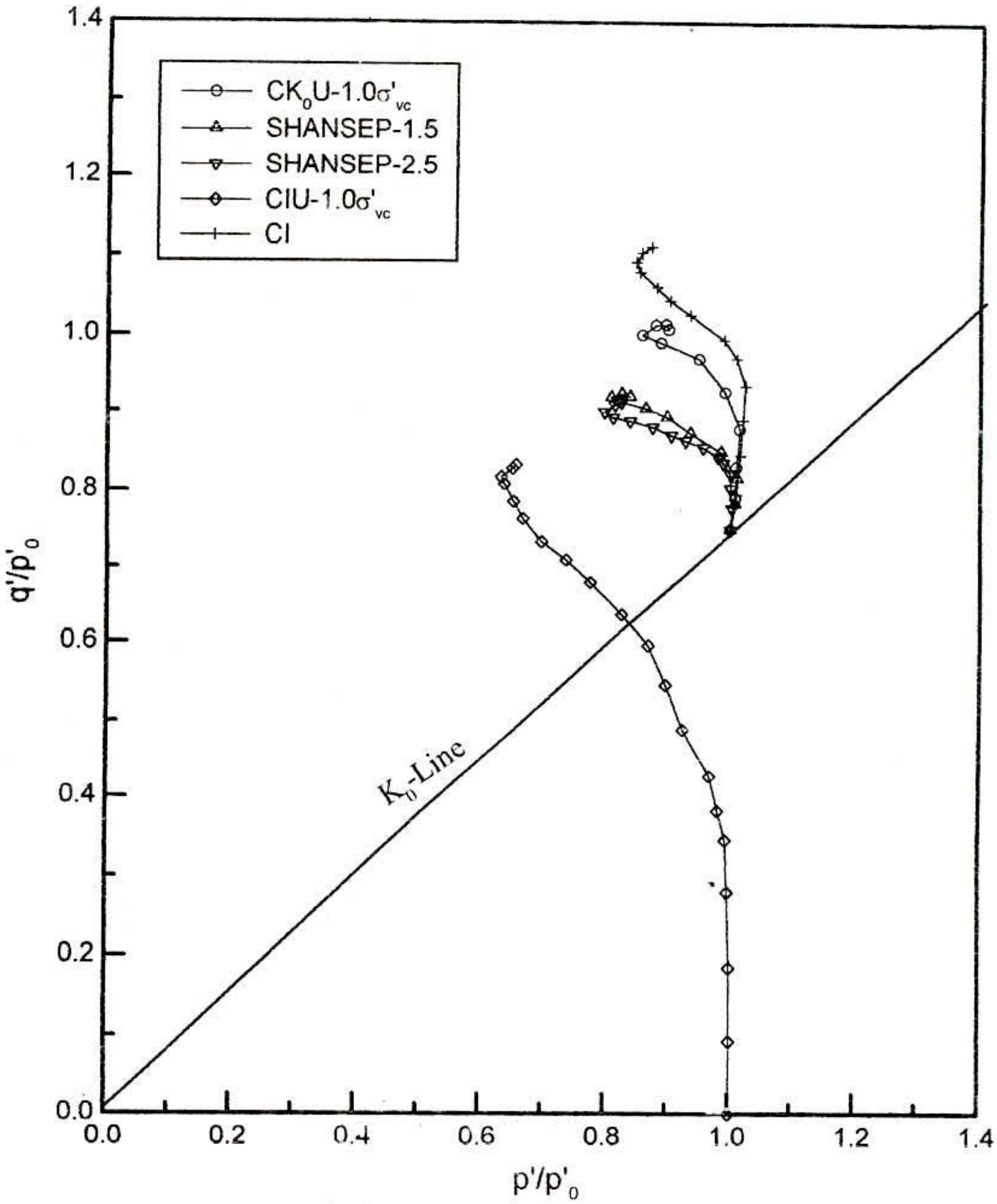


Fig. 8.15 Comparison of Normalized Effective Stress Paths Between "In Situ" and Reconsolidated "Tube" Samples from Chandanaish

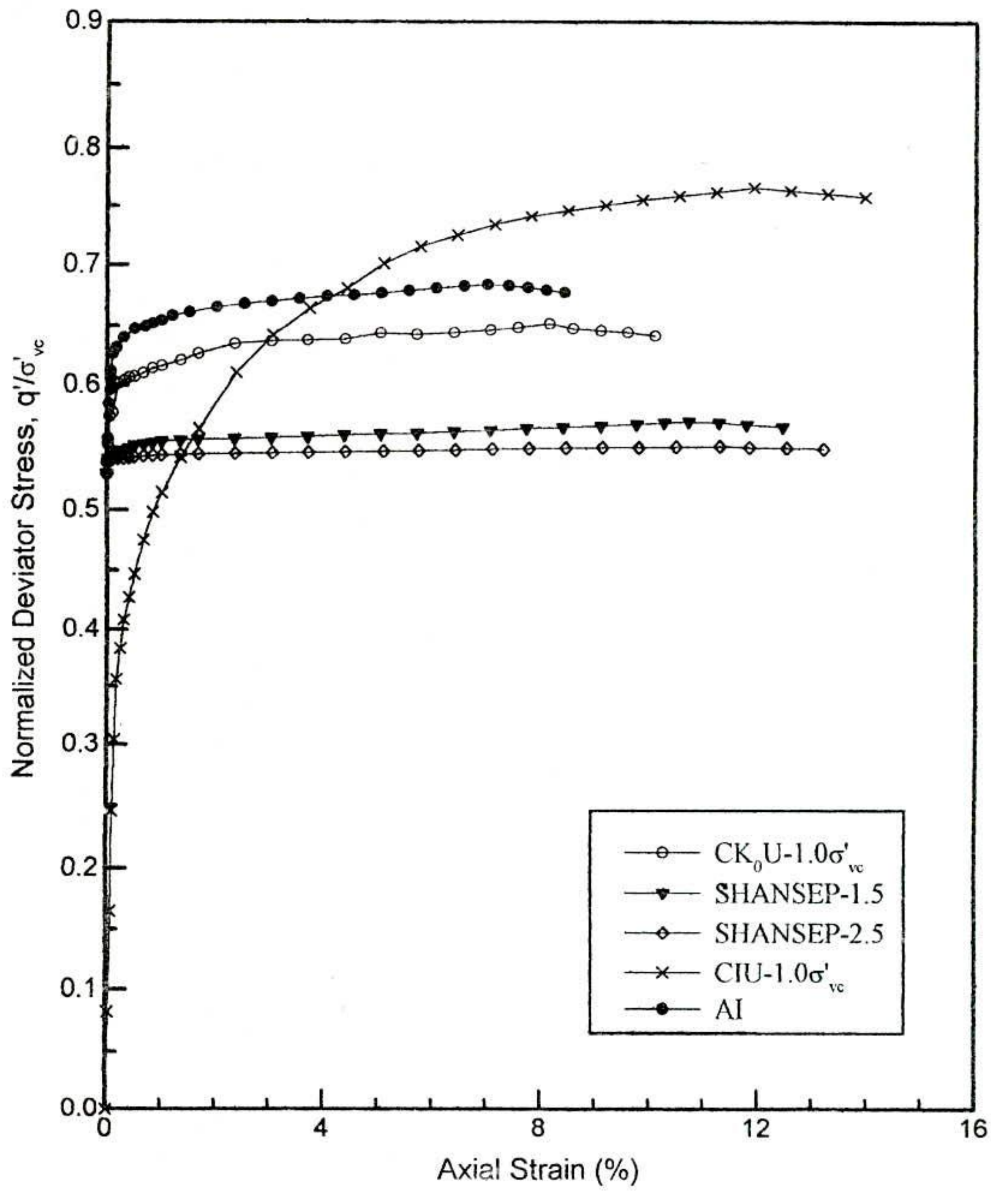


Fig. 8.16 Comparison of Normalized Deviator Stress vs. Axial Strain Plots for "In Situ" and Reconsolidated "Tube" Samples from Banskhali

