

**STUDIES ON STRESS-STRAIN BEHAVIOUR AND  
CREEP RESPONSE OF SOFT CLAYEY SOILS OF  
NORMAL KOLKATA DEPOSIT**

**Thesis submitted by  
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Kolkata, India  
2023**

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"STUDIES ON STRESS-STRAIN BEHAVIOUR AND CREEP RESPONSE OF SOFT CLAYEY SOILS OF NORMAL KOLKATA DEPOSIT"

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- (2) Chowdhury, A., Sahu, R.B. Creep potential of soft clays of Normal Kolkata Deposit. *Arab J Geosci* 14, 1354 (2021). <https://doi.org/10.1007/s12517-021-07677-0>

4. List of Patents: Nil.

5. List of Presentations in National/Inter National/ Conferences/Workshops:


- (1) Chowdhury A. (2019) "*Long term deformations of soft clays: A case study*", 7th Indian Young Geotechnical Engineers Conference – 7IYGEC 2019, NIT Silchar, Assam, India. [https://doi.org/10.1007/978-981-16-6456-4\\_43](https://doi.org/10.1007/978-981-16-6456-4_43).

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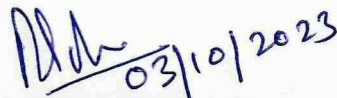
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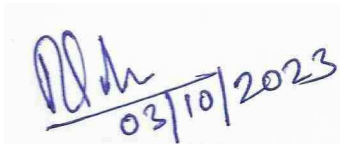


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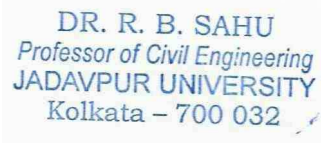
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## **ACKNOWLEDGEMENT**

I express my sincere gratitude and respect to my supervisor, Dr. R.B. Sahu, Department of Civil Engineering for the excellent support and guidance throughout the research work. This research would not have been possible without his support and guidance. I am extremely thankful for his readiness for academic discussions and general help which were of immense value to me.

I am extremely appreciative for the research experience at the Jadavpur University, which would not have been possible without the support of Professors, Professor Partha Bhattacharya, Head of Civil Engineering Department, Faculty members of Civil Engineering Department Jadavpur University. I am also thankful to Soil Mechanics Laboratory supporting staff Sri. Rabin Paul, Sri. Apurba Banerjee, Ranjit Kushari and others for their support and cooperation during my experimental work in the laboratory. I would like to thank my fellow researchers Biplab Adhikary, Saptarshi Roy, Poulami Ghosh, Sayak Nandi and Arunashis Majumder for being part of my research journey.

No words or form of appreciation is sufficient to express my gratitude towards my parents and Tanusri who have been a pivotal part of my life. I dedicate this to the three of you.

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## Symbols and Notations

$c$	cohesion
$\phi$	angle of internal friction
$s_u$	undrained shear strength
$E_u$	Young's secant modulus
$m$	Singh – Mitchell Creep potential parameter
$A$	Singh – Mitchell Creep potential parameter
$\alpha$	Singh – Mitchell Creep potential parameter
$\dot{\epsilon}_a$	axial strain rate
$C_\alpha$	coefficient of secondary compression
$C_c$	Coefficient of compression
$t$	time
D or Dr	Stress ratio
SL	Stress level
$\sigma_f$	normal stress acting on the failure plane
$\tau_f$	shear strength
$\theta_f$	angle between potential shear failure and major principle stress $\sigma_1$
$u$	pore water pressure
$c_e$	true cohesion
$\phi_e$	true angle of friction
$c'$	effective cohesion
$\phi'$	effective angle of friction
$c_u$	undrained cohesion
$\sigma_1$	major principal stress
$\sigma_3$	minor principal stress
$\phi_d$	angle of internal friction under drained condition
$\sigma_d$	deviator stress
$\epsilon$	strain
$B$	pore pressure Coefficient
$e$	Void ratio

$e_o$	Initial void ratio
$\sigma'_v$	effective vertical stress
$e^*_{100}$	intrinsic void ratio at 100kPa effective vertical pressure
$e^*_{1000}$	intrinsic void ratio at 1000 kPa effective vertical pressure
$I_v$	void index
$I_{vo}$	in situ void index
$\mu$	Poisson's ratio
$w_L$	Liquid limit
$\epsilon_r$	radial strain
$\epsilon_a$	axial strain
LL	Liquid limit
$A_f$	pore water parameter at failure
$A'$	pore water pressure parameter, A-value
$\Delta v$	Change in volume
$\Delta l$	Change in length
$p_o$	Effective confining pressure
$\beta$	Modified Singh-Mitchell Equation parameter
B	Modified Singh-Mitchell Equation parameter
$\lambda$	Modified Singh-Mitchell Equation parameter
$m_v$	coefficient of volume change
$I_\sigma$	influence factor

## ABSTRACT

The safety of any construction depends significantly on the type of foundation/substructure it rests on. The nature of existing sub-soil conditions at a particular construction site plays a crucial role in the choice of foundation. One of the most important properties of soils is its shear strength. In this study a laboratory investigation to evaluate creep characteristics and shear strength parameters ( $c$ ,  $\phi$ ) of remoulded (artificially consolidated) inorganic and organic clay using triaxial compression and multistage triaxial creep tests was conducted. An attempt has been made to investigate the variations in magnitude of shear strength parameters ( $c$ ,  $\phi$ ) due to difference in testing procedures and drainage conditions in order to predict the realistic and most appropriate ( $c, \phi$ ) values for evaluating soil strength for construction, estimating the creep potential and creep parameters of soft clays of Normal Kolkata Deposit. For this purpose conventional triaxial compression and multistage triaxial creep tests under undrained and drained conditions on remoulded inorganic and organic clays of Normal Kolkata Deposit have been conducted. The creep potential and creep parameters of soft clays of Normal Kolkata Deposit have been estimated using the triaxial test data. The compression behaviour of natural clays and remoulded clays have been compared using available data from oedometer tests conducted on natural and remoulded clays of this region. The results indicate that for natural and remoulded clays of Kolkata region the compression behaviour and parameters governing compression behaviour are similar and that the effect of soil fabric in case of Kolkata clays is not prominent at all. The effective friction angles obtained from undrained tests are quite higher than that obtained from drained tests,  $4-6^\circ$  and  $5-8^\circ$  for inorganic and organic clays respectively. The maximum deviator stress and also effective friction angle obtained from stress-controlled undrained tests are higher than those obtained from strain-controlled undrained tests for both inorganic and organic clays. The creep potential calculated using Singh and Mitchell (1969) parameters revealed that under undrained conditions the effect of time dependent evolution of

behaviour was evident for inorganic and organic clays. Under drained conditions the effect of creep was not as significant as evidenced during undrained tests.

**Keywords:** Normal Kolkata Deposit, Strain-controlled tests; stress-controlled tests; Singh and Mitchell (1969), creep , undrained tests, effective friction angle, drained tests.

## **INTRODUCTION**

### **1.0 General**

The safety of any construction depends significantly on the type of foundation/substructure it rests on. The nature of existing sub-soil conditions at a particular construction site plays a crucial role in the choice of foundation. One of the most important properties of soils is its shear strength. Shear strength plays a crucial role in the estimation of bearing capacity of soils required for designing the foundation of structures. Shear strength is also critical for assessing the stability of slopes, embankments, retaining structures and highway pavements. Shear strength of soil can be defined as the inherent capacity of the soil mass to resist failure under the action of external forces. Soft clays as a foundation soil is encountered all over the world. Soft clays are found in many areas of Scandinavia, Mexico, Japan, China, Northern United States and Canada, India and the Middle-east. Soft clays are generally recent sediments laid down by sea (marine), river (alluvial) or lakes (lacustrine). Most river valleys of the world have abundant deposits of soft clays of alluvial origin. The Mississippi in USA, the Nile in Egypt, Yangtze in China, Euphrates and Tigris in Iraq, Mekong in Indo-China, Ganga-Brahmaputra in India are examples of alluvial deposits. Marine clays are found along coastal plains of the world. Soft clays are fine-grained soils with moderate to high clay fraction (30%) having high compressibility and shear strength less than  $2.5 \text{ t/m}^2$ . For structure resting over soft clayey soils, deformation consists of two components, consolidation which occurs due to expulsion of pore water and creep which occurs after or during primary consolidation due to reorientation of soil particles. For completed structures and man-made or natural slopes, magnitude as well as rate of deformation due to creep depend upon a number of factors like mobilized shear stresses, properties of soil and its variation with space and time.

Consequently, roads, harbours, bridges, buildings and underground constructions in soft clays pose a challenge to design and construction which has to be assessed and solved with attention and foresight.

### **1.1 Importance of creep deformation in geotechnical engineering practice**

It is important to assess the “long-term” condition of soil to understand and predict the performance of the structure erected. The “end of construction” conditions of a soil change with time due to variation in pore pressure and shear strength of the soil. Creep is the time dependent deformation phenomenon of soil. Over the years, there have been several instances where creep of soil has led to instability or failure of a structure. Skempton (1964) presented detailed discussions and analysis of landslides caused due to creep of soils which led to failure of retaining wall at Kensal Green in 1941, Sudbury Hill failure in 1949 and Northolt Station in 1955. Vyalov (1986) discussed the effect of soil creep on dams, retaining walls, intake structure of hydropower project, sheet pile walls and a reinforced concrete arched viaduct of an abutment. These are cases which highlight the effect of horizontal load on structures induced due to creep which led to failure of the structure in certain instances and in others the structure was damaged. A classic example of creep settlement is the Leaning Tower of Pisa. The long term settlements of San Jacinto Monument, Texas (Briaud et al.2007) and plain earth dams of the Kakhovka hydro-power station on the Dnieper resting on clayey silts (Karpyshev et al.1972) are some examples of vertical load induced creep settlements. Bjerrum (1967) found that settlement of six buildings resting over Drammen clay deviated from predictions using Terzaghi’s theory with consideration of “secondary” consolidation. The concept of instantaneous compression and delayed compression was then highlighted for prediction of settlement behaviour of the buildings. Drammen clay has loose structure evident from the fact that the water content is relatively high and is almost equal to the liquid limit, throughout its profile with very low undrained shear strength and high compressibility. This layer situated

below the drying crust had a shear strength of  $1t/m^2$ . The shear strength was found to linearly increase with depth. The net pressure acting on the soil below the foundation of the six buildings considered for this study were comparable. The settlements were found to be dependent upon the depth at which the plastic clay layer was located and also the thickness of the clay layer. The above case studies indicate that a major portion of the deformation of structures and embankments over soft clay is due to creep. Foott and Ladd (1981) reviewed the case histories of Cross river embankments constructed on thick organic plastic clays, Atchafalaya flood control levees and storage tanks on plastic organic clays in New Jersey, U.S.A. The measured field settlements in these cases exceeded the predicted primary consolidation settlement. The investigations revealed that in case of plastic or organic clays initial settlements due to undrained shear deformations can have large values compared to settlement due to consolidation owing to unusually low normalized modulus  $E_u/s_u$  ( $E_u$  being the Young's secant modulus and  $s_u$  the undrained shear strength). Moreover if the rate of consolidation is slow, continuing creep deformations can lead to substantial additional settlements. Tavenas et al. (1979) analyzed the lateral displacements occurring in clay foundations under 21 different embankments during construction and consolidation. On the basis of measured centerline and lateral deformations the authors highlighted that at the beginning of construction prior to application of stress exceeding the pre-consolidation stress rapid partial consolidation occurs. Post pre-consolidation stress the loading is essentially undrained and major portion of the settlement is due to lateral deformation. Following the end of construction during consolidation the lateral deformations continue to increase. Delayed lateral deformations can be significant and develop linearly with the consolidation settlement, at least during the first years after the end of construction.

## 1.2 Experimental investigations of creep behaviour of clays

Extensive experimental research investigating the creep behaviour of clays has been conducted by researchers over the years. In the literatures significant consideration has been given to evaluation of reference time as it plays a crucial role estimating the creep settlement in low permeability soils like clays. There exist two well documented approaches which have been adopted by researchers. Hypothesis A adopted by Ladd et al. (1977); Leonards (1977); Mesri and Choi (1985a,b) assumes that thickness of sample and pre-consolidation pressure has no effect on end of primary consolidation. This hypothesis assumes secondary consolidation (creep) occurs after pore pressure dissipation is complete and both processes are mutually exclusive. Hypothesis B on the other hand assumes that creep occurs throughout the consolidation process and that time dependent strains exists even during primary consolidation. This approach has been suggested by Bjerrum (1967), Leroueil et al. (1985) Crawford (1986), Kabbaj et al. (1986), and Yin (1999). There is still no general agreement over which approach is closest to reality as highlighted in the works of Duncan et al. (1996) and Leroueil et al. (1985). Real soil behaviour practically lies in between since these two hypotheses correspond to two extreme cases. This has been affirmed by researchers like Aboshi (1973) who conducted an experimental investigation to assess the phenomenon of similitude in normally consolidated clays. Singh and Mitchell (1969) proposed a three parameter general function for soils which expressed the strain rate as a function of time after application of sustained deviator stress. These parameters help is estimation of settlement and assessing the creep susceptibility of soils. Singh and Mitchell (1969) proposed the following equation for prediction of strain rate of soil during creep under undrained condition.

$$\dot{\epsilon}_a = Ae^{\alpha D} \left( \frac{t_1}{t} \right)^m \quad (1.1)$$

Where  $A$ ,  $\alpha$  and  $m$  are a measure of the creep potential of a soil, for values of  $m < 1$  the chances of creep of soil exists. Lower the value higher the creep potential. This expression has been used by several researchers to predict long term deformation characteristics of soils. Some of the other notable researches have been conducted by Campbella and Vaid (1974), Tian et al. (1994), Fedaa (1992), Zhu et al. (1999) and Lacasse and Berre (2005). These investigations carried out on soft clays, marine deposits across the world such as Hong Kong marine deposits, Mexico clay, Norwegian clay, Boston Blue clay, Saint Alban clay, London clay, Osaka clay, Pancone clay are well documented. In the past decade some of the notable experimental investigation to assess the creep behaviour, i.e. rate-dependent and long-term yield stress-strain and strength of clays have been conducted by Lai et al. (2014), Yin et al. (2015), Kharanaghi et al. (2017), Wang et al. (2017), Fu et al. (2018), Lai et al 2010 and Zhao et al. 2018. Feng et al. (2020) emphasised on the importance of variable compressibility in calculating settlement of soft soil strata, particularly for thick soil layers, and offered a more simplified approach to determine settlement of a thick soil layer with creep.

### **1.3 Background of present study**

The above case studies indicate that a major portion of the deformation of structures and embankments over soft clay is due to creep. This phenomenon is also observed for structures constructed over soft clay deposit in Kolkata region. The subsoil deposit in Kolkata region basically consists of (a) Normal Kolkata Deposit (b) River Channel Deposit which mainly lies along / both the sides the Adi Ganga / Tolly Nallah channel consisting of a deep deposit of silty sand / sandy silty down to 20 – 30m below existing ground level. Normal Kolkata deposit consists of a top soft clay / silty clay / clayey silt down to a depth of about 14-16m with intermittent layers of organic clay / peat at a depth of 4-5 m and at some location at 9-10m having varying percentages of organic matter / decomposed wood / decayed vegetation. A typical soil stratification along a bore hole of Normal Kolkata Deposit is depicted in figure 1.1.

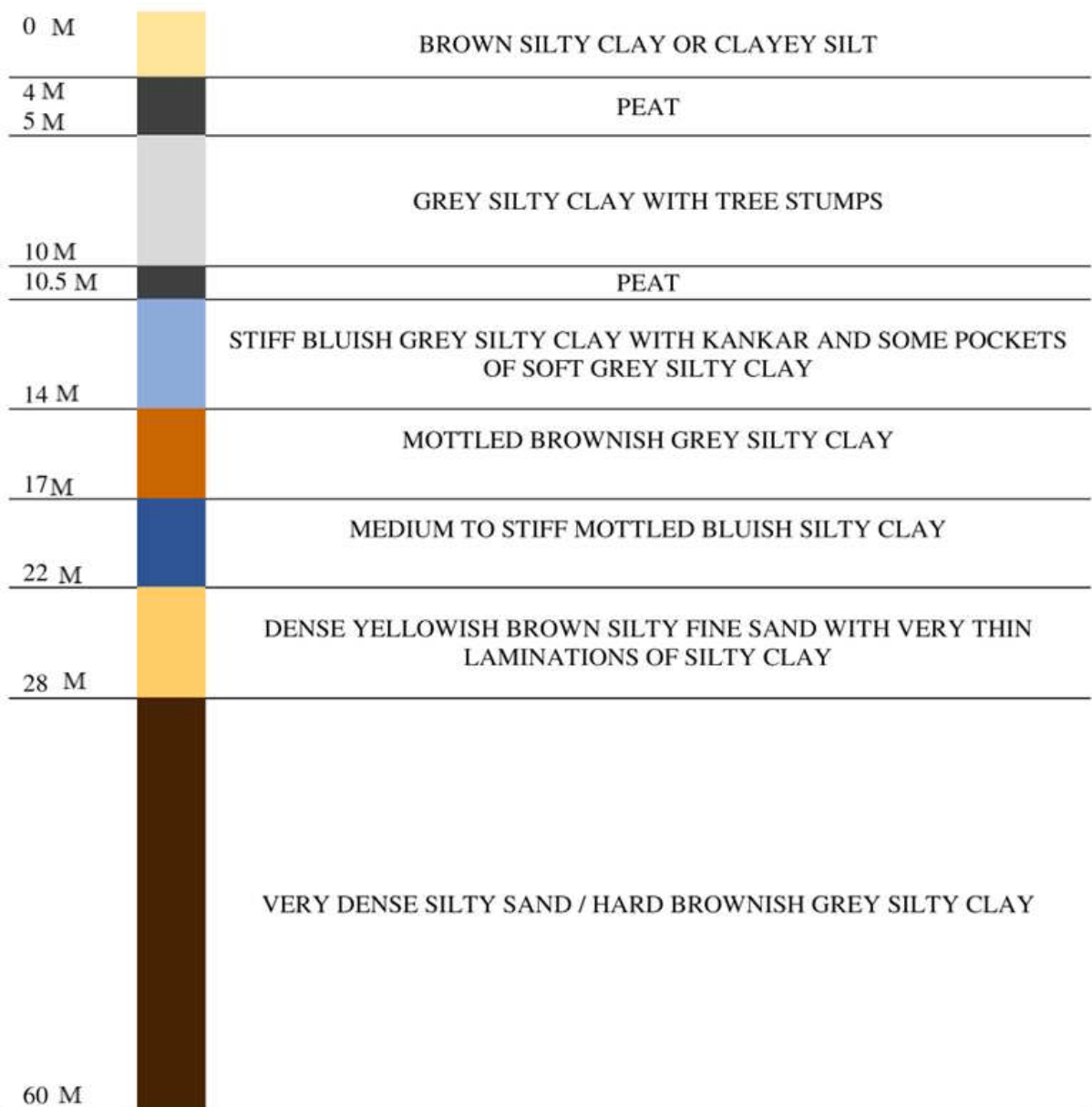


Fig 1.1: Soil stratification of a bore log along Normal Kolkata Deposit

(Dastidar and Ghosh1967)

One typical example is High Court building at Kolkata which was constructed about 150 years and is still undergoing settlement due to creep under high foundation pressure. The structure experienced excessive settlement which led to the development of distress cracks of certain structural members (Mittal et al. 2018). Koti Reddy et al. (2014) conducted incremental loading (IL) consolidation tests on organic clays of Kolkata region. From IL test results it was found

that the rate and magnitude of settlement increase with amount of organic content. Pore-water pressure measured using modified consolidation cell recorded a significant reduction in the generation when compared with applied stress increment. A significant time lag between application of effective stress and generation of peak pore pressure is also observed which in turn depend on type of organic soil.  $C_a/C_c$  concept which was used to suggest practical values of secondary compression index and the values in the range of 0.058 - 0.040 for organic clays of this region. It was observed from long duration consolidation tests that there was no significant effect of increasing loading duration on consolidation parameters of these soils but tertiary creep was noticed. Further, a number of failure were reported during construction of embankments required for widening and strengthening of the existing national highways near Kolkata in recent past. One such instance of embankment failure along a national highway was investigated by Roy et al. (2008). These are predominantly due to presence of soft clayey soils in the upper reaches of subsoil deposit in and around Kolkata which indicates a possibility of creep and a need to assess the behaviour of this clay post application of sustained loading.

#### **1.4 Motivation of work**

The above instances and soil behaviour highlight the need to investigate the creep characteristics of soft clayey soils of Kolkata. There is lack of published data investigating the shear strength and creep behaviour of soft soils of this region despite the presence of such detailed investigations on other types of clays across the globe such as London clay, Hong Kong marine deposits, Boston Blue clay, Pancone clay of Pisa, Norwegian clay. Further no significant study has been conducted on the effect of stress-controlled triaxial test influencing the magnitude of shear strength parameters. Therefore it has been attempted to evaluate both shear strength parameters and creep characteristics using creep triaxial test on remoulded clayey soils collected from a location near Jadavpur. In this context it may be noted that prior to assessing the strain strain response of the soil, the influence of soil structure on the

engineering behaviour of the soils available in this region was also studied using available test results conducted on remoulded and natural clays of this region.

### **1.5 Objective of present study**

1. To investigate the variations in magnitude of shear strength parameters ( $c$ ,  $\phi$ ) due to difference in testing procedures and drainage conditions using conventional triaxial compression and multistage triaxial creep tests under undrained and drained conditions on remoulded inorganic and organic clays of Normal Kolkata Deposit.
2. Estimating the creep potential and creep parameters of soft clays of Normal Kolkata Deposit, remoulded inorganic and organic clays, using multistage creep triaxial data under both drained and undrained condition.

### **1.6 Scope of work**

1. To achieve the above-mentioned objective inorganic and organic soil samples of Normal Kolkata Deposit were collected.
2. Physical properties of the inorganic and organic soils were determined.
3. As the present investigation was conducted on artificially consolidated locally available inorganic and organic clay, analysis was conducted using available test data to study the effect of remoulding and soil structure on the behaviour of locally available soils. The data were collected from unpublished sub-soil investigation reports of Soil Mechanics and Foundation engineering laboratory of Jadavpur University and some test data on artificially prepared soil conducted at River Research Institute Haringhata (unpublished data).
4. Conventional triaxial compression tests under undrained and drained conditions (CU and CD) along with multistage creep triaxial tests under undrained and drained conditions were conducted on remoulded inorganic and organic soils.

5. Setting up of a framework containing a lever system for application of sustained loads for conducting both multistage and single stage triaxial creep testing.

## 1.7 Organisation of thesis

This thesis has been organized in **eight** different chapters as follows:

**Chapter 1** briefly discusses the necessity for determination of shear strength parameters ( $c, \phi$ ) using triaxial and creep triaxial tests, creep of soft soil, its significance in geotechnical engineering and necessity to investigate the creep behaviour of soft clays of Kolkata region laying the objective and scope of the study.

**Chapter 2** reviews the **previous studies** conducted on shear testing of soil, time-dependent behaviour of soft clays including some case studies, experimental studies on creep behaviour.

**Chapter 3** provides **description of the apparatus** used for **testing and techniques** adopted for conducting the experimental study. The basic soil properties of remoulded inorganic and organic clays of Normal Kolkata Deposit is determined.

**Chapter 4** investigates the effect of soil structure on the compression behaviour of soils.

**Chapter 5** investigates the variations in **shear strength parameters** ( $c, \phi$ ) obtained from strain controlled and stress controlled triaxial tests.

**Chapter 6** evaluates the **creep potential parameters** developed by Singh and Mitchell(1969) for Normal Kolkata soil using multistage creep testing.

**Chapter 7** **comparison of** results obtained from **single stage creep triaxial testing and** with obtained multistage creep and conventional testing data.

**Chapter 8** brings out the **Summary, Conclusions and Suggestions for Further Research.**

An **Abstract** of the dissertation has been presented at the beginning of this dissertation.



## **REVIEW OF LITERATURE**

### **2.0 General**

In this chapter the literatures related to the shear strength of clays, the determination of shear strength parameters using triaxial tests are presented. The case studies on creep of soft clays along with laboratory investigations of creep behaviour of soils has also been presented in this section to highlight the need and motivation of the current study.

### **2.1 Shear strength parameters of soft clays**

The shear strength of soil is defined as the magnitude of shear stress that a soil mass can sustain without undergoing failure. The shear strength of soil is thus the most important criteria for assessing the performance of the soil when subjected to external loading. In the following sections literatures are presented in order to understand the fundamental aspects related to shear strength of soil and explore the current laboratory techniques used to determine the shear strength of soil.

Soil derives its shear strength primarily due to three aspects viz. interlocking of particles, frictional resistance between individual soil grains and cohesion i.e. adhesion between soil particles. Over the years researchers have extensively studied the shear strength of soils.

**2.1.1 Coulomb(1776)** expressed the failure strength of soil in terms of two variables namely the cohesion ( $c$ ) and angle of internal friction ( $\phi$ ) as presented in Fig. 2.1.

$$\tau_f = c + \sigma_f \tan \phi \quad (2.1)$$

Where  $\sigma_f$  is the normal stress acting on the failure plane. This is the first expression established for quantifying the shear strength  $\tau_f$  of a soil. He differentiated the compressive stress with respect to  $\theta$  to obtain the minimum value which was

$$\theta_f = +45^\circ - \phi/2 \text{ or } \theta_f = -45^\circ + \phi/2 \quad (2.2)$$

Where  $\theta_f$  is the angle between potential shear failure and major principle stress  $\sigma_1$ .

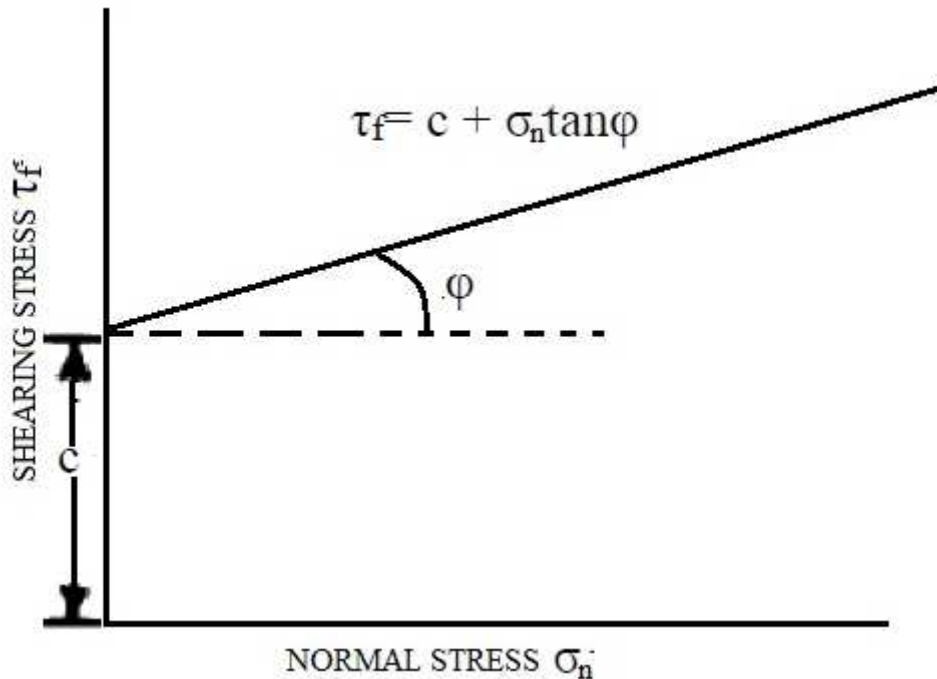


Fig. 2.1: Coulumb's Law

**2.1.2 Mohr(1900)** examined the failure criteria using the empirical data available to him and found none of the failure criterion conformed well to the data. He postulated that the normal pressure on a material and the characteristic properties of a material influence the maximum shear stress acting on a plane. This criterion was applicable to failure shear strength at ductile and brittle state of a material. Further he stated that material properties themselves depended on the state of stresses acting on a material. In 1882, Otto Mohr had developed a graphical method for analyzing stress known as Mohr's circle, using which he proposed an early theory of strength based on shear stress. The Mohr–Coulomb failure criterion is represented by a linear envelope obtained from a two dimensional plot of the shear strength of a material versus

the applied normal stress( as shown in Fig. 2.2). When the applied shear stress exceeds the shear strength of soil, the soil is considered to have failed.

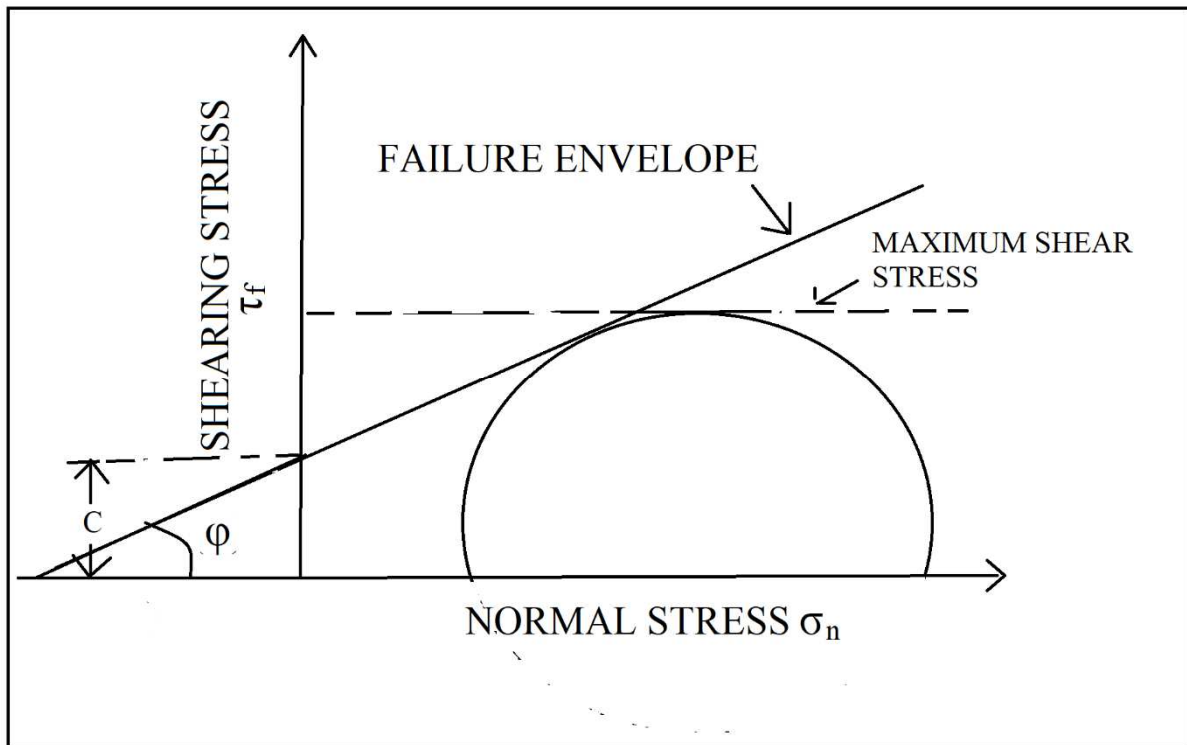


Fig. 2.2: Failure stress relationships using Mohr's Circle.

The concept of pore pressure was presented by Terzaghi in the beginning of 1920s. In 1933 Rendulic was the first researcher to measure pore pressure inside a clay specimen Skempton(1960). Bell (1915) presented the results of the systematic study on shear strength and reported that  $\phi$  value was small for clays. For a clay having  $c=32\text{kPa}$  the  $\phi$  value was close to  $0.5^\circ$ . Terzaghi in the 1920s investigated the physical properties and nature of friction of clays. His test data indicated that surface tension was proportional to the contact area between clay particles. The effect of capillarity forces attributed to the apparent cohesion of clays. Shear strength was thus expressed as

$$\tau_f' = c' + (\sigma_f - u_f) \tan \phi' \quad (2.3)$$

where  $(\sigma_f - u) = \sigma'_f$  effective stress on failure plane, was formulated by Terzaghi. He recognised the importance of drainage on the shear strength of clays. The concept of drained shear strength was proposed by A. Casagrande in 1930-32 (Skempton 1960).

Significant advances were made in understanding the nature of shear strength of clays by Hvorslev (1937). His investigations during 1934 to 1936 in Vienna led to the development of terms such as true cohesion  $c_e$  and true angle of friction  $\phi_e$ .

$$\tau'_f = c_e + (\sigma_f - u_f) \tan \phi_e \quad (2.4)$$

He evaluated parameter  $c_e$ ,  $\phi_e$  by using direct shear tests. Hvorslev found  $c_e$  for clays was only a function of water content and that  $c_e$  was proportional to consolidation pressure. Skempton (1948c) summarized the then available data of  $c_e$  and  $\sigma'_c$  and reported  $c_e / \sigma'_c$  varied between 0 and 0.15.

## **2.2 Laboratory methods for evaluation of shear strength parameters of soft clays**

To determine the shear strength of soils under different loading and drainage conditions laboratory and field methods were developed. Owing to the extensive care and judgement required during sampling, transportation, storage and handling of samples prior to laboratory testing, in-situ tests are often preferred over laboratory tests. Especially in case of cohesionless soil which get severely disturbed during sampling and handling. Correlating laboratory results with actual field performance becomes questionable. For granular soils  $c'$  is often taken as zero and field results are correlated with only  $\phi'$ . In case of granular soil undrained failure can be ruled out owing to high permeability. The most common in-situ tests are vane shear tests which are used to measure  $c_u$  for very soft clays and for sand Standard penetration test is used. For cohesive soils the long term parameters cannot be determined satisfactorily using in-situ tests

and there is a need to conduct laboratory investigation to determine physical and shear strength characters. Even though the  $c$  and  $\phi$  are not truly constant in practice they are considered as constant value at a particular depth below the ground surface. Thus the testing should be carried out at stress magnitudes applicable to the solution and location under consideration. The shear strength of soil can be determined using laboratory tests such as unconfined compression tests, direct shear tests and triaxial tests. Vane shear tests are also used to determine the shear strength of very soft / soft clays in both field and laboratory. Amongst these, triaxial tests provide close simulation of the stress condition present in the ground and thus the shear strength parameters are more realistic. The triaxial setup and shear box equipments are the mainstay of a commercial laboratory especially where both  $c$  and  $\phi$  are required. A special form of triaxial test where  $\sigma_3=0$  is Unconfined compression tests. The stresses acting on the soil sample in a triaxial setup is shown in Fig. 2.3.

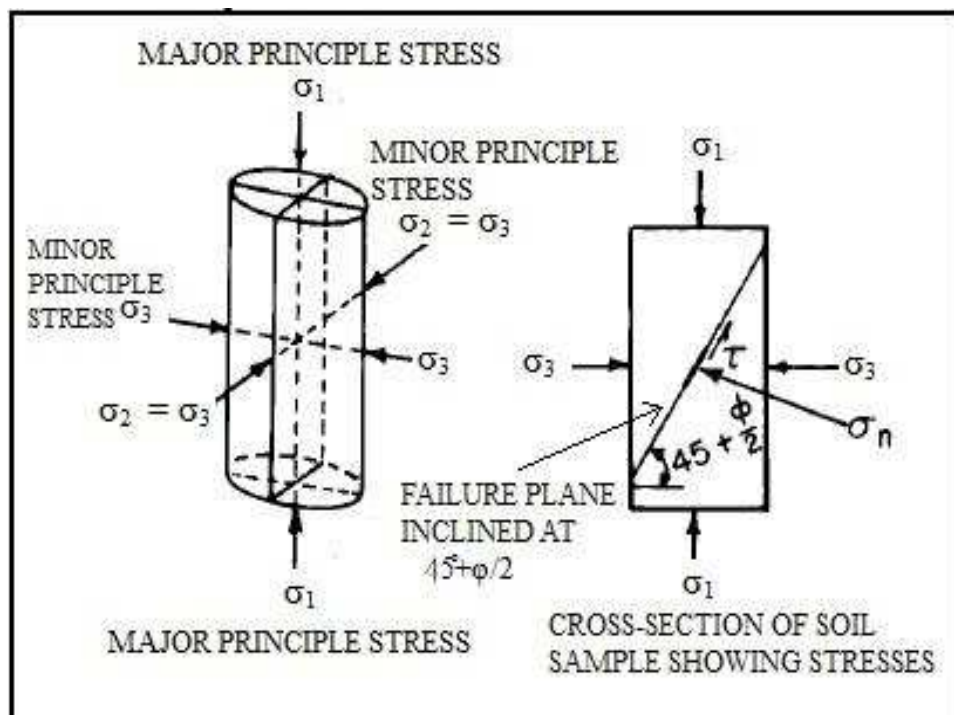


Fig. 2.3: Stresses acting on triaxial compression sample (Raymond 1997)

### 2.2.1 Types of shear testing

According to the manner in which the pore pressure from the test specimens dissipates, there are primarily three types of tests done on soils.

Undrained Tests or Quick Tests: The samples are subjected to an applied pressure and are quickly sheared thereafter. During this entire process no drainage is allowed. Any dissipation of pore water is prevented as the results assume none occurs. The findings are especially relevant to low permeability clays, where drainage is extremely slow or virtually non-existent. If the soil samples are fully saturated and are tested under same stress history then the typical results obtained are shown in Fig 2.4.

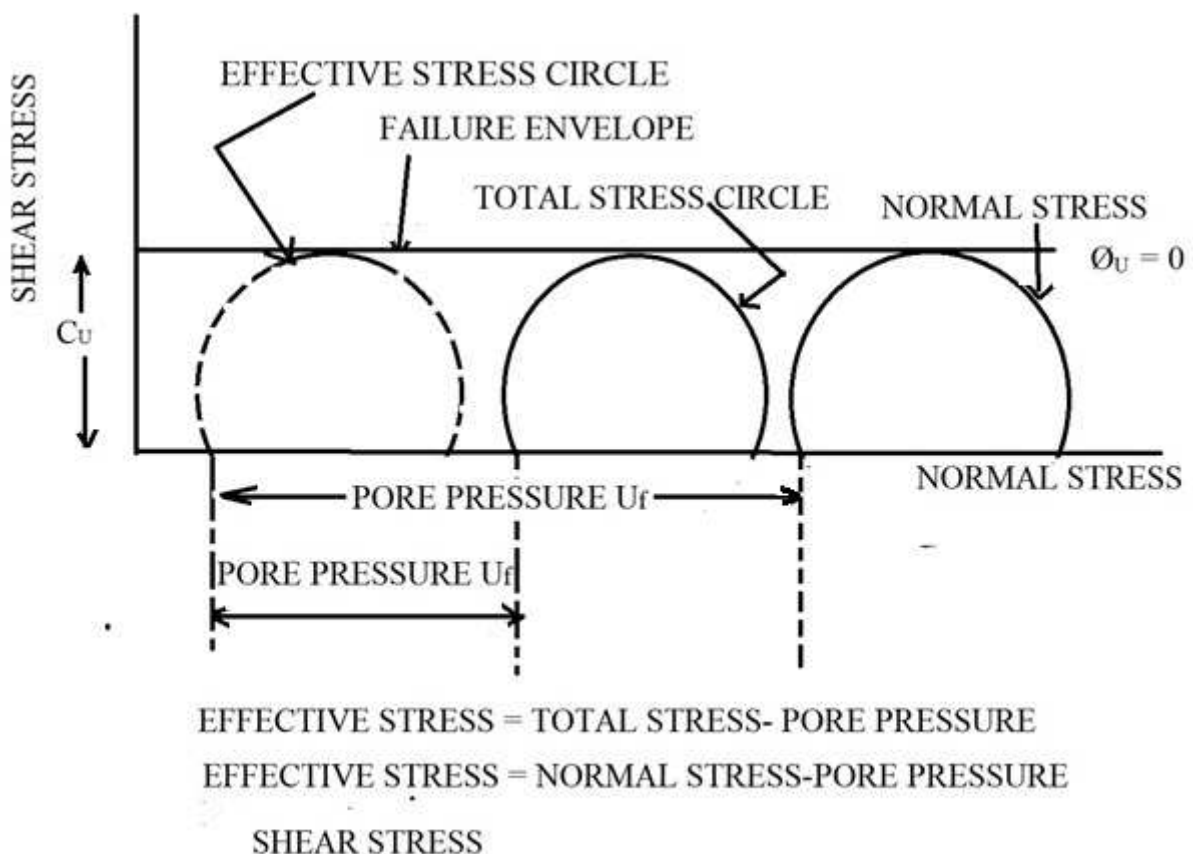


Fig 2.4: Typical results obtained from Unconsolidated Undrained tests.