Table 8.6 Comparison of Undrained Shear Characteristics of "In Situ" and Reconsolidated "Tube" Samples of the Three Coastal Soils

Sample Designation	Test Type	s_u/σ'_{vo}	ε_p (%)	E_i/σ'_{vo}	E_{50}/σ'_{vo}	A_p
BM	CIU-1.0 σ'_{vc}	0.383	11.92	242.5	159.15	0.802
	CK_0U-1.0 σ'_{vc}	0.31	8.15	141.23	102.4	0.745
	SHANSEP - 1.5	0.285	10.72	117.58	89.53	1.12
	SHANSEP - 2.5	0.276	11.3	97.53	84.49	1.24
	BI	0.34	7.0	163.8	124.8	0.76
AM	CIU-1.0 σ'_{vc}	0.408	12.7	256.4	170.67	0.772
	CK_0U-1.0 σ'_{vc}	0.327	8.9	150.3	112.5	0.723
	SHANSEP - 1.5	0.302	11.25	125.2	101.17	1.18
	SHANSEP - 2.5	0.29	13.5	101.73	89.10	1.26
	AI	0.357	7.8	175.2	132.3	0.74
CM	CIU-1.0 σ'_{vc}	0.42	12.62	276.0	189.0	0.74
	CK_0U-1.0 σ'_{vc}	0.34	9.72	161.0	127.0	0.726
	SHANSEP - 1.5	0.31	11.47	133.0	110.0	1.26
	SHANSEP - 2.5	0.304	13.10	107.0	91.0	1.42
	CI	0.37	8.8	184.0	147.0	0.71

overestimated up to 14.3% and 70.3%, respectively. Compared with the "in situ" samples, the values of stiffness ratios, E_i/σ'_{vo} and E_{50}/σ'_{vo} increased significantly for the "tube"samples reconsolidated using isotropic pressure equal to σ'_{vc}. Values of E_i/σ'_{vo} and E_{50}/σ'_{vo} increased up to 50% and 29%, respectively due to reconsolidation using an isotropic pressure equal to σ'_{vc}. Siddique et al. (2000) also reported that s_u/σ'_{vo}, ε_p and E_i/σ'_{vo} increased due to isotropic reconsolidation in two normally consolidated soft samples of coastal soils. Similar results have been found by Siddique and Rahman (2000) for firm Dhaka clay.

Compared with "in situ" samples, the ratios s_u/σ'_{vo}, E_i/σ'_{vo} and E_{50}/σ'_{vo} of "tube" samples reconsolidated using Bjerrum (1973) procedure, SHANSEP-1.5 and SHANSEP-2.5 reduced significantly. The undrained strength ratio (s_u/σ'_{vo}) for the "tube" samples from Banskhali reconsolidated using Bjerrum (1973), SHANSEP-1.5 and SHANSEP-2.5 procedures reduced by 8.82%, 16.2% and 18.82%, respectively. The respective reductions for the samples from Anwara are 8.4%, 15.4% and 18.77%, respectively while those for the samples from Chandanaish are 8.11%, 16.2% and 17.84%, respectively. The undrained stiffness ratio (E_i/σ'_{vo}) reduced by 13.78%, 28.2% and 40.5% for "tube" samples from Banskhali reconsolidated using Bjerrum (1973), SHANSEP-1.5 and SHANSEP-2.5 procedures, respectively. The respective reductions for the samples from Anwara are 14.2%, 28.54% and 41.9%. The stiffness ratio (E_i/σ'_{vo}) reduced by 12.5%, 27.72% and 43.95% for "tube" samples from Chandanaish reconsolidated using Bjerrum, SHANSEP-1.5 and SHANSEP-2.5 procedures, respectively. The undrained stiffness ratio (E_{50}/σ'_{vo}) reduced for samples from Banskhali by 17.95%, 28.3% and 32.3%; for samples from Anwara by 14.97%, 23.53% and 32.65%; and, for samples from Chandanaish by 13.61%, 25.2% and 38.1% due to reconsolidation using the above Bjerrum, SHANSEP-1.5 and SHANSEP-2.5 procedures, respectively.

The values of ε_p of all the "perfect" samples reconsolidated using above Bjerrum (1973), SHANSEP-1.5 and SHANSEP-2.5 procedures, however, increased. The values of ε_p increased for samples from Banskhali by 16.4%, 53.14% and 61.4%; for samples from Anwara by 12.36%, 44.23% and 73.1%; and for samples from Chandanaish by 10.34%, 30.34% and 48.86% due to reconsolidation using the Bjerrum, SHANSEP-1.5 and SHANSEP-2.5 procedures, respectively.

From the above comparisons of the values of s_u/σ'_{vo}, E_i/σ'_{vo}, E_{50}/σ'_{vo} and ε_p between the "in situ" and reconsolidated "tube" samples, it is evident that for each soil, despite all the anisotropic reconsolidation procedures including SHANSEP procedures provided a lower bound strength and stiffness, and an upper bound axial strain at peak deviator stress, the values of s_u/σ'_{vo}, E_i/σ'_{vo}, E_{50}/σ'_{vo} and ε_p of the "tube" sample reconsolidated using Bjerrum procedure (i.e., reconsolidation under K_0-condition to vertical effective stress equal to in situ vertical effective stress, σ'_{vc}) produced the best agreement between the "tube" and "in situ" samples in terms of the strength, deformation and stiffness properties than the "tube" samples reconsolidated using SHANSEP-1.5 and SHANSEP-2.5 procedures. Siddique and Rahman (2000) reported that compared with SHANSEP reconsolidation procedures, the soil parameters of "tube" samples reconsolidated using Bjerrum procedure (CK_0U-$1.0\sigma'_{vc}$) agrees more closely with those of the respective "in situ" samples than those of the samples reconsolidated using SHANSEP procedures. Siddique et al. (2000) reported that reconsolidation of "tube" samples using SHANSEP-1.5 procedure produced better agreement with the characteristics of the respective "in situ" samples for the reconstituted normally consolidated two coastal soils. Sarker (1994) reported that K_0-reconsolidation of "tube" samples using the SHANSEP procedures could not restore "in situ" behaviour of soft Dhaka clay. Siddique et al. (2000) and Sarker (1994), however, did not adopt the reconsolidation of "tube" samples using Bjerrum procedure in their investigations.