In-situ vane shear testing can also be used to determine the undrained immediate strength. Clays that are soft and wet are best for this test. The value of  $\phi$  for such soils is assumed to be zero. Similarly when unconfined compression tests ( $\sigma_3=0$ ) is conducted in the laboratory  $\phi=0$  is assumed and undrained cohesion  $c_u$  is obtained as  $(\sigma_1-\sigma_3)/2$ . Samples from different depths are tested under suitable/applicable pressure to obtain  $c_u$  values corresponding to a particular depth.

Consolidated Undrained Tests (CU) :the soil sample is allowed to consolidate under applied pressure. Once the equilibrium condition is attained the drainage lines are closed. Thereafter the sample is sheared without allowing any drainage. The generated pore pressure in the sample due to application of pressure is not allowed to dissipate during shearing. The total stress parameters  $c_{cu}$ ,  $\phi_{cu}$  obtained have little practical application, hence there is a need to measure the pore pressure developed in order to estimate the effective stress parameters  $c'$  and  $\phi'$  which are useful to predict the long term behaviour of clays. Typical results of total and effective analysis are shown in Fig 2.5. In order to allow equalisation of the generated pore fluid the test should be conducted at a very slow rate. For clayey soils tested in CU triaxial tests, owing to the very low coefficient of permeability of clays, the samples need to be sheared very slowly so as to allow the pore pressure to develop uniformly throughout the sample. The effect of rate of strain on mechanical properties of soils can have significant engineering implications. In engineering systems the strain rates generally experienced are in the range of  $10^{-2}$  and  $10^{-3}$  %/hr( Bjerrum 1972, Prapaharan et al 1989).

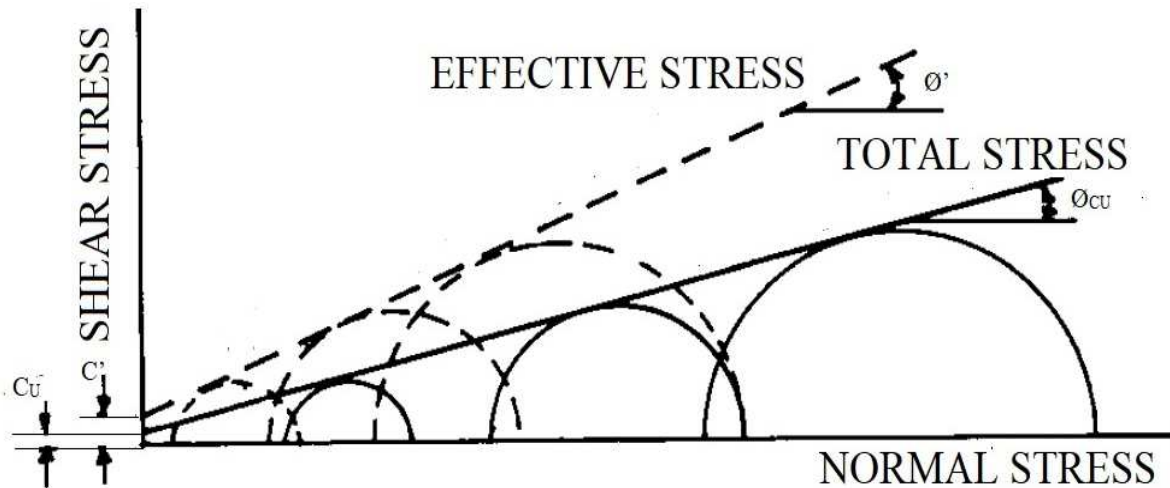


Fig. 2.5 : Typical results obtained from Consolidated Undrained tests

Consolidated Drained Tests (CD): The samples in this case are consolidated under an applied pressure by allowing drainage similar to CU tests. During shearing of the samples the drainage line is kept open thereby allowing drainage to occur even during shearing stage. The test should be conducted slow enough to allow the dissipation of pore fluid. Since drainage is allowed even during shearing stage, permeable materials such as granular soil which allow pore fluid to escape are sheared quicker than clays which have low permeability. Since the effective and total stresses are same, theoretically results are expected to be similar to effective stress analysis obtained from CU tests.

Drained and undrained triaxial tests were introduced to demonstrate the principle of effective stress and for determination of effective shear strength parameters (Rendulic 1935, Skempton 1960, Bishop 1950). The first triaxial test was conducted by Theodore von Kármán in 1910 (Vásárhelyi 2010). He conducted triaxial tests on Carrara marble and Mutenberg sandstone with varying confining pressures up to 600 MPa. He demonstrated transition of the soil from brittle behaviour to ductile behaviour in addition to hardening. His main objective was to check the validity of the two principle hypotheses of Mohr which stated that the limit of elastic

behaviour of a material is independent at the value of intermediate main stress and limit of elasticity and failure stress is determined by  $\tau = f(\sigma)$  for brittle materials. Former experiments had established that these hypotheses were valid for elastic and plastic materials but were questionable for brittle materials. The CU triaxial tests are generally used for short term behaviour of clays under undrained conditions to assess the stability of slopes, foundations, retaining walls and excavations. The CD test results highlight the long-term behaviour of clayey soils. The CU test is a more popular triaxial test as the undrained shear strength in addition to the effective shear parameters can be calculated using these tests. In case of sands the CU tests require significantly less time compared to CD tests.

### **2.3 Comparison of test results obtained from CU and CD**

The conventional strain controlled undrained and drained tri-axial tests are conducted in the laboratory to determine the shear strength parameters for analysis and design of foundations, embankments, dams, excavations etc. Theoretically the effective stress parameters obtained from consolidated drained and undrained tests are identical but practically differences in results have been reported by various researchers. Bjerrum and Simons (1960) reported consolidated drained and undrained tests on normally consolidated clays of Canada, Sweden, New Zealand, Oslo, Sarpsborg etc. and indicated a difference of 2-3° in  $\phi'$  values obtained from undrained and drained triaxial tests. The  $\phi$  values obtained from consolidated undrained tests were higher than corresponding drained tests results. Triaxial consolidated drained and undrained tests were conducted by Casagrande and Wilson (1953) on organic clays and Boston Blue Clay, Casagrande and Rivard (1959) on highly plastic soils. Hirschfeld (1959) conducted tests on Weymouth clay and New Haven clay and found  $\phi'$  (undrained) appreciably greater than  $\phi_d$ (drained) values. This behaviour has also been reported by Ting and Ooi(1976) and Taha(1998). Salih and Kassim (2012) compared the effective shear strength parameters

obtained from CU and CD tests conducted on highly plastic silts of Malaysia. The shear parameters obtained from CU were higher than corresponding CD test results. The authors suggested that CD triaxial test results were more reliable and also proposed a reduction factor in order to reduce the  $c'$  and  $\phi'$  values obtained from CU tests. Some of the literatures reporting a difference in  $\phi'$  (CU triaxial) and  $\phi_d$  (CD triaxial) of clayey soils have been presented in Table 2.1.

Table 2.1: Difference in  $\phi'$  and  $\phi_d$  values

<b>Soil type</b>	<b>State</b>	<b>Difference in <math>\phi</math> values from CU and CD triaxial (<math>\phi' - \phi_d</math>) in degrees</b>	<b>Reference</b>
Weald Clay	Remoulded	0.5-1	Henkel(1956)
London clay	Remoulded	0.5-1	Henkel(1956)
Fredrikstad clay	Undisturbed	1-2	Bjerrum and Simons (1960)
Fornebu clay	Undisturbed	0.5-1	Bjerrum and Simons (1960)
Seven sisters clay	Undisturbed	2-3	Casagrande&Rivard(1959)
North Ridge clay	Undisturbed	2-3	Casagrande&Rivard(1959)
Organic clay	Undisturbed	9	Casagrande& Wilson(1959)
Boston Blue Clay	Undisturbed	3	Casagrande
Weymouth clay	Undisturbed	3	Hirschfeld (1959)
New Haven clay	Undisturbed	5-6	Hirschfeld (1959)
Malaysian residual granite soils	Not reported	3	Taha(1998)
Malaysian residual granite soils	Not reported	2-3	Ting &Ooi(1976)
Silt (Malaysia)	Remoulded	3	Salih and Kassim (2012)

## 2.4 Stress controlled shear strength parameters

The classical failure theories, i.e. the Mohr-Coulomb theory expresses the shear strength of clays as a function of inter-granular normal stress and the slope, intercept of the shearing resistance-effective stress relationship (Mohr 1882). These terms do not indicate mechanism of shearing resistance which depends on a number of factors such as stress history, temperature, rate of deformation and structure of soil. Time dependency of stress-strain behaviour is a very important aspect to be considered for geotechnical problems. Due to particulate nature of soil, deformation involves time dependent structural rearrangement of soil particles (Erol 1977). The actual mechanism of deformation of the soil under different field conditions is thus stress dependent. The soil deforms under the action of load imposed on it as a result of various structures/constructions carried out over the soil. Stress dependent tests are thus necessary to investigate the stress-strain-time aspects and understand the influence of time and rate of strain on strength and deformation. The behaviour exhibited by soils is dependent on the stress-levels [Taylor(1948), Leonards and Ramiah(1959), Lade and Duncan (1976) Tanevas et al.(1983), Duncan(1993), Terzaghi et al(1996). Elkateb (2018) conducted tests on marine clay of the Northern Arabian Gulf to investigate the dependence of the coefficient of consolidation on stress level. The test results indicated that the coefficient of consolidation ( $c_v$ ) and coefficient of permeability ( $k$ ) were stress-dependent. As most of the geotechnical problems involves the application of sustained loading, there emerges a necessity to examine the stress dependent behaviour of soils especially clayey soils. In many cases significant long-term residual settlement of structures constructed on clayey soil (e.g. Leroueil and Hight, 2003) has been reported. This issue has mainly been investigated for clayey soil (Murayama and Shibata, 1958; Bishop, 1966; Leroueil et al., 1985; Christensen and Wu 1964; Singh and Mitchell 1969; Borja and Kavazanjian 1985, Enamoto,2016). The main cause of long-term residual settlement is

attributed to the creep behaviour, caused by the viscous properties of geomaterials, in which deformation occurs under the constant effective stress state.

## 2.5 Creep of soil

It is very important to understand the phenomenon of creep of soil and how it is defined so as to understand its implications in geotechnical application. A creep test is depicted in Fig. 2.6.

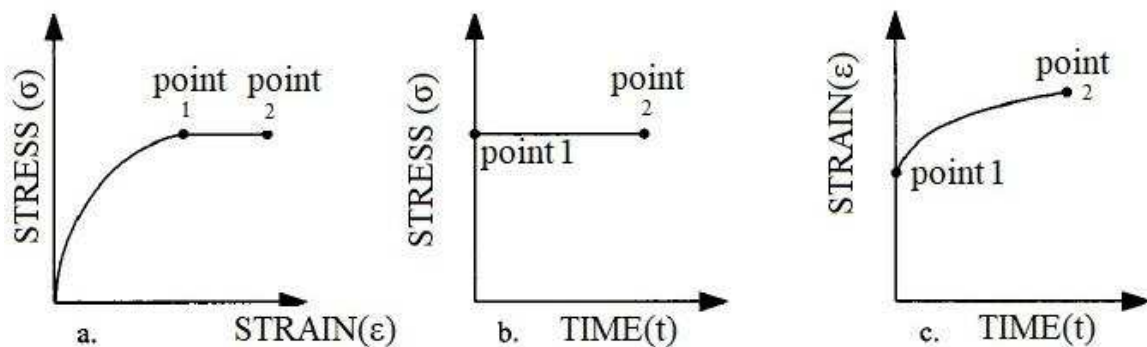


Fig 2.6 : (a) Stress-strain relationship (b) stress history (c) strain history.

The soil is sheared till it reaches a point 1 as depicted in the Fig 2.6(a), post which a creep process is initiated by keeping the stress acting on the soil constant over a period of time as depicted in Fig 2.6 (b). During this process a gradual increase in strain is observed as illustrated in Fig 2.6 (c). It can be summarized that during a creep test the rate of strain gradually increases when subjected to constant stress. The results of a constant stress triaxial test depicting the various stages of creep are presented in Fig 2.7 (a) and (b). The creep process is divided in three parts namely primary creep, secondary creep and tertiary creep characterized by a decreasing, constant and increasing strain rate respectively. The variation in strain rate with time is depicted in Fig 2.7 (b). Tertiary creep or acceleration creep eventually leads to creep failure or creep rupture. Primary, secondary and tertiary creep is different from primary, secondary and tertiary compression evidenced during oedometer tests. In oedometer tests

primary, secondary and tertiary compression is defined by plotting strain rates against logarithmic time as shown in Fig 2.7 (c). Primary compression phase is where excess pore pressure dissipates, secondary consolidation corresponds to pure creep, i.e., deformation occurs due to deformations in the soil skeleton and tertiary phase is subsequent to secondary phase characterized by non-linear relationship between  $\log(\text{time})$  and strain [ Fig 2.7 (d)]. From the figure below it can be inferred that primary, secondary and tertiary creep are associated with decreasing, constant and increasing strain rate whereas primary, secondary and tertiary consolidation are all associated with decreasing strain rate.

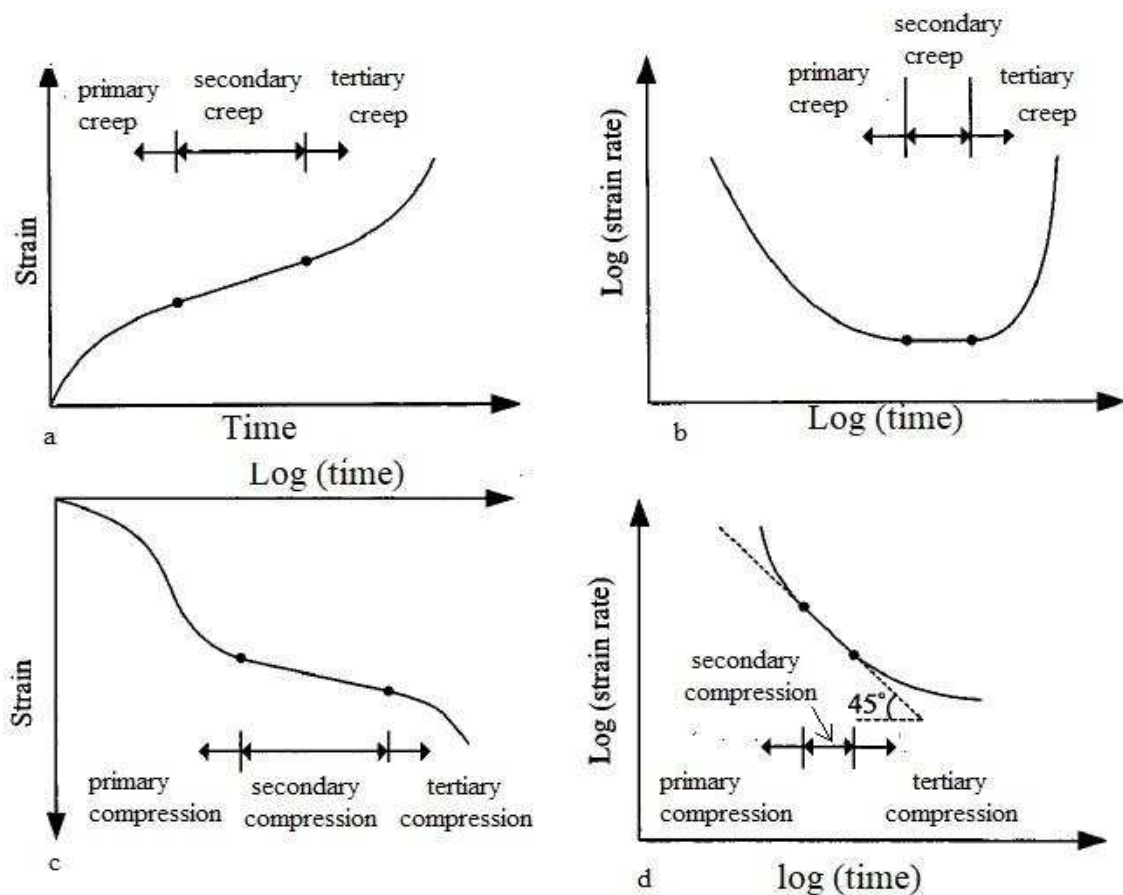


Fig 2.7:(a) Strain versus time for creep test and (b)  $\log(\text{strain rate})$  versus  $\log(\text{time})$  for creep test(c) Strain versus  $\log(\text{time})$ for oedometer testand (d)  $\log(\text{strain rate})$  versus  $\log(\text{time})$  for oedometer test (Augusteen et al. 2004)

One of contentious issues over the years has been the selection of the reference time which is concerned with deciding when the creep strain begins. Selection of reference time plays an important role in determining the magnitude of creep strain. There are two ways to establish selection of reference time. The first scenario where reference time is taken as the time at end of primary consolidation and thus it is dependent on thickness of the soil or drainage length. The second case where reference time is considered as an intrinsic parameter for a given soil and hence independent of drainage conditions and soil thickness. These two aspects led to the two well-known approaches adopted in estimation of secondary compression; Hypothesis A and Hypothesis B (Ladd et al. 1977). Hypothesis A provides a unique value of strain at EOP, which corresponds to the fact that the soil does not show any time dependent creep behaviour during during pore pressure dissipation that affects the strains at EOP. This hypothesis is suggested by (Ladd et al. 1977; Leonards 1977; Mesri and Choi 1985a,b) and that assumes secondary compression occurs only after primary compression and both processes are mutually exclusive. Hypothesis B assumes creep occurs throughout the consolidation process and the strain at the end of EOP is not unique. Time dependent strains thus take place during primary consolidation as suggested by Bjerrum (1967), Leroueil et al. (1985), Crawford (1986), Kabbaj et al. (1986), and Yin (1999) among others. Still there is no general agreement over which approach is closest to reality as highlighted in the works of Duncan et al. (1986) and Leroueil et al. (1985) who pointed out that experimental evidence is not convincing as Leonards(1977) justified the theory A using the same consolidation test results obtained by Berre and Iversen (1972) on specimens of different heights to validate method B. Real soil behaviour practically lies somewhere in between since the two hypotheses correspond to two extreme cases. This has been affirmed by researchers like Aboshi (1973) who conducted an experimental investigation to assess the phenomenon of similitude in normally consolidated clays.

For triaxial creep tests, in connection with creep process two types of tests exist namely drained creep and undrained creep. In drained creep the mean effective stress  $p'$  and deviator stress  $q'$  are kept constant as drainage is allowed. In undrained creep tests the pore pressure builds up thus the mean effective stress  $p'$  decreases and deviator stress  $q'$  remains constant. As per definition of creep (development of strains under constant effective stress), drained creep is considered as pure creep and undrained creep test does not represent a pure creep process. Undrained creep deformation from triaxial tests consists of plastic deformation (due to changes in effective stress) and inelastic (creep) deformations. However in literatures both processes are described as creep.

## **2.6 Case studies on creep settlement**

A major portion of the land area on the surface of the earth comprises of soft soils like marine and estuarine clays. In geotechnical engineering terms, these soils are problematic and construction over soft soils poses a challenge to engineering. The difficulty in construction is attributed to the low strength, high compressibility and high natural moisture content of these soils. Due to the rapid expansion in infrastructure to cope with the growth in population there is a need to construct over these soft soils. Embankments remain an integral part of development thus the construction of embankments over soft soils is inevitable. In this section the behaviour of embankments constructed over soft soils has been studied and an attempt has been made by citing available case studies to understand the problems affecting the performance of these embankments. In this section the creep (time-dependent) behaviour and its importance is discussed with the help of certain case histories.

### **2.6.1 The Leaning Tower of Pisa**

The leaning tower of Pisa, Italy is a classic example of creep settlement of soil. It is an architectural marvel constructed during the years 1174 to 1370. The tower was constructed in

three phases. The first four floors were constructed between the years 1173 to 1178 following which the construction stopped. The next three floors were constructed in the second phase of construction during the years 1272 to 1278. The third and final phase of construction began in 1360 and the tower was completed in 1370. The area of ring foundation provided was 280m<sup>2</sup> with outer diameter of 19.35m and inner diameter of 4m. The net pressure on the soil below foundation was 505kPa resting on soil having an undrained shear strength of 55kPa (Jamiolkowsky 2006). The tower began to tilt during construction period when the pressure reached 331kPa due to inadequate undrained strength below the foundation in the southern side. The sectional view of the tower is presented below (Fig. 2.8).

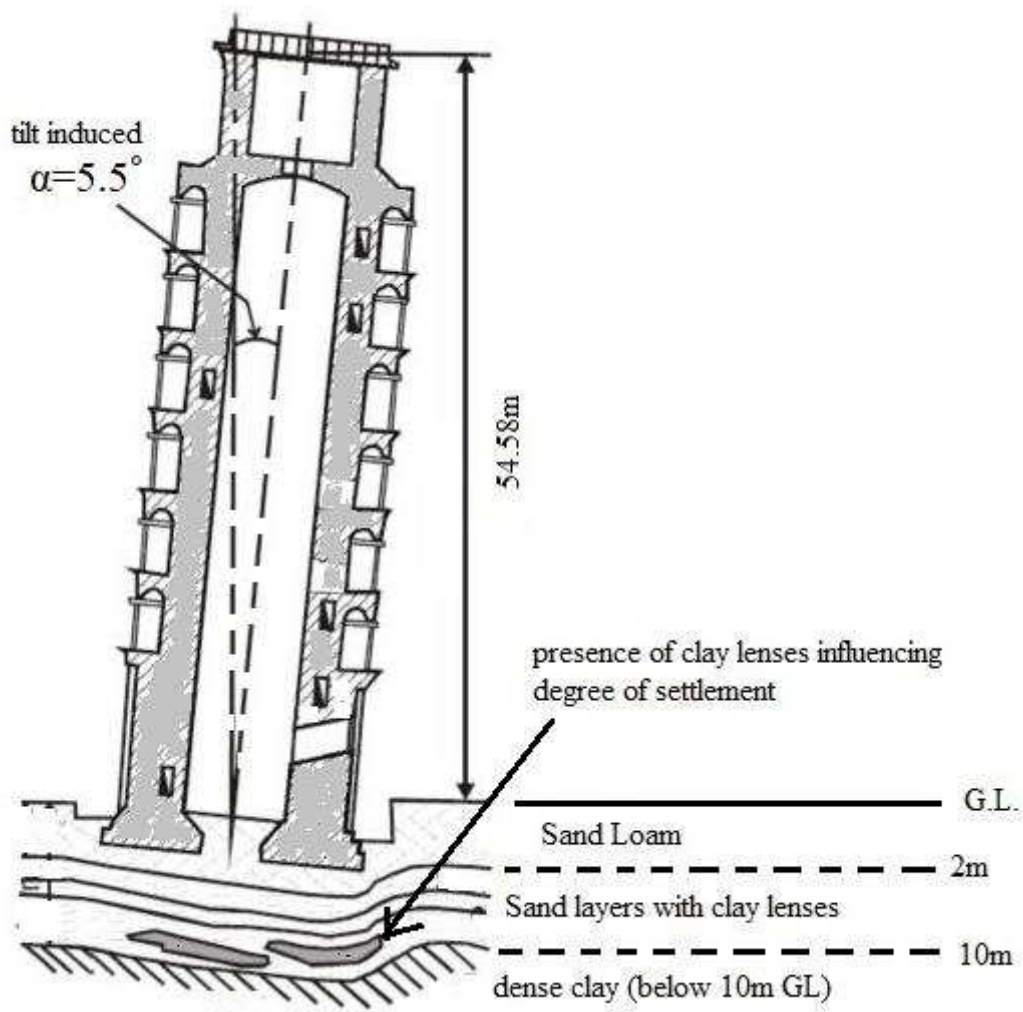


Fig. 2.8: The Leaning Tower of Pisa sectional view (Meschyan 1995)

### *Subsoil Conditions*

The subsoil below the tower consisted of three main strata belonging to the Iolocene and Pleistocene age (Jamiolkowski 1993). The top layer is made of slightly clayey and sandy yellow silt with interbedded lenses and layers of sand and clay. The subsequent stratum is predominantly clayey in nature and the third consists of slightly silty sand, which extends at least to a depth of 65 to 70 m below ground level. The unit weight of the soil in these strata were  $19\text{kN/m}^3$ ,  $16\text{-}20\text{kN/m}^3$  and  $21\text{kN/m}^3$ . The liquid limits varied in the range of 28-42%, 34-78% for stratum one and two. The plasticity indices ranged from 22 to 100% for the top stratum and was greater than 80% for the second stratum. Several researchers (e.g Mitchell et al., 1977, Lenonards 1979, Jamiolkowski 1993, Meschyan 1995, Mitchell and Soga 1995, Jamiolkowsky (2006) have extensively studied the settlement in order to determine the deformation mechanism. Due to the presence of soft plastic clay strata below the tower which was subjected to low overburden pressure pre-construction, the tower started to settle on application of superstructure loading. The tower continued to tilt for decades before the construction was completed. By the end of construction in the 14<sup>th</sup> century the structure had already tilted by 2.1 m on one side. The tower continued to settle over the centuries after construction and is presently settling at 2mm per year. The average settlement has been recorded over 1.5m and the tilt is 5.58m. The tower is 54.583 metres high from the ground on the low side and 56.29metres on the high side.

#### **2.6.2 San Jacinto Monument**

The San Jacinto Monument, the tallest column monument in the world according to the Guinness Book of World Records was built in 1936 to honour the 100-year anniversary of the victory of the Texan army over the Mexican army. It consists of a 176.5-m-high column founded on a large steel reinforced square mat with each side of 37.8 m. The mat is 4.6m thick at the centre having a width of 14.3 tapering to 1.8m at the edges. The total weight including

the column, a star at the top of the column, a museum at the base and mat foundation was estimated to be 312.7 MN (Cummins 1937a). The foundation was designed by Raymond Dawson considering an average pressure of 223.8 kPa on the soil under the mat. The mat was laid at a depth of 4.6m below ground level resulting in the net pressure acting on the soil below foundation being 141kPa.(Unit weight of the soil removed from above foundation was  $18\text{kN/m}^3$ ). Dawson had setup settlement points on the mat foundation along with benchmarks away from the monument. The settlements recorded over the years were reported by Briaud et al.(2007). The settlements continued at a constant rate of 6mm/year even 45 years after construction when the average settlement measured was 0.3m. The settlement of the structure over 70years was measured to 0.329m.

#### *Subsoil Conditions*

The subsoil at the site consisted of clay upto a depth of 20m, a thin strip of clay with sand seams of average thickness 2m was present in between this layer at depths 10-12m. The borehole data collected at various time after construction till date depict a variation in ground water level was found to vary between depths of 0.61m to 11.9m. The moisture content of the soil strata at depths 0-10 m was found to be close to 35% and reduced to 20% at greater depths. The plasticity index of the top soil was as high as 55%. A reasonable estimate of a lower bound of undrained shear strength of soil at the site is about 100 kPa as obtained from soil investigations carried out in 1953, 1954, 1958, 1959, and 1962. The average of coefficient of compression determined from analysis was 0.255 and average  $c_v$  value was  $1.3 \times 10^{-7} \text{ m}^2 / \text{s}$  (Briaud et al.2007).

#### **2.6.3 Reported cases of failure of structures**

Failure of a retaining wall took place in Kensal Green, London in the year 1941. The wall was constructed in the year 1912 for the widening of the London-Birmingham Railway. A failure

had occurred at the highest section of the wall in 1929 following which repairs were conducted and several observation points were setup. Owing to one of these observation points being close to maximum movement during 1941 slip 12 years of displacement data were recorded continuously. The initial rate of movement was thus first recorded in 1929, 17 years after construction of wall. The rate was found to be 6.35mm/year which gradually increased finally resulting in a displacement of 0.4572m. This is a perfect example of progressive failure. Post failure analysis by Skempton (1964) revealed that the average strength had decreased by 60% at the slip surface in a span of 29years after completion of construction. The values of average peak and average residual strength for the soil was 29.2kN/m<sup>2</sup> and 11.2 kN/m<sup>2</sup>. The average shear stress along failure plane was 18.55kN/m<sup>2</sup>. From the design point of view the factor of safety using peak value of strength was 1.6 and factor of safety corresponding to residual strength was 0.6. Factor of safety of 1 is considered appropriate.

#### Sudbury Hill

The third slope failure considered exhibiting similar loss of strength with time was evidenced in Sudbury hill excavation failure in 1949. The excavation was completed in 1900. Intermittent movements of small magnitudes occurred even when the failure was been investigated in the years between 1949 and 1956. The water levels were measured using three piezometers and using slip circle method the effective stress and shear stress were estimated. The angle of friction  $\phi' = 15^\circ$  was estimated and using back analysis the value of friction angle during failures was estimated as  $\phi' = 17^\circ$ . The residual strength during failure was 80%.

#### Northolt Station, England

A similar failure to Kensal Green retaining occurred in a 10m excavation in Northolt Station, England. The failure occurred in 1955, 19 years after the completion of the excavation. Post

failure analysis carried out by Skempton(1964) considering the slip surface to be approximately circular revealed a 56% loss in strength. The values of average peak and average residual strength for the soil was  $36.6 \text{ kN/m}^2$  and  $10.5 \text{ kN/m}^2$ . The average shear stress along failure plane was  $18.55 \text{ kN/m}^2$  thus reducing the factor of safety below 1. A similar phenomenon was evidenced in 1952 when a landslide occurred at the village of Jackfield, Shropshire, on the River Severn 114 miles downstream of Iron Bridge; destroying several houses and causing major dislocations in a railway and road. Factor of safety for the slope using peak shear strength was 2.06 while that using residual strength was 1.11 which was reported to trigger the large movement of the slope ultimately leading to its collapse.

#### Failure of two dams in France

A 520m X 22 m dam was commissioned in 1895 in France, constructed on a 6m bed of argillaceous sandstone. The structure collapsed 11 years after its construction The cause of failure was its incapability to resist creep induced progressive displacement. In another instance a 550m X 28.3m dam was constructed on compact fissured clay at depths 2m to 10m. The structure failed due to significant deformations from its origin (Maslov 1968b).Haefeli et al. 1953 reported the creep induced deformation of an r.c. arched viaduct in Switzerland. Immediately after the construction of the bridge the abutment started shifting at a steady rate of 37 mm per year. The total deformation amounted to 480 mm in 13 years time. The vertical displacement during this period was only 14.9mm. Luga (1964) described a situation encountered in stiff and hard clay soils where creep caused movement 3 abutments of a railway bridge through horizontal distances ranging from 9.3 to 46 cm in a span of 38 years (1916-1954). Land abutments of bridges over the Danube in Budapest, as well as those of the Reichsbrücke, etc., have been known to suffer from long-term displacements (Vyalov 1986).

At the 2nd ICSMFE (Rotterdam, 1948), Peck reported the effect of creep on retaining walls in U.S.A constructed to arrest long term deformations of slopes. Out of the retaining walls studied 18% of the walls failed due to creep. In 53 cases the walls were in a state of progressive displacement, 11 % were in the state of stabilized displacement. 4% of the walls inspected showed insignificant displacements.

Bhandari and Roy Chowdhury 2005 discussed the failure of a highway embankment constructed over soft to very soft marine clays. A 10.5 km stretch of highway linking Vizag port to NH-5 was planned by the National Highway Authority of India (NHAI) under infrastructure development programme launched by Govt. of India. The subsoil conditions comprised of soft to very soft clays extending up to considerable depths. A 3.33m high embankment above the original ground level was constructed in a 170m long stretch at a section between 4.250 km and 4.420 km. For constructing the remaining height of the embankment 30 truck load of fill material were deposited on April 23, 2003. During evening inspection a longitudinal crack running along the center of the embankment was observed. A week after the development of the crack, the left side of the embankment was found to settle by 500mm more relative to the right side which had only settled by 100mm. A waste water drain with crack was observed. No such cracking was observed on the right side of the embankment.

The reasons for absence of cracking on right side of the embankment were pointed out in the following points

1. Rotational slip of the right toe was prevented by the presence of haul road which was well compacted under traffic close to the right side of the embankment.
2. Absence of water body on the right side and presence of a water body 60m at a distance of 60m from left toe.

The subsequent measurements made with the help of a reference pillar showed a horizontal displacement of about 80cm and a heaving of 20cm. The crack developed on the water drain was measured to be 25mm-50mm.

Huat 1995 discussed the failure of three embankments during construction of a highway stretch in northern Peninsular Malaysia. The first embankment was constructed on untreated soft clay with a proposed height of 5m and a side slope of 2 horizontal to 1 vertical. A tension crack developed on left side of the embankment, 155 into the construction with the height of embankment being 4.6m. Another embankment was constructed for a rubber and palm oil state, having a height of 3.2m including 1.5m surcharge. 145 days into the construction the embankment collapsed without any prior development of tension crack. A detailed sub soil investigation was conducted to analyse the failure of the embankment. The laboratory investigation revealed that the liquid limits varied in the range of 50%-120% and natural moisture contents of 30-80%. The values of  $c_v$  were low, typically ranging between 1-10m<sup>2</sup>/yr. In case of embankment 1 cracks extended over a longitudinal distance of 80m. Differential settlement of 250mm was observed between the failed and intact sections. Toe heaving was observed 7m from the toe. The details of the failure and the factor of safety at the time of failure has been represented in have been presented in the following Tables.

Table 2.2: Details of the failure

Embankment number	Longitudinal Distance of crack (m)	Distance beyond toe at which heaving was observed(m)	Differential settlement on either side of crack(m)
1	80	7	0.25
2	110	5	—
3	140	4	1

Table 2.3: Details of the factor of safety.

Embankment number	Fill height above OGL (m)	Fill strength		Factor of safety
		C' (kPa)	$\Phi'$ degree	
1	80	17	27	0.91
2	110	42	26	1.03
3	140	25	23	1.04

The geotechnical analysis highlighted the causes of failure of the embankment. The failure was preceded by soft response of the foundation with regards to generation of excess settlements, lateral deformation and excess pore water pressure.

Roy and Singh 2008 investigated the failure of two embankments constructed over soft clays near Kolkata, India. The embankments were constructed on soft and compressible soils of intertidal flats and back swamps of the Ganges delta. The failure of the first embankment occurred on July 15, 2003. The side slope of the embankment was 2 horizontal to 1 vertical supported by a mechanically stabilized earth wall at the sides. Nearly the complete MSE wall underwent a deep seated failure in early hours of rainfall and an additional 0.4mm rainfall the following day. The height of embankment was between 8.9 to 9.8m. The second instance of failure involved a 30 year old, 9 m high embankment with side slope 3H:1V. The failure occurred on 9 February 2006 one month after the opening of the highway for vehicular traffic. During the failure the 2 lane approach settled vertically by 3m and translated outward by 1m. The embankments appeared to fail due to external instability rather than internal distress. Sub surface investigation and monitoring was undertaken to understand the causes of failure. Upon detailed investigation the following reasons were attributed as the cause of failure of the embankment

The rapid rate of construction prevented completion of consolidation and hence the affected the development of undrained shear strength of the soil.

The driving force was underestimated because of the use of smaller embankment heights and material unit weight for stability analysis during initial designing.

## **2.7 Laboratory studies investigating creep of soil**

Over the years researchers across the world have performed several creep tests to assess the problem of creep in clayey soils. (Bishop & Lovenbury, 1969; Mesri, 1973; Tavenas et al., 1978; Tian et al., 1994; Yin, 1999; Zhu et al., 1999). However the phenomenon of creep has not yet been completely assessed owing to the complex elasto-viscoplastic properties of clayey materials. This section highlights some of the experimental investigations carried out which have provided an insight into creep behaviour of clays.

Creep behaviour of clays have been assessed using one-dimensional compression, direct shear creep tests, pressuremeter creep tests, rheometer creep tests, triaxial shear and large scale field creep tests. Laboratory creep tests remain the main approach for understanding the creep behaviour as it is easier to observe the behaviour over a longer time period, it offers better control over testing conditions and is less costly. This accompanied with the fact the repeated trials can be conducted is a primary reason why laboratory tests are preferred to field tests.

### **2.7.1 One-dimensional compression tests**

The vertical effective stress in one-dimensional creep tests is maintained constant and the corresponding development in axial deformation with time is monitored. It is the most basic test which helps understand the creep behaviour of clays by separating the total settlement into 2 parts: the primary compression and secondary compression. The coefficient of secondary compression can be calculated and its evolution over time can be quantified.

In one-dimensional consolidation tests, for each load increment the deformation is due to primary consolidation i.e., the pore water pressure dissipation and the secondary compression (Bjerrum, 1967; Graham et al., 1983). Yin (1999) summarized the volumetric creep strain mechanism under the drainage condition in three components:

- (a) The primary stage, under the vertical load the compression deformation causes a decrease in the volume whereas pore water pressure increases. The deformation caused due to the dissipation of the pore water pressure is significantly less than that caused by compression.
- (b) The secondary stage, the pore water pressure dissipated to its initial value and the volume variation of soil is mainly caused by dissipation of pore water pressure.
- (c) The third stage, is the pure creep stage when deformation of soil occurs after the dissipation of the pore water pressure.

Bishop (1966) first highlighted that under sustained loading the available strength of a clay stratum changes from the undrained strength to the drained strength. The rate of this change is dependent on the length of drainage path and the coefficient of swelling or consolidation. In majority of the foundation problems the rate of change increases with increase in load and in case of excavations or cutting scenarios owing to the reduction in load the rate of change also reduces. However, the shear strength parameters estimated from actual slips on the basis of knowledge regarding the field pore pressure varies drastically from the peak strength values measured in the laboratory in the case of overconsolidated clays. This observation was pointed out by Skempton during the fourth Rankine Lecture. The following aspects may be the reason for this discrepancy:

- (a) The shear strength parameter estimate is influenced by the orientation and size of the test specimens during off-field testing.

(b) Progressive in-situ failure is most likely occur in soils which are brittle in nature or the soils which exhibit work-softening stress-strain behaviour. In such scenarios the ultimate strength will not be mobilized simultaneously along the complete slip path.

(c) For highly over-consolidated clays and clay shales the peak values of the shear strength parameter under drained conditions might be significantly time-dependent.

Bishop (1966) reported drained creep triaxial test results on block samples of London Clay continued for about seven months. Samples were subjected to constant shear stresses of magnitude 90%, 80%, 70%, 60%, 40%, and 16% of the peak drained strength. It was observed that under the loading of 90% of peak strength the sample underwent failure after two days, post sufficient warning of its failure. The 60% loading test exhibited a reducing rate of strain for the first three and a half months (100 days), but the rate subsequently increased steadily prior to its failure. In undisturbed clay the creep rate is not constant under sustained loading at any stage of the testing. There is either a steady decreases or increases.

The undrained creep behaviour of undisturbed San Francisco Bay Mud was investigated by means of triaxial tests by Singh and Mitchell (1969). It was derived that logarithm of strain rate decreases with logarithm of time and the slope of the  $\log \dot{\epsilon}$  vs.  $\log t$  relationship is essentially independent of creep stress. It was observed that there is a linear relationship between the logarithm of strain rate and the deviatoric stress at a given time with the exception of very high or very low stress levels. On the basis of these results a simplified model of the time dependent behaviour of clays was achieved with the combined concepts of limit state and isotaches. The equation for generalized creep of soil was proposed is as follows

$$\dot{\epsilon}_a = Ae^{\alpha D} \left( \frac{t_1}{t} \right)^m \quad (2.5)$$

Where A,  $\alpha$  and m are a measure of the creep potential of a soil. The strain rate is denoted by  $\dot{\epsilon}$ . D is the deviatoric stress or stress level. The reference time is denoted by  $t_1$  and t is the real

elapsed time. This equation has been used by several researchers subsequently to determine the creep parameters of various clays. The parameter  $m$  quantifies the creep potential. The probability of creep rupture is inversely proportional with the value of  $m$ . Most of the creep ruptures were evidenced for values of  $m \leq 1$ . Singh and Mitchell stated that the parameter  $m$  is independent of stress level. Some of values of the parameter  $m$  is presented in Table 2.4.

Table 2.4 : Creep potential of soils.

Soil	Stress history	Test type	M	Reference
Redwood City Clay	Undisturbed	Consolidated undrained	0.75	Singh & Mitchell 1969
Bangkok Mud	Undisturbed	Consolidated undrained	0.70	Singh & Mitchell 1969
Bentonite Sand Mixture	Remolded	Consolidated undrained	0.70	Singh & Mitchell 1969
Kaolinite Sand Mixture	Remolded	Consolidated undrained	1.05	Singh & Mitchell 1969
Osaka Clay	Undisturbed	Consolidated undrained	1	Singh & Mitchell 1969
Sault Ste. Marie Clay	Remolded	Consolidated undrained	1.30	Singh & Mitchell 1969
Pacific Palisade Silt	Compacted	Consolidated undrained	1.25	Singh & Mitchell 1969
Bayer Red Mud	Remolded	Consolidated drained	0.986	Zhao et al 2018
Qianjiangping soil	Remolded	Unsaturated drained	0.89	Lai et al 2010
Hong kong marine deposits	Remoulded	consolidated undrained	0.57	Zhu et al. 1999

An experimental research was done by Bishop and Lovenbury (1969) to pinpoint the main creep properties of weathered, over-consolidated London Clay. The study's goal was to identify the onset of deformation under continuous load over extended periods of time and the drained strength's temporal dependence. According to the authors, the majority of studies conducted before this used short-term experiments conducted in undrained settings as a basis for evaluating soil behaviour over the long term. The development of specialised spring-loaded testing and hydraulically-loaded equipment (based on the triaxial cell) was done in order to address the significant technological challenges associated with carrying out long-term sustained stress tests. The authors were only able to conduct a small number of experiments, some of which lasted nearly three years. Following loading, several stress paths have been tested to determine the features. The results were fully disclosed, but regrettably there wasn't enough information to make straight forward generalizations. All stress levels showed indications of ongoing axial deformation, but at a slower pace and without the "steady-state" phase of creep. The logarithmic law had a narrow range of applications. Even though it was only temporary, an instability in axial strain rate that was seen in multiple tests for weeks following the application of load was thought to reflect a fundamental alteration of the undisturbed soil structure. The maximum loss in the drained strength of the clays under study is less than 15%, and for London Clay, this value was within the range of the recorded 5-day peak strengths. In London Clay, creep seems to have no practical application. However, it might have a sizable impact on the long-term deformation of soft, normally-consolidated clays.

Vaid, Robertson and Campanella (1978) conducted comparative study of the undrained creep rupture characteristics of saturated, normally consolidated, undisturbed marine clay under triaxial and plane strain conditions. Creep rupture tests were performed on samples which were consolidated both isotropically and under  $K_0$  conditions to the same vertical effective stress.

The authors concluded that for a given test type, linear relationships exist between log minimum creep rate and log rupture life and also between log current tertiary creep rate and log remaining time to rupture. However, for the same creep rate, the conventional isotropic triaxial test would result in an unconservative estimate of rupture life or remaining time to rupture by a factor of about four if the real situation corresponds to  $K_0$  consolidation or plane strain. For creep tests the rupture failure time dependent on type of type of consolidation, i.e. the time to rupture varied under isotropic consolidation condition and under  $K_0$  conditions. The authors suggested that in-situ tests were the best option to accurately monitor the behaviour of soils and determine creep rupture.

Zhu et al. (1999) performed both single stage and multi-stage triaxial tests on Hong Kong marine deposits to evaluate the creep behaviour under both undrained and drained conditions. Single stage undrained tests were conducted on soils isotropically consolidated under 400kPa pressure and stress levels of 0.49 – 0.89. Multistage test samples were isotropically consolidated under 200 kPa and stress levels varying from 0.10 to 0.99. Consolidation pressure range for drained triaxial tests was 100 – 400 kPa and a back pressure of 100kPa under deviator stress ranging between 61 – 619 kPa. The Singh and Mitchell parameters calculated on the basis of the results. Using the semi-empirical equation suggested by Singh and Mitchell (1969) the creep potential parameters were calculated from results of multistage loading undrained creep triaxial tests. The volumetric strain of creep was found to be dependent on time and stress ratio. The time dependency of deformations was evident in results obtained from drained triaxial creep tests. Two empirical hyperbolic equations relating the deviator stress and volumetric strain with time were proposed using results of drained creep triaxial tests.

The authors concluded that under undrained conditions the behaviour was influenced by the stress history and empirical equations taking stress history into account was proposed. For

drained creep an empirical equation applicable for all stress levels was proposed. The need for the same was felt by the authors as the results of their study highlighted certain shortcomings in the Singh and Mitchell equation whose applicability was restricted to a particular stress level range.

Zhao et al. 2018 conducted a series of drained creep triaxial tests to investigate the creep behaviour of Bayer Red Mud (BRM) in turn assessing the long term stability and safety of BRM disposal field. The results established that confining pressure and deviatoric stress were critical factors in determining the creep behaviour of BRM. Using two creep models and the test results obtained a comparative analysis of creep of BRM was carried out. A modified version of Burgers creep model was proposed which better predicted the deformation, creep attenuation, steady state creep and accelerated creep of BRM.

Tian et. al 1994 conducted long-term triaxial creep under drained conditions and direct simple shear creep tests on undisturbed marine specimens from the region of the north-central Pacific and the Gulf of Mexico. Results of the study indicated that time-dependent deformations can be represented by a power law equation and the creep strain rate can be estimated by an equation similar to that suggested earlier by other researchers. However, it was observed that the creep potential parameter 'm' suggested by Singh and Mitchell (1969) was not a constant value for a particular soil and is dependent on stress level and sediment plasticity. The value of m was observed found to increase with stress levels. Ageing effects were evidenced in triaxial specimens in the form of increase of shear strength which is dependent on the test duration and stress level.

## 2.8 Observation from creep triaxial tests.

The data obtained from creep triaxial tests is generally plotted in the  $\log \dot{\epsilon} - \log t$  space for visualization of creep behaviour under triaxial conditions. The slope of a straight line in the  $\log \dot{\epsilon} - \log t$  diagram is denoted as the parameter  $m$  and is calculated as shown in equation 2.6

$$m = - \frac{\Delta \log \dot{\epsilon}}{\Delta \log t} \quad (2.6)$$

The characteristics of  $m$  values for three different cases is presented in Fig. 2.9. For  $m \neq 1$  slope values are not constant (line is curved) in  $\epsilon - \log t$  diagrams whereas in  $\log \dot{\epsilon} - \log t$  the slope is constant and is quantified as the parameter  $m$ .

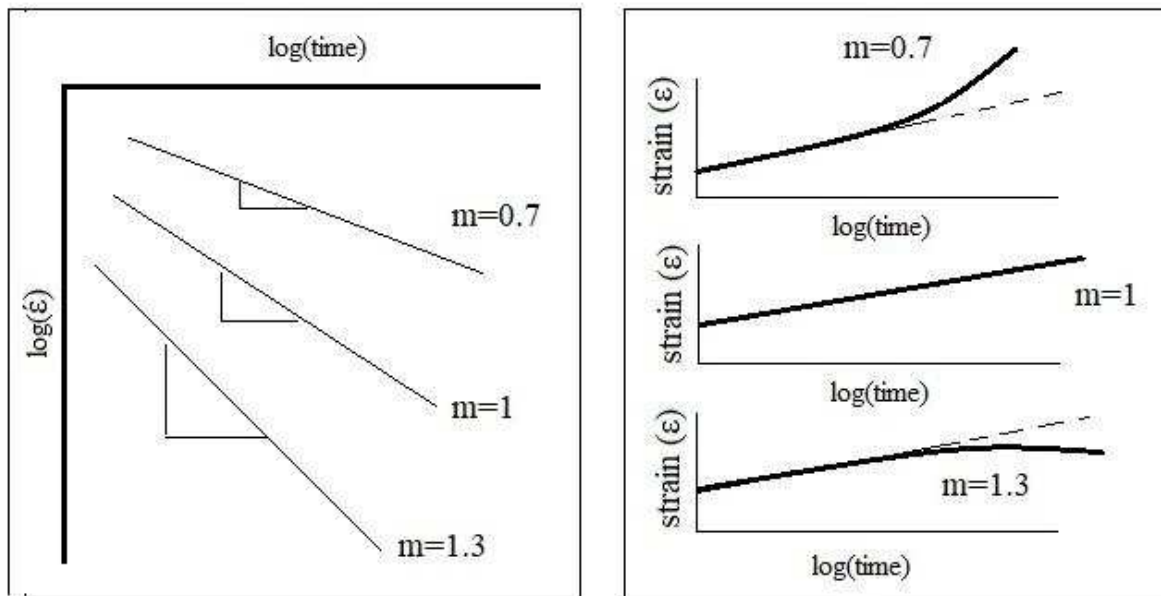


Fig 2.9 : The characteristics of  $m$  values (Augustesen et al. 2004)

A summary of the observations obtained from creep triaxial tests on normally consolidated clays is presented in Table 2.5

Table 2.5 Summary of results

Author	Soil type	Test type	m value	dependence of m on deviator stress level	m value with increasing Deviator stress level
Singh and Mitchell (1969)	Normally consolidated clays (NC)	Drained and Undrained Triaxial	0.75 to a value slightly > 1	Assumed to be independent	Not affected.
Bishop and Lovenbury (1969)	Normally consolidated Pancone clay	Drained Triaxial tests	0.8 to >1.2. For SL= 50% m=0.8 and SL=85% m>1.2	Dependent.	Increases
Tian et. al (1994)	Gulf of Mexico and the North-Central Pacific (undisturbed marine sedimented Clays)	Drained triaxial		Dependent	Increases
Den Haan (1994)	Peat and organic clays	Drained and Undrained Triaxial	Peat : m≈0.7-0.9 Organic soil m≈0.84-0.90	Largely independent	Not affected.
Zhu et. al (1999)	Hong Kong Marine deposits	Drained and Undrained Triaxial	M decreased from 0.91 to 0.57 for q=14 kPa to 121 kPa	dependent	Decreases

The value of  $m$  for NC clays generally varies between 0.7 to 1.2 and in most cases it is less than 1. Several writers have reported that  $m$  is dependent of deviator stress level contrary to the assumption by Singh and Mitchell (1969). In certain cases,  $m$  decreases with increasing deviator stress; most other cases show the opposite. Generally the strain rate increases with increasing deviator stress or stress level. Most of the reported studies of creep under triaxial conditions involve determination of  $m$  for axial strains only. Some studies reported the  $m$  value for the volumetric strains. In the work of Tavenas et al. (1978), it was suggested that  $m$  may be taken as a constant, i.e., the same value for the volumetric and axial strain development. This however cannot be generalized. Feda (1992) and Tian et al. (1994) found different values of  $m$  for the axial and volumetric parts of the creep tests. Note that the determination of  $m$  for the volumetric part is associated with some uncertainty compared with determination of  $m$  for the axial part.

## **2.9 Summary**

Any material subjected to constant loading will deform in due course of time. The magnitude of time-dependent deformation depends on the properties of individual materials. Geotechnical projects are long term installations and soil is the most sensitive construction material as compared to other construction materials whose properties remain more or less constant and their behaviour can be determined to a high level of accuracy. Soil is never uniform and the omnipotence of theories ceases to exist. For designing any structure supported on soil it is of utmost importance to evaluate the strength of the soil properly. In geotechnical problems soil may be subjected to different types of loading. Time of loading may vary from a few seconds to long period of time intervals. The stress strain characteristics and shear strength parameters of soil are highly dependent on rate of loading. In geotechnical engineering events a wide range of strain rates are encountered. The in-situ soil conditions are time dependent and changes in

these conditions can affect the strength characteristics of soil thereby jeopardizing the stability of the structure constructed with or over it. In this regard it is very important to evaluate the shear strength soil properly and address the uncertainties related to time dependent behaviour of soft clays. Even though such detailed investigations on other types of clays across the globe such as London clay, Hong Kong marine deposits, Boston Blue clay, Pancone clay of Pisa, Norwegian clay are available in the literature no such study is yet to be conducted on Kolkata clays. Owing to developmental activities undertaken in this region various types of structures are being constructed for creating infrastructure facilities. Hence there is a need to conduct strain controlled and stress controlled consolidated drained and consolidated undrained triaxial tests on inorganic and organic soils available in Kolkata to study the variations in shear strength parameters in an attempt to understand the difference in shear strength parameters due to testing mechanism. Moreover assessing the time dependent behaviour of the soils in this region is essential.

## **MATERIALS AND METHODS**

### **3.0 General**

Test methods adopted to carry out the present investigation are described in this chapter. This chapter describes the sampling details of materials collected in section 3.1. Detailed test program is presented in section 3.3. The testing methodology is basically divided into three parts viz: tests for determination of index properties and classification, conventional triaxial tests and creep triaxial tests with multistage loading and single stage loading. Section 3.2 describes the tests used for determination of index properties and classification. Section 3.3 describes the sample preparation for triaxial testing along with the test procedure and method adopted for both conventional and creep triaxial tests.

### **3.1 Sampling Details**

Locally available inorganic and organic soils, collected from 2-4m below ground level were used in the present study. Undisturbed block soil samples were collected through open excavations. The location of samples was carefully selected such that inorganic and organic soil layer of considerable thickness exist and the stratum had uniformity in formation. Top loose soil was removed and the ground was made flat at sampling location. The soil was cut in the cubical shape of size 300 x 300 x 300 mm on the ground with the help of sharp knife and thin wire. The formed cube was covered with two polythene covers to prevent evaporation of water from soil sample. A wooden box with internal thermocol lining was inverted over the plastic sheet and sample was cut at bottom with a thin wire. Finally top cover was fitted and

transported to the Soil Mechanics and Foundation Engineering laboratory, Jadavpur University, Kolkata.

Undisturbed block samples were collected from Jadavpur, Pal bazaar area where a open excavation was made for an underground bridge work.

### 3.2 Determination of index properties

Test conducted in the laboratory to determine index properties were water content, Specific gravity, Liquid limit, Plastic limit, and organic content. All these tests are carried out by following the Indian standard code of practice. Details of standards used in the determination of physical properties are presented in the Table 3.1

Table 3.1 Details of standards used in the determination of physical properties

Sl no	Description of test	Standard used
1	water content (oven dry method)	IS 2720 (Part – 2)
2	Specific gravity (Density bottle method)	IS 2720 (Part – 3/sec I)
3	Grain size distribution (wet sieving, pipette)	IS 2720 (Part – 4)
3	Atterberg’s limits (Casagrande’s method)	IS 2720 (Part – 5)
4	Organic content	ASTM D2974-07
5	Triaxial Testing	IS 2720- (Part 12)

The inorganic soil was brownish in colour and according to A-line it can be classified as clayey soil having intermediate compressibility (CI). In case of organic soil the sample was Blackish Grey and its position is above A-line having high compressibility (CH). The index properties of soil for both Organic and Inorganic samples were determined as per IS 2720 Part 5:1985. The results are listed in table no. 3.2.

The particle size distribution curves has been presented in Fig 3.1.

Table 3.2 Index properties of soil

Type of Soil	Organic Content (%)	Clay (%)	Silt (%)	Sand (%)	Liquid Limit (%)	Plastic Limit (%)	Specific Gravity
Inorganic Soil	0	27	65	8	46	23	2.65
Organic Soil	26	38	58	4	61	28	2.4

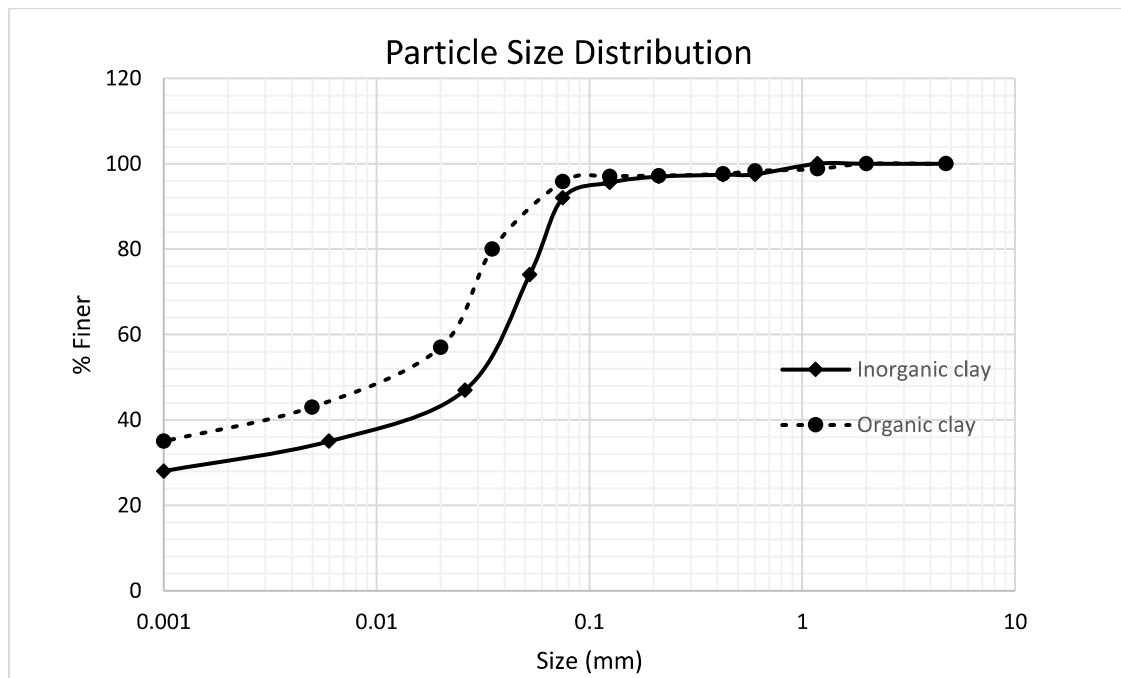


Fig 3.1 : Particle size distribution

Remoulded clay samples were used for this investigation work as studies and analyses provided in the following chapter show that effect soil structure on the mechanical behaviour of Kolkata clays is negligible. Artificially consolidated clays were thus adopted for this study because it gave greater control over sample preparation in turn leading to uniformity in results essential for the parametric studies conducted based on the experimental results.

### **3.3 Triaxial Test**

#### **3.3.1 Sample preparation**

To prepare the sample for triaxial tests, about 3kg of soil was grounded and clay clods were broken to form a powdery form. The soil was subsequently mixed with water to form soil slurry at water content greater than liquid limit. To prepare the samples for testing a cylindrical mould of diameter 130mm and height of 450mm was used. A 130mm diameter porous stone was provided at the bottom of the mould to enable drainage through it and a porous stone of diameter slightly less than 130mm was provided at the top (so as to reduce the friction between the stone and the inner walls of the mould in order to allow smooth downward movement during consolidation). The porous stones were washed in boiling hot water and saturated before use. Moist filter papers were provided on surface of porous stone in contact with the soil slurry to prevent the migration of soil particles into the porous stone during consolidation. The soil slurry was then poured into the cylinder. Loading piston was then used to apply the required load for consolidation of the soil. A total load of 96 kg was applied axially during consolidation to simulate the pressure which would give the sample sufficient strength/undrained cohesion so that there would be least change in behaviour during sampling/handling of the test samples. The sample preparation set up is shown in Fig.3.2. The entire load was allowed to act on soil slurry for two weeks. Subsequently the loads were removed and the consolidated soil mass was taken out very carefully. It was then cut into equal four (4) pieces without disturbing the soil

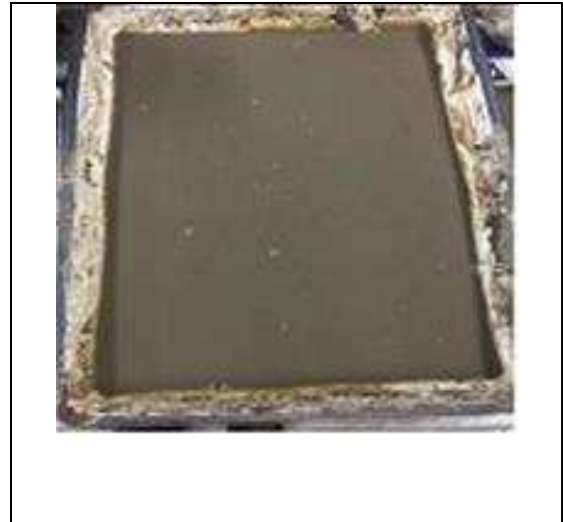
sample (as minimum as possible) and subsequently trimmed using a wire saw very gently to form specimens of diameter 38mm and height 76mm. Before setting up of the sample distilled water was circulated from one of the drainage outlets present through the second drainage outlet to ensure smooth passage of water and remove any air bubbles present in the channel. To measure the pore pressure in case of undrained tests or volume change in case drained tests the pore pressure transducer (shown in Fig. 3.4 a) and volume gauge (shown in Fig. 3.5) was connected. When the connection was made the water was passed slowly and continuously through the channel so that no air bubbles are introduced at the connection. Porous stones were placed over the base of the pedestal on which the specimen was to be mounted. The porous stones of diameter 38mm were boiled, saturated before being placed on the pedestal. A moist filter paper of diameter 38mm were placed on them. The sample was mounted on the porous stone placed over the pedestal and surrounded by a vertical paper drain. The top cap along with a porous stone and filter paper was placed centrally on top of the specimen and a sheath stretcher was used to place two rubber membranes (each 0.05mm thick) around the samples. The membrane was gently stroked upward against the sample to remove any air trapped between the membrane and the specimen. Two o-rings were stretched around the mounting cylinder and rolled off onto the pedestal first and then two o-rings were provided on top cap to make the specimen water tight. The axial load piston was brought into contact with the specimen cap. The final setup is shown in Fig 3.2.

Standardization of physical properties of remoulded soil samples was crucial for ensuring consistency and comparability in geotechnical testing. Prior to testing the dimensions of each sample along with the density was noted to maintain parity. The initial weights of the samples were in the range of 155-159 grams. The average weight of the samples was 156.8 grams as calculated taking the weight of all the samples used for testing for this study. The post testing weights of the samples were also noted. The difference in sample weights post testing was due

to effect of conditions under which they were tested mainly the drainage conditions which was primarily the point of difference along with the effective pressure under which the samples were tested. The water content of the samples were noted prior to testing. The water content of the samples were found in the range of 30% to 31.75%.



a) Dried soil to be grounded



b) Slurry form of soil (grounded soil and added water)



c) Mould and consolidation setup



d) Soil slurry loaded for consolidation

Fig 3.2 Sample Preparation



a. Consolidated specimen for sample preparation



b. Using wire and knives the consolidated sample being shaped for triaxial testing



c. Desiccator for storing samples

Fig 3.3 : Sample post consolidation and sample storing apparatus



Fig 3.4 a: Pore pressure Transducer



Fig 3.4 b: HEICO S-type digital load cell



Fig 3.5 Volume Gauge.

### 3.3.2 Test Programme

The list of consolidated undrained and drained tests performed on remoulded Normal Kolkata organic and inorganic clay soil under different loading conditions are given below in Table 3.3 and Table 3.4. 1. The soft clay deposit as available in Normal Kolkata Deposit is up to 15-20 metres below ground surface. For analysis and design of various structures; shallow foundation, embankments the shear strength parameters/behaviour of this soil has a major role. Further effective consolidation pressures at a depth of 8-10m is in the order of 100kPa and for 15-20m is in the order of 200kPa considering ground water table at 2-3m depth below ground level. This is the logic behind selecting the effective pressures of 100kPa and 200kPa respectively. Also some studies were conducted at effective pressures of 50kPa.

Table 3.3: Test Programme.

Type of Test Triaxial test	Type of Soil	Drainage Condition	CellPr. (kPa.)	Back Pr. (kPa)	Soil sample Weight (grams)
Strain controlled	Inorganic	Consolidated Undrained (CU)	300	100	157
			200	100	156
Strain controlled	Inorganic	Consolidated Drained (CD)	300	100	156.4
			200	100	158.2
Strain controlled	Organic	Consolidated Undrained (CU)	300	100	157.3
			200	100	155.2
Strain controlled	Organic	Consolidated Drained (CD)	300	100	155.4
			200	100	156
Stress controlled	Inorganic	Consolidated Undrained (CU)	300	100	156.8
			200	100	157.4
Stress controlled	Inorganic	Consolidated Drained (CD)	300	100	158
			200	100	155.1
Stress controlled	Organic	Consolidated Undrained (CU)	300	100	157
			200	100	157.8
Stress controlled	Organic	Consolidated Drained (CD)	300	100	156.7
			200	100	158.9

Note : A load of 4.5kgs was applied on the sample at every increment for stress controlled tests.

Table 3.4: Test Programme for Single Loading Creep Tests.

<b>Type of Test</b>	<b>Type of Soil</b>	<b>Drainage Condition</b>	<b>Cell Pr. (kPa.)</b>	<b>Back Pr. (kPa)</b>	<b>Magnitude of load intensity (kPa)</b>	<b>Soil sample Weight (grams)</b>
Stress controlled	Inorganic	Consolidated Undrained (CU)	100	50	16	156.7
					22	157.2
					35	158
					44	156
Stress controlled	Inorganic	Consolidated Undrained (CU)	200	100	38	157.3
					56	156.6
					70	155.9
Stress controlled	Organic	Consolidated Undrained (CU)	100	50	16	156.2
					22	156.4
					33	157.8
					41	158.2
Stress controlled	Organic	Consolidated Undrained (CU)	200	100	26	155.6
					40	156.2
					70	156.4
					88	157.1

Note: In case of single stage tests only one load depending upon stress level to which the soil specimen is subjected was applied.

### **3.3.3 Procedure:**

#### **Saturation Stage:**

The objective of the saturation phase of the test was to fill all air voids, if present in the specimen with water without allowing the specimen to swell. The codal provision for application of back pressure is presented IS 2720(Part XII) :1981 sections 3.4.5. Saturation was accomplished by applying back pressure to the specimen pore water to drive air into solution. A small back pressure and cell pressure was applied with cell pressure approximately greater than back pressure. The difference of cell pressure and back pressure was kept in the range of 15 – 20 kPa. The cell and back pressure were increased in steps with specimen drainage valves opened so that de-aired water may flow into the specimen. While applying back pressure, the pressure was applied incrementally with adequate time between increments to permit equalization of pore-water pressure throughout the specimen. The degree of saturation was checked by closing the drainage valve and increasing the confining pressure. Corresponding increase in pore pressure were measured. The ratio of increase in pore pressure and increase in confining pressure is expressed as  $B = \Delta U / \Delta \sigma_3$ . For a saturated sample is the value of Pore Pressure Coefficient  $B = \Delta U / \Delta \sigma_3 = 1.0$ . The process as mentioned in step 1 is repeated until the magnitude of pore pressure response indicated as B becomes greater than or equal to 0.95. The sample is then considered to be ready for consolidation.

#### **Consolidation Stage:**

The consolidation stage was used to bring the specimen to the effective stress state required for shearing. The codal provision for application of back pressure is presented IS 2720(Part XII) :1981 sections 6.4.1 for consolidation processes respectively prior to shearing stage. Each soil sample was then allowed to consolidate isotropic condition for 24 hours by applying cell pressure and back pressure 300 and 100, 200 and 100 kPa simultaneously, as required, to

maintain the effective confining stress of 200 and 100 kPa. During consolidation volume gauge were connected to the back pressure line and the change in volume with time was recorded. The volume change – time recorded was plotted to obtain rate of deformation of soil specimen during shearing as per Bishop and Henkel (1962).

### **Shearing Stage:**

#### Strain controlled tests

The shearing procedure was as follows:

1. During shear, the chamber pressure was kept constant
2. Initial reading of the submersible load cell attached to the triaxial set up was recorded. The dial gauge was set to zero.
3. For CU tests the drainage valve were closed and pore water pressure transducer was used to measure increase in pore pressure during shearing. Drainage valve was kept open in case of CD test and volume gauge was used to measure the change in volume during shearing. Loading machine was then run at a strain rate 0.048mm/min calculated using the volume change - time data obtained during the consolidation stage as described in Bishop and Henkel (1962). The strain rate calculated for inorganic samples was 0.044mm/min and for organic samples it was 0.062mm/min.
4. Load and deformation readings were recorded at sufficiently small intervals to accurately define the stress-strain curve.

#### Stress-controlled tests

1. Load was applied on the soil specimen through lever system.
2. Initially a very small seating load of about 1 kg was applied for about 2 hours.

3. Then the axial loads were applied in increments corresponding to different stress level. Stress level (SL) is the ratio of deviator stress acting on the soil during a particular load application to the deviator stress at failure obtained from the conventional compression triaxial tests under similar conditions (i.e. drainage condition, effective pressures and type of soil).
4. For each load increment vertical deformation of the soil sample was recorded with time for 24 hours. A load of 4.5kgs was applied on the sample at every increment. In case of single stage tests only one load depending upon stress level to which the soil specimen is subjected was applied.
5. For multistage testing the loads acting on the sample at the penultimate stage prior to the application of the failure load increment was considered the ultimate load carrying capacity of the sample.
6. Initially the rate of deformation was less and became higher with the increase in the deviator stress. The test was continued till the rate of deformation was very high so as to cause failure of the specimen or till 20% of axial strain was achieved.

The setup for conventional triaxial test, its pictorial representation and the pneumatic pressure control panel have been presented in Fig 3.6 to Fig 3.8. The stress-controlled triaxial setup is shown in Fig 3.9.



Fig 3.6: Conventional (strain controlled) Triaxial Test Setup

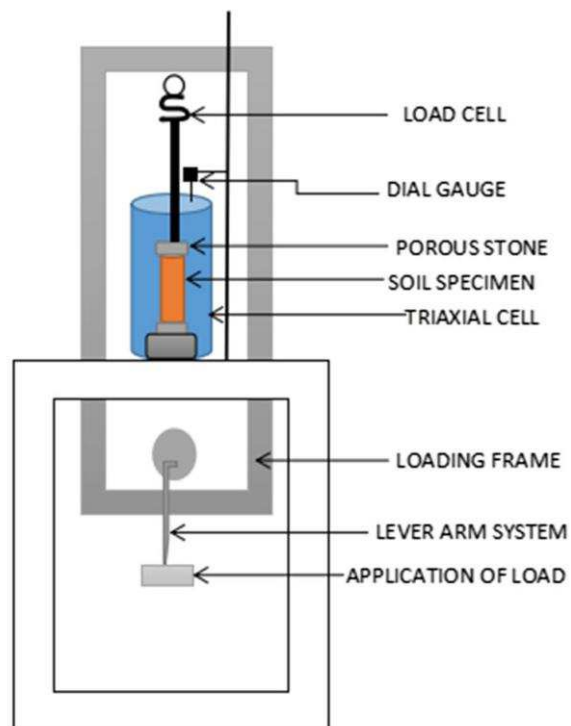


Fig 3.7: Pictorial representation of (strain controlled) Triaxial Test Setup



Fig 3.8 : The Pneumatic Pressure Panel Control System



Fig 3.9: Creep (stress controlled) Triaxial Test Setup

**EFFECT OF SOIL STRUCTURE ON THE COMPRESSION BEHAVIOUR OF  
SOILS**

**4.0 General**

The change/irregularity of soil structure in remoulded clays is a well documented fact which is why reconstituted clays are used as a basic frame of reference to examine the effect of soil structure on mechanical behaviour of natural clays (Skempton & Northey, 1953; Houston & Mitchell, 1969; Nagaraj & Srinivasa Murthy, 1986; Hight et al., 1987; Burland, 1990; Liu & Carter, 1999; Hong & Tsuchida, 1999; Chandler, 2000; Cotecchia & Chandler, 2000; Liu & Carter, 2000). Owing to the presence of a defined soil structure the compression curve of natural clays (in terms of void ratio and effective vertical stress) lies above that of reconstituted clays (Leroueil et al., 1979; Leroueil et al., 1985; Locat & Lefebvre, 1986; Leroueil & Vaughan, 1990; Schmertmann, 1991; Cotecchia & Chandler, 1997; Hong & Tsuchida 1999). The presence of fabric and bond between constituent particles of natural clays restricts deformation of natural clays under vertical stresses resulting in lower compressibility in comparison to reconstituted clays (Holtz et al., 1986; Burland, 1990).

In case of soft clays, soil structure is believed to play a significant role in the degree of rate dependence on the stress-strain behaviour of clay. At the thirteenth Rankine Lecture of the British Geotechnical Society, Professor J.B. Burland discussed on the compressibility and shear strength of natural clays (Burland, 1990). He presented a framework to interpret the characteristics of natural sedimentary clays in their current void ratio-vertical effective stress state using the compressibility and strength characteristics of reconstituted clays as a basic frame of reference. Based on Burland's void index framework Sheahan (2003) proposed a

methodology to predict the type and degree of dependence of soil fabric on the soil's stress-strain behaviour and developed a systematic method to quantify the same.

In this section, an attempt has been made to examine the difference in compression behaviour of reconstituted and natural clays of Normal Kolkata deposits. The effect of liquid limit of the soil on its compressibility has also be examined along with the variation of compression characteristics with depth. Finally, following the framework proposed by Sheahan (2003) as a reference, an attempt has been made to quantify the soil fabric/structure on compression behaviour of Normal Kolkata deposit.

#### **4.1 Background Framework**

##### **4.1.1 Burland's(1990) framework relating compressibility to the structure of natural clays**

Burland (1990) defined “intrinsic properties” as the properties obtained from reconstituted clay having water content in the range of 100% to 150% of its liquid limit, without any drying. Further the strength and compressibility properties of reconstituted clays was used as a reference to interpret characteristics of natural sedimentary clays. To obtain the intrinsic properties, the reconstituted clays were consolidated under one-directional conditions and the  $e$ - $\log \sigma'_v$  was plotted. The void ratios corresponding to  $\sigma'_v = 100$  kPa and 1000 kPa were called intrinsic void ratios denoted by symbols  $e^*_{100}$  and  $e^*_{1000}$  respectively. The compression curve was normalized by defining void index ( $I_v$ ) as:

$$I_v = \frac{e - e^*_{100}}{C_c^*} \quad (4.1)$$

where

$$C_c^* = e_{100}^* - e_{1000}^* \quad (4.2)$$

If the liquid limit of the soil is known, then the value of  $e_{100}^*$  and  $C_c^*$  can be calculated using the equations 4.3 and 4.4 respectively.

$$e_{100}^* = 0.109 + (0.679 * e_L) - (0.089 * e_L^2) + (0.016 * e_L^3) \quad (4.3)$$

$$C_c^* = 0.256 e_L - 0.04 \quad (4.4)$$

where  $e_L$  is the void ratio at liquid limit.

Burland concluded that the normalized intrinsic compression curve obtained from the plot of  $I_v$  vs.  $\log \sigma'_v$  was reasonably unique for most reconstituted clays and termed the curve as the intrinsic compression line (ICL). The ICL was expressed as follows:

$$I_{v,ICL} = 2.45 - 1.285x + 0.015x^3 \quad (4.5)$$

where

$x = \log \sigma'_v$  in kPa.

Using the void index as a normalizing parameter, Burland compared the sedimentation compression curves of soils with the corresponding ICL. For a normally consolidated clay with void ratio  $e_0$  under effective overburden pressure  $\sigma'_{v0}$  the in situ void index  $I_{v0}$  is expressed as:

$$I_{v0} = \frac{e_0 - e^*_{100}}{C_c^*} \quad (4.6)$$

Thus successive values of  $e_0$  and  $\sigma'_v$  down a soil profile were used to plot a graph of  $I_{v0}$  against  $\log \sigma'_v$  to obtain the sedimentation compressive curve which could be directly compared with the ICL. Having plotted the in situ void ratios versus their respective in situ  $\sigma'_v$  Burland found that various sedimentation curves condensed into a reasonably unique line, which he termed as

the sedimentation compression line SCL. In the range of  $\sigma'_v = 10$  kPa to 1000 kPa the ICL and the SCL were approximately parallel with the SCL expressed as:

$$I_{v,SCL} = 3.2436 - 0.6239 \ln \sigma'_v + 0.0244 (\ln \sigma'_v)^2 - 0.0012 (\ln \sigma'_v)^3 \quad (4.7)$$

In the Fig 4.1 the normalized ICL and SCL for a hypothetical soil has been depicted.

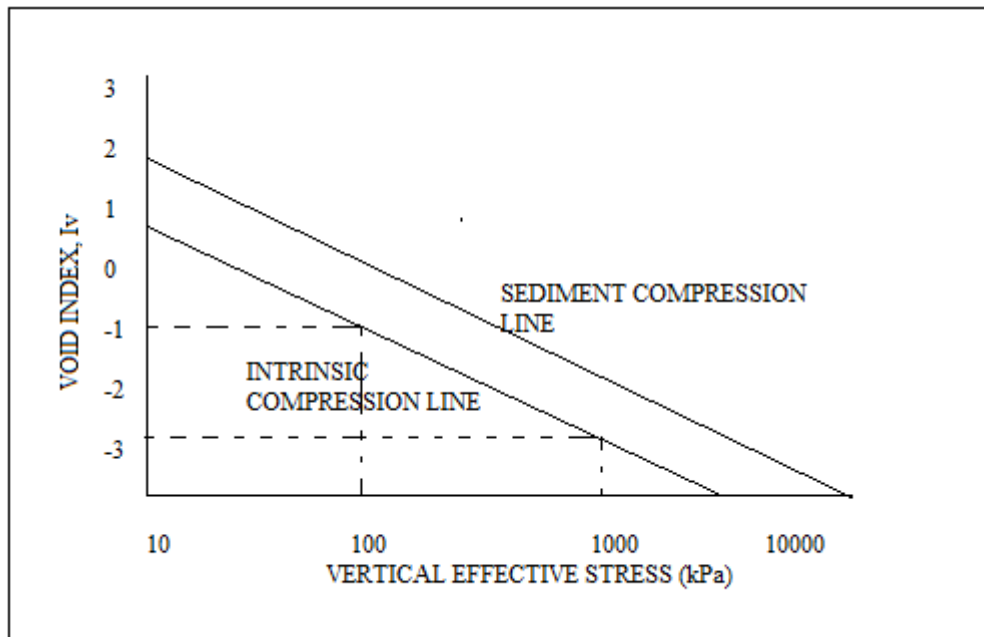


Fig. 4.1 : Void Index Compression Curve framework

#### 4.1.2 Sheahan's (2003) Structure Number(SN) Relating Rate Dependent behaviour to Structure

Sheahan (2003) developed a systematic method to relate the structure of a soil to the stress-strain characteristics of the soil at that current state of the soil. Sheahan (2003) introduced the concept of Structure Number (SN) using Burland's framework to indicate how far the soil's natural state is from its remoulded or intrinsic compression curve and sediment compression curve. The ICL was defined as SN=0 and the original sedimentation curve (SCL) as SN=1.