8.7.3 Pore Pressure Responses

Typical plottings of the variation of Skempton's pore pressure parameter, A with axial strain are presented in Fig. 8.17 for reconsolidated "tube" samples from Anwara. In Fig. 8.17, the corresponding plot for the"in situ"sample are also shown for comparison with the reconsolidated samples. It can be seen from Fig. 8.17 that the pore pressure response (as evaluated in terms of Skempton's pore pressure parameter, A) of the "tube" sample reconsolidated using Bjerrum (1973) procedure (i.e., consolidation under K_0-condition to vertical effective stress equal to in situ vertical effective stress, σ'_{vc}) produced the best agreement with the "in situ" sample than the "tube" samples reconsolidated using other techniques. Skempton's pore pressure parameter, A at peak deviator stress (A_P) has been determined from stress-strain and pore pressure data. Similar results were also found for

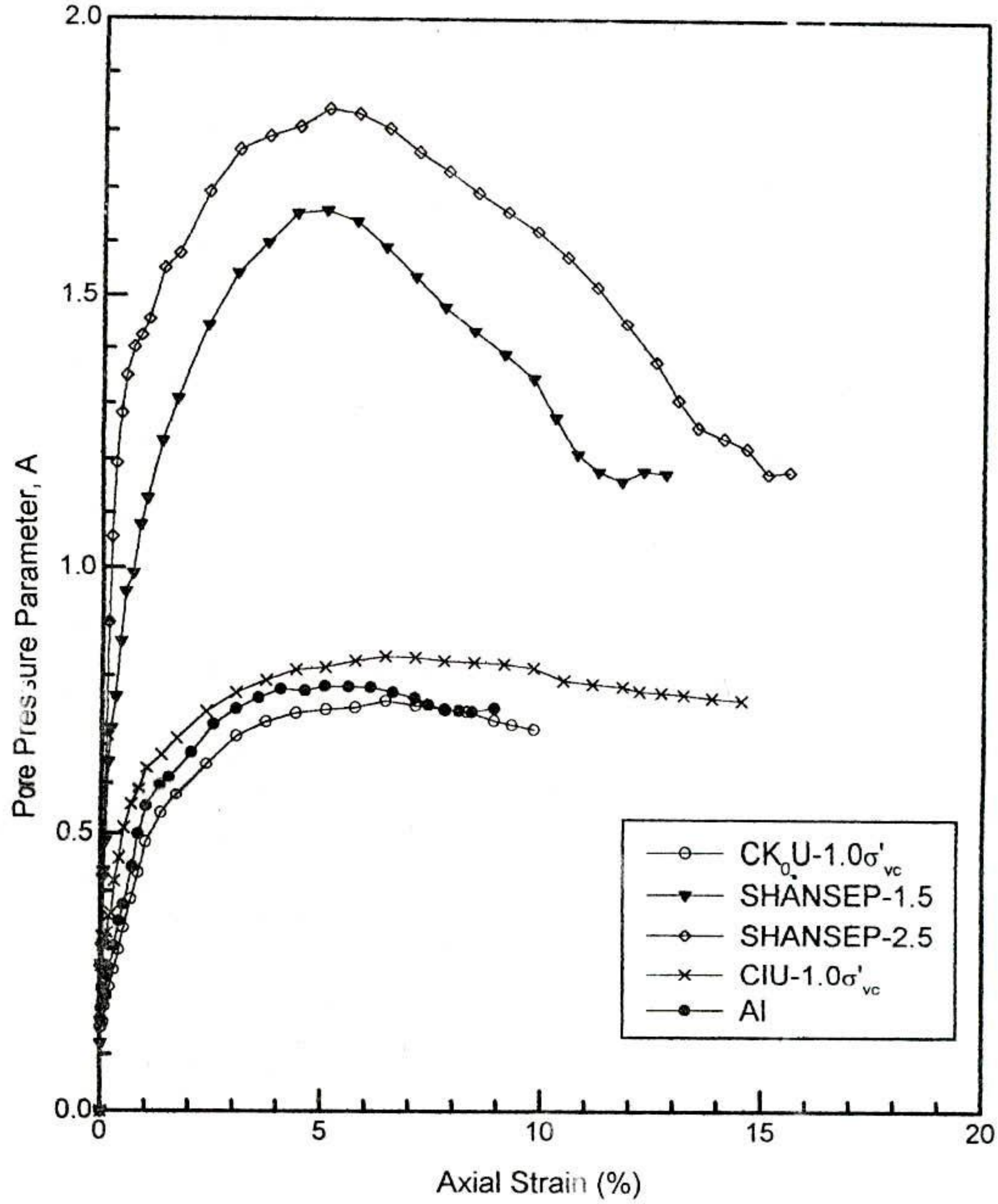


Fig. 8.17 Pore Pressure Parameter, A vs. Axial Strain Plots for "In Situ" and Reconsolidated "Tube" Samples from Anwara

other soils from Banskhali and Chandanaish. The values of A_p for all samples of the three soils have already been shown in Table 8.5. The following points can be summarized from Table 8.5:

- Isotropic reconsolidation with pressure equal to σ'_{vc}, overestimated (up to 5.53%) the values of A_p.
- Anisotropic reconsolidation using Bjerrum (1973) procedure slightly underestimated or overestimated (up to 2.25%) the values of A_p.
- Anisotropic reconsolidation using SHANSEP-1.5σ'_{vc} and SHANSEP-2.5σ'_{vc} procedures grossly overestimated (up to 77.5% and 100%, respectively) the values of A_p.

From the above comparisons, it is evident that the values of A_p of the "tube" samples reconsolidated using Bjerrum (1973) procedure provided the better estimate of "in situ" sample than the "tube" samples reconsolidated using other techniques.

Siddique and Rahman (2000) reported that the value of A_p increased significantly due to isotropic reconsolidation of "tube" samples of Dhaka clay, while Siddique et al. (2000) reported that the value of A_p decreased due to isotropic reconsolidation of "tube" samples of two coastal soils. The contradictory results might be possible due to variation of reconsolidation pressure. Siddique and Rahman (2000) considered effective overburden pressure as a isotropic reconsolidation pressure, whereas Siddique et al. (2000) considered p'_0 as isotropic reconsolidation pressure. Siddique and Rahman (2000) reported that compared with reconsolidation procedures, the value of A_p of "tube" samples reconsolidated using Bjerrum procedure (CK_0U-1.0σ'_{vc}) agrees more closely with the respective value of "in situ" samples of Dhaka clay. Siddique et al. (2000) reported that reconsolidation of "tube" samples using SHANSEP-1.5 procedure produced better agreement with the respective value of A_p of the "in situ" samples for the reconstituted normally consolidated two coastal soils. Sarker (1994) reported that K_0-reconsolidation of "tube" samples using the SHANSEP procedures could not restore "in situ" behaviour of soft Dhaka clay in case of A_p. Siddique et al. (2000) and Sarker (1994), however, did not adopt the reconsolidation of "tube" samples using Bjerrum procedure in their investigations.

CHAPTER 9

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE STUDY

9.1 Conclusions

In the present study, undrained strength-deformation, compressibility and intrinsic properties of three reconstituted samples of Chittagong coastal soils of Bangladesh have been investigated. Attempts have been made to evaluate normalized soil engineering properties for these soils. Models to predict undrained shear strength were also developed for normally consolidated and overconsolidated soils. Deformation characteristics of K_0 and isotropically consolidated samples have been evaluated for reconstituted soils. Using Burland's (1990) concept, a general intrinsic compression curve has been established which may be used to determine compressibility of soils at any depth of known overburden pressure. From the stress paths of normally consolidated and overconsolidated reconstituted samples of the three soils, State Boundary Surfaces were established. Some constitutive models showing the relationship between critical state parameters and plasticity index of the soils were also developed.

In addition to evaluation of stress deformation properties, sampling effects in these three coastal soils have also been assessed. The effects of "perfect" sampling disturbance and "tube" sampling disturbance on undrained shear characteristics have been examined for reconstituted normally consolidated soil samples. The influence of the sampler characteristics, namely area ratio and D_e/t (external diameter to wall thickness) ratio on the measured soil parameters has also been examined. Attempt has been made to examine the suitability of different reconsolidation procedures to minimize the "perfect" and "tube" sampling disturbance effects. The main findings have been outlined into three sections relating to the following areas:

(a) Stress-deformation-strength properties, compressibility and swelling characteristics and intrinsic properties of reconstituted samples of three coastal soils.

(b) Effects of "perfect" and tube sampling on undrained shear properties of reconstituted samples of three coastal soils and also the influence of sampler geometry on the measured undrained shear parameters of the reconstituted samples of the three coastal soils.

(c) Assessment of various reconsolidation techniques to minimize "perfect" and tube sampling disturbance effects in the three coastal soils.

On the basis of experimental results, the following major findings and conclusions can be drawn:

(i) The general equation $s_u^{(oc)}/\sigma'_{vo} = [s_u^{(nc)}/\sigma'_{vo}] \times (OCR)^{\Lambda}$ proposed by Mitachi and Kitago (1976) has been found to be valid for the three coastal soils to determine the undrained shear strength of overconsolidated samples under isotropic stress condition.

(ii) The intrinsic compression line for the three coastal soils found from the intrinsic values almost coincided with the ICL values for most clays furnished by Burland (1990). This curve can be used as a basic frame of reference for the compressibility of natural soils.

(iii) For the reconstituted normally consolidated samples of the soils s_u, ε_p, E_i, E_{50} increased with the increase of plasticity while A_p decreased with the increase of plasticity of the soils.

(iv) For the reconstituted overconsolidated soils, s_u, ε_p, E_i/s_u, E_{50}/s_u, A_p decreased with the increase of OCR for each soil. Strength ratio (s_u/σ'_{vo}) and Stiffness ratios (E_i/σ'_{vo} and E_{50}/σ'_{vo}), however, increased with the increase of OCR. At higher OCR, the rate of increase of s_u/σ'_{vo}, E_i/σ'_{vo} and E_{50}/σ'_{vo} was highly pronounced. On the other hand, E_i/s_u and E_{50}/s_u decreased with the increase of OCR for each soil.

(v) The relations $E_i = k\ s_u$ and $E_{50} = \psi\ s_u$ can be applied to coastal soils to represent their initial tangent modulus and secant modulus.

(vi) From the comparative study between the experimental results and those predicted using theoretical models, it has been found that the Modified Cam clay model compares more favourably with the experimental results than the Cam clay model in terms of both stress-strain curves and effective stress paths for each soil.

(vii) Compared with the "in situ" samples, undrained strength, s_u, the initial tangent modulus, E_i, secant modulus at half the peak deviator stress, E_{50}, pore pressure parameter at peak deviator stress, A_p of the "perfect" and "tube" samples decreased while the values of axial strain at peak deviator stress, ε increased for the three samples. It is also evident that the initial effective stress σ'_i decreased in case of "tube sampling disturbance and mean effective stress, p' decreased in case of "perfect" sampling in comparison with the "in situ" samples.

(viii) It was found that the nature of the effective stress paths and pore pressure responses of the "perfect" and "tube" samples were markedly different from the "in situ" samples. The "perfect" and "tube" samples showed stress paths and pore pressure responses similar to those for overconsolidated samples.

(ix) The extent of disturbances due to "perfect" and "tube" sampling has been found to depend on the type of soil. The less plastic samples from Banskhali (PI = 10) suffered larger reductions in s_u, E_i, E_{50} and p' and σ'_i than the more plastic samples from [illegible]danaish (PI = 20). The values of D_d of the "tube" samples have also been found to be higher in less plastic samples than in more plastic samples.

(x) The findings of the previous and present investigations on coastal soils clearly demonstrate that the design of a sampler tube has profound influence on sample disturbance. In order to minimize disturbance due to sampling in coastal soils, area ratio of sampler should be kept as low as possible. From practical point of view, the area ratio of a tube sampler should not exceed 10%.

(xi) The experimental results of the present investigation agree favourably with those reported previously by Farooq (1995), Siddique and Farooq (1996) and Siddique et al. (2000), for three coastal soils of higher plasticity and thereby confirm and validate the previous works on coastal soils of Bangladesh. The present results therefore illustrated that the behaviour of the coastal soils studied is, in general, similar to those of other coastal soils of Bangladesh. The results of the present investigation also indicated that the effects of "perfect" and "tube" sampling disturbance on undrained stress-strain-strength, stiffness and pore pressure characteristics in the coastal soils are similar to those in other soils.

(xii) The experimental results of the present investigation indicated that the assessment of the effects of tube sampling disturbance in the coastal soils is important and should be recognized specially in connection with the evaluation of undrained shear strength for these soils. A method for correcting undrained shear strength suggested in this research work may be utilized for this purpose.

(xiii) In each soil, isotropic reconsolidation of "perfect" and "tube" samples using a pressure equal to vertical effective consolidation pressure (σ'_{vc}) has the effect of grossly overestimation of "in situ" strength, stiffnesses E_i and E_{50}, ε_p and A_p, while isotropic reconsolidation of "perfect" samples using a pressure equal to σ'_{ps} results in substantial underestimation of "in situ" strength, E_i, E_{50} and A_p and overestimation of ε_p for the three coastal soils.