SN=2 was defined as the contour parallel to both the ICL(SN=0) and SCL (SN=1) lying above the SCL with the distance between ICL and SCL equal to the distance between SCL and SN=2.

Fig. 4.2 shows the arrangement as proposed. Sheahan (2005) quantified soil's state in Burland's void index framework by defining the Structure Number SN as follows:

$$SN = \frac{I_v - I_{vICL}}{I_{vSCL} - I_{vICL}} \quad (4.8)$$

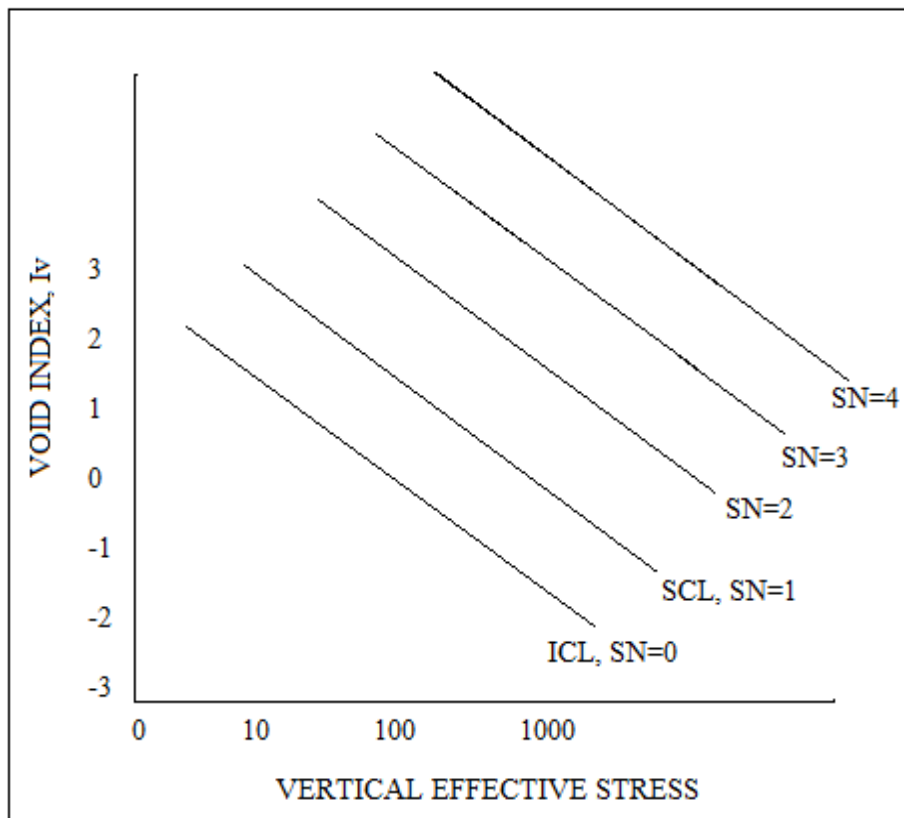


Fig. 4.2 :Sheahan's structure number framework in Burland's Void Index space.

## **4.2 Test Results**

### **4.2.1 Results of Oedometer tests on natural and reconstituted clays**

The compression behaviour of remoulded clays of Kolkata region and artificially prepared soils were studied on the basis of 25 oedometer test results based on their varying liquid limits. The artificially prepared soil were made by adding sand with bentonite, kaolin and other clays in various proportions and combinations to natural clays of Kolkata region in order to study their compression behaviour alongside remoulded and natural clays. The variation is shown in Fig. 4.4. For natural clays of Normal Kolkata deposit compression behaviour was examined by using documented results of oedometer tests conducted in the Soil Mechanics and Foundation Engineering laboratory on undisturbed samples collected from different locations having varying liquid limits and depths. The locations are presented in Fig 4.3. The  $e$ - $\log \sigma_v$  curves are shown in Fig 4.4 and Fig 4.5.

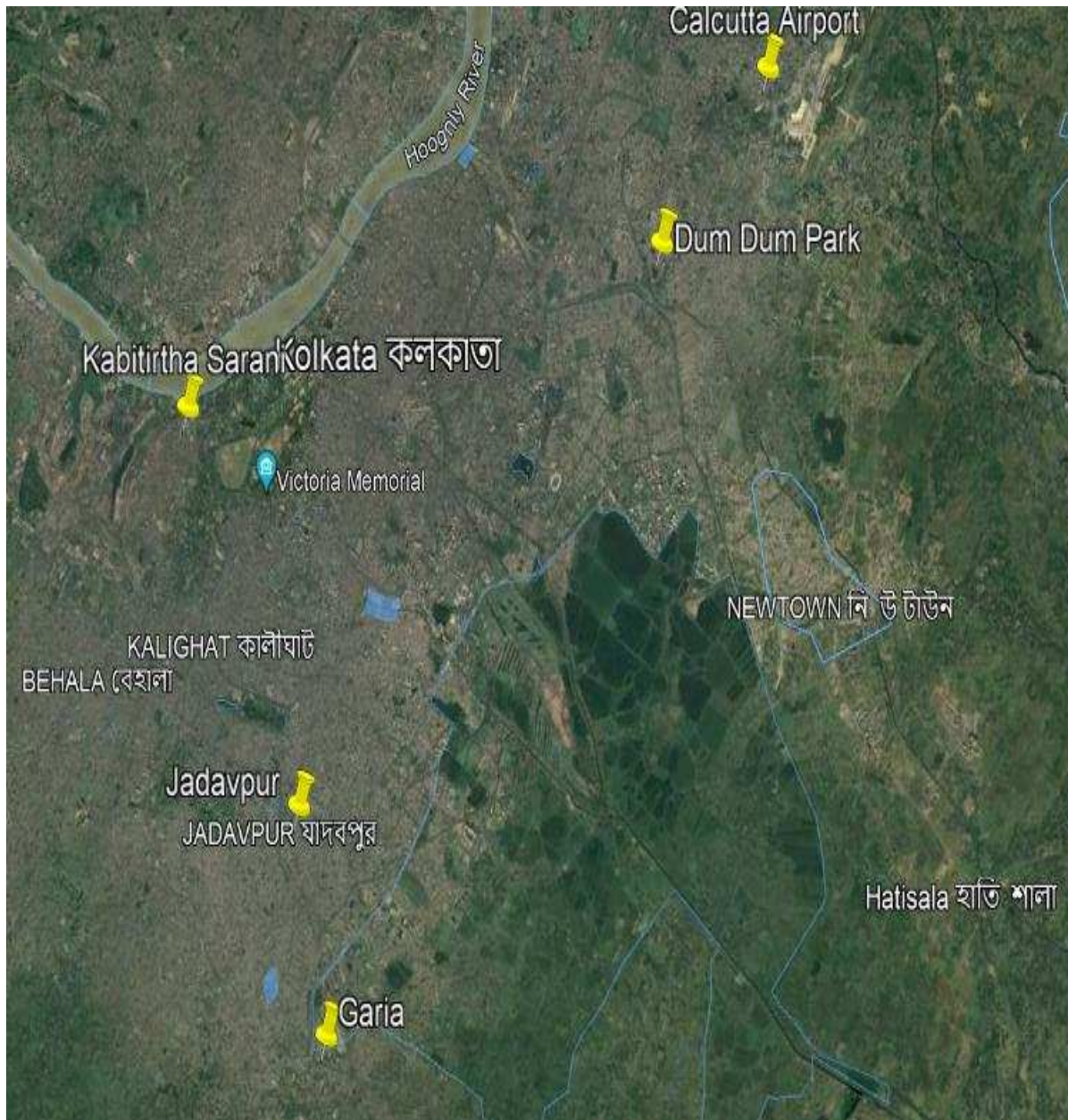


Fig 4.3 : Locations of Undisturbed Soil Samples

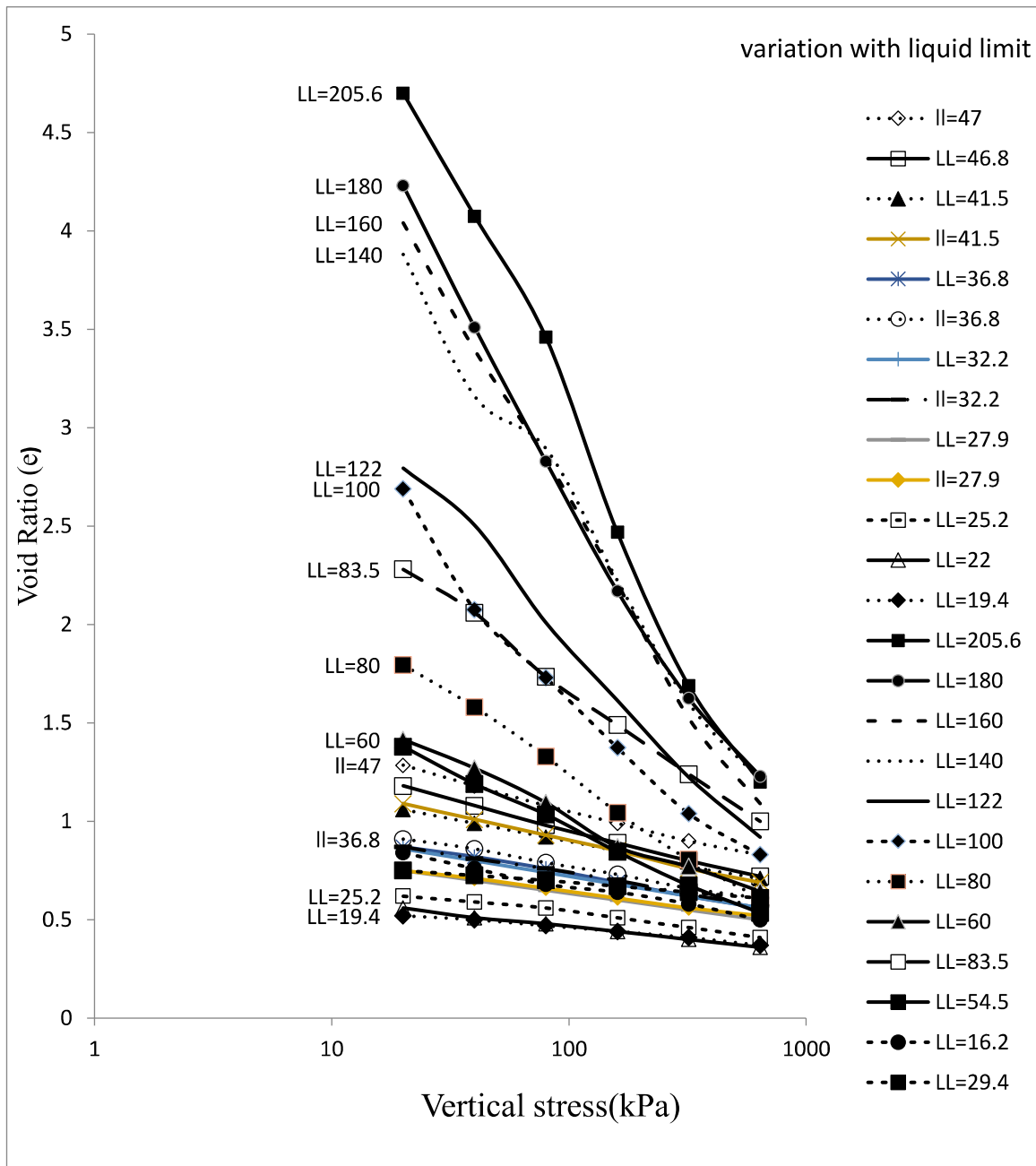


Fig. 4.4: Variation in compression behaviour of remoulded clays with liquid limit

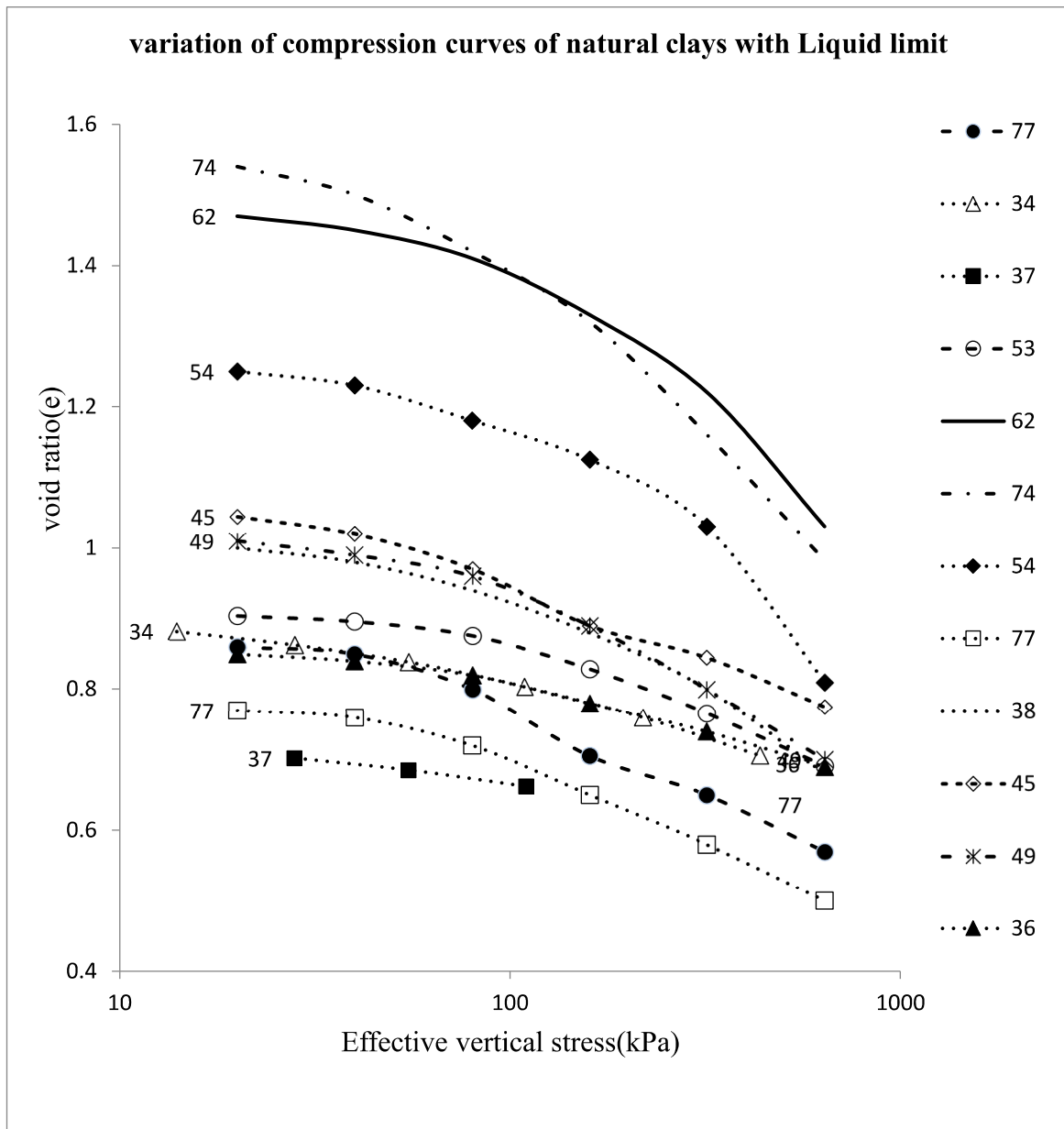
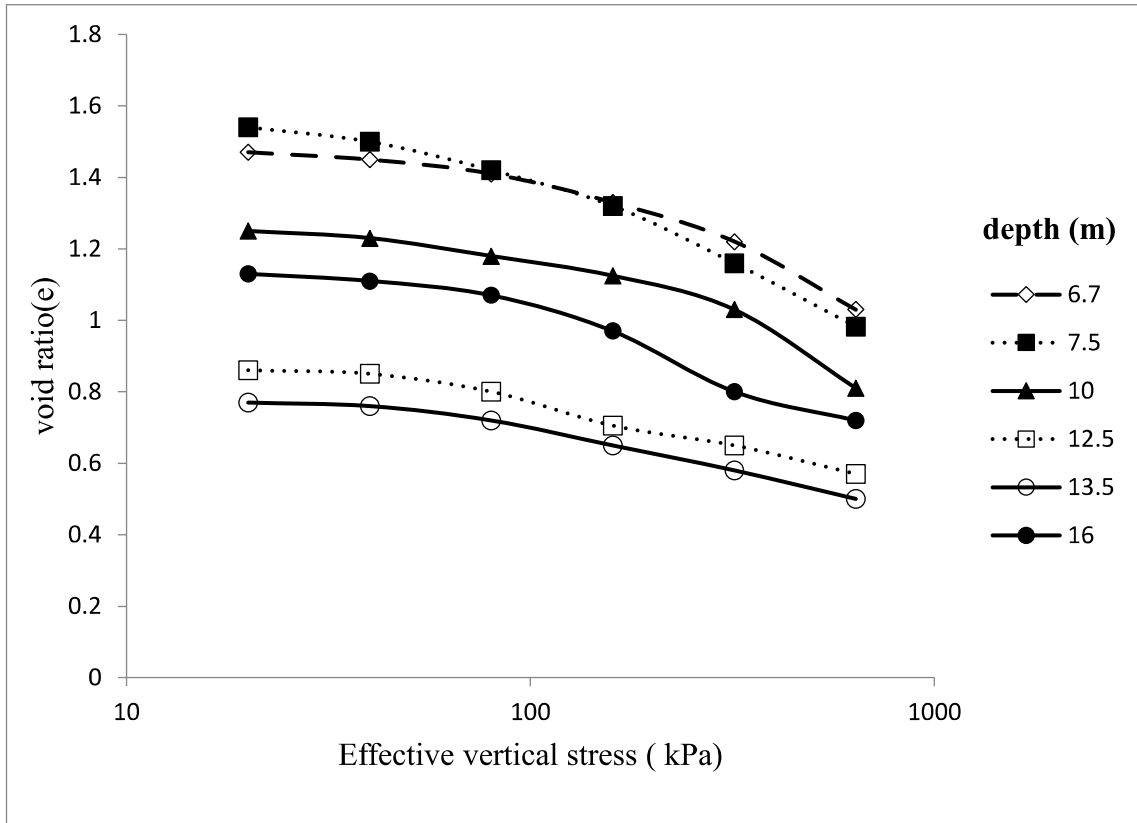
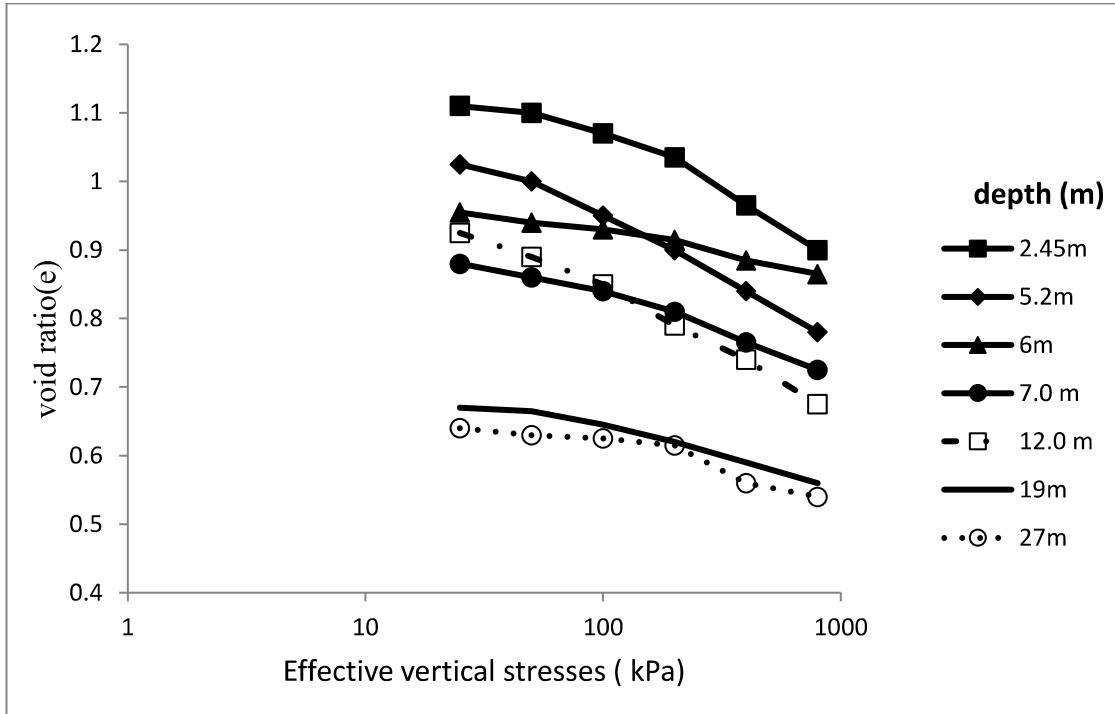


Fig 4.5: Variation in compression behaviour of natural clays with liquid limit

To understand the compression characteristics of natural clays a depth wise variation in compressibility of the clays was examined. The variation of compression along the depth at a particular location was plotted by conduction oedometer tests on soil samples collected from different depths of a particular borehole. The results are depicted in Fig 4.6a and 4.6b.



(a): Variation in compression behaviour of natural clays with depth (Dum Dum Park)



(b): Variation in compression behaviour of natural clays with depth (Calcutta Airport)

Fig 4.6 : Variation in compression behaviour of natural clays with depth

The compression curves of remoulded and natural clays are dependent on liquid limit and void ratio increases with the increase in water content. In case of remoulded clays it was evident that greater the initial water content of the soil, greater is the void ratio thus increasing the compressibility of the soil. The variation in compression patterns of natural clays indicates that soils at less depths have greater void ratios and are thus more compression than soils which are located at greater depths. This might be because of the soils closer to the surface having a more flocculated structure than the soils at greater depths that tend to have a dispersed structure as they are subjected to greater overburden pressure. This effect is pronounced for soils situated closer to the top surface, with greater depths the effect is less evident, for example in Fig.4.5b the compressibility of soils located at 19m and 27m below ground level may not vary by much. The compression curves of natural clays of Kolkata soil with varying liquid limits have been depicted in Fig.4.5, it can be seen that more the liquid limit, greater are the voids so more is its compressibility with some exceptions. For two soils with liquid limit of same order the compression behaviour is a function of depth. The compressions curves of a soil having liquid limit 77% and collected from a depth of 12m is comparatively less compressible with respect to that of a soil having similar liquid limit collected from a depth of 4.5m.

#### **4.2.2 Variation of coefficient of compression with Liquid Limit**

From the oedometer tests results presented in 4.2.1 the variation of coefficient of compression for remoulded and natural clays of Kolkata region are presented in Fig 4.7.

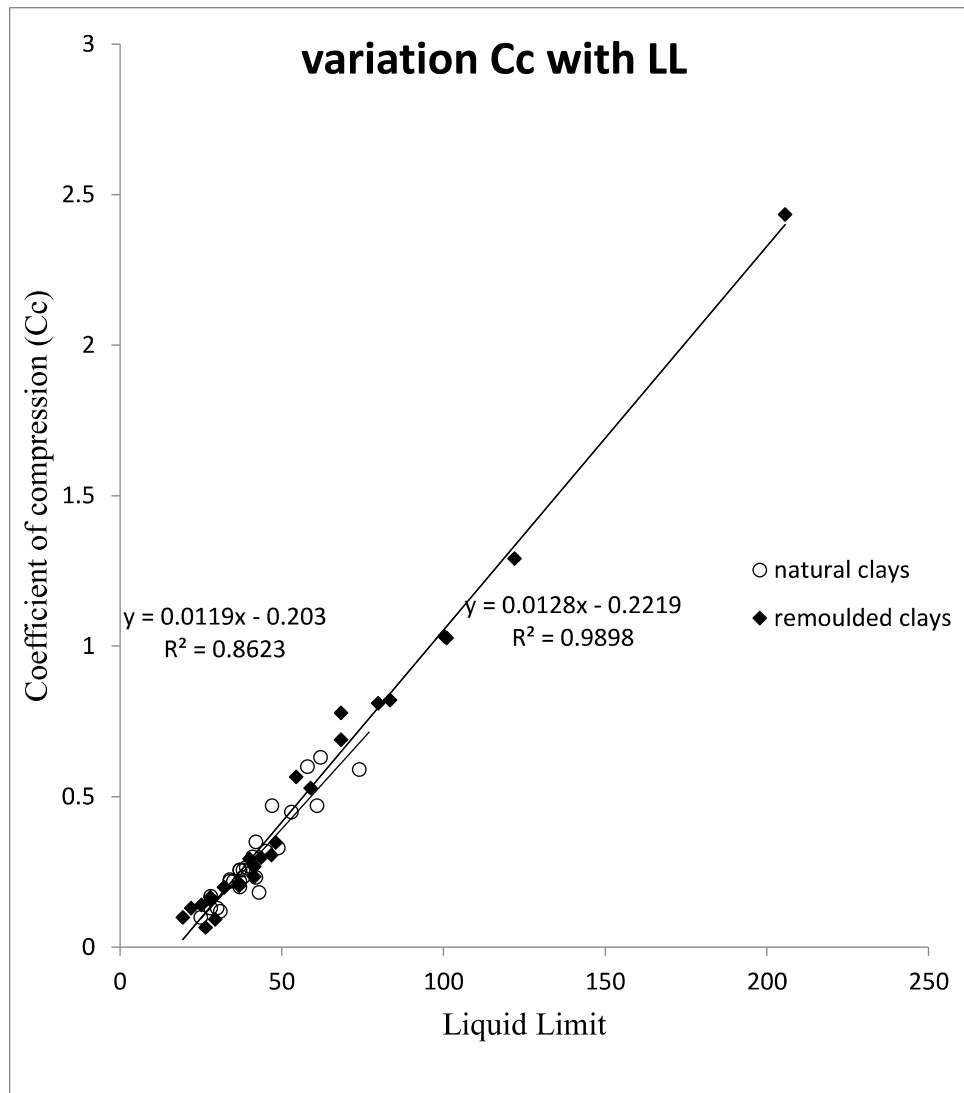


Fig. 4.7: Comparative variation of coefficient of compression for remoulded and natural clays

It can be seen that the coefficient of compression (Cc) for both remoulded and natural clays are dependent on liquid limit and the behaviour pattern for both the clays are almost identical. This shows that natural clays of Normal Kolkata deposit primarily exist in post transitional regime. The relationship between Cc and liquid limit  $w_L$  were studied and it is observed that Cc is a function of liquid limit of the soil. Cc increases with the increase in liquid limit as shown in Fig4.7. Using regression analysis unique equation for remoulded and natural clays with the correlation coefficient  $r=0.99$  and  $0.86$  respectively are obtained

$$C_c = 0.0128w_L - 0.222 \quad (4.9)$$

$$C_c = 0.011w_L - 0.203 \quad (4.10)$$

### 4.2.3 Variation of $e_{100}$ with Liquid Limit

It can be seen that  $e^*_{100}$  is an important parameter in normalizing compression curves. The variation of  $e^*_{100}$  with liquid limit ( $w_L$ ) was studied and it was found to be dependent on liquid limit of the soil as evident in Fig 4.8.  $e^*_{100}$  is directly proportional to the liquid limit and using regression analysis an equation with correlation coefficient  $r=0.98$  and  $r=0.76$  for remoulded and natural clays respectively was formulated. The equation for remoulded and natural clays can be respectively expressed as:

$$e^*_{100} = 0.0144 w_L + 0.2379 \quad (4.11)$$

$$e_{100} = 0.019w_L + 0.132 \quad (4.12)$$

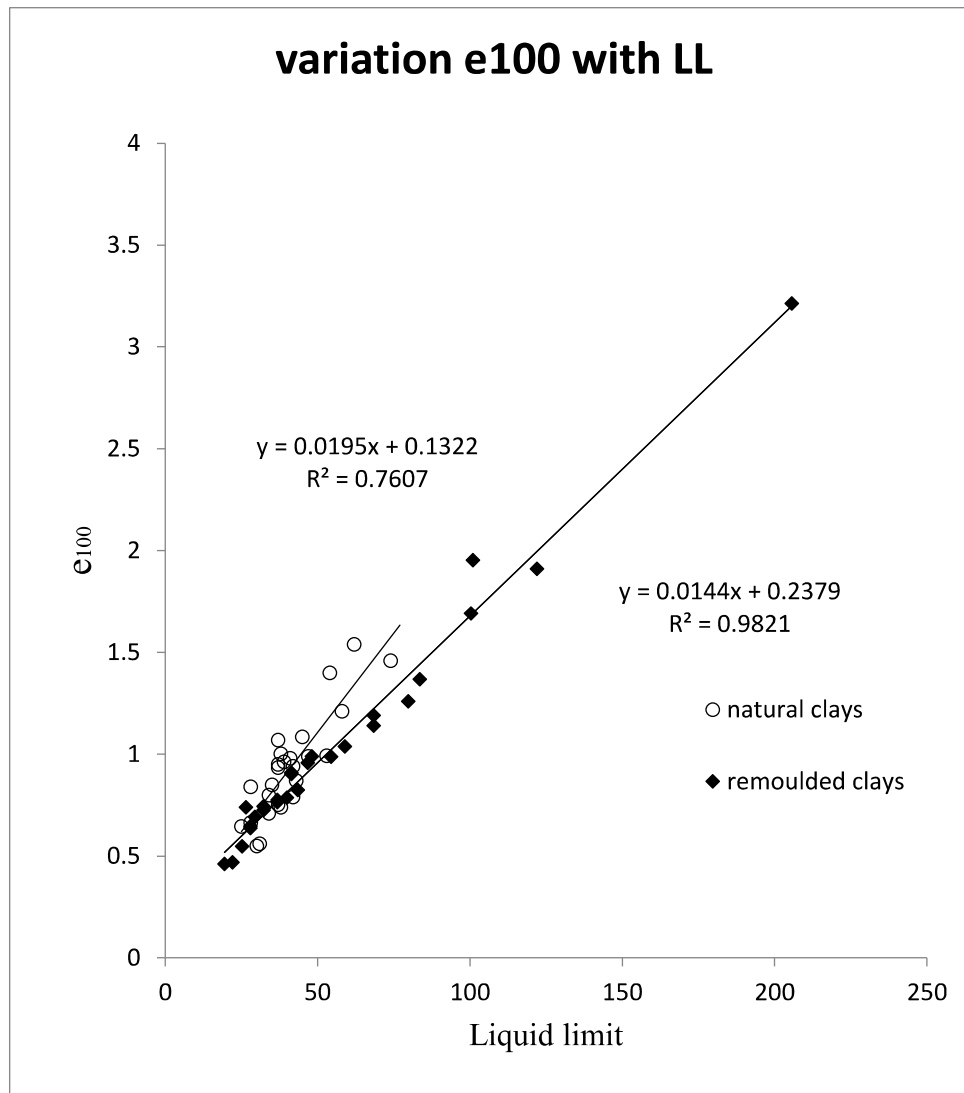


Fig.4.8: Comparative variation of void ratio corresponding to 100kPa effective vertical stress for remoulded and natural clays

#### 4.2.4 Comparison of compression curves

The compression curve of natural clays of Kolkata soil lies very slightly above that of remoulded clays. The curve drawn using regression analysis to plot the normalized compression curve for reconstituted clays was found to be in agreement with the ICL. The normalized compression curves of natural clays beyond the consolidation yield stresses lie on or just above the ICL. The reason for these lying well below the SCL and closer to the ICL can be attributed to the depositional characteristic as pointed out in Burland (1990). On the basis

of Sheahan's (2003) framework and the compression characteristics of the natural clays being very similar to those of reconstituted clays, it may be inferred that the effect of soil structure on the mechanical properties of NKD is not prominent. Sheahan concluded that soils with high degree of structure SN 2.5 or greater (SN=4.43, 3.50, 2.82 as was determined) have a pronounced effect of structure and strain rate on the mechanical properties of such soils whereas soils with low degree of structure i.e. below 2.5 (SN=1.17 or 1.93) display no associated dependence on soil structure.

The structure number of the natural clays as defined by Sheashan (2003) were found to be negative as the natural compression curves lie well below the SCL, SN=1. Hence the effect of soil fabric on mechanical behaviour of Normal Kolkata Deposit on soil structure is not pronounced. The curves are presented in Fig 4.9 and Fig 4.10. Fig. 4.11 a and b represents the curves in terms of Burland and Sheahan framework respectively.

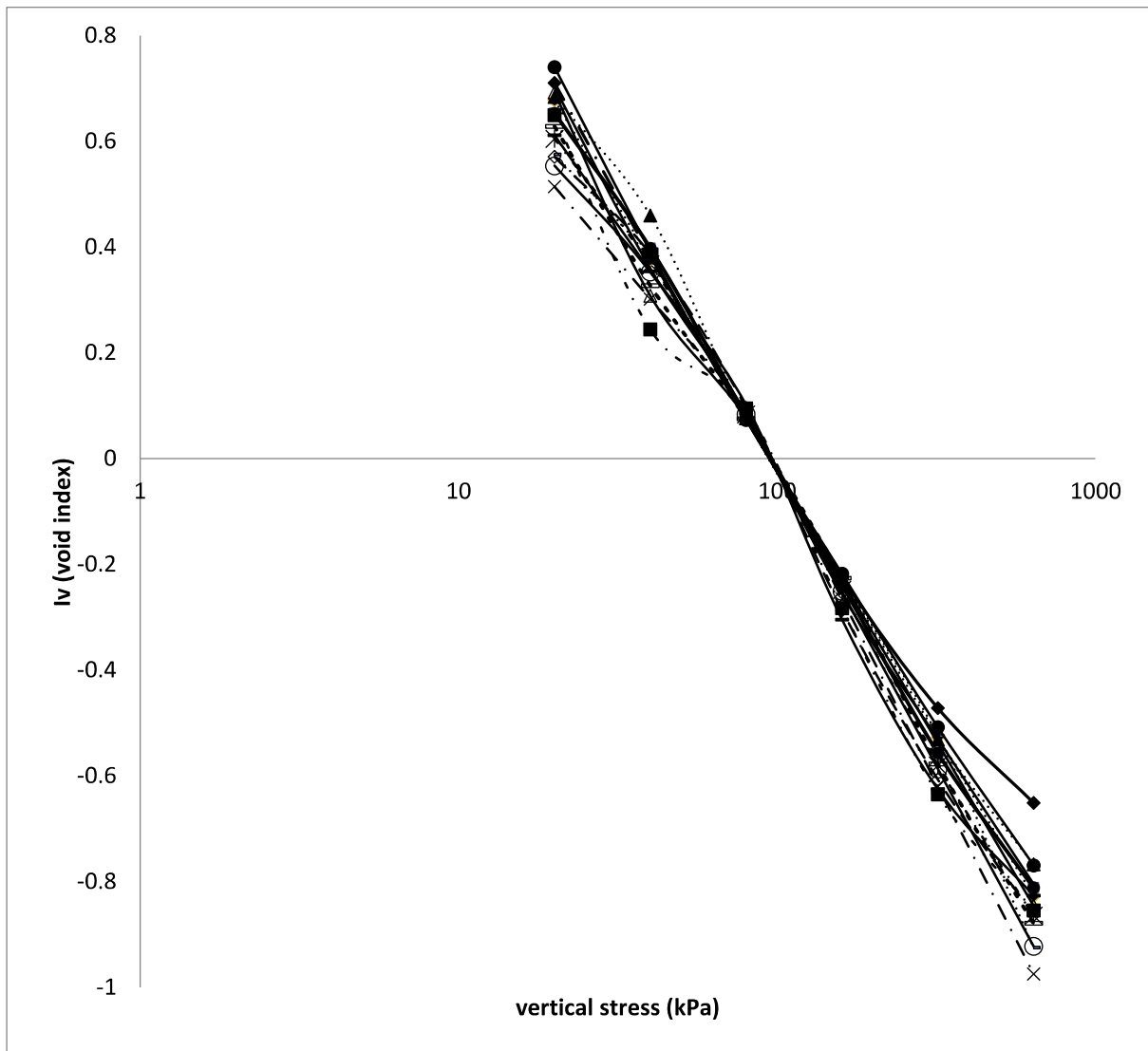


Fig 4.9: Normalized compression curves of remoulded clays.

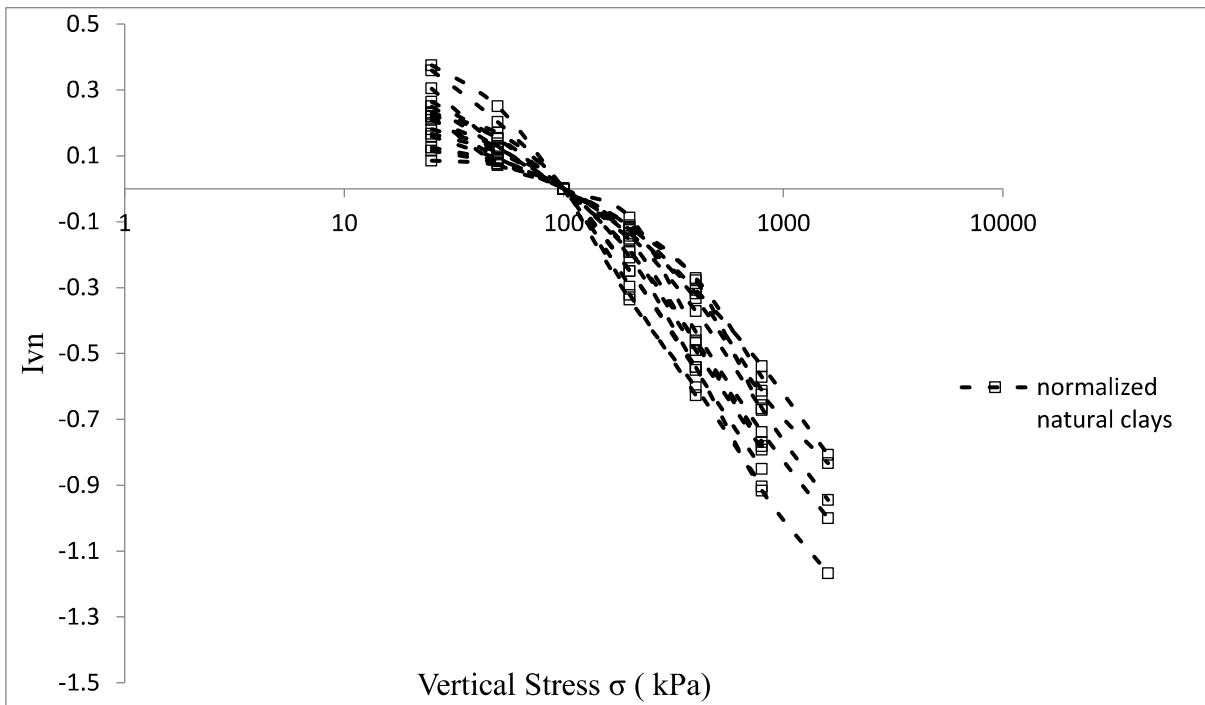


Fig 4.10 : Normalized compression curves of natural clays.

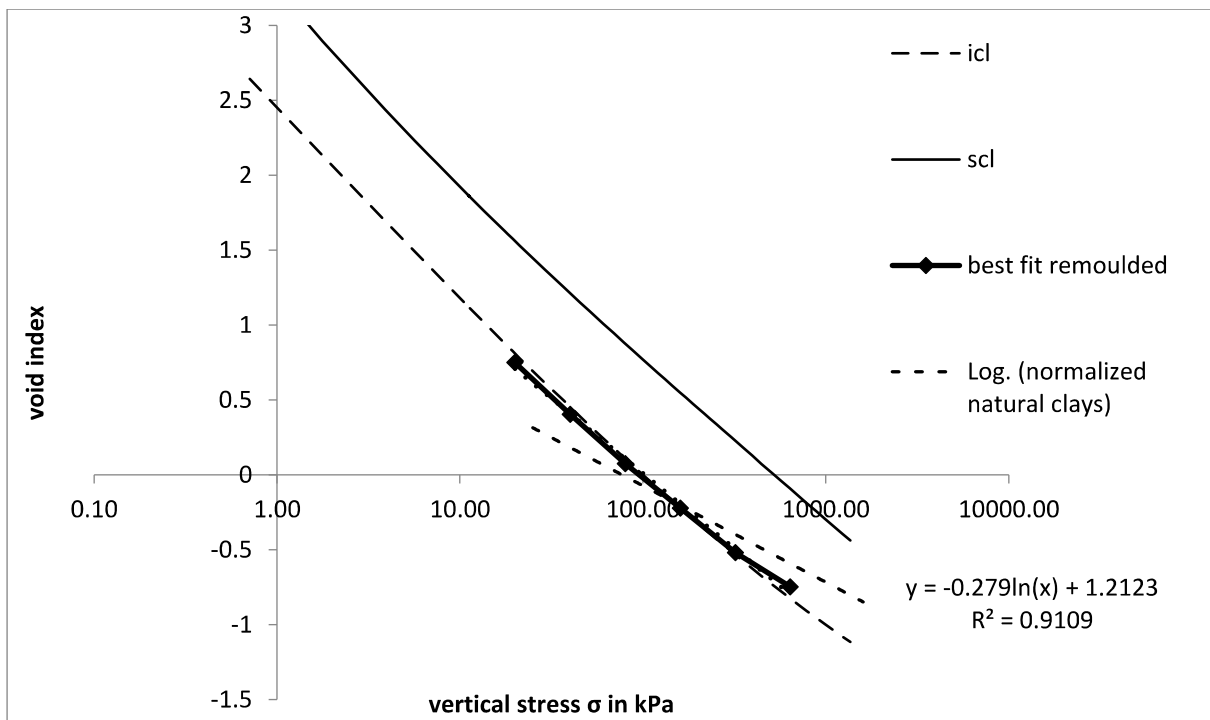


Fig. 4.11 a: Comparison of the ICL and SCL in Burland (1990) with the corresponding curves of NKD

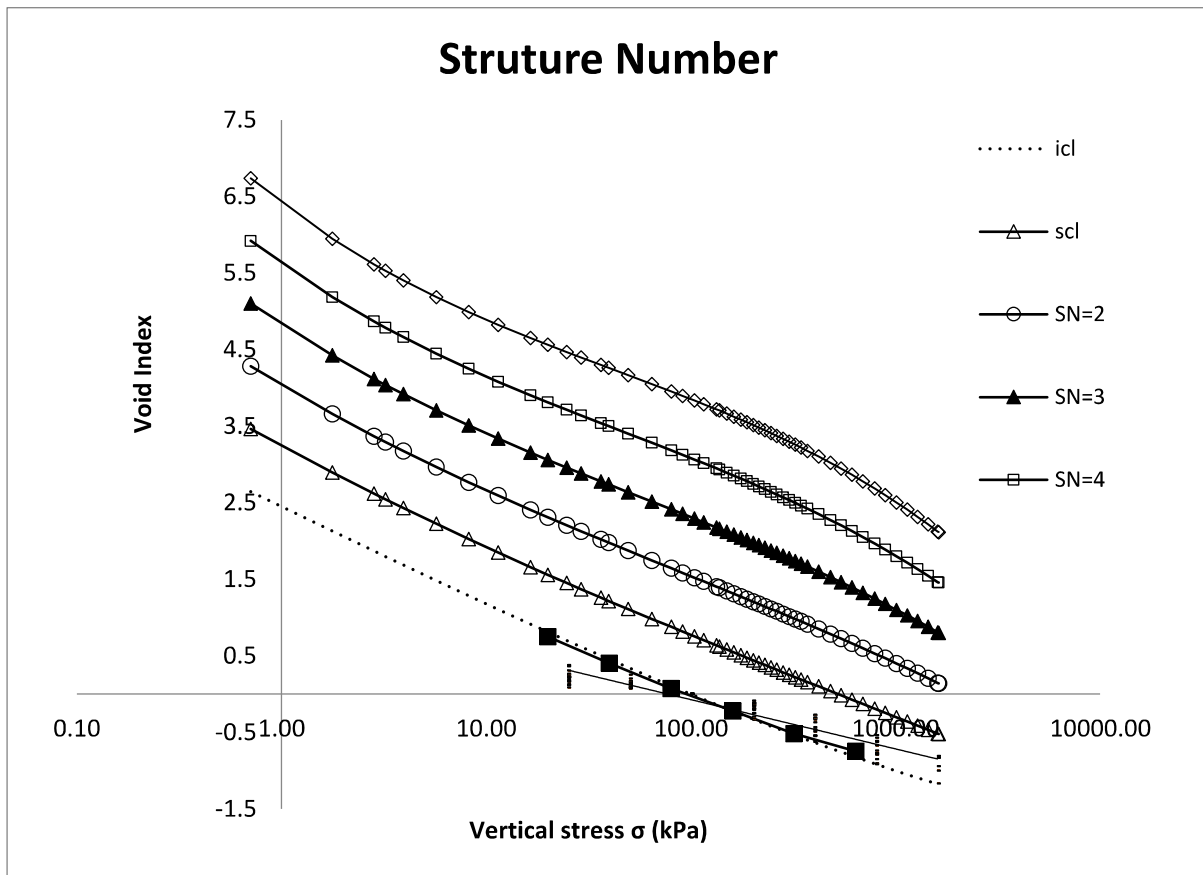


Fig. 4.11 b : Sheahan Framework of structure number to calculate strain rate dependence

### 4.3 Triaxial Test Results

In order to further investigate the effect of soil structure on mechanical properties of Normal Kolkata clay some consolidated undrained triaxial tests were conducted on undisturbed and remoulded clay specimens. The results are presented in Fig 4.12 and Fig 4.13.

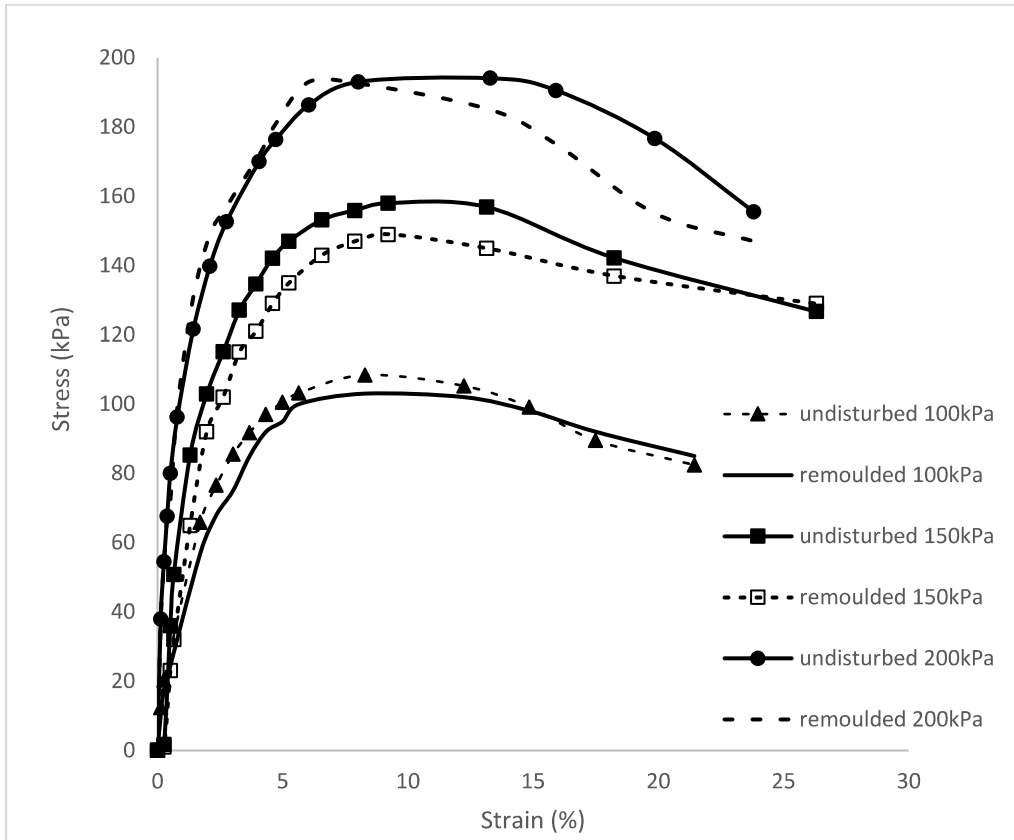


Fig. 4.12 Stress-Strain response of remoulded and natural organic clays under consolidated undrained triaxial tests

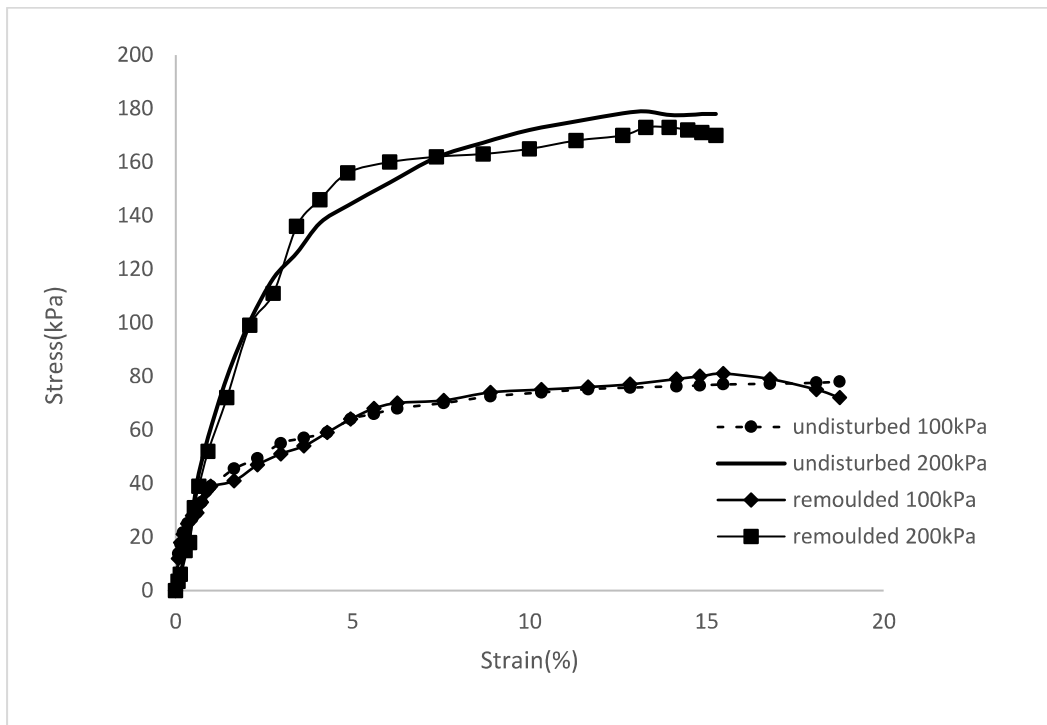


Fig. 4.13 Stress-Strain response of remoulded and natural inorganic clays under consolidated undrained triaxial tests

From the above results it can be observed that the stress-strain response of natural and remoulded clays under a confining pressure of 100kPa, 150kPa and 200kPa were similar.

#### 4.4 Inference

The effect of soil structure or soil fabric on the mechanical behaviour of clays of Normal Kolkata region were examined. It was inferred that for natural and remoulded clays the compression behaviour and parameters governing compression behaviour are similar and that the effect of soil fabric in case Kolkata clays is not prominent at all. The results obtained formed the basis of selection of remoulded inorganic and organic clays for conducting the time dependent study as it was expected to provide greater control over sample preparation, testing and hence a better consistency in studying the behaviour of soils.

## **STRAIN- AND STRESS CONTROLLED SHEAR STRENGTH BEHAVIOUR OF SOFT CLAYS OF KOLKATA**

### **5.0 General**

In this section results of strain controlled and stress controlled consolidated drained and consolidated undrained triaxial tests conducted on remoulded inorganic and organic clay soils available in Kolkata to study the variations in shear strength parameters in an attempt to examine and understand the difference in results due to testing procedures are presented. The conventional strain controlled undrained and drained triaxial tests are conducted in the laboratory to determine the shear strength parameters for analysis and design of foundations, embankments, dams, excavations etc. Theoretically the effective stress parameters obtained from consolidated drained and undrained tests are identical but practically differences in results have been reported by various researchers as highlighted in chapter 2. Time dependency of stress-strain behaviour is a very important aspect to be considered for geotechnical problems. Due to particulate nature of soil, deformation involves time dependent structural rearrangement of soil particles (Erol 1977). The actual mechanism of deformation of the soil under different field conditions is thus stress dependent. The soil deforms under the action of load imposed on it as a result of various structures/constructions carried out over the soil. Stress dependent tests are thus necessary to investigate the stress-strain-time aspects and understand the influence of time and rate of strain on strength and deformation. The stress and strain controlled tests have been conducted two to three times in order to account for reliability of results and reduce variability of data.

## **5.1 Comparison of Strain-controlled and stress-controlled tests**

### **5.1.1 *Inorganic soil-Undrained tests***

The results of undrained strain and stress-controlled tests conducted on inorganic clays are presented in Fig 5.1 to Fig 5.3. In Fig 5.1 the sequence of load increment for stress-controlled tests are shown. From the stress-strain curves presented in Fig. 5.2 it is seen that nature of curves for strain-controlled tests follow the standard hyperbolic pattern and become asymptotic at higher strain. For stress-controlled tests stresses are found to reduce with strain for a particular load increment due to the increase in area of the specimen, however, the average pattern appear to be quite similar to that of strain-controlled tests. It is also observed that till 50% of the ultimate deviator stress obtained from strain-controlled tests, corresponding to a maximum strain of say 2 %, the stress-strain response for strain-controlled and stress-controlled tests are similar. Beyond this, there is a pronounced increase in stress-strain response for stress-controlled tests due to the change in soil particle arrangement owing to the difference in duration of load application. Further, from the stress-strain curves it is evident that the load carrying capacity is more for stress-controlled tests under similar conditions with higher values for higher confining pressure. At confining pressure of 100 kPa and 200 kPa the load carrying capacity increases by 60 % and 52 % respectively in case of stress-controlled test.

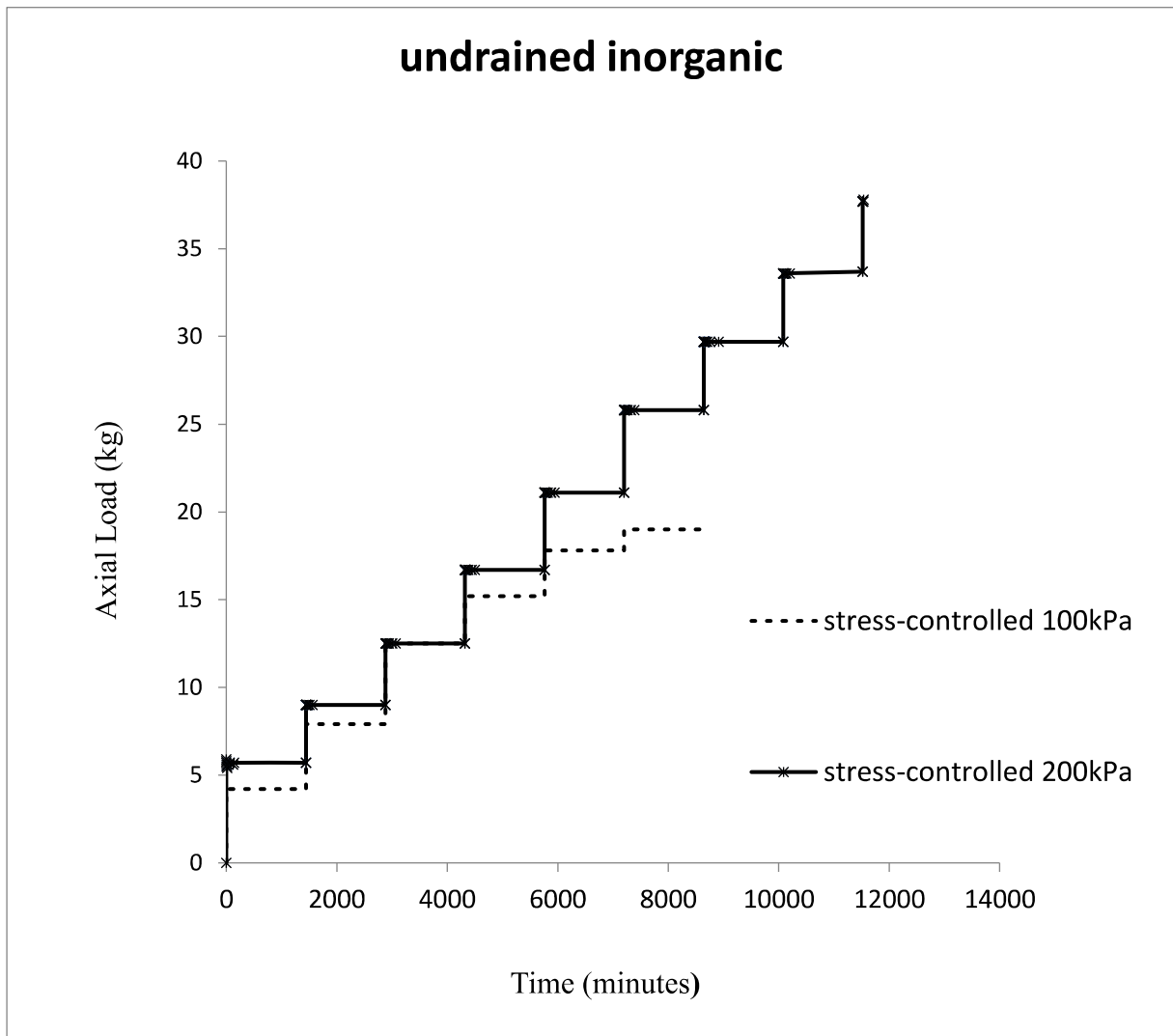


Fig. 5.1. Load vs. time curves for inorganic clays during undrained stress-controlled tests

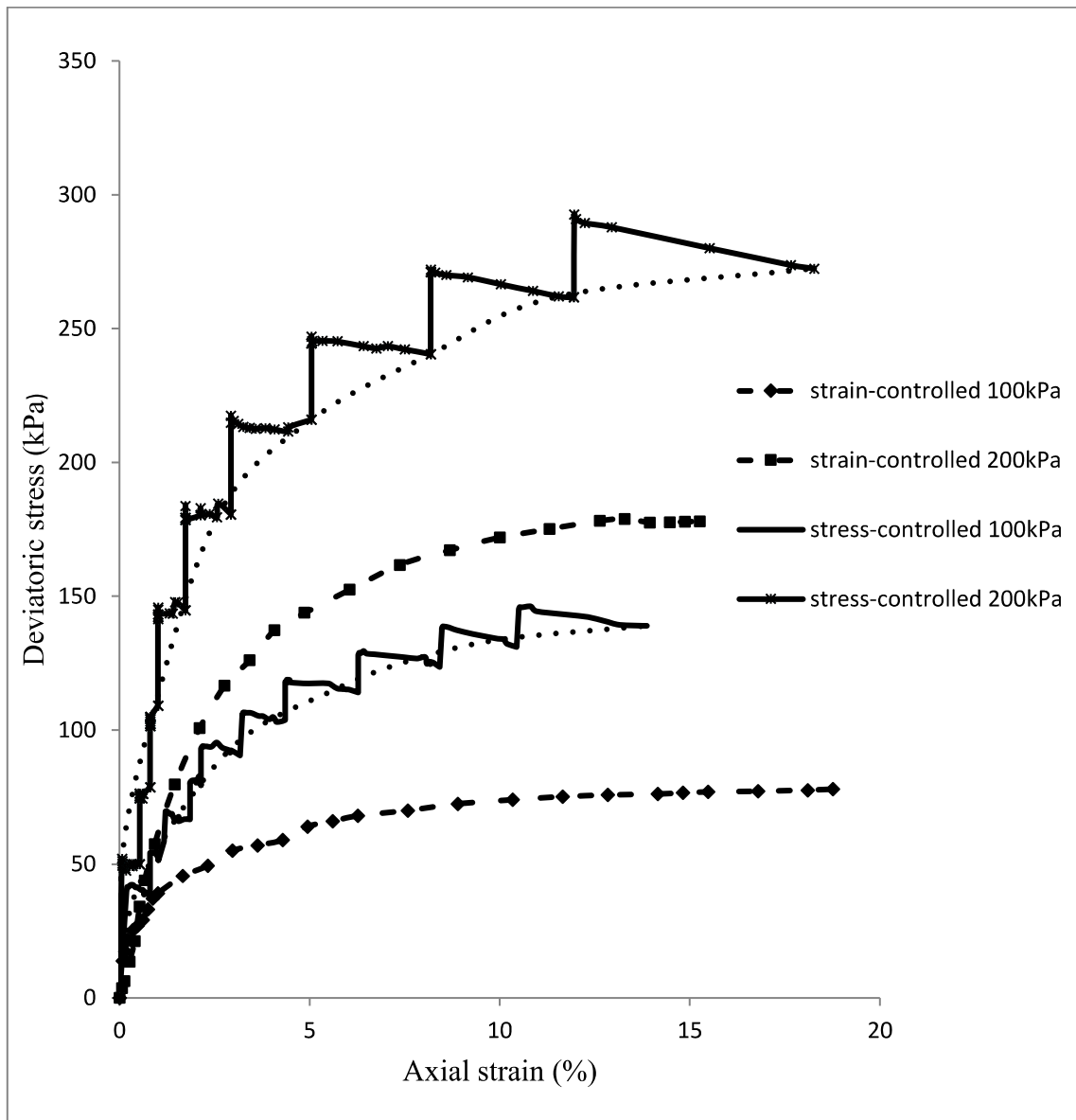


Fig. 5.2 :Stress-strain response for undrained inorganic clays

Fig. 5.3 presents the variations in pore pressure with axial strain for inorganic clay soil of Normal Kolkata Deposit. The maximum excess pore pressure developed during strain-controlled tests was more as compared to stress-controlled tests. The initial pore pressure developed after application of load during stress-controlled tests is higher but with deformation the rate of increase reduces and the maximum pore pressure is lesser than the corresponding values observed during strain-controlled tests under similar conditions.

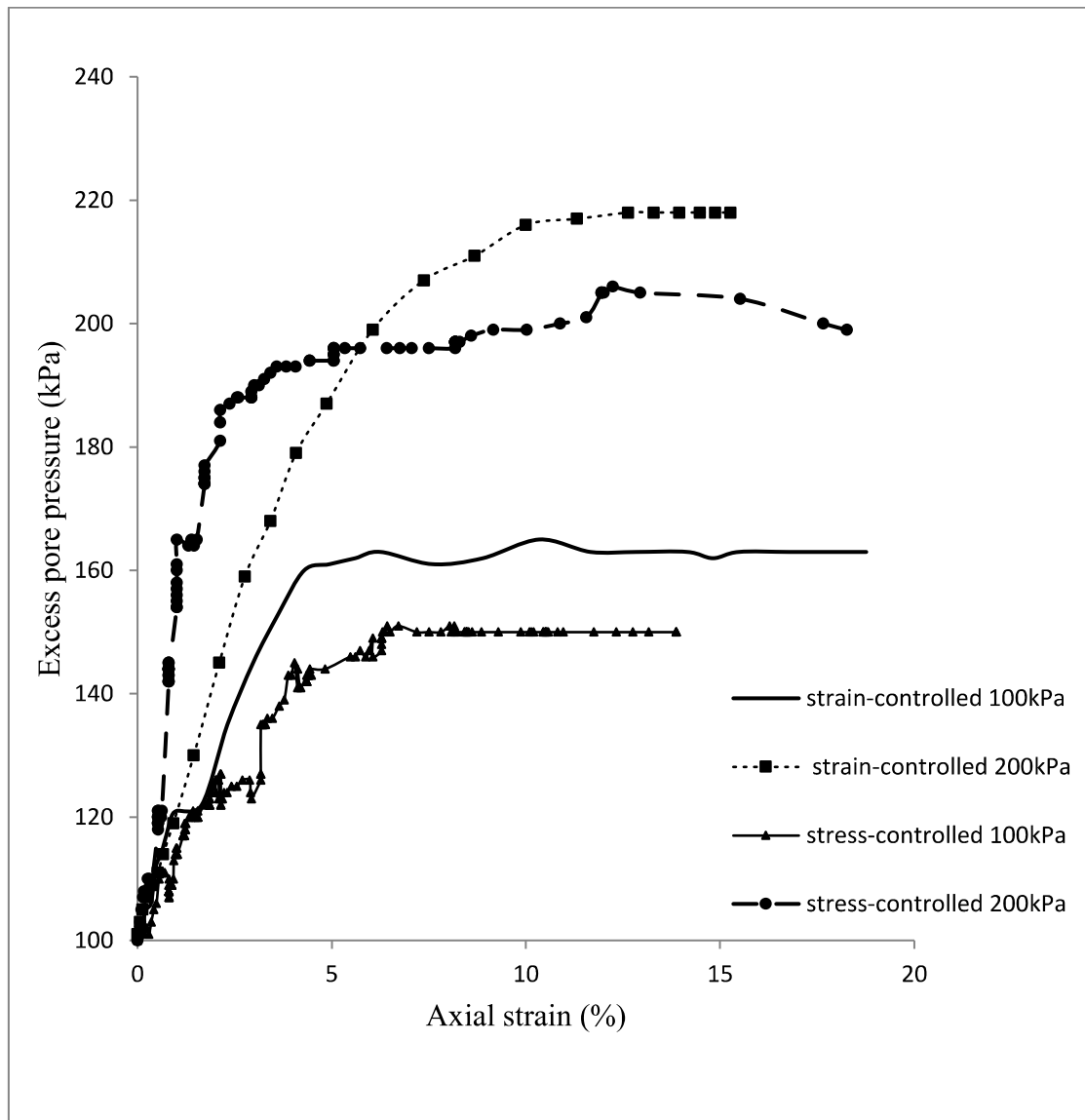


Fig.5.3 : Pore pressure response for undrained inorganic clays

### 5.1.2 Inorganic soil-Drained tests

The test results of strain-controlled and stress-controlled drained triaxial tests conducted on inorganic clays of Normal Kolkata deposit have been presented in Fig. 5.4 to Fig. 5.6. In case of drained tests with effective cell pressure of 200 kPa the deviator stress was higher by 14% for stress-controlled tests. This may be due to increase in strength as a result of consolidation under drained condition during different load increment. This is also evident from the volume change diagrams plotted subsequently. At lower confining pressure the stress-strain curve for

both strained controlled and stress-controlled tests produced similar response of the soil due to soft consistency of the soil.

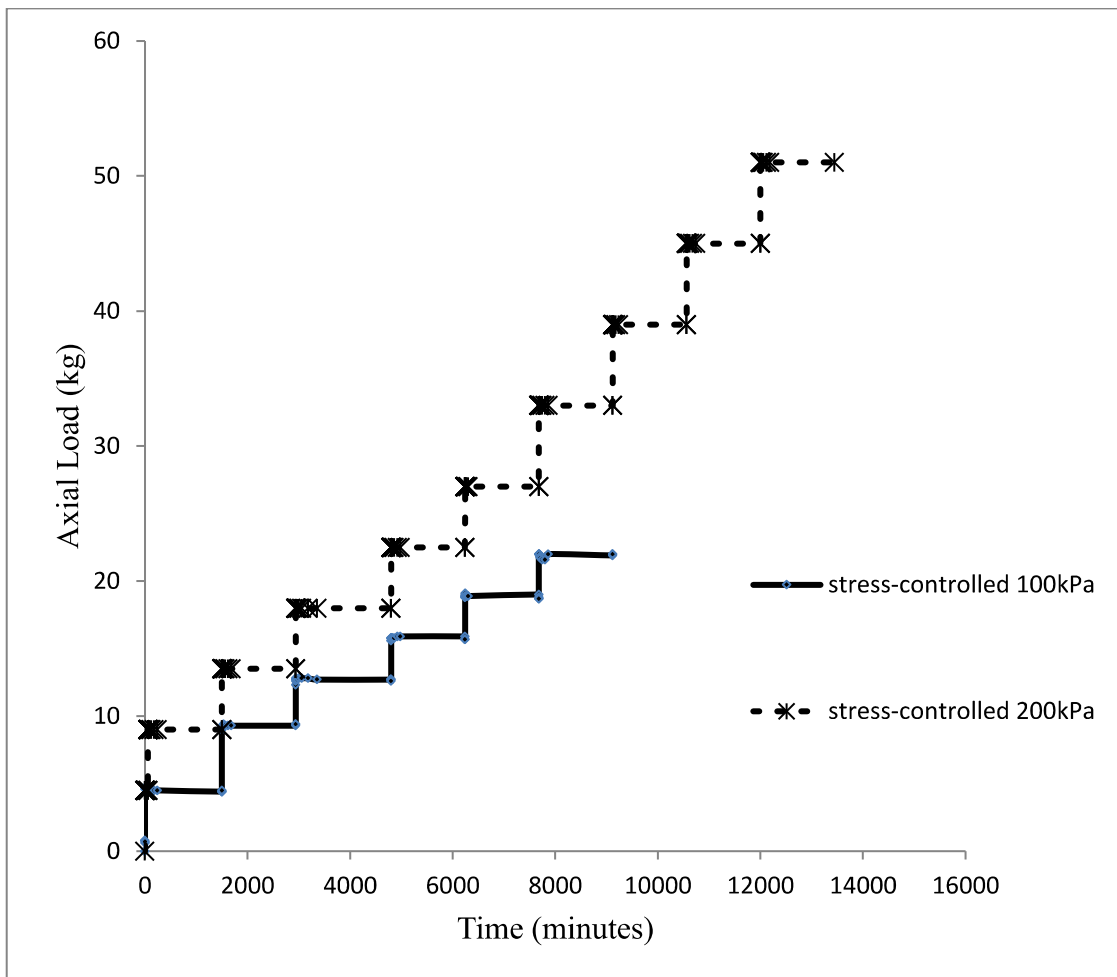


Fig. 5.4 : Load vs. time curves for inorganic clays during drained stress-controlled tests

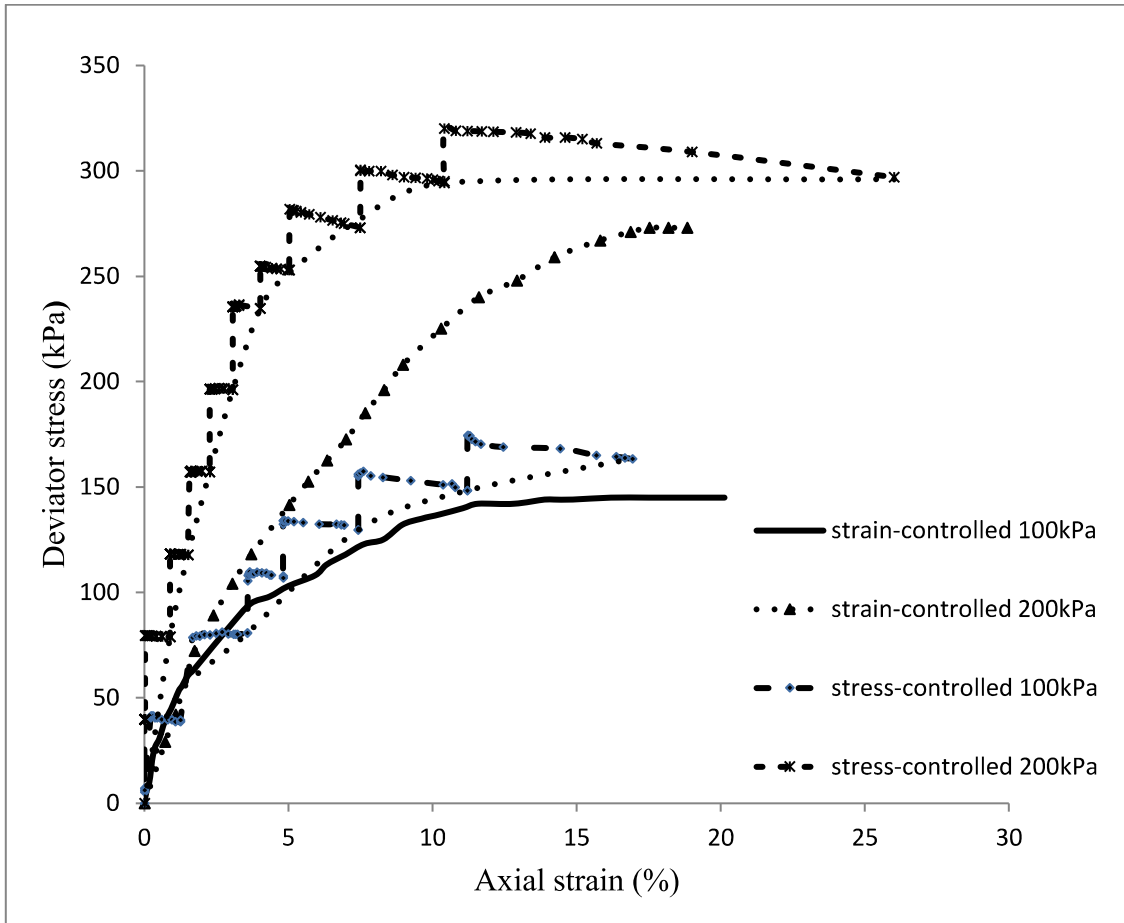


Fig. 5.5 : Stress-strain response for drained inorganic clays

The volume changes during the triaxial tests have been presented in Fig. 5.6. The volume change in case of stress-controlled test under 200 kPa was more than that for strain-controlled test. The trend at lower effective pressure is more or less similar for both the types of tests.

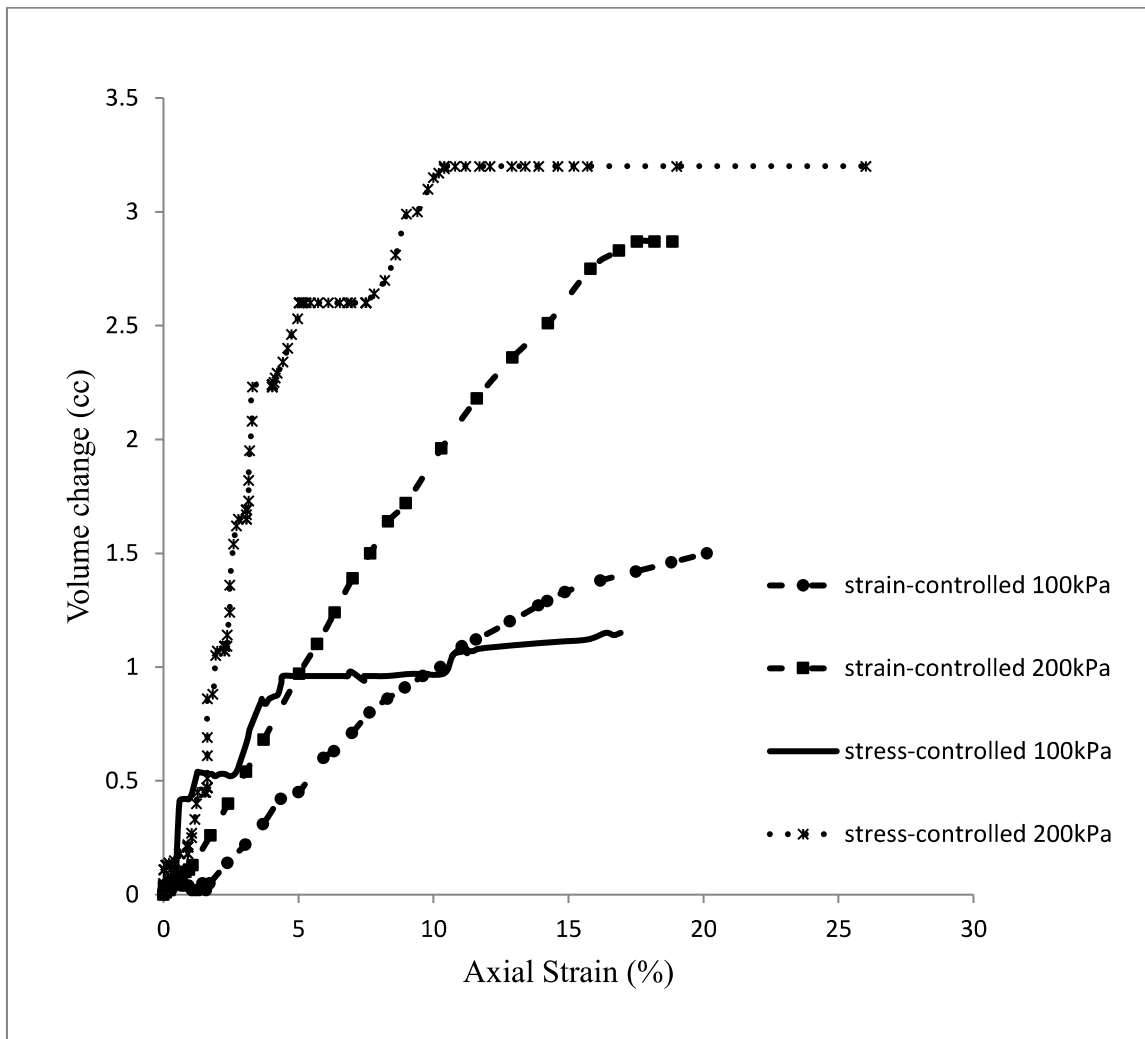


Fig. 5.6 : Volume change vs. axial strain curves for drained inorganic clays

### 5.1.3 Organic soil-Undrained tests

The results of undrained strain-controlled and stress-controlled tests conducted on organic clays are presented in Fig. 5.7 to Fig. 5.9. From the stress-strain curves obtained it is evident that the behaviour is similar to that observed in case of inorganic soil. The maximum deviator stress is found to increase by 68 % and 47 % at confining pressure of 100 and 200 kPa respectively for stress-controlled tests. Further, the trend for pore water pressure variation is similar to that of inorganic clay (Figure 5.3). The pore pressure generation response is more or less same up to a strain of about 3 % but thereafter a difference is noticed and the maximum

excess pore pressure developed during strain-controlled tests more than that of stress-controlled tests.