(xiv) The results indicate that all the anisotropic reconsolidation techniques provided lower bound strength and stiffness ratios than those of "in situ" samples. Reconsolidation of "perfect" and "tube" samples using SHANSEP procedures (K_o-consolidation to vertical effective stress equal to 1.5 times and 2.5 times the in situ vertical effective stress, σ'_{vc}) considerably underestimated the values of strength and stiffness ratios, and overestimated the values of ε_p and A_p. However, reconsolidation of the "perfect" and "tube" samples using Bjerrum (1973) procedure slightly underestimated in situ strength ratio and stiffness ratios, slightly underestimated or overestimated the values A_p, and overestimated to some extent the values of ε_p. Compared with the "in situ" samples Bjerrum (1973) procedure provided the results with the lesser variation than all other reconsolidation procedures in all respects. So it may be concluded that the anisotropic reconsolidation to σ'_{vc} under K_0-condition, i.e., Bjerrum procedure produced the best agreement between the "in situ" and "perfect" or "tube" samples from Banskhali, Anwara and Chandanaish in terms of strength, deformation, stiffness, and pore pressure responses. It, therefore, appears from the present investigation that reconsolidation using SHANSEP procedures may not be applicable to these coastal soils. In order to determine the undrained shear parameters of these coastal soils from triaxial testing on samples retrieved from ground, the samples should first be reconsolidated using Bjerrum (1973) procedure before being sheared up to failure.

9.2 Recommendations for Future Study

Several aspects of the work presented in this thesis require further study. Some of the important areas of further research may be listed as follows:

(i) In this research, reconstituted samples have been used to develop models of undrained shear strength of overconsolidated and normally consolidated soils. This research may be extended using natural undisturbed coastal soils in order to make comparisons between the behaviour of reconstituted and intact soils.

(ii) Three coastal soils have been used in this research to develop constitutive models for critical state soil parameters. Research on more soils from different coastal belts of Bangladesh may be carried out to verify and validate these models. Such studies would also lead to develop a generalized frame work of behaviour of coastal soils of Bangladesh.

(iii) The scope of present research has been limited to investigating the effects of sampling disturbance in reconstituted unaged or young coastal soils. The fabric of natural soils may have a significant influence on the behaviour of soils and hence, further research is required on natural soils to identify any special features associated with fabric, composition, bonding and ageing.

(iv) The scope of the testing programme has been limited to investigating only the "perfect" and tube sampling disturbance effects in reconstituted normally consolidated unaged coastal soils. Further study can be carried out to investigate the influence of "ideal" sampling disturbance in coastal soils.

(v) Effects of area ratio or D_e/t ratio of sampler on disturbance have been investigated. Besides this, the effects of other design parameters on sampling disturbance can be investigated. Such design parameters may include variation of outside cutting edge angle, inside clearance ratio, outside clearance ratio, inside cutting edge angle and variation of sampler diameter.

(vi) To observe and identify the important effects of stress history on soil disturbance, it is perhaps desirable to extend similar investigation on overconsolidated samples having a wide range of overconsolidation ratios.

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APPENDIX - I

LISTING OF COMPUTER PROGRAM TO PREDICT THE VALUES OF STRESS AND STRAIN USING CAM CLAY MODEL FOR ISOTROPICALLY NORMALLY CONSOLIDATED SOIL

/* Typical Program to Predict the Values of Stress and Strain by Using Cam Clay Model for Isotropically Normally Consolidated soil */

```
#include<stdio.h>       // Including Standard Input Output Header File
#include<conio.h>       // Including Configuring Input Output Header File
#include<math.h>        // Including Math Header File
/* Header files contains the built-in functions/keywords of C Compiler */

#define pe 150          // Constant pe declaration
#define k 0.019         // Constant k declaration
#define lam 0.115       // Constant lam declaration
#define Mc 1.34         // Constant Mc declaration

/*
 pe = preshear consolidation pressure,
 k = slope of average line swelling and recompression line,
 lam = slope of normal consolidation line, critical state parameter,
 Mc = ratio of deviator stress and mean effective normal stress at
 failure point.
*/

void calculation();     // Function prototype declaration

/* Float type Variable declaration */
float
j,dEs[40],sdEs[40],c[40],W,G,Vo,o[40],ita[40],dita[40],q0p[40],p0p[40],pp[40],qp[40
],
dpp[40],m[40],v[40],l[40],xx[40],y[40],z[40],blam;

/*
 dEs=differential strain
 sdEs=summation of differential strain
 W=moisture content
 G=specific gravity
 Vo=specific volume
 dita=differential ita
 p0p=pnot prime
 q0p=qnot prime
 dpp=differential pprime
 blam=big lamda
*/
```

main() // This is the main function from where program execution starts
{
calculation(); // Function names calculation is being called
}

void calculation() // Function details
{
FILE *fout; // FILE is a built-in structure
FILE *fin;
int i1,n; // Integer type Variable declaration

fin=**fopen**("c:\\in_cl.doc","w"); // fopen function opens a document file in the path shown
fout=**fopen**("c:\\out_cl.doc","w");

/* fin is the input file pointer in which all the input data of the program will be stored
/* fout is the output file pointer in which all the output data of the program will be stored

if (fin==**NULL**)
{
puts ("Cannot open file");
}

if (fout==**NULL**)
{
puts ("Cannot open file");
}

printf ("\t\t\t Enter the number of data's"); // Writes to the standard output I,e monitor
scanf ("%d",&n); // Cursor waits here for the user-input from the standard input I,e keyboard

fprintf (fin,"\n\t\tThis is the input List\n"); // fprintf writes to a function
/* Sending the text inside the double quotation to the input file pointer names fin */

fprintf (fin,"\n\t Total Number of data's are : %d\n",n+1);