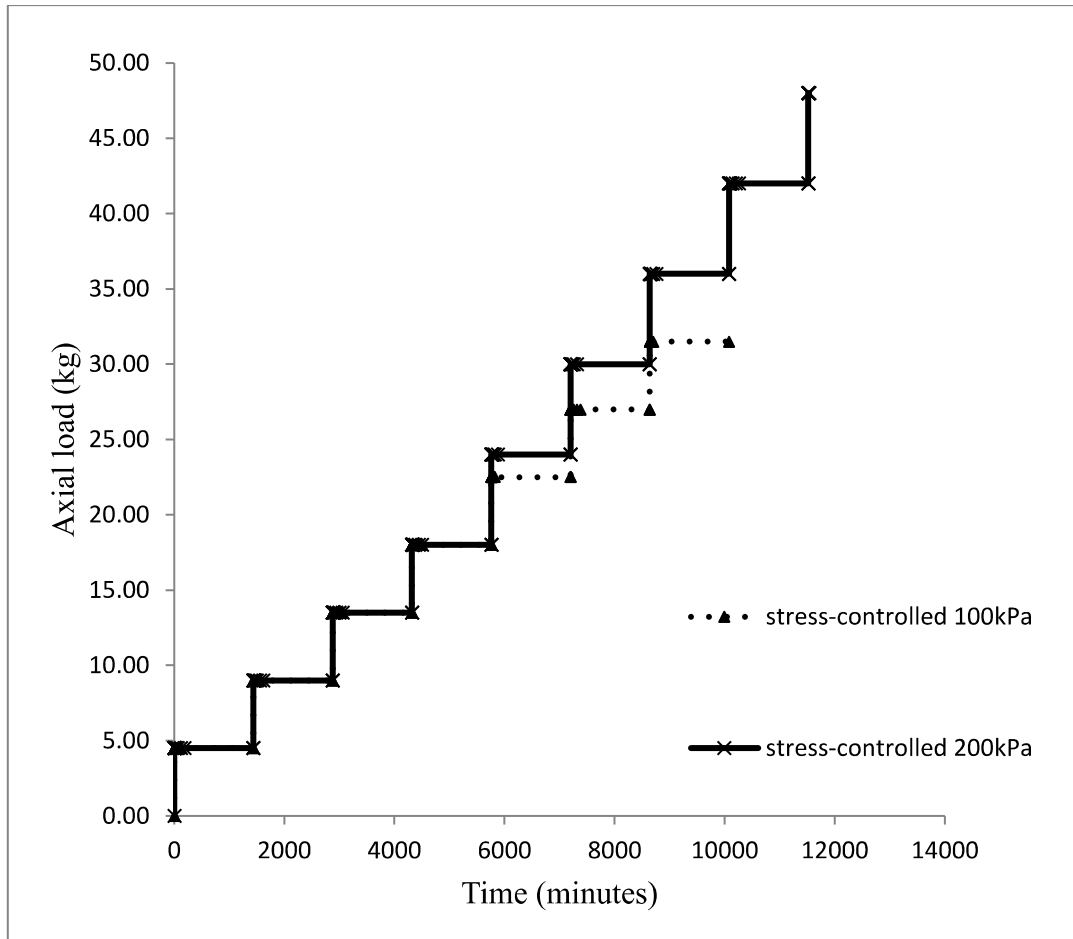


Fig. 5.7 : Load vs. time curves for organic clays during undrained stress-controlled tests

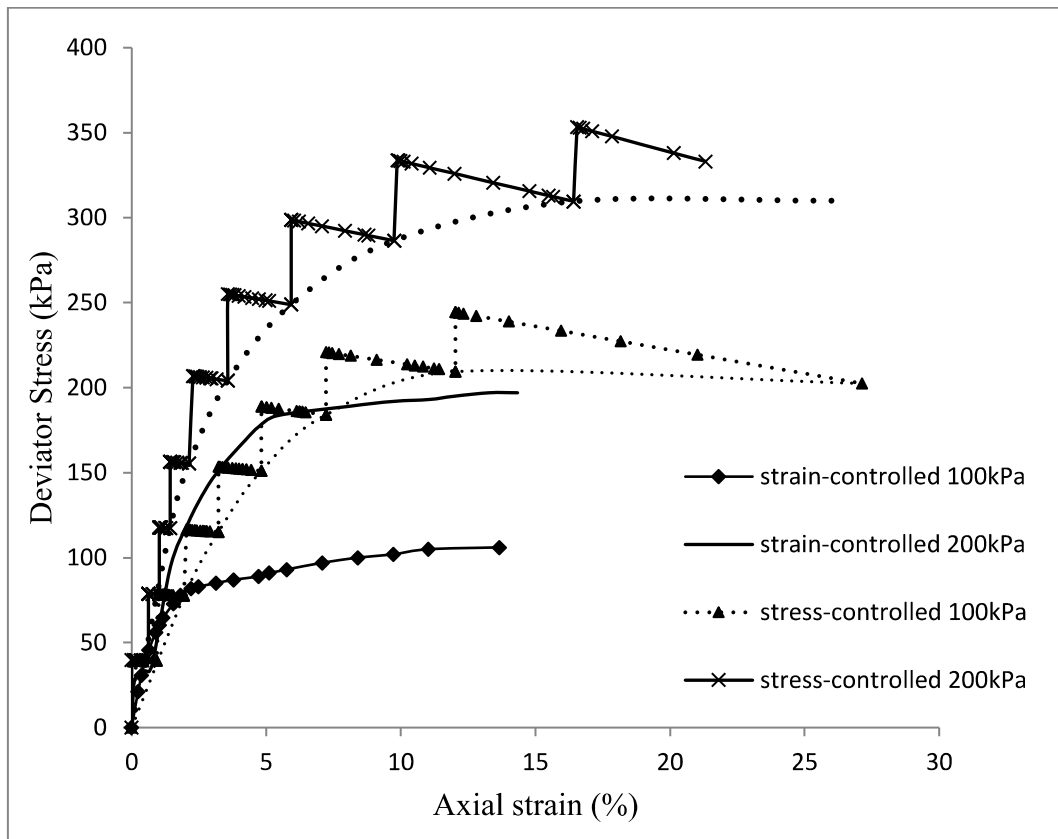


Fig. 5.8 : Stress-strain response for undrained organic clays

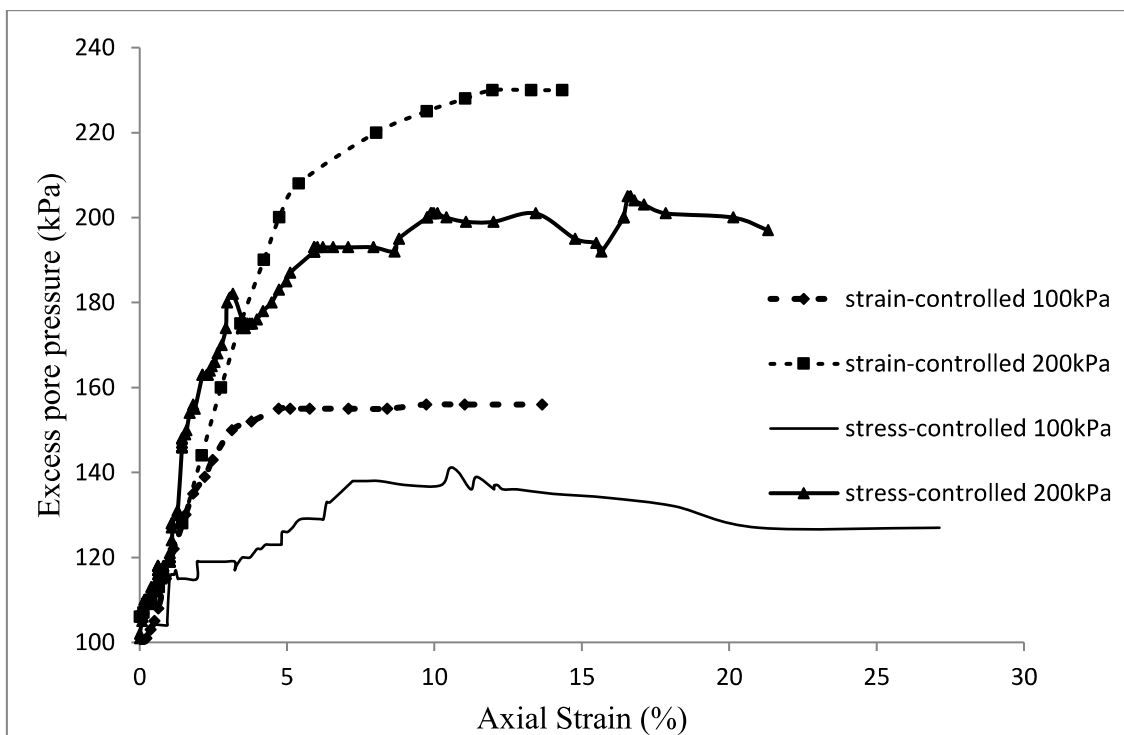


Fig. 5.9 : Pore pressure response for undrained organic clays

#### 5.1.4 Organic soil-Drained tests

The test results of strain-controlled and stress-controlled drained triaxial tests conducted on organic clays of Normal Kolkata deposit have been presented in Fig. 5.10 to Fig. 5.12. In case of drained tests with effective pressure of 200 kPa and 100 kPa the soil strength was marginally higher for stress-controlled tests. The deviator stress at failure increased by 0.38 % and 9.4 % corresponding to 200 kPa and 100 kPa in case of stress-controlled tests as compared to strain-controlled tests.

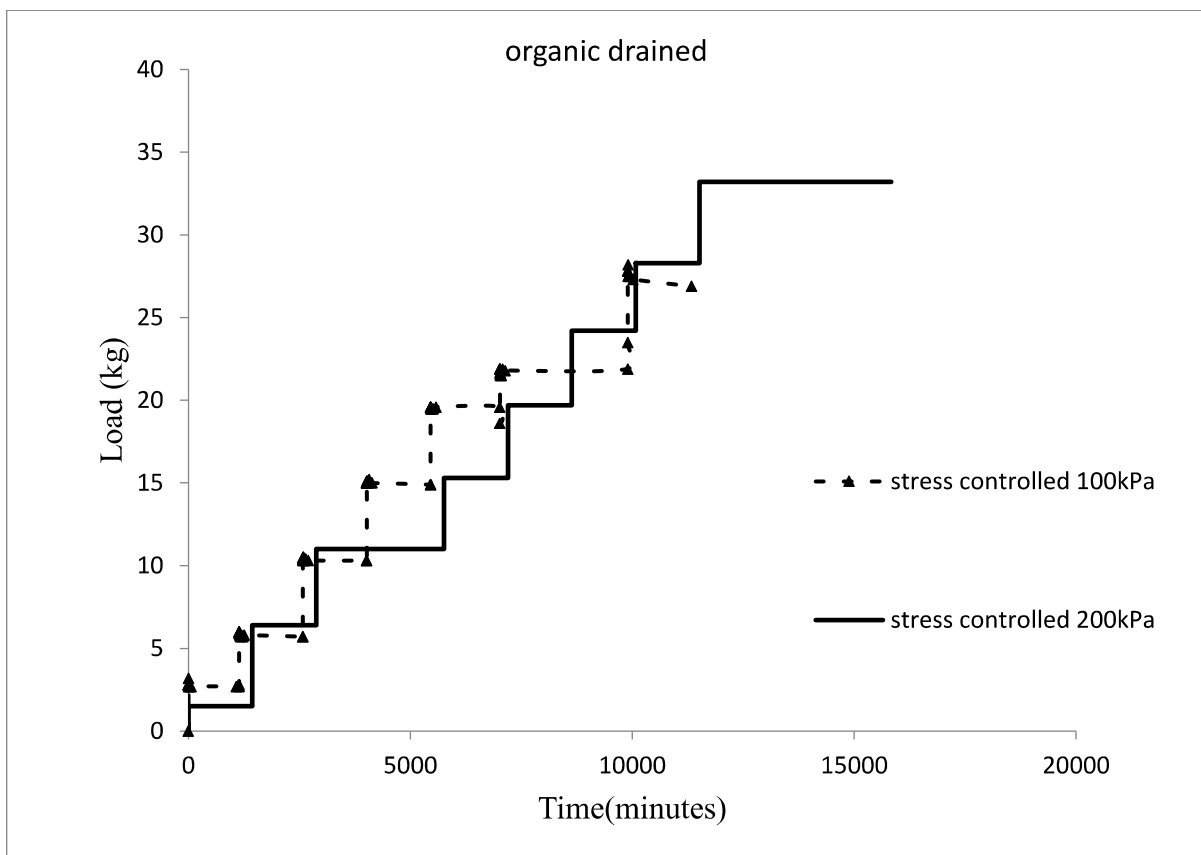


Fig. 5.10 : Load vs. time curves for organic clays during drained stress-controlled tests

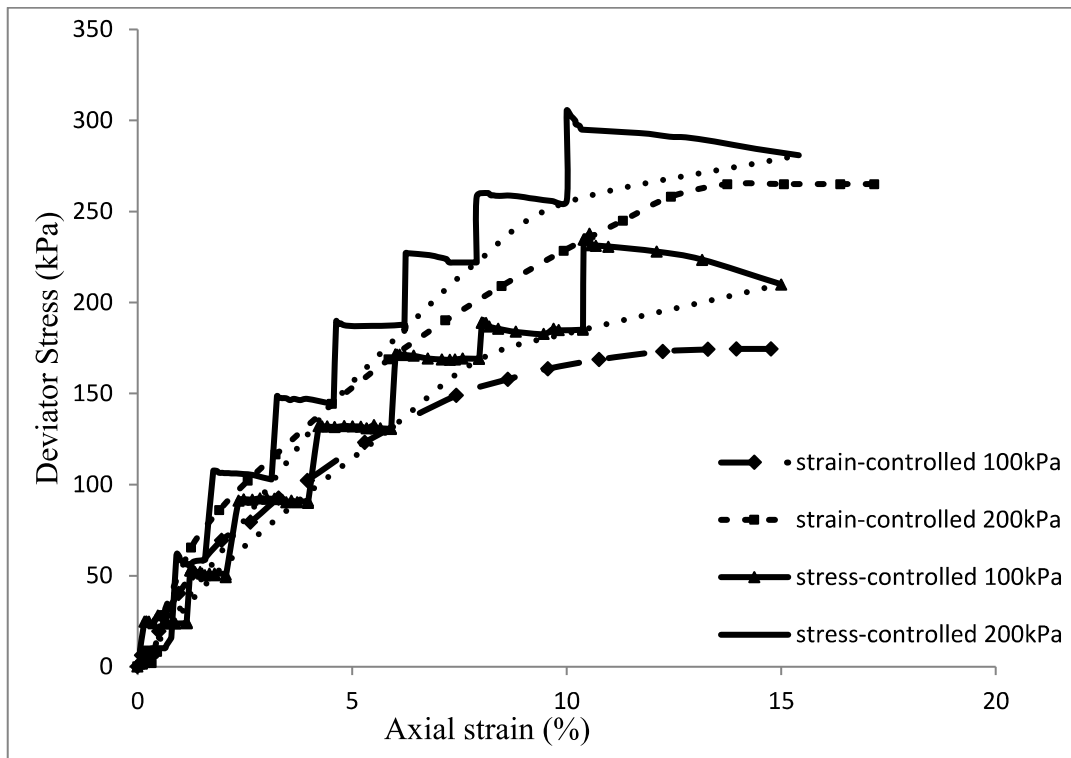


Fig. 5.11 : Stress-strain response for drained tests on organic clays

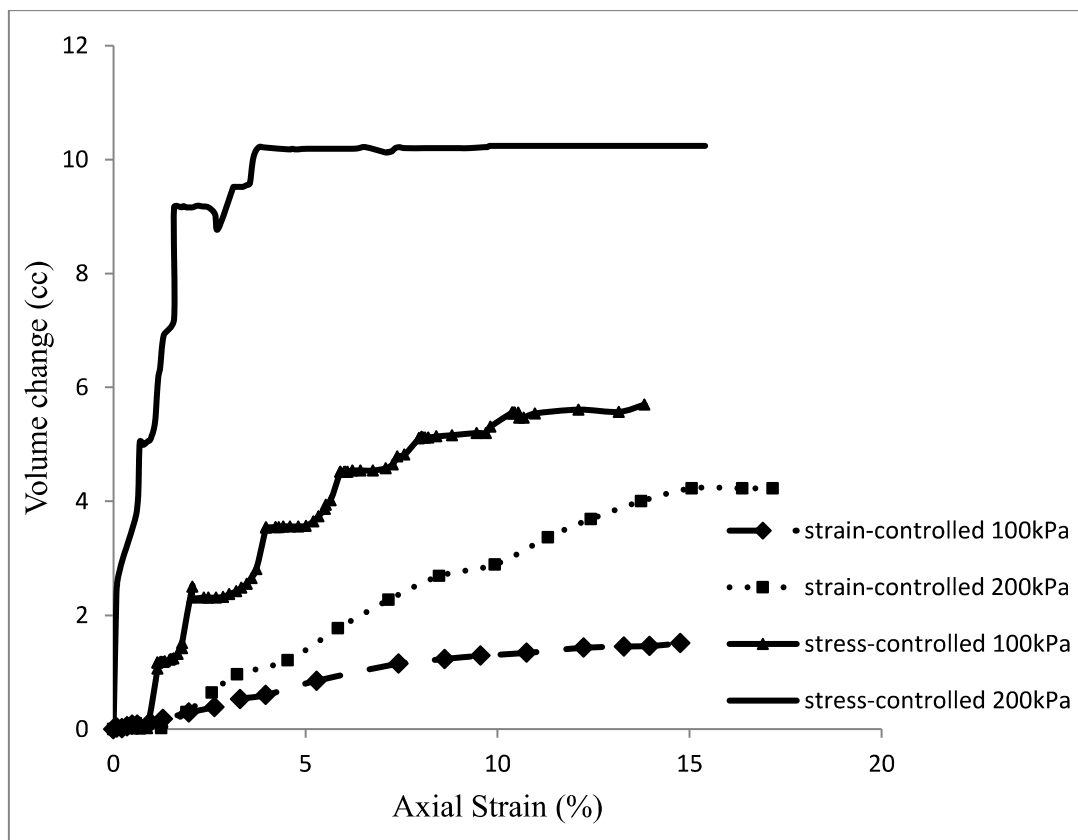


Fig. 5.12 : Volume change vs. strain curves for drained tests on organic clays

The volumetric strain in case of stress-controlled is higher than in strain-controlled tests (Figure 5.12). A part of this volumetric strain may contribute to the axial strain of the soil in the initial stress-strain response. At higher effective pressure the soil becomes stiff due to consolidation and, therefore, the axial strain becomes lesser.

## 5.2 Pore pressure parameter and poisson's ratio

### *Pore pressure parameter and Poisson's ratio*

The  $A_f$ , pore water parameter at failure, for undrained triaxial tests were calculated as the ratio of excess pore water pressure at maximum deviator stress and corresponding deviator stress. In case of stress-controlled tests deviator stress corresponding to penultimate load increment was the maximum deviator stress. The results are presented in Table 5.1. From this table it may be seen that for strain-controlled tests the magnitude of  $A_f$  lies between 0.52 to 0.83 while that for stress-controlled test is 0.20 – 0.35. This indicates that there is a change in grain arrangement or soil structure imparting a pseudo over-consolidation effect due to the increase in loading period.

To check for the consistency and the reliability of the test results the Poisson's ratio for drained triaxial tests were calculated for different ranges of strain and stresses in strain-controlled and stress-controlled tests. The Poisson's ratio for a soil specimen was calculated as the ratio of radial strain to axial strain. The radial strain can be obtained as volumetric strain minus the axial strain of the sample whole divided by two.

$$\epsilon_r = \frac{\epsilon_v - \epsilon_a}{2} = \frac{\frac{\Delta v}{v} - \frac{\Delta l}{l}}{2} \quad (5.1)$$

$$\text{Poisson's ratio, } \mu = -\frac{\epsilon_r}{\epsilon_a} \quad (5.2)$$

Where,  $\varepsilon_v$  is the volumetric strain,  $\varepsilon_r$  is the radial strain,  $\varepsilon_a$  is the axial strain,  $v$  is the volume,  $l$  is the length of the sample and  $\Delta v$  and  $\Delta l$  are the change in volume and change in length respectively.

Further, the Poisson's ratio, as obtained from drained tests for both strain-controlled and stress-controlled tests, lying in the range of 0.36 – 0.43 (Table 5.1) indicates that there is no significant change in behaviour due to the change in loading methodology.

Table 5.1 Pore water pressure parameter,  $A_f$ , and Poisson's ratio,  $\mu$

Type of triaxial test	Drainage condition	Type of soil	Cell pressure (kPa)	Back pressure (kPa)	$A_f$	$\mu$ (Poisson's ratio)
Strain-controlled	Consolidated undrained (CU)	Inorganic	300	100	0.67	
			200	100	0.83	
		Organic	300	100	0.65	
			200	100	0.52	
Strain-controlled	Consolidated drained (CD)	Inorganic	300	100		0.36-0.40
			200	100		0.37-0.43
		Organic	300	100		0.38-0.40
			200	100		0.36-0.42
Stress-controlled	Consolidated undrained (CU)	Inorganic	300	100	0.35	
			200	100	0.35	
		Organic	300	100	0.33	
			200	100	0.20	
Stress-controlled	Consolidated drained (CD)	Inorganic	300	100		0.39-0.42
			200	100		0.38-0.40
		Organic	300	100		0.40-0.42
			200	100		0.38-0.40

### 5.3 Shear strength parameters from Strain and Stress controlled tests

#### *Shear strength parameters from Strain and Stress-controlled tests*

The maximum deviator stress as obtained from the stress-strain diagram for different cases are found to be quite close to the failure deviator stress corresponding to  $(\sigma_1'/\sigma_3')_{\max}$ . These are summarized in Table 5.2 along with the normalized undrained cohesion,  $C_u/p_o$ . Here,  $C_u$  is the undrained cohesion of the soil estimated as the half of the maximum deviator stress acting on the soil specimen prior to failure and  $p_o$  is the initial effective confining pressure acting on soil specimen i.e. 100kPa or 200kPa. For stress-controlled tests penultimate deviator stress is considered to be the maximum deviator stress. From this table it may be seen that  $C_u/p_o$  is higher for stress-controlled tests for both inorganic and organic soil.

Table 5.2: Maximum deviator stress (kPa).

Type of clay	Effective pressure (kPa)	Strain-controlled tests			Stress-controlled tests		
		CU	$C_u/p_o$	CD	CU	$C_u/p_o$	CD
Inorganic	100	79	0.395	145	133	0.63	148
Inorganic	200	178	0.44	273	262	0.655	294
Organic	100	106	0.53	174	180	0.90	185
Organic	200	197	0.49	255	309	0.775	257

The effective stress parameters are determined from Mohr's circles which gives the effective cohesion  $c'$  value as zero with only internal friction, given in Table 5.3, contributing to the strength as the soils studied are normally consolidated in nature. From these results it is clearly evident that in case of strain-controlled undrained tests the effective friction angle is higher

than drained tests. For inorganic clays of Kolkata clay the difference in effective friction angle is 4 to 6 degrees whereas for organic clays of Kolkata clay the difference in effective friction angle is around 5 to 8 degrees. These trends are in line with some of the previous literatures as indicated in Fig 5.13. This is because during drained tests there is a reduction in volume of the soil specimen due to expulsion of pore water which causes an increase in repulsive forces as the clay particles move closer to one another, thus reduces the resistance to external forces, thereby reducing the value of friction angle (Sridharan 1991; Sachan and Penumadu 2007). In case of undrained tests, the repulsive force between the soil particles are less as they are separated by the pore water resulting in increase in resistance to the external forces, leading to a relative increase in friction angle.

Table 5.3 .Effective angle of internal friction ( $\phi'$  and  $\phi_d$ ) in degrees

Type of clay	Effective pressure (kPa)	Strain-controlled tests		Stress-controlled tests	
		CU	CD	CU	CD
Inorganic	100	31	27	36	25
Inorganic	200	31	25	37	26
Organic	100	32	27	37	27
Organic	200	33	25	38	25

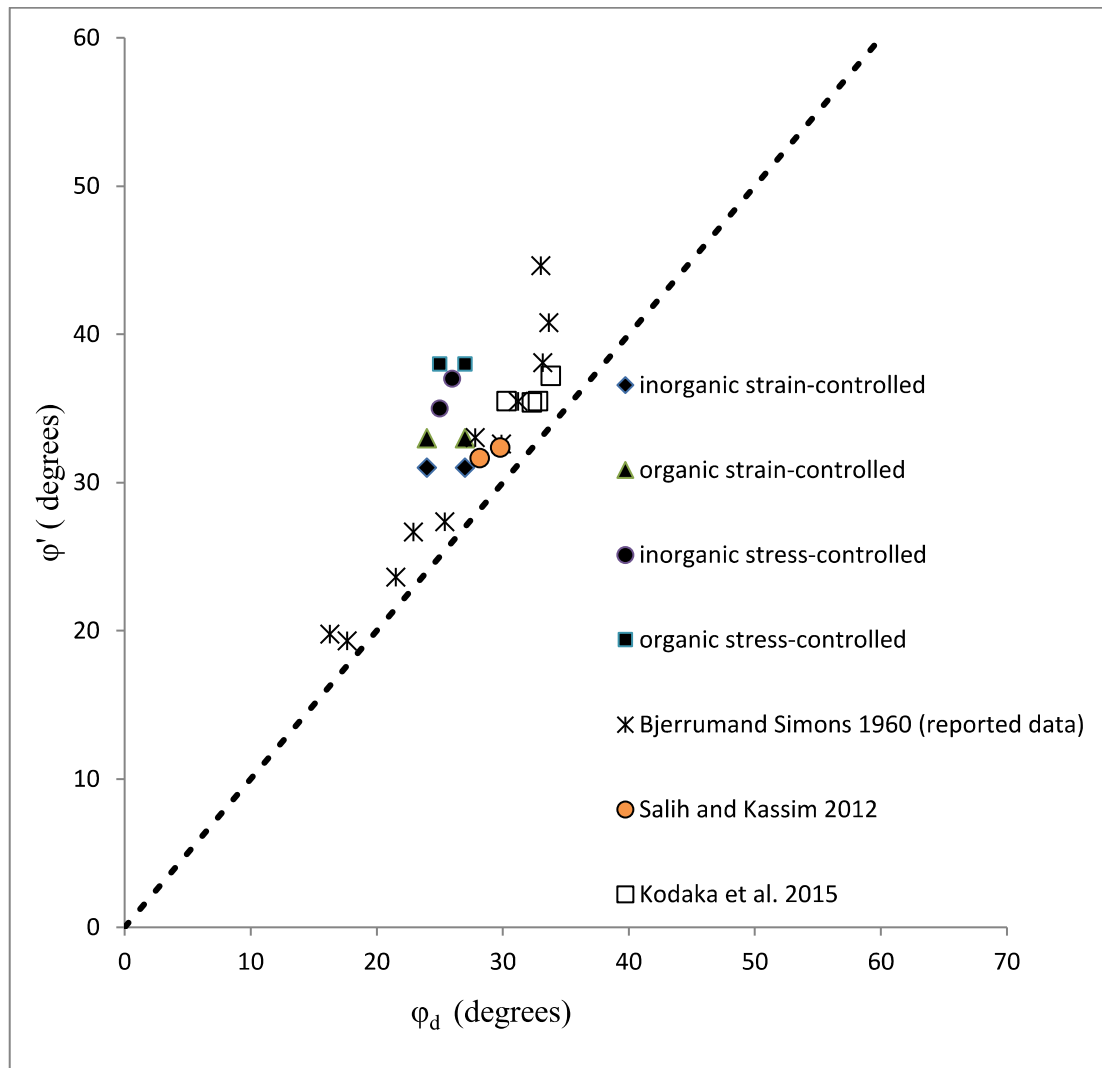


Figure 5.13. Variations in effective friction angle due to change in drainage conditions and method of load application (stress/strain-controlled)

Further, from Table 5.3 it is observed that for undrained tests, friction angle in case of stress-controlled tests are significantly higher than those obtained from strain-controlled tests. This is because under sustained loading there is a decrease in  $A_f$  value as evident from Table 5.1 possibly due to the increase in strength and stiffness due to ageing as a result of rearrangement of particles, may be from flocculated to dispersed with time.

From previous studies (Casagrande and Wilson 1951, Bjerrum 1972, Lefebvre and LeBoeuf 1987, Sheahan et al. 1996) it is clear that the undrained shear strength of normally consolidated

(NC) clay increases at rate of 5-15% per log cycle increase in strain rate. The effective friction angle ( $\phi'$ ) also increases with increase in strain rate as a result of reduction in shear-induced pore pressures (Lo and Morin 1972, Tavenas et al. 1978, Vaid et al. 1979, Leroueil and Tavenas 1979, Sheahan et al. 1996). But the results obtained from the present study indicates that normalized undrained shear strength ( $C_u/p_o$ ) as well as friction angle ( $\phi'$ ) is greater for stress-controlled tests though the samples experienced strain rates much lesser than those of strain-controlled tests. It is difficult to compare these results on the basis of strain rate as there exists a fundamental difference in load application / strain inducing mechanism between stress-controlled and strain-controlled tests which is evident from the pore pressure generated during the tests. The increase in undrained shear strength and effective friction angle in case of stress-controlled tests may be due to the effect of ageing and rearrangement of soil particles leading to increased interlocking under sustained loading (Schmertmann 1991). However, the  $\phi_d$  values obtained from stress-controlled drained tests are more or less equal to  $\phi_d$  values from strain-controlled drained tests and the difference is not as pronounced as in case of undrained tests.

#### **5.4 Inference**

On the basis of the present study the following conclusions can be made:

1. In case of strain-controlled tests, maximum deviator stress obtained from drained tests are higher than that from undrained tests for both inorganic and organic clays. However, friction angle obtained from undrained tests are quite higher than that obtained from drained tests,  $4-6^\circ$  and  $5-8^\circ$  for inorganic and organic clays respectively.
2. The maximum deviator stress and also effective friction angle obtained from stress-controlled undrained tests are higher than those obtained from strain-controlled

undrained tests for both inorganic and organic clays. However, for drained tests no significant difference is observed.

3. Pore pressure generated from strain-controlled tests is higher than corresponding values from stress-controlled tests.  $A_f$  for strain-controlled and stress-controlled tests were in the range of 0.52 - 0.83 and 0.20 - 0.35 respectively.



## **CREEP POTENTIAL OF SOFT CLAYS OF NORMAL KOLKATA DEPOSIT**

### **6.0 General**

Creep behaviour of clayey soils across the globe has been extensively studied and documented indicating their detrimental effects on the stability of structures, embankments and dams, excavations etc. using a large number of case studies since the middle of last century at various locations all over the world. A number of similar cases were also reported on distressing / failure of structures in and near Kolkata due to the presence of soft clay in the upper region as highlighted in the literature. The creep/long term behaviour of soft inorganic and organic clays of Normal Kolkata Deposits has been examined in this section by conducting multistage creep tri-axial tests on artificially consolidated soil specimens under undrained conditions. The creep parameters developed by Singh and Mitchell have been calculated to quantify the creep potential of soft clays of Kolkata. Chapter 4 presented a preliminary study conducted on undisturbed samples locally collected which established that the effect of soil structure in case of clays of Normal Kolkata deposit is minimal. The intrinsic properties (i.e the properties of remoulded, unstructured clays) of Kolkata clays is similar to properties of naturally sedimented clays (undisturbed structure) of this region. To maintain uniformity in sample preparation and to understand the variation in creep behaviour and its dependence on various parameters, artificially remoulded clays were thus considered for this study. Each triaxial tests was conducted twice, at times thrice in order to account for reliability of obtained results and reduce variability of data. The applicability of the creep parameters obtained from these tests were assessed with the help of a design example for a typical foundation of the High Court building resting over soft Normal Kolkata deposit.

## 6.1 Test results and discussion – undrained analysis

### *Creep triaxial tests*

The axial deformation has been normalised in order to represent the accumulation of deformation under the action of applied loads corresponding to specific stress levels. The stress level is defined as the ratio of load applied axially to the ultimate load obtained from conventional monotonic triaxial tests under similar conditions as given in Fig.6.1 and the pore pressure response has been presented in Fig 6.2. The have been summarised in Table 6.1.

Further,  $\sigma_d/p_o$  is the ratio of deviator stress to the effective confining pressure.

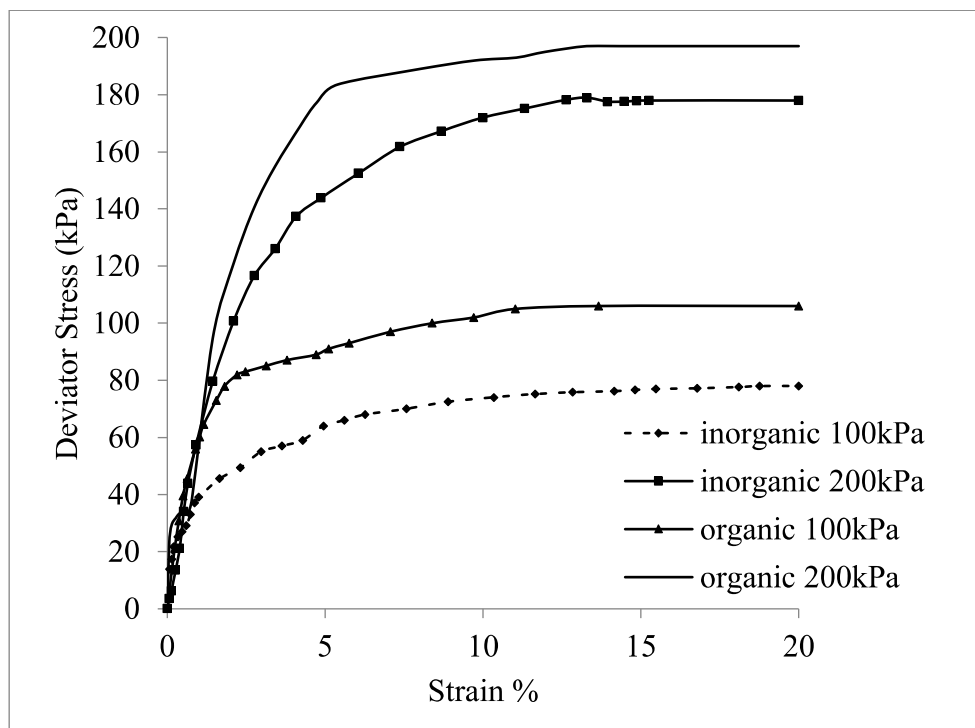


Fig. 6.1: Stress-strain response from conventional triaxial undrained tests.

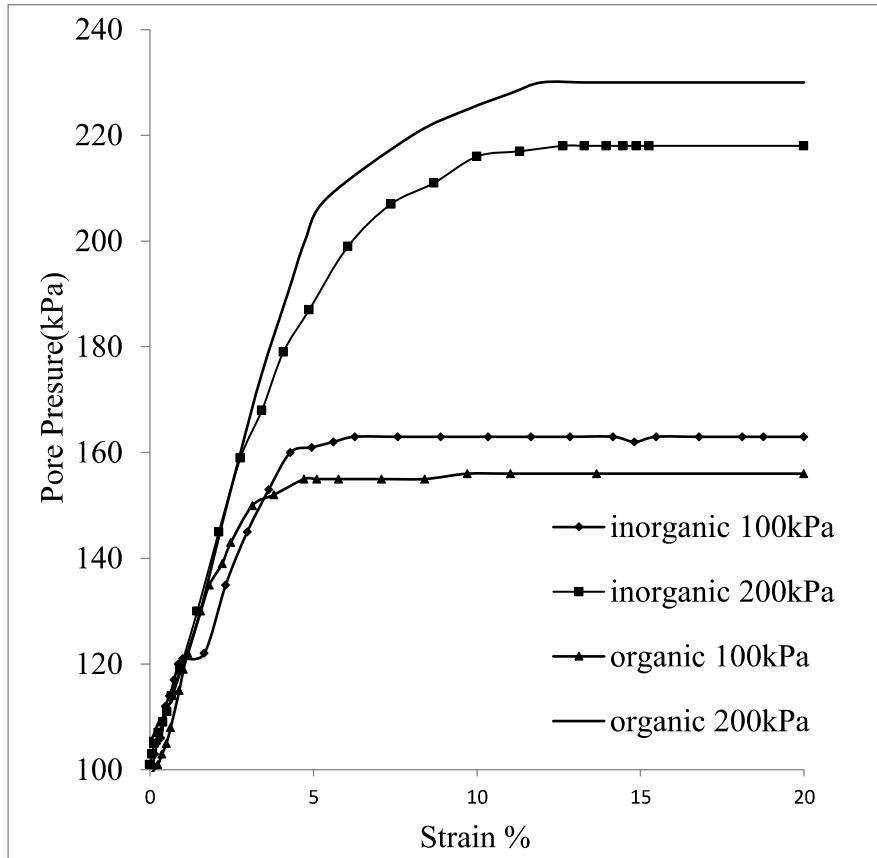


Fig. 6.2 : Pore pressure response from conventional triaxial undrained tests.

Table 6.1 Maximum deviator stress and pore water pressure

		Inorganic Soil			Organic Soil		
Confining pressure (kPa)	Back pressure (kPa)	Maximum Deviator stress	Maximum excess pore water pressure	A-value	Maximum Deviator stress	Maximum excess pore water pressure	A-value
200	100	76	63	0.82	106	56	0.52
300	100	177	118	0.67	197	130	0.66

Fig 6.3 presents axial deformation vs time. Axial deformation normalized with respect to failure deformation as obtained from conventional monotonic triaxial tests vs. time and excess pore pressure vs. time response under different load increments (deviator stress, kPa) corresponding to different stress levels for inorganic and organic clayey soils tested under undrained conditions for confining pressure of 100 and 200 kPa are presented in Figs. 6.4 and 6.5.

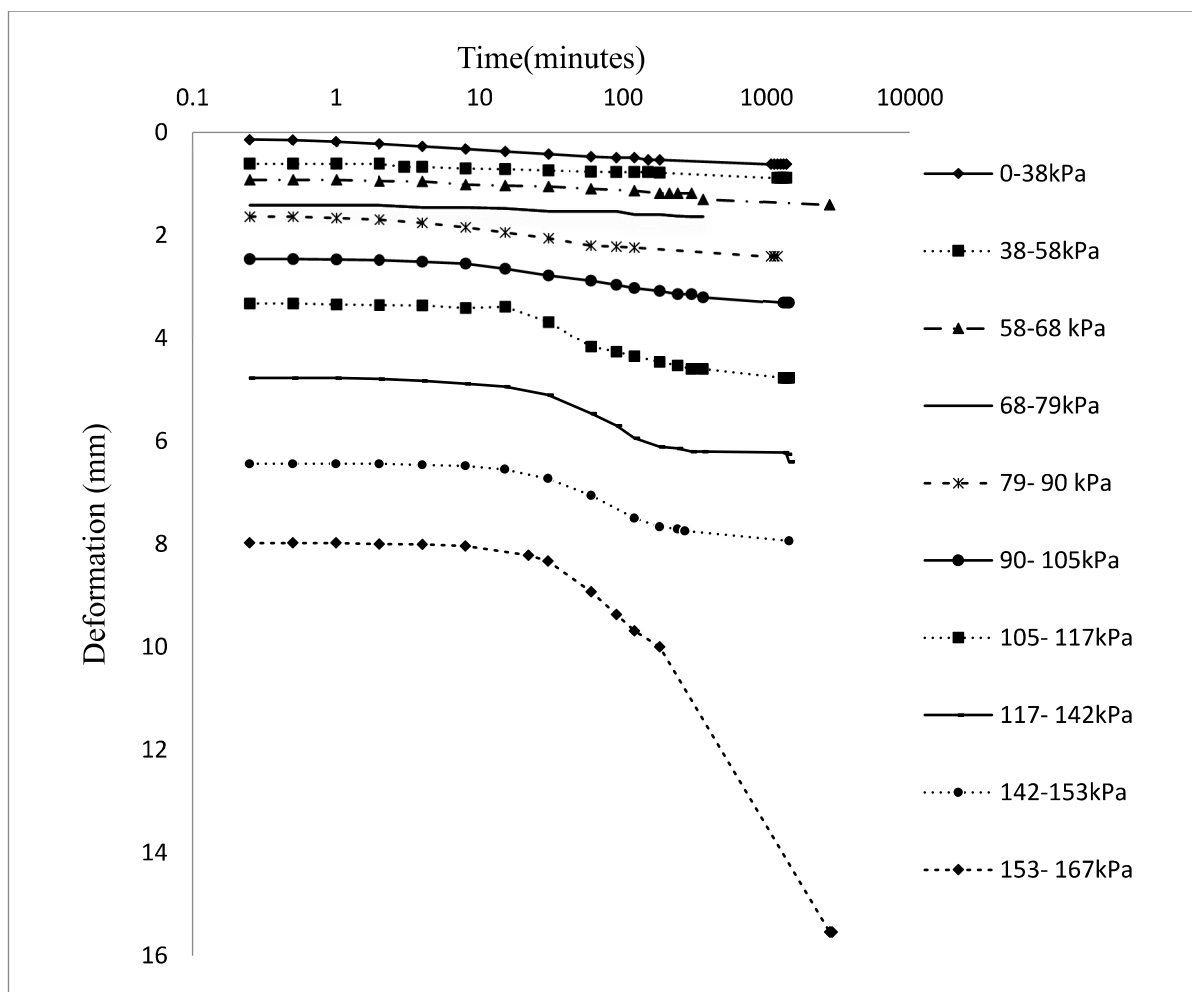


Fig. 6.3(a) : Axial deformation vs. time curve Inorganic clay-100kPa

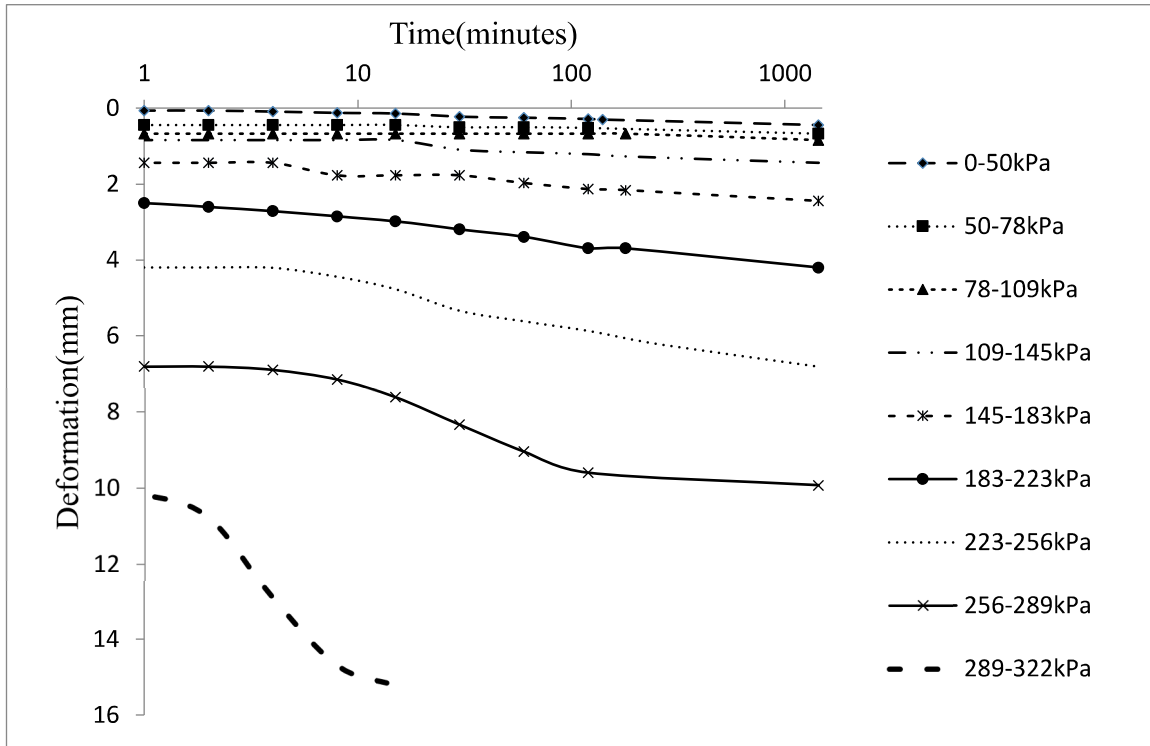


Fig. 6.3(b) : Axial deformation vs. time curve Inorganic clay-200kPa

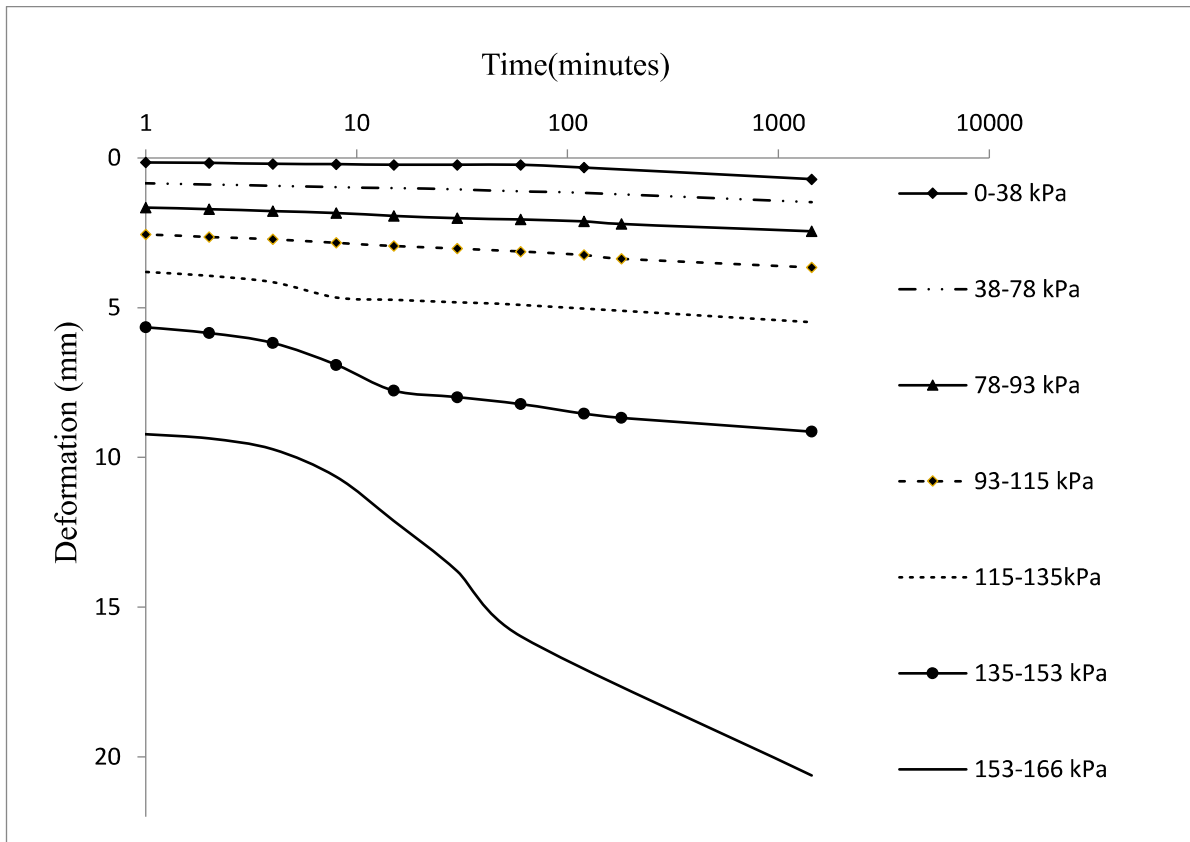


Fig. 6.3(c) : Axial deformation vs. time curve Organic clay-100kPa

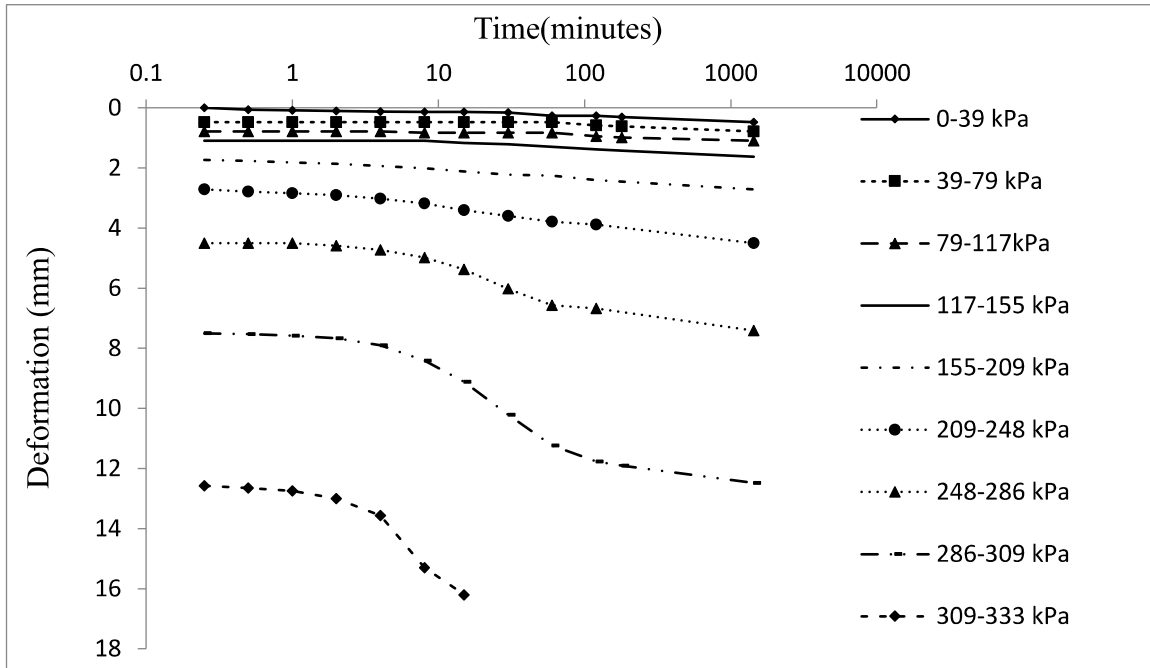


Fig. 6.3(d) : Axial deformation vs. time curve Organic clay-200kPa

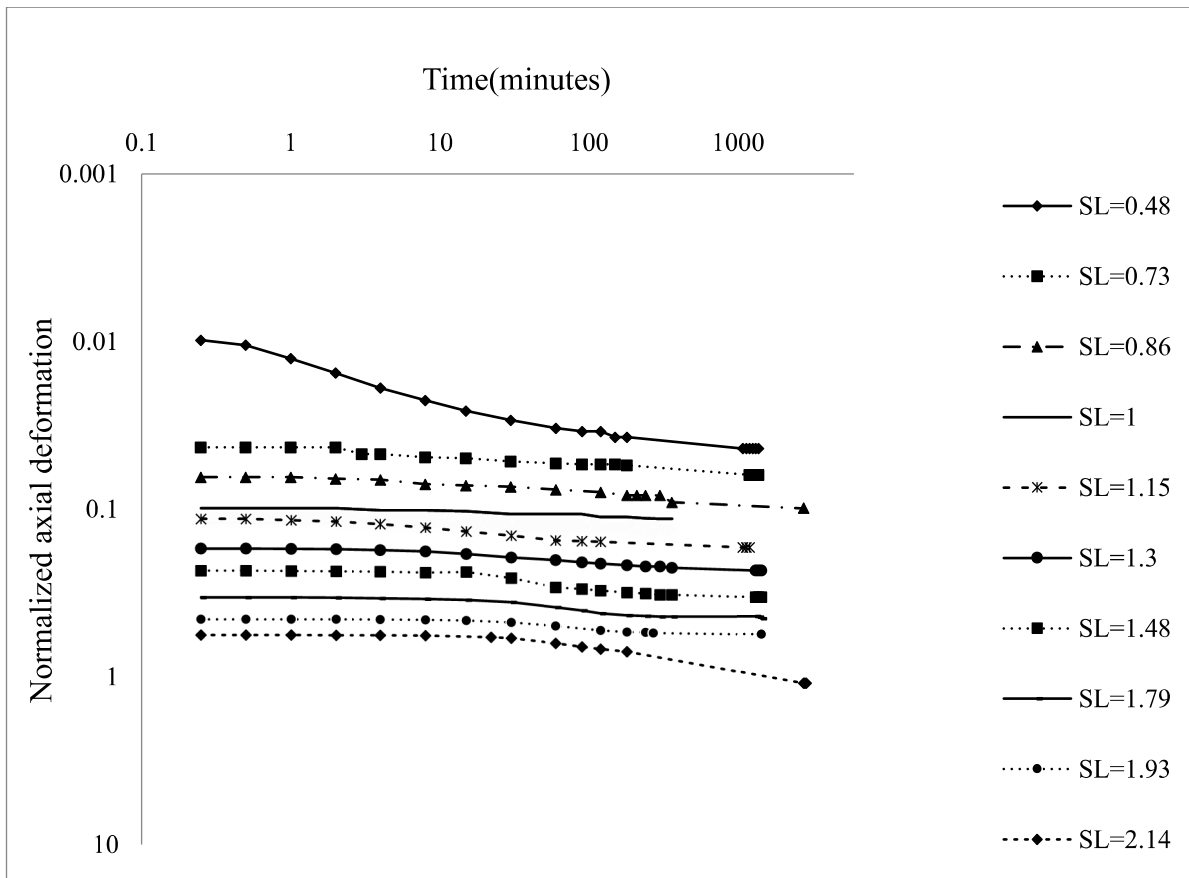


Fig. 6.4(a) : Normalized Axial deformation vs. time curve Inorganic clay-100kPa

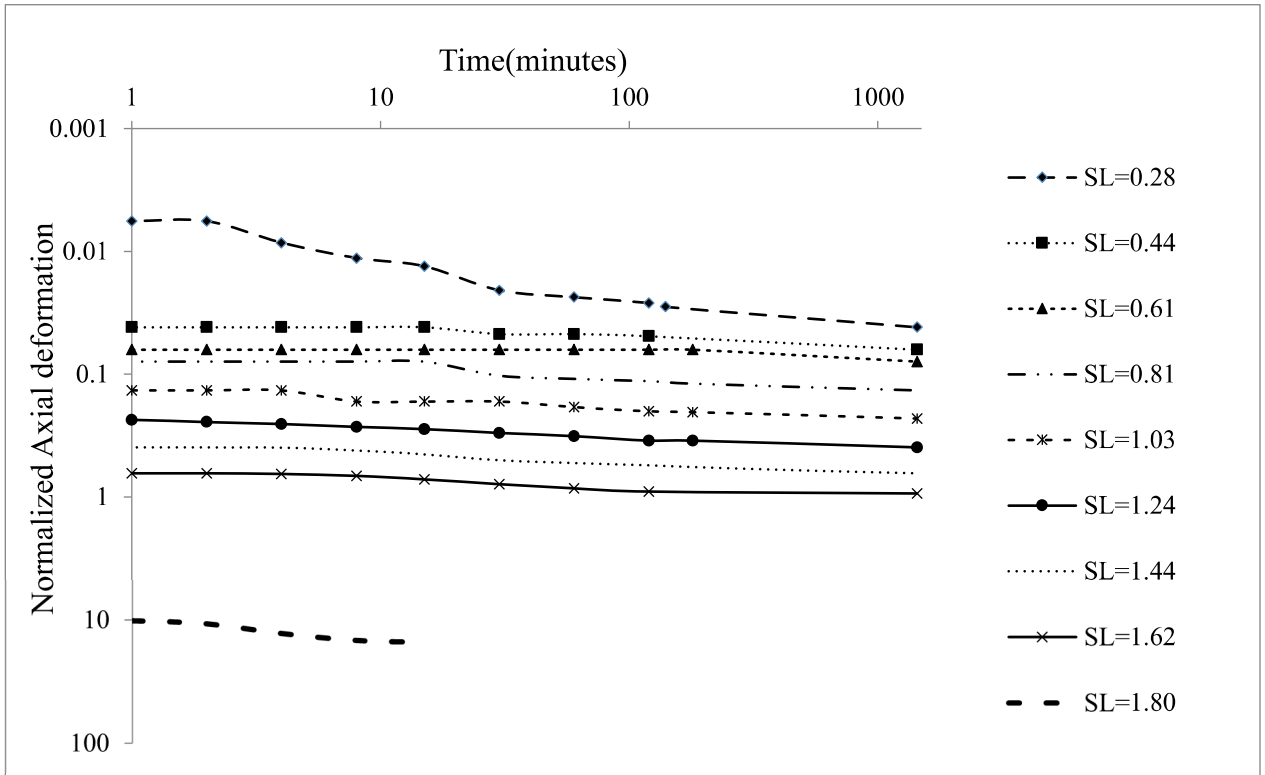


Fig. 6.4(b) : Normalized Axial deformation vs. time curve Inorganic clay-200kPa

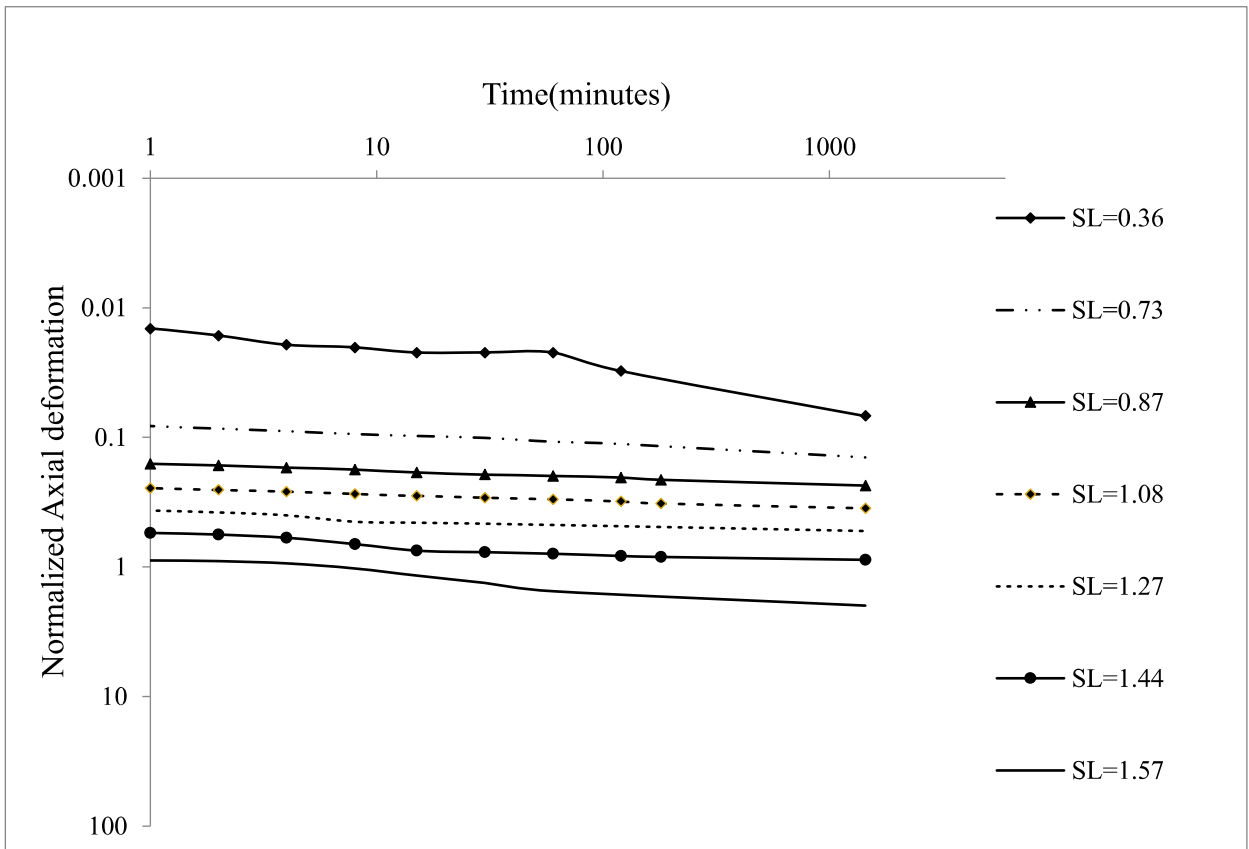


Fig. 6.4(c) : Normalized Axial deformation vs. time curve Organic clay-100kPa

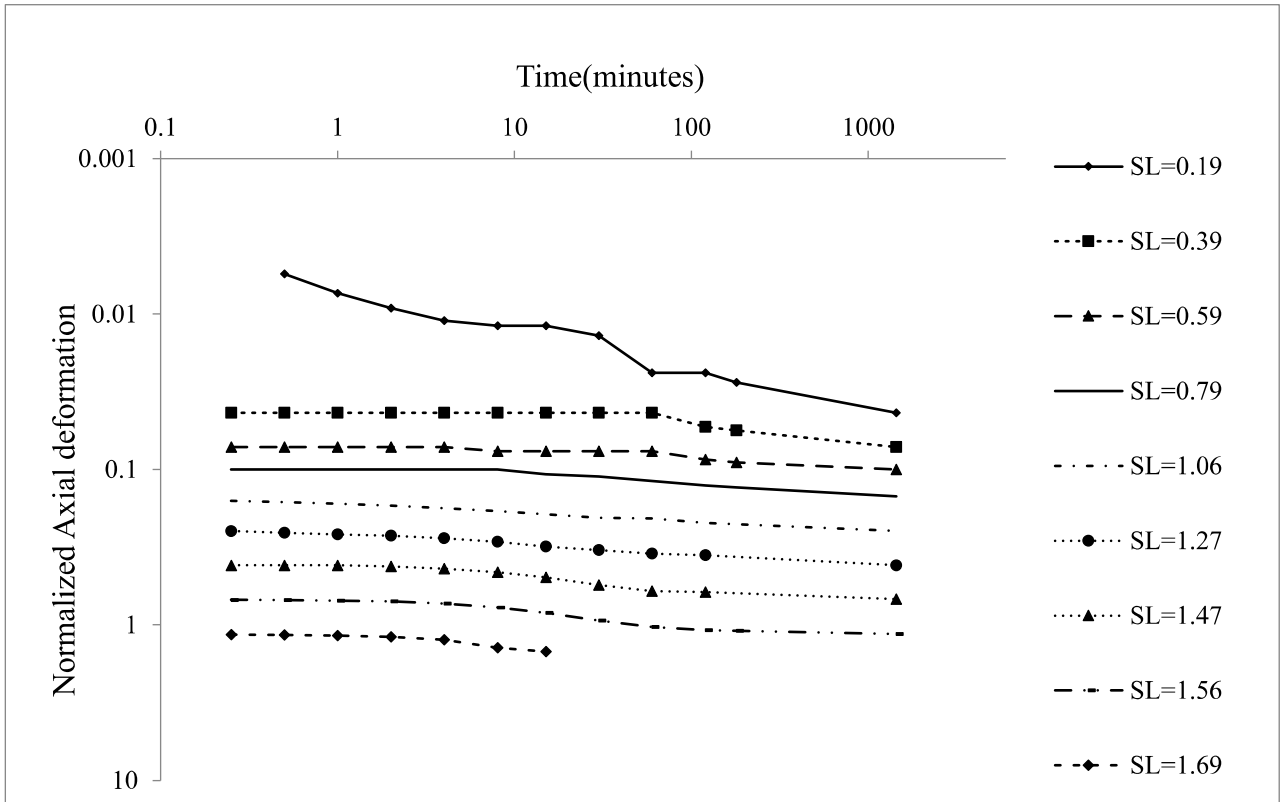


Fig. 6.4(d) : Normalized Axial deformation vs. time curve Organic clay-200kPa

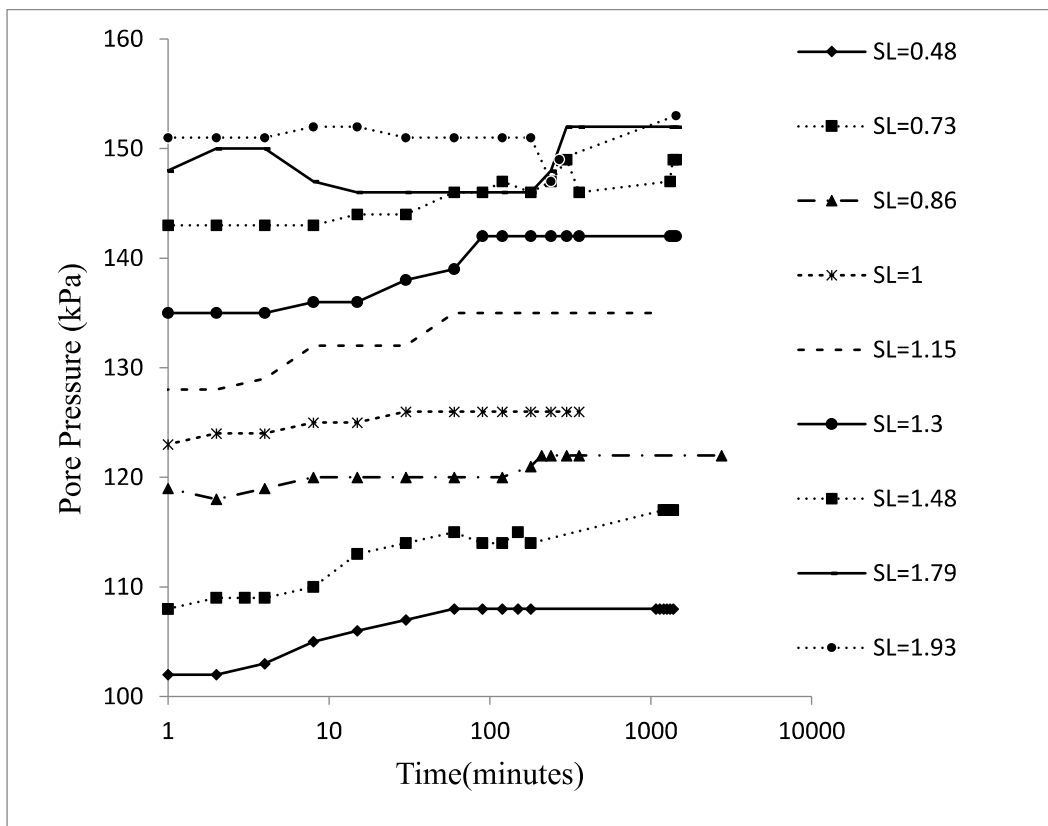


Fig. 6.5 (a) : Excess pore pressure vs. time curve Inorganic clay-100kPa

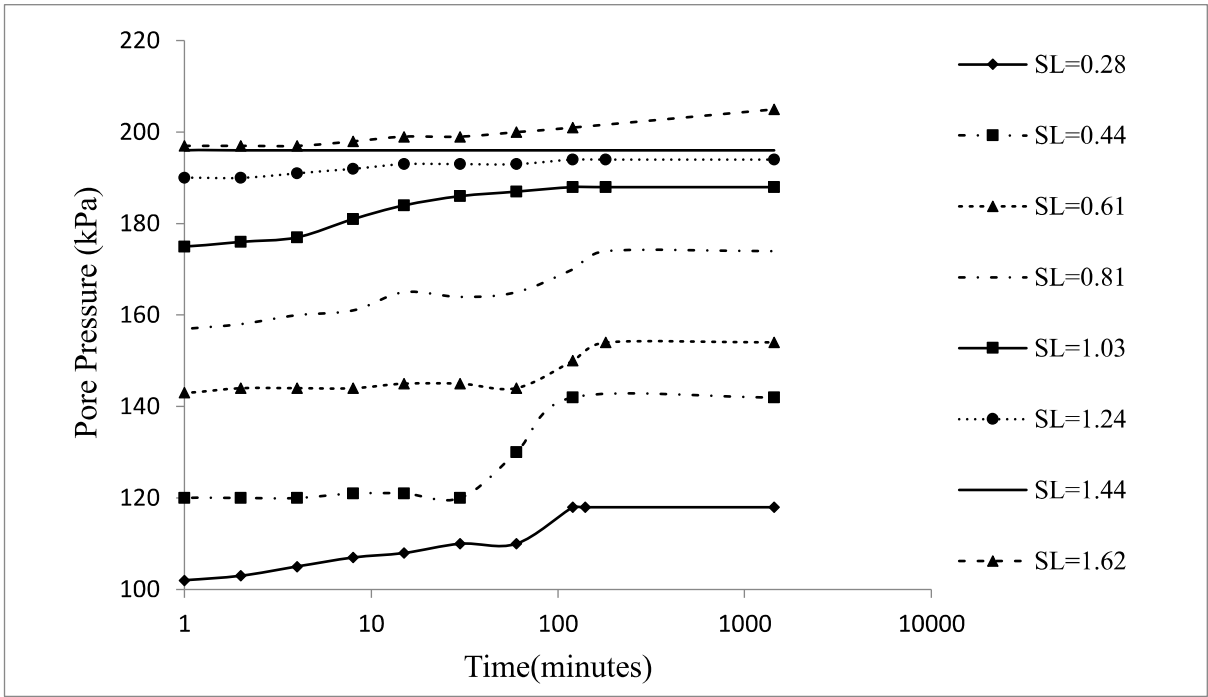


Fig. 6.5 (b): Excess pore pressure vs. time curve Inorganic clay-200kPa

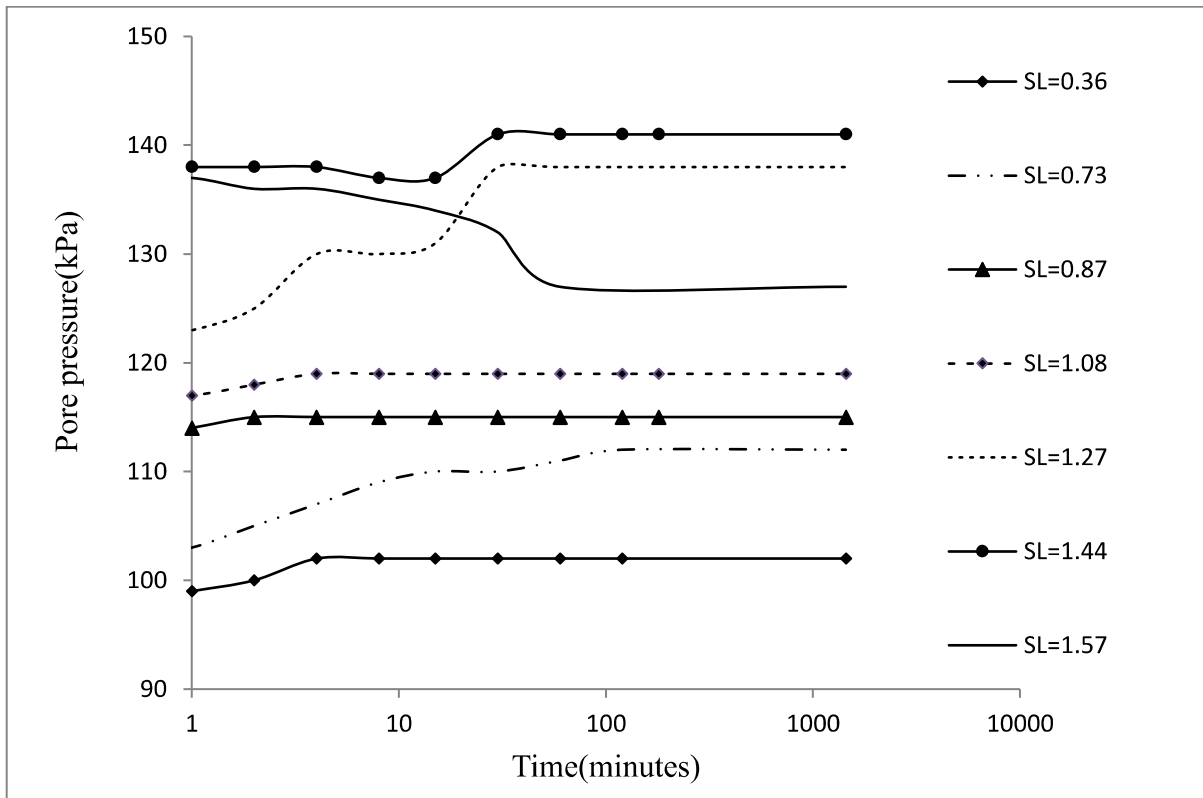


Fig. 6.5 (c) : Excess pore pressure vs. time curve Organic clay-100kPa

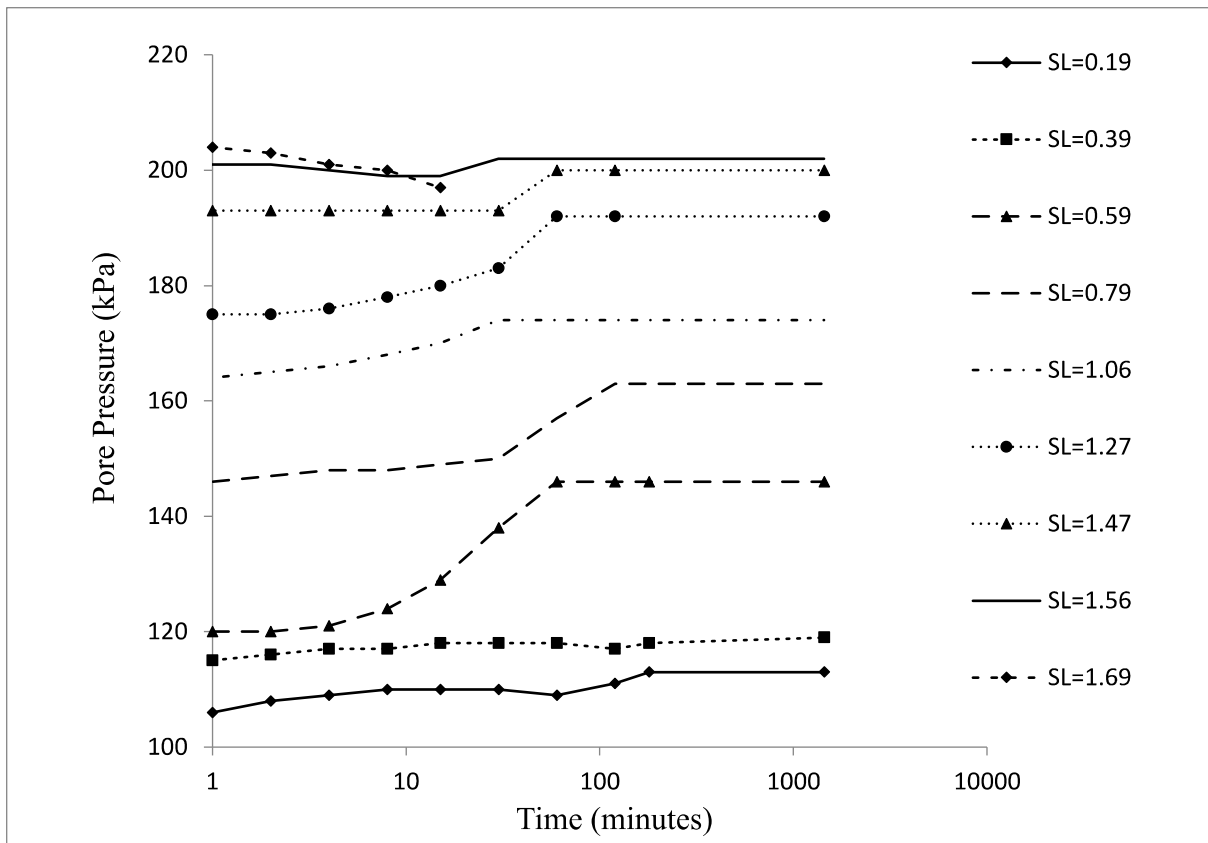


Fig. 6.5(d) : Excess pore pressure vs. time curve Organic clay-200kPa

Increase in axial strain, incremental undrained, creep and total, along with corresponding increase in pore water pressure and conventional pore water pressure parameter, A-value (denoted as A') for different stress level and normalized deviator stress, for both inorganic and organic soils are tabulated in Table 6.2 – 6.5. Creep strain was considered to develop after the excess pore pressure becomes more or less constant and the strain prior to this was categorized as the undrained strain which is to some extent in line with hypothesis 'A'.