printf ("\t\t\t enter the value of W");
scanf ("%f",&W);

```
fprintf (fin,"\n\t The value of W is: %.3f\n",W);

printf ("\t\t\t enter the value of G");
scanf ("%f",&G);

fprintf (fin,"\n\t The value of G is: %.2f\n",G);

//calculation of specific volume, Vo
Vo=(1+W*G);

            //assigning the value of qnotprime//

fprintf(fin,"\n\t The value of qnotprime is\n");
fprintf(fin,"\t -------------------------\n");

for(i1=0;i1<=n;i1++)
 {
 printf("\t\t\t enter the value of q0prime");
 scanf("%f",&q0p[i1]);
 fprintf(fin,"\t\t\t %.2f\n",q0p[i1]);
 }

            //assigning the value of pnotprime//

fprintf(fin,"\n\t The value of pnotprime is: \n");
fprintf(fin,"\t --------------------------\n");
for(i1=0;i1<=n;i1++)
 {
 printf("\n\t\t\t enter the value of p0prime");
 scanf("%f",&p0p[i1]);
 fprintf(fin,"\t\t\t %.2f\n",p0p[i1]);
 }

fclose(fin);    // closing the input file pointer names fin

             // calculation for ita //

for(i1=0;i1<=n;i1++)
 {
 ita[i1]=q0p[i1]/p0p[i1];
 }
```

```
//calculation for big lambda(blam)//

blam=(1-k/lam);
        //finding the value of ita/Mc which is y//

for(i1=0;i1<=n;i1++)
 {
 y[i1]=ita[i1]/Mc;
 }
       //finding the value of z=ln(pe/pp)which is ita/Mc*blam //

for(i1=0;i1<=n;i1++)
 {
 z[i1]=y[i1]*blam;
 }
            //finding the value of pe/pp=xx //

for(i1=0;i1<=n;i1++)
 {
 xx[i1]=exp(z[i1]);
 }
            // finding the value of pprime,pp=pe/xx //

for(i1=0;i1<=n;i1++)
 {
 pp[i1]=pe/xx[i1];
 }
            //finding the value of qprime,qp=ita*pp //

for(i1=0;i1<=n;i1++)
 {
 qp[i1]=ita[i1]*pp[i1];
 }

      //finding the value of differential pprime,dpp=dpp1-dpp0 //

dpp[0]=0;
for(i1=1;i1<=n;i1++)
 {
 dpp[i1]=pp[i1]-pp[i1-1];
 }
```

```
        //finding the value of v=dpprime/pprime=dpp/pp //

for(i1=0;i1<=n;i1++)
 {
 v[i1]=dpp[i1]/pp[i1];
 }
        //finding the value of differential ita,dita=dita1-dita0 //

dita[0]=0;
for(i1=1;i1<=n;i1++)
 {
 dita[i1]=ita[i1]-ita[i1-1];
 }

        //finding the value of dita/Mc=l//

for(i1=0;i1<=n;i1++)
 {
 l[i1]=dita[i1]/Mc;
 }

        //finding the summation of l and v i,e m=dita/Mc+(dpp/pp) //

for(i1=1;i1<=n;i1++)
 {
 m[i1]=l[i1]+v[i1];
 }

                //finding the value of j=(lam-k)/Vo //

j=(lam-k)/Vo;
                //finding the value of o=1/(Mc-ita) //

for(i1=0;i1<=n;i1++)
 {
 o[i1]=1/(Mc-ita[i1]);
 }

   //finding the value of dEs=1/(Mc-ita)*(lam-k)/Vo*(dita/Mc)+(dpp/pp) //

for(i1=0;i1<=n;i1++)
 {
 dEs[i1]=o[i1]*j*m[i1];
 }
```

```
//finding the summation of dEs=sdEs //

c[0]=dEs[0];
sdEs[0]=dEs[0];
for(i1=1;i1<=n+1;i1++)
 {
 c[i1]=(dEs[i1]+c[i1-1]);
 sdEs[i1]=c[i1]*100;
 }

                 //out put table//
 clrscr();
 printf("\n\t\t q0prime  p0prime   ita    ita/Mc    ln(p0p/pp)\n");
 printf(" \t\t ------  ------   ------  ------  ------\n");

 for(i1=0;i1<=n;i1++)
 {
 printf("\t\t %7.4f %7.4f %7.4f %7.4f %7.4f\n",q0p[i1],p0p[i1],ita[i1],y[i1],z[i1]);
 }

 getche();     // wait for any hit from keyboard
 clrscr();      // clears the screen

 printf("\n\t  p0p/pp pprime   qprime    dpprime  dpprime/pprime\n");
 printf("\t   ----  ------  ------    -------  -----------\n");

 for(i1=0;i1<=n;i1++)
 {
 printf("\t   %7.4f %7.4f %7.4f %7.4f
%9.6f\n",xx[i1],pp[i1],qp[i1],dpp[i1],v[i1]);
 }

 getche();
 clrscr();

 printf("\n\t    dita/Mc   dita/Mc+dpp/pp    1/(Mc-ita)    dEs     sdEs\n");
 printf("\t   ------  ---------------    --------    -------    -------\n");

 for(i1=0;i1<=n;i1++)
 {
 printf("\t  %7.4f  %7.4f\t   %7.4f  %11.8f
%9.6f\n",l[i1],m[i1],o[i1],dEs[i1],sdEs[i1]);
 }
```

INPUT LIST FOR CAM CLAY MODEL

Total Number of data are: 16

The value of W is: 0.305

The value of G is: 2.69

The value of qnotprime is:

0.0	6.26	12.54	18.57	25.04	35.44	48.94	59.6	61.52
65.45	75.14	93.25	104.32	110.28	115.50	117.63	119.20	

The value of pnotprime is:

150.0	149.83	149.33	148.75	148.25	146.85	145.20	142.33	141.51
139.41	132.74	120.58	111.11	104.32	97.01	94.34	88.95	

OUTPUT LIST FOR CAM CLAY MODEL

q0prime	p0prime	ita	ita/Mc	ln(p0p/pp)
0.0000	150.0000	0.0000	0.0000	0.0000
6.2600	149.8300	0.0418	0.0312	0.0260
12.5400	149.3300	0.0840	0.0627	0.0523
18.5700	148.7500	0.1248	0.0932	0.0778
25.0400	148.2500	0.1689	0.1260	0.1052
35.4400	146.8500	0.2413	0.1801	0.1503
48.9400	145.2000	0.3371	0.2515	0.2100
59.6000	142.3300	0.4187	0.3125	0.2609
61.5200	141.5100	0.4347	0.3244	0.2708
65.4500	139.4100	0.4695	0.3504	0.2925
75.1400	132.7400	0.5661	0.4224	0.3526
93.2500	120.5800	0.7733	0.5771	0.4818
104.3200	111.1100	0.9389	0.7007	0.5849
110.2800	104.3200	1.0571	0.7889	0.6586
115.5000	97.0100	1.1906	0.8885	0.7417
117.6300	94.3400	1.2469	0.9305	0.7768
119.2000	88.9500	1.3401	1.0001	0.8348

p0p/pp	pprime	qprime	dpprime	dpprime/pprime
1.0000	150.0000	0.0000	0.0000	0.000000
1.0264	146.1461	6.1061	-3.8539	-0.026370
1.0537	142.3546	11.9542	-3.7915	-0.026634
1.0809	138.7763	17.3249	-3.5783	-0.025785
1.1110	135.0186	22.8052	-3.7576	-0.027831
1.1622	129.0617	31.1471	-5.9570	-0.046156
1.2336	121.5908	40.9825	-7.4709	-0.061443
1.2981	115.5576	48.3892	-6.0332	-0.052210
1.3111	114.4118	49.7394	-1.1457	-0.010014
1.3397	111.9624	52.5639	-2.4494	-0.021877
1.4228	105.4240	59.6772	-6.5384	-0.062020
1.6189	92.6531	71.6529	-12.7709	-0.137835
1.7948	83.5741	78.4668	-9.0790	-0.108635
1.9320	77.6391	82.0748	-5.9349	-0.076442
2.0995	71.4448	85.0621	-6.1943	-0.086701
2.1744	68.9836	86.0137	-2.4613	-0.035679
2.3044	65.0921	87.2286	-3.8914	-0.059783

dita/Mc	dita/Mc+dpp/pp	1/(Mc-ita)	dEs	sdEs
0.0000	0.0000	0.7463	0.00000000	0.000000
0.0312	0.0048	0.7703	0.00019537	0.019537
0.0315	0.0049	0.7962	0.00020380	0.039917
0.0305	0.0047	0.8229	0.00020447	0.060364
0.0329	0.0051	0.8539	0.00022752	0.083116
0.0541	0.0079	0.9102	0.00037904	0.121020
0.0714	0.0100	0.9971	0.00052516	0.173536
0.0610	0.0088	1.0855	0.00050116	0.223652
0.0119	0.0019	1.1047	0.00011198	0.234850
0.0259	0.0040	1.1487	0.00024517	0.259367
0.0721	0.0101	1.2921	0.00068561	0.327928
0.1547	0.0168	1.7647	0.00156799	0.484727
0.1235	0.0149	2.4931	0.00195963	0.680690
0.0882	0.0118	3.5352	0.00219950	0.900640
0.0996	0.0129	6.6934	0.00455383	1.356023
0.0420	0.0063	10.7380	0.00357687	1.713710
0.0696	0.0098	12.7046	0.00600415	2.314124

APPENDIX - II

LISTING OF COMPUTER PROGRAM TO PREDICT THE VALUES OF STRESS AND STRAIN USING MODIFIED CAM CLAY MODEL FOR ISOTROPICALLY NORMALLY CONSOLIDATED SOIL

```c
/* Typical Program to Predict the Values of Stress and Strain by Using Modified
   Cam Clay Model for Isotropically Normally Consolidated soil */

#include<stdio.h>        // Including Standard Input Output Header File
#include<conio.h>        // Including Configuring Input Output Header File
#include<math.h>         // Including Math Header File

/* Header files contains the built-in functions/keywords of C Compiler */

#define pe 150           // Constant pe declaration
#define k 0.019          // Constant k declaration
#define lam 0.115        // Constant lam declaration
#define Mc 1.34          // Constant Mc declaration

/*
  pe = preshear consolidation pressure,
  k = slope of average line swelling and recompression line,
  lam = slope of normal consolidation line, critical state parameter,
  Mc = ratio of deviator stress and mean effective normal stress at
  failure point.
*/

 void calculation();          // Function prototype declaration

/* Float type Variable declaration */
float
j,dEs[40],sdEs[40],c[40],d[40],q[40],W,G,Vo,o[40],ita[40],dita[40],q0p[40],p0p[40],p
p[40],
qp[40],dpp[40],m[40],v[40],l[40],xx[40],y[40],z[40],blam;


/*
  dEs=differential strain
  sdEs=summation of differential strain
  W=moisture content
  G=specific gravity
  Vo=specific volume
  dita=differential ita
  p0p=pnot prime
  q0p=qnot prime
  dpp=differential pprime
  blam=big lamda
 */
```

```
main()                  // This is the main function from where program execution
starts
{
calculation();          // Function names calculation is being called

}

void calculation()      // Function details
{
FILE *fout;             // FILE is a built-in structure
FILE *fin;
int i1,n;               // Integer type Variable declaration


fin=fopen("c:\\in_mcl.doc","w");    // fopen function opens a document file in the
path shown
fout=fopen("c:\\out_mcl.doc","w");

/* fin is the input file pointer in which all the input data of the program will be stored
/* fout is the output file pointer in which all the output data of the program will be
stored

if(fin==NULL)
{
puts("Cannot open file");
}

if(fout==NULL)
{
puts("Cannot open file");
}

printf("\t\t\t Enter the number of data's");       // Writes to the standard output I,e
monitor

scanf("%d",&n);        // Cursor waits here for the user-input from the standard input
I,e keyboard


fprintf(fin,"\n\t\tThis is the input List\n");    // fprintf writes to a function
/* Sending the text inside the double quotation to the input file pointer names fin */

fprintf(fin,"\n\t Total Number of data's are : %d\n",n+1);
```

```
printf("\t\t\t enter the value of W");
scanf("%f",&W);

fprintf(fin,"\n\t The value of W is: %.3f\n",W);

printf("\t\t\t enter the value of G");
scanf("%f",&G);

fprintf(fin,"\n\t The value of G is: %.2f\n",G);

//calculation of specific volume, Vo
Vo=(1+W*G);



                //assigning the value of qnotprime//

fprintf(fin,"\n\t The value of qnotprime is\n");
fprintf(fin,"\t -------------------------\n");

for(i1=0;i1<=n;i1++)
 {
 printf("\t\t\t enter the value of q0prime");
 scanf("%f",&q0p[i1]);
 fprintf(fin,"\t\t\t %.2f\n",q0p[i1]);
 }

              //assigning the value of pnotprime//

fprintf(fin,"\n\t The value of pnotprime is: \n");
fprintf(fin,"\t --------------------------\n");

for(i1=0;i1<=n;i1++)
 {
 printf("\n\t\t\t enter the value of p0prime");
 scanf("%f",&p0p[i1]);
 fprintf(fin,"\t\t\t %.2f\n",p0p[i1]);
 }

fclose(fin);              // closing the input file pointer names fin

                //calculation for ita//

for(i1=0;i1<=n;i1++)
```

```
{
ita[i1]=q0p[i1]/p0p[i1];
}

        //calculation for big lambda//

blam=(1-k/lam);

 //finding the value of {(ita square)+(Mc square)}/(Mc square) which is y//

for(i1=0;i1<=n;i1++)
 {
 y[i1]=(ita[i1]*ita[i1]+Mc*Mc)/(Mc*Mc);
 }

      //finding the value of z=y to the power blam = pe/pprime //

for(i1=0;i1<=n;i1++)
 {
 z[i1]=pow(y[i1],blam);
 }

        //finding the value of pprime=pe/z //

for(i1=0;i1<=n;i1++)
 {
 pp[i1]=pe/z[i1];
 }

        // finding the value of qprime,qprime=ita*pprime //

for(i1=0;i1<=n;i1++)
 {
 qp[i1]=pp[i1]*ita[i1];
 }

        //finding the value of pprime/peprime = d //

for(i1=0;i1<=n;i1++)
 {
 d[i1]=1/z[i1];
 }

   //finding the value of differential pprime,dpp=dpp1-dpp0 //
```

```
dpp[0]=0;
for(i1=1;i1<=n;i1++)
 {
 dpp[i1]=pp[i1]-pp[i1-1];
 }

              //finding the value of v=dpprime/pprime //

for(i1=0;i1<=n;i1++)
 {
 v[i1]=dpp[i1]/pp[i1];
 }

      //finding the value of differential ita,dita=dita1-dita0 //

dita[0]=0;
for(i1=1;i1<=n;i1++)
 {
 dita[i1]=ita[i1]-ita[i1-1];
 }

      //finding the value of 2*ita*dita/(Mc*Mc+ita*ita)=l//

for(i1=0;i1<=n;i1++)
 {
 l[i1]=2*ita[i1]*dita[i1]/(ita[i1]*ita[i1]+Mc*Mc);
 }

      //finding the summation of q and v i,e m=q+(dpp/pp) //

for(i1=1;i1<=n;i1++)
 {
 m[i1]=l[i1]+v[i1];
 }

              //finding the value of j=(lam-k)/Vo //

j=(lam-k)/Vo;

              //finding the value of o=2*ita/(Mc*Mc-ita*ita)

for(i1=0;i1<=n;i1++)
 {
```

```
 o[i1]=2*ita[i1]/(Mc*Mc-ita[i1]*ita[i1]);
 }

  //finding the value of dEs=2*ita/(Mc*Mc-ita*ita)*(lam-k)/Vo*(q+dpp/pp) //

for(i1=0;i1<=n;i1++)
 {
 dEs[i1]=o[i1]*j*m[i1];
 }

            //finding the summation of dEs=sdEs //

c[0]=dEs[0];
sdEs[0]=dEs[0];
for(i1=1;i1<=n+1;i1++)
 {
 c[i1]=(dEs[i1]+c[i1-1]);
 sdEs[i1]=c[i1]*100;
 }

                  //out put table//

 clrscr();
 printf("\n\t\t q0prime   p0prime    ita      y     ypowblam\n");
 printf(" \t\t ------   ------    ------   ------   ------\n");

 for(i1=0;i1<=n;i1++)
 {
 printf("\t\t %7.4f  %7.4f  %7.4f  %7.4f  %7.4f\n",q0p[i1],p0p[i1],ita[i1],y[i1],z[i1]);
 }

 getche();              // wait for any hit from keyboard

 clrscr();              // clears the screen

 printf("\n\t    pprime    qprime   pp/pe   dpprime   dpp/pp\n");
 printf("\t    ------  ------   -------   -------  --------\n");

 for(i1=0;i1<=n;i1++)
 {
 printf("\t   %7.4f %7.4f  %7.4f  %7.4f  %9.6f\n",pp[i1],qp[i1],d[i1],dpp[i1],v[i1]);
 }

 getche();
```

```
clrscr();

printf("\n\t    l    l+dpp/pp      o      dEs     sdEs\n");
printf("\t   ------  -------------  --------   -------  -------\n");

for(i1=0;i1<=n;i1++)
{
printf("\t  %7.4f  %7.4f\t   %7.4f   %11.8f
%9.6f\n",l[i1],m[i1],o[i1],dEs[i1],sdEs[i1]);
}
getche();
clrscr();

/* Sending output to the output file pointer names fout */
fprintf(fout,"\n\t\t q0prime  p0prime   ita     y    ypowblam\n");
fprintf(fout," \t\t ------  ------   ------  ------  ------\n");

for(i1=0;i1<=n;i1++)
{
fprintf(fout,"\t\t %7.4f %7.4f %7.4f %7.4f
%7.4f\n",q0p[i1],p0p[i1],ita[i1],y[i1],z[i1]);
}

fprintf(fout,"\n\t   pprime   qprime  pp/pe  dpprime  dpp/pp\n");
fprintf(fout,"\t    ------  ------   -------  -------  --------\n");

for(i1=0;i1<=n;i1++)
{
fprintf(fout,"\t   %7.4f %7.4f %7.4f %7.4f
%9.6f\n",pp[i1],qp[i1],d[i1],dpp[i1],v[i1]);
}

fprintf(fout,"\n\t    l    l+dpp/pp      o      dEs     sdEs\n");
fprintf(fout,"\t    ------  ------------  --------   -------  -------\n");

for(i1=0;i1<=n;i1++)
{
fprintf(fout,"\t  %7.4f   %7.4f\t   %7.4f   %11.8f
%9.6f\n",l[i1],m[i1],o[i1],dEs[i1],sdEs[i1]);
}
fclose(fout);         // closing the output file pointer names fout
}

/* Bold words refer to the key words of the standard C library */
```

INPUT LIST FOR MODIFIED CAM CLAY MODEL

Total Number of data are: 17

The value of W is: 0.305

The value of G is: 2.69

The value of qnotprime is:

0.00	6.26	12.54	18.57	25.04	35.44	48.94	59.60	61.52
65.45	75.14	93.25	104.32	110.28	115.50	117.63	119.20	

The value of pnotprime is:

150.00	149.83	149.33	148.75	148.25	146.85	145.20	142.33	141.51
139.41	132.74	120.58	111.11	104.32	97.01	94.34	88.95	

OUTPUT LIST FOR MODIFIED CAM CLAY MODEL

q0prime	p0prime	ita	y	ypowblam
0.0000	150.0000	0.0000	1.0000	1.0000
6.2600	149.8300	0.0418	1.0010	1.0008
12.5400	149.3300	0.0840	1.0039	1.0033
18.5700	148.7500	0.1248	1.0087	1.0072
25.0400	148.2500	0.1689	1.0159	1.0132
35.4400	146.8500	0.2413	1.0324	1.0270
48.9400	145.2000	0.3371	1.0633	1.0525
59.6000	142.3300	0.4187	1.0977	1.0809
61.5200	141.5100	0.4347	1.1053	1.0871
65.4500	139.4100	0.4695	1.1228	1.1015
75.1400	132.7400	0.5661	1.1785	1.1469
93.2500	120.5800	0.7733	1.3331	1.2712
104.3200	111.1100	0.9389	1.4909	1.3957
110.2800	104.3200	1.0571	1.6224	1.4977
115.5000	97.0100	1.1906	1.7894	1.6254
117.6300	94.3400	1.2469	1.8658	1.6831
119.2000	88.9500	1.3401	2.0001	1.7837

pprime	qprime	pp/pc	dpprime	dpp/pp
150.0000	0.0000	1.0000	0.0000	0.000000
149.8784	6.2620	0.9992	-0.1216	-0.000811
149.5100	12.5551	0.9967	-0.3684	-0.002464
148.9217	18.5914	0.9928	-0.5883	-0.003950
148.0391	25.0044	0.9869	-0.8826	-0.005962
146.0557	35.2483	0.9737	-1.9834	-0.013580
142.5116	48.0339	0.9501	-3.54[illegible]	-0.024869
138.7751	58.1114	0.9252	-3.73[illegible]	-0.026925
137.9777	59.9844	0.9199	-0.797[illegible]	-0.005779
136.1808	63.9339	0.9079	-1.7970	-0.013195
130.7857	74.0337	0.8719	-5.3951	-0.041251
117.9956	91.2514	0.7866	-12.7901	-0.108395
107.4712	100.9030	0.7165	-10.5244	-0.097927
100.1525	105.8740	0.6677	-7.3187	-0.073076
92.2840	109.8733	0.6152	-7.8685	-0.085263
89.1192	111.1203	0.5941	-3.1649	-0.035513
84.0960	112.6953	0.5606	-5.0232	-0.059731

l	l+dpp/pp	o	dEs	sdEs
0.0000	0.0000	0.0000	0.00000000	0.000000
0.0019	0.0011	0.0466	0.00000278	0.000278
0.0039	0.0015	0.0939	0.00000727	0.001004
0.0056	0.0017	0.1403	0.00001245	0.002250
0.0082	0.0022	0.1912	0.00002216	0.004465
0.0189	0.0053	0.2778	0.00007733	0.012198
0.0338	0.0089	0.4008	0.00018868	0.031066
0.0347	0.0078	0.5169	0.00021226	0.052293
0.0070	0,0012	0.5412	0.00003507	0.055800
0.0162	0.0030	0.5961	0.00009381	0.065181
0.0517	0.0104	0.7675	0.00042202	0.107382
0.1339	0.0255	1.2916	0.00173944	0.281326
0.1161	0.0182	2.0543	0.00197037	0.478363
0.0858	0.0127	3.1181	0.00209496	0.687859
0.0989	0.0136	6.2982	0.00453256	1.141115
0.0419	0.0064	10.3515	0.00347949	1.489064
0.0696	0.0098	12.7049	0.00658275	1.636682