Table 6.2 Incremental strain and Pore pressure parameters – Inorganic soil -100kPa

Deviator stress(kPa)	0-38	38-58	58-68	68-79	79-90	90-105	105-117	117-142	142-153
SL	0.48	0.73	0.86	1	1.15	1.33	1.48	1.80	1.93
$\sigma_d/p_o$	0.38	0.58	0.68	0.79	0.90	1.05	1.17	1.42	1.53
Incremental undrained axial strain (%)	0.16	0.205	0.46	0.62	0.74	0.71	1.69	1.88	2.11
Incremental creep axial strain (%)	0.134	0.15	0.2	0.22	0.29	0.47	0.22	0.27	9.91
Incremental total axial strain (%)	0.29	0.36	0.66	0.84	1.03	1.18	1.91	2.15	12.02
Cumulative axial strain (%)	0.29	0.65	1.31	2.14	3.17	4.35	6.27	8.42	20.44
Pore pressure	100-108	108-117	117-122	122-126	126-135	135-142	142-145	145-149	149-152
Time to reach constant pore pressure (post loading)	60	60	180	30	60	120	270	360	60
$A' [= \Delta u / \Delta(\sigma_1 - \sigma_3)]$	0.21	0.45	0.50	0.36	0.81	0.50	0.25	0.11	0.27
Average $A'$	0.21	0.29	0.32	0.33	0.39	0.40	0.38	0.35	0.34

Table 6.3 Incremental strain and Pore pressure parameters – Inorganic soil -200kPa

Deviator stress(kPa)	0-50	50-78	78-109	109- 145	145- 183	183- 223	223- 256	256- 289
SL	0.29	0.46	0.64	0.85	1.08	1.31	1.51	1.70
$\sigma_d/p_o$	0.25	0.39	0.55	0.725	0.915	1.11	1.28	1.45
Incremental undrained axial strain (%)	0.21	0.29	0.28	0.52	0.86	1.5	2.595	3.36
Incremental creep axial strain (%)	0	0	0.12	0.21	0.34	0.61	0.55	0.39
Incremental total axial strain (%)	0.21	0.29	0.40	0.73	1.2	2.11	3.145	3.75
Cumulative axial strain (%)	0.21	0.60	1	1.73	2.93	5.04	8.19	11.94
Pore pressure	100- 118	118- 142	142-154	154- 174	174- 188	188- 194	194- 198	198- 202
Time to reach constant pore pressure (post loading)	120	120	180	180	120	120	120	120
$A' [= \Delta u/\Delta(\sigma_1 - \sigma_3)]$	0.36	0.85	0.39	0.56	0.37	0.15	0.12	0.12
Average $A'$	0.36	0.53	0.50	0.51	0.48	0.42	0.38	0.35

Table 6.4 Incremental strain and Pore pressure parameters – organic soil -100kPa

Deviator stress(kPa)	0-38	38-78	78-93	93-115	115-135	135-153
SL	0.35	0.73	0.87	1.08	1.27	1.44
$\sigma_d/p_o$	0.38	0.78	0.93	1.15	1.35	1.53
Incremental undrained axial strain (%)	0.24	0.26	0.77	0.35	1.53	3.29
Incremental creep axial strain (%)	0.31	0.67	0.97	1.24	0.88	1.51
Incremental total axial strain (%)	0.55	0.97	1.74	1.59	2.41	4.8
Cumulative axial strain (%)	0.55	1.48	3.22	4.81	7.22	12.02
Pore pressure	100-105	105-112	112-115	115-119	119-138	138-141
Time to reach constant pore pressure (post loading)	4	120	2	4	30	30
$A' [= \Delta u/\Delta(\sigma_1 - \sigma_3)]$	0.13	0.2	0.2	0.18	0.95	0.16
Average $A'$	0.13	0.15	0.16	0.17	0.28	0.26

Table 6.5 Incremental strain and Pore pressure parameters – Organic soil -200kPa

Deviator stress(kPa)	0-39	39-79	79-117	117-155	155-209	209-248	248-286	286-309
SL	0.19	0.39	0.59	0.79	1.04	1.27	1.56	1.56
$\sigma_d/p_o$	0.19	0.39	0.59	0.78	1.05	1.24	1.43	1.54
Incremental undrained axial strain (%)	0.25	0.26	0.21	0.38	0.79	1.4	2.75	3.67
Incremental creep axial strain (%)	0.15	0.26	0.30	0.32	0.65	0.95	1.11	3
Incremental total axial strain (%)	0.40	0.52	0.51	0.7	1.44	2.35	3.86	6.67
Cumulative axial strain (%)	0.40	0.92	1.43	2.13	3.57	5.92	9.78	16.45
Pore pressure	101-113	113-119	119-146	146-163	163-174	174-192	192-200	200-202
Time to reach constant pore pressure (post loading)	180	15	60	120	30	60	60	30
$A' [= \Delta u/\Delta(\sigma_1 - \sigma_3)]$	0.30	0.15	0.71	0.45	0.2	0.46	0.16	0.09
Average $A'$	0.30	0.23	0.38	0.40	0.35	0.37	0.70	0.33

The normalized deviator stress versus cumulative axial strain as well as its undrained component for both inorganic and organic clays has been presented in Figs. 6.6(a)-(b) respectively. From these figures it may be seen that difference of total and its undrained component increases with the increase in  $\sigma_d/p_o$  for both the soils with higher difference for organic soil indicating its higher creep behaviour.

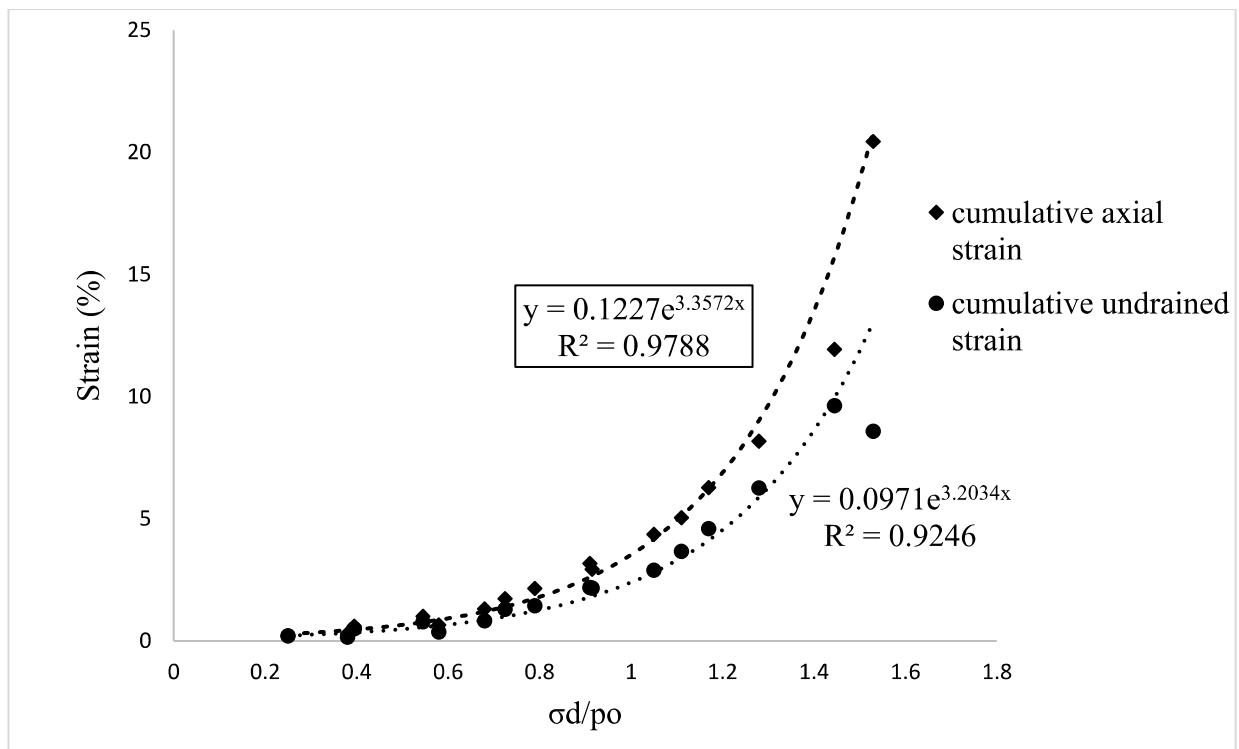


Fig. 6.6(a): Normalized deviator stress versus axial strain Inorganic clay

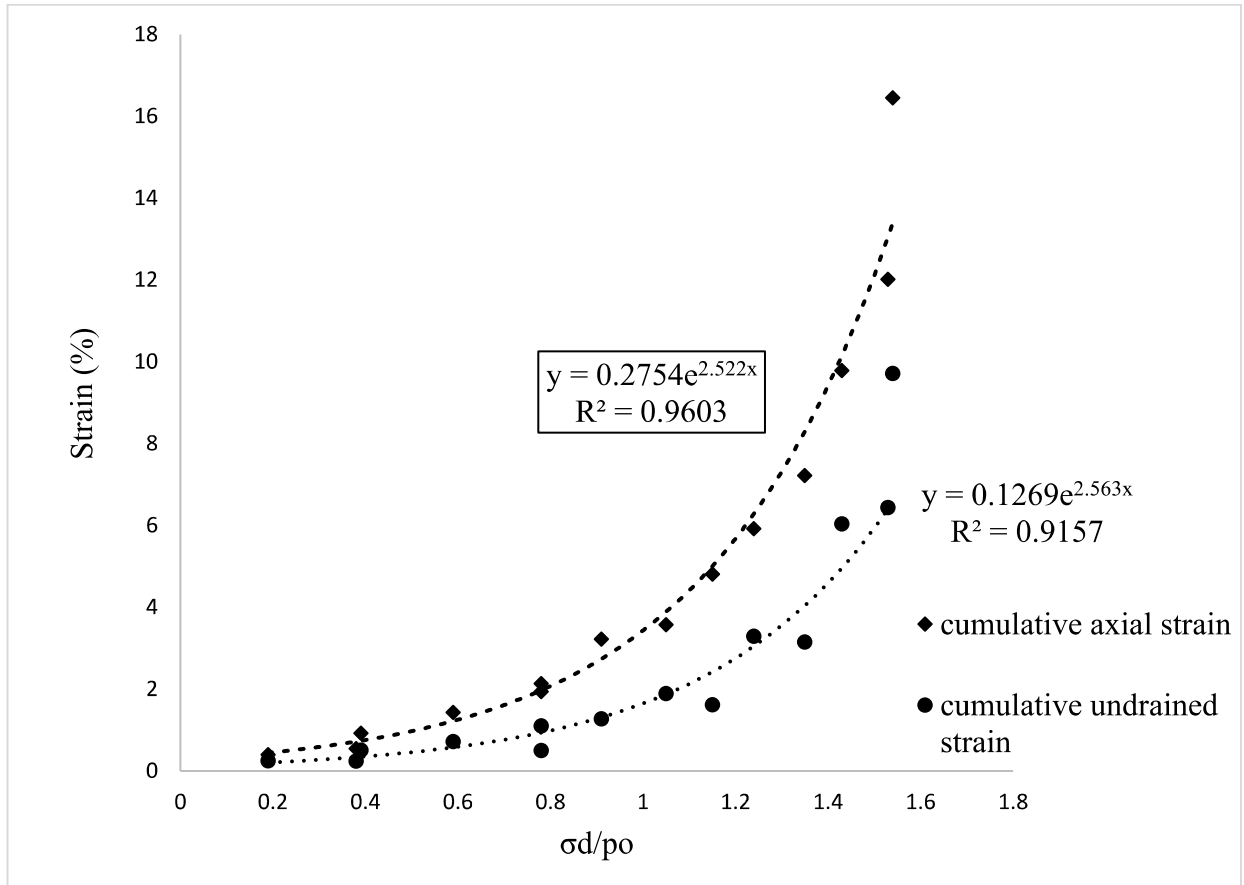


Fig. 6.6 (b): Normalized deviator stress versus axial strain Organic clay

Further, comparing the increase in pore pressure and corresponding incremental / average ‘A’ - value calculated by dividing the incremental / cumulative increase in pore water pressure by corresponding deviator stress given in Tables 6.2 -6.5 with that presented in Table 6.1 for conventional CU tests it may be noted that increase in pore water pressure is comparatively less in case of creep triaxial tests for both inorganic and organic soils. In case of creep tests the average ‘A’-value is 0.33 and 0.48 for inorganic soils and 0.17 and 0.35 for organic soils at stress level nearly 1.0 for effective confining pressure of 100kPa and 200kPa respectively. These are much less than the corresponding values of 0.82 and 0.67 for inorganic soils and 0.52 and 0.66 for organic soils. This may be due to growth in bonding of the soils under sustained loading in spite of their creep deformation.

## 6.2 Creep parametric study for undrained tests

Singh and Mitchell (1969) proposed the following three parameter equation for prediction of strain rate of soil during creep under undrained condition at any time  $t$  after application of load.

$$\dot{\varepsilon}_a = Ae^{(\alpha D)} \left(\frac{t_1}{t}\right)^m \quad (6.1)$$

Where,  $A$ ,  $\alpha$  and  $m$  are the parameters which are to be determined to assess the creep characteristics of a soil. The parameter  $\alpha$  is the value of slope of the mid-range linear plot between logarithmic strain rate and deviator stress, all points corresponding to same time after application of load. The term  $t_1$  represents a unit/reference time and  $D$  is the deviator stress at any time  $t$ . The reference time for this study was considered to be 1 minute. The rate of decrease in strain rate with time is established using the parameter  $m$ ; the strain rate dependency on stress level is given by the parameter  $\alpha$ ;  $A$  is the extrapolated strain rate at a reference time for a stress level of zero. The parameter  $m$  is the slope of a logarithmic strain rate versus logarithmic time straight line. Without considering initial strains the strain at any time can be calculated by integrating the equation (1) thus strain  $\varepsilon$  is given by the following equation:

$$\dot{\varepsilon}_a t = A t_1^m e^{(\alpha D_r)} t^{(1-m)} \quad (6.2)$$

Where,  $D_r$  is the stress ratio which is the ratio of deviator stress to mean normal stress. A sample plot to calculate the parameters is shown in Fig. 6.8.

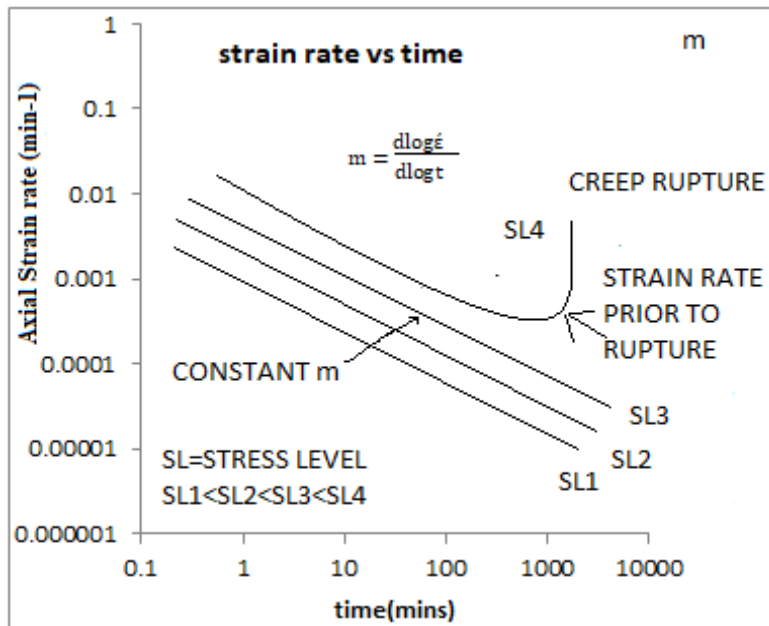


Fig. 6.7: Singh and Mitchell (1969) creep parameters (a) Creep parameter m

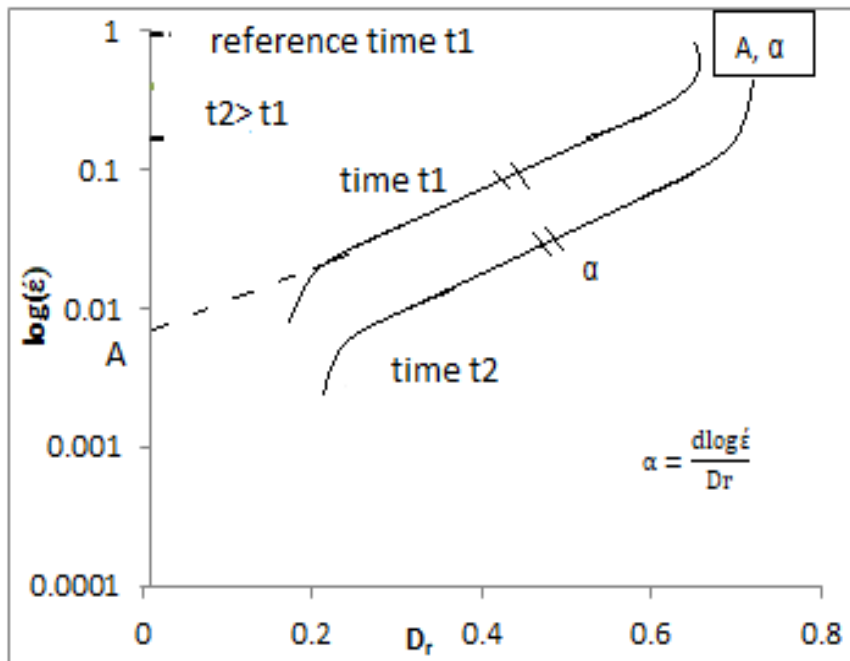


Fig. 6.8: Singh and Mitchell (1969) creep parameters (b) parameter A and  $\alpha$

Parameter A is the value of strain rate at reference time  $t_1$  and  $D=0$  and by extrapolating the values from Fig 6.8 the average A value for inorganic clay of Kolkata deposit is estimated as 0.00026/min and for organic Kolkata clays it is 0.00052/min. Lacerda and Houston (1973)

conducted a series of undrained stress relaxation tests and concluded that the parameter A is a function of clay type which is a constant value for a particular type of soil depending upon its index properties and is calculated using the equations (3) – (4) given below.

$$h_o = 13.2 A^{1.1} \quad (6.3)$$

$$h_o = 10^{(0.044Ip - 3.82)} \quad (6.4)$$

Where,  $h_o$  is a parameter dependent on the plasticity index of the soil. For inorganic and organic Kolkata soil the values of A are 0.00029/min and 0.00072/min which validates the test results obtained. The values of  $\alpha$  obtained from Fig. 6.9(a) – (d) are presented in table 6.6.

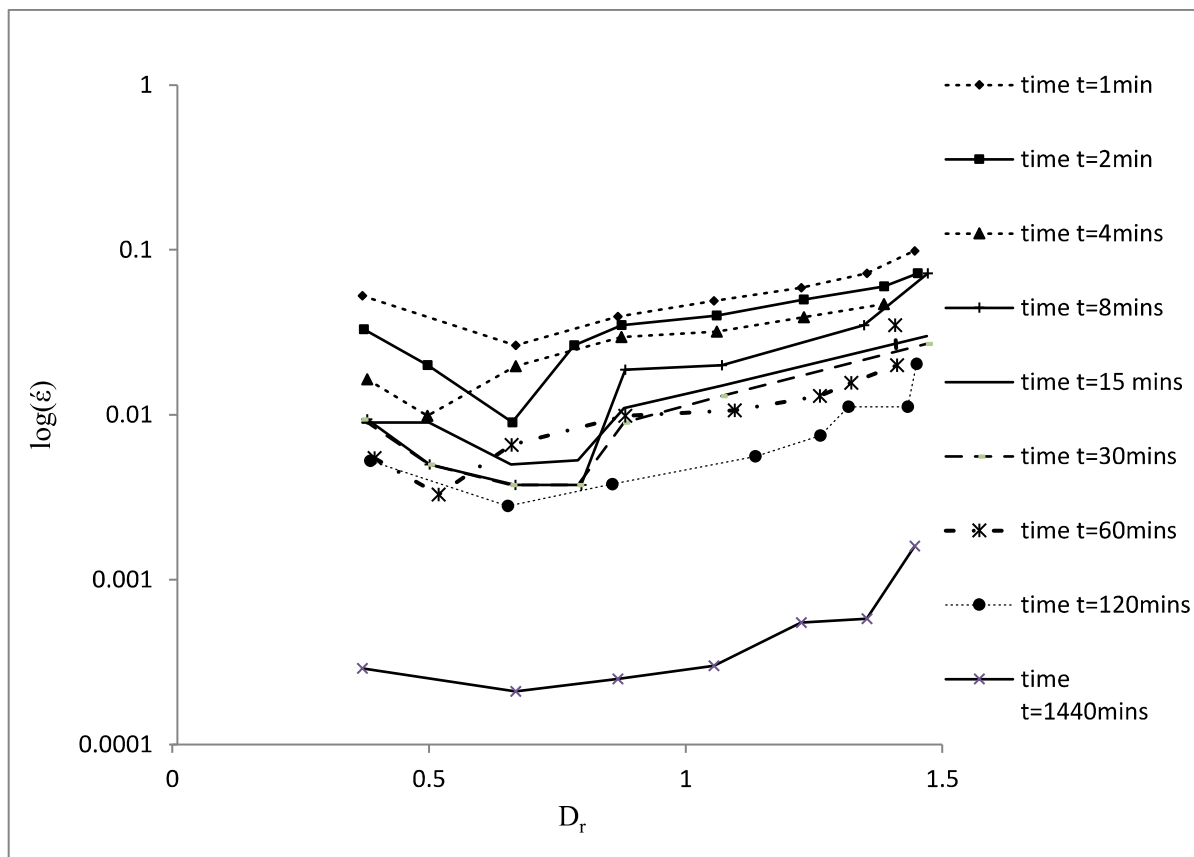


Fig. 6.9 a: Axial strain rate % vs stress ratio Inorganic clay-100kPa

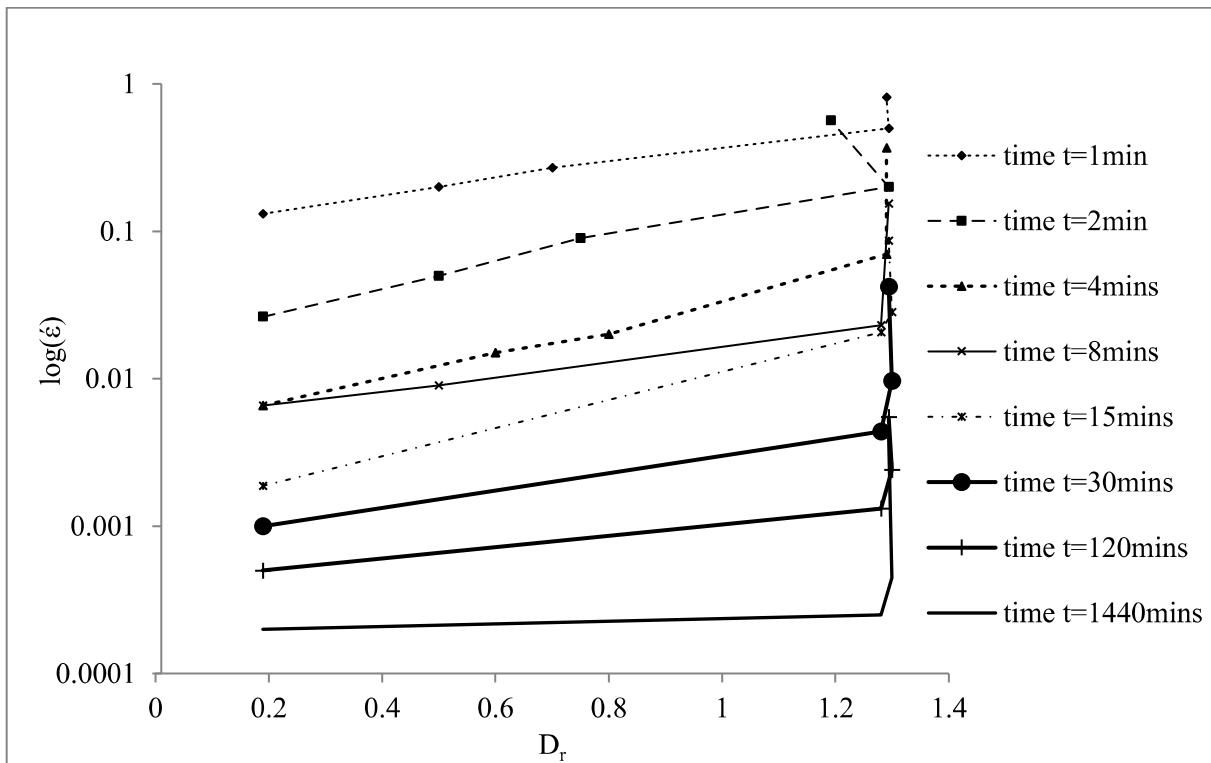


Fig. 6.9 b: Axial strain rate % vs stress ratio Inorganic clay-200kPa

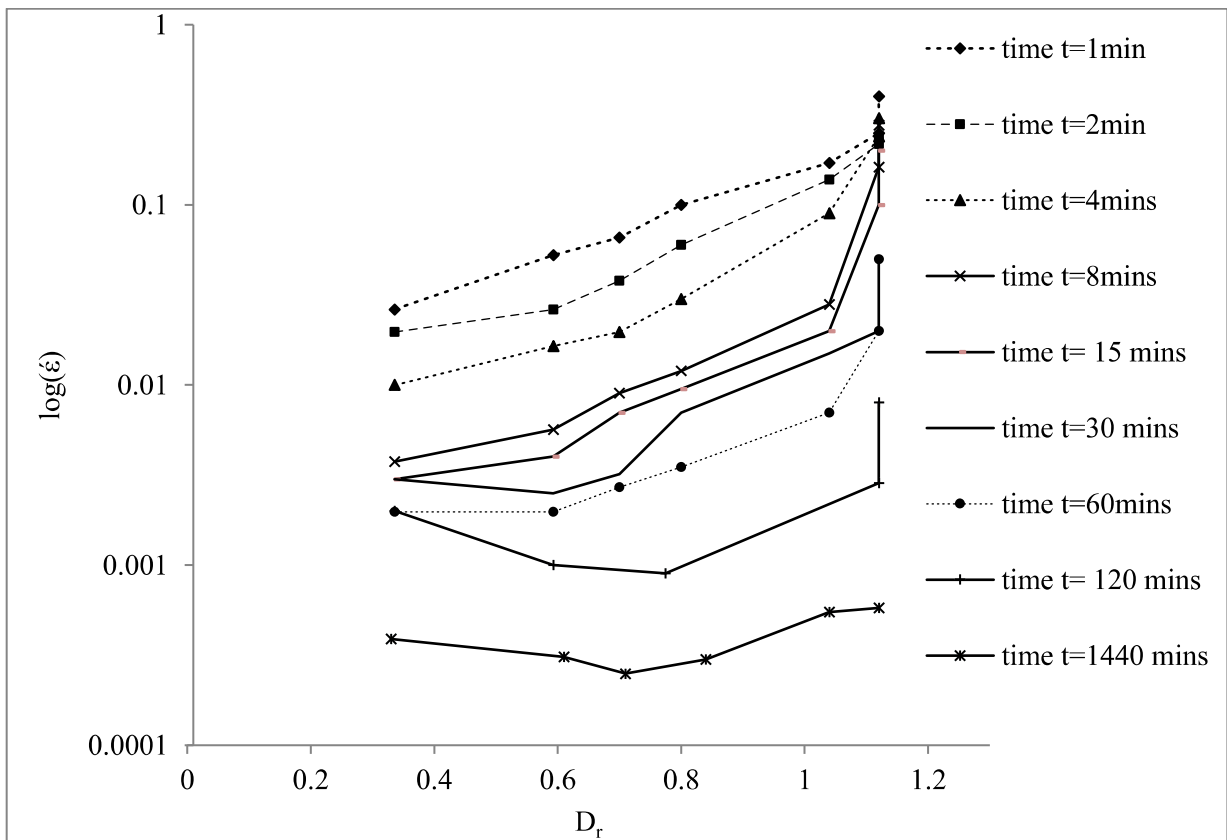


Fig. 6.9 c: Axial strain rate % vs stress ratio Organic clay-100kPa

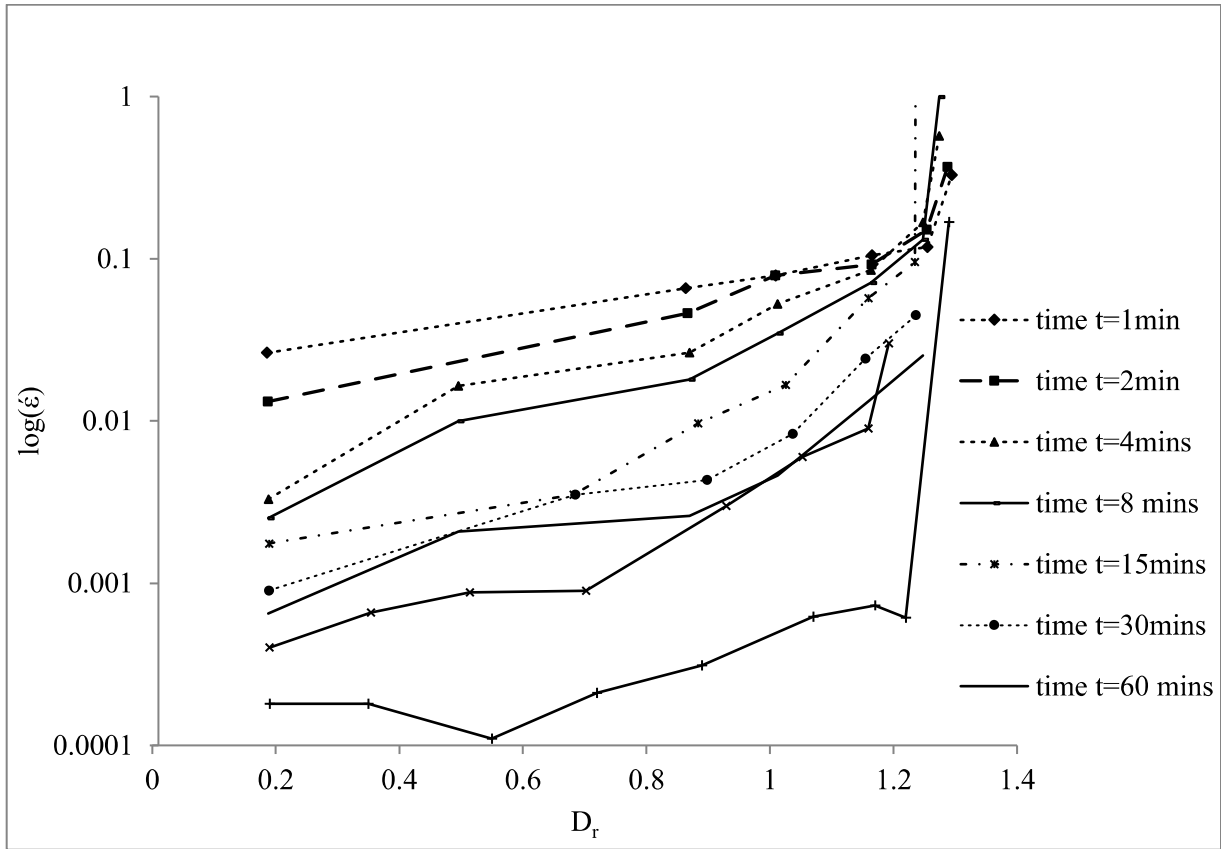


Fig. 6.9 d: Axial strain rate % vs stress ratio Organic clay-200kPa

Table 6.6 The values of  $\alpha$

Time (mins)	1	2	4	8	15	30	60	120	1440
Soil type									
inorganic clay -100kPa	0.066	0.027	0.050	0.032	0.022	0.022	0.014	0.015	0.00122
inorganic clay -200kPa	0.12	0.07	0.048	0.015	0.017	0.015	0.01375	0.0125	0.000243
organic clay - 100kPa	0.32	0.29	0.48	0.055	0.043	0.03	0.06	0.007	0.00125
organic clay - 200kPa	0.30	0.23	0.20	0.18	0.17	0.077	0.03	0.005	0.00117

For visualisation of creep behaviour in triaxial conditions, the creep triaxial data was presented in  $\log(\dot{\epsilon}_a)$ - $\log(t)$  plots. The consequences of varying  $m$  have been presented in Fig. 6.7a by taking  $\dot{\epsilon}$  as the axial strain rate. Singh and Mitchell (1969) conducted one of first studies of creep under drained and undrained conditions and reported that the values of  $m$  varied between 0.75 and a value slightly greater than 1. They suggested that the value of ' $m$ ' is a soil specific parameter independent of the deviator stress level i.e. the creep lines for different deviator stress levels have the same slope in the  $\log(\dot{\epsilon}_a)$ - $\log(t)$  plot. For soils susceptible to creep the value of  $m < 1$ . Lower the value of  $m$ , greater is the creep potential as the parameter  $\dot{\epsilon}_a t$  in equation (2) is an increasing function of time for  $m < 1$ , a constant function for  $m = 1$  and decreasing function for  $m > 1$  as elaborated using equation (5).

$$\dot{\epsilon}_a t = \frac{d\epsilon}{d\log_c(t)} = \frac{d\epsilon}{2.73 d\log_{10}(t)} \quad (6.5)$$

It may be seen that  $\dot{\epsilon}_a t$  is a measure of the first derivative of strain with respect to logarithm of time. The lumped parameter  $\dot{\epsilon}_a t$  may be considered as an instantaneous creep coefficient and is a measure of the increase of strain per log cycle of time Singh and Mitchell (1969).

The undrained creep results as obtained in the present study for inorganic and organic soil are plotted in the form of logarithmic strain rate vs. logarithmic time in Fig. 6.10(a) – (d). From the plots, best fit lines are drawn for different load increments and from which the parameter  $m$  is evaluated. These are given in Table 6.7 – 6.10. From these Tables and Fig. 6.10 it may be seen that for inorganic clay magnitude of ' $m$ ' is higher for higher confining pressure while for organic clays it is independent of confining pressure till a stress level of about 0.5 as the  $m$  values obtained for corresponding stress levels under both confining pressure were similar. Further, for both the soils ' $m$ ' decreases with the increase in stress level which indicates that the magnitude of deformation is higher at higher stress level. This is contradictory to the observations reported by Singh and Mitchell (1969) where  $m$  was considered independent of

stress level. For normally consolidated clays Tian et al. (1994) and Feda (1992) reported  $m$  values increases with increases of stress levels. The analyses conducted by Tian et al. (1994) were based on the drained triaxial creep tests on undisturbed marine clays of Gulf of Mexico and the North-Central Pacific. Den Haan and Edil (1994) studied the parameter  $m$  for low permeability soils and reported that for triaxial undrained and drained tests performed on peat and clays, it is common to find  $m$  in the range of 0.7-0.9 and is independent of stress level. The observations in the present study are similar to those reported by Zhu et al. (1999) where the value of  $m$  was reduced with increase in stress levels ( $m$  decreased from 0.91 to 0.57 for stress 14 kPa to 121 kPa) .

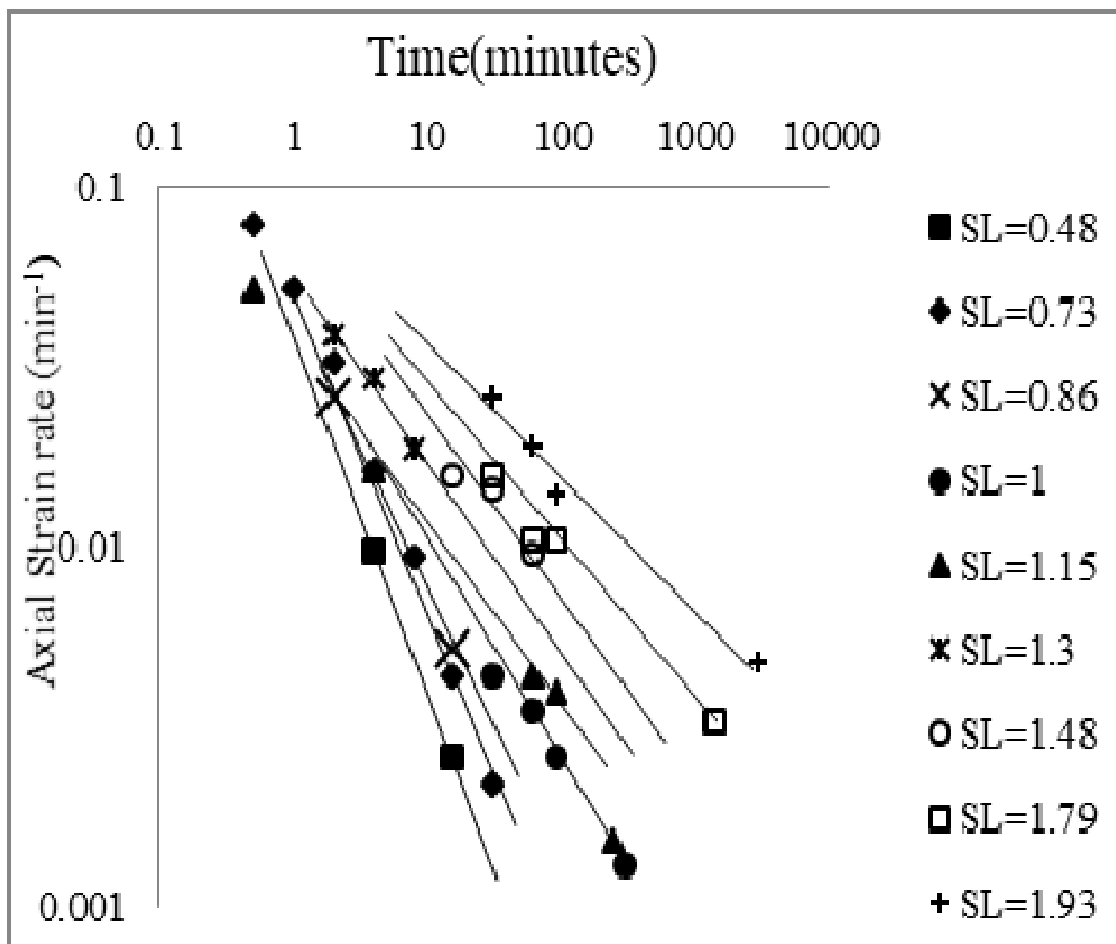


Fig. 6.10a: Strain rate vs. time from undrained creep tests Inorganic clay-100kPa

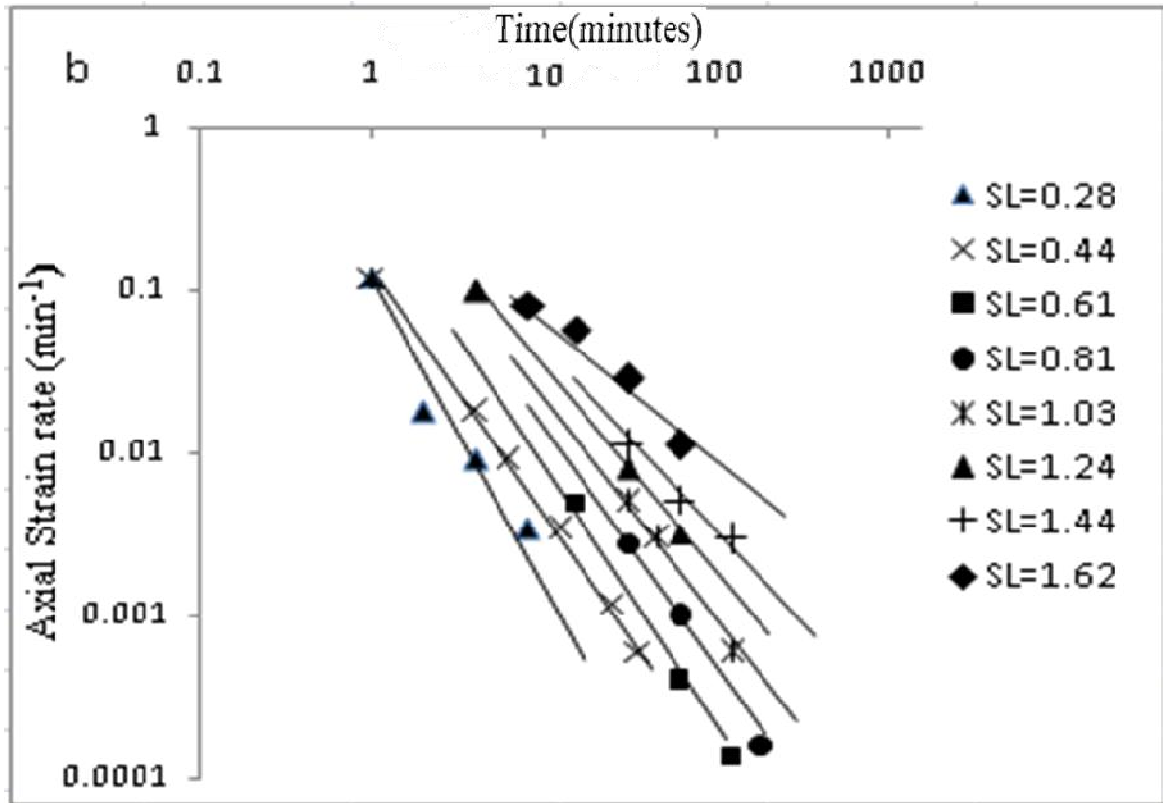


Fig. 6.10 b: Strain rate vs. time from undrained creep tests Inorganic clay-200kPa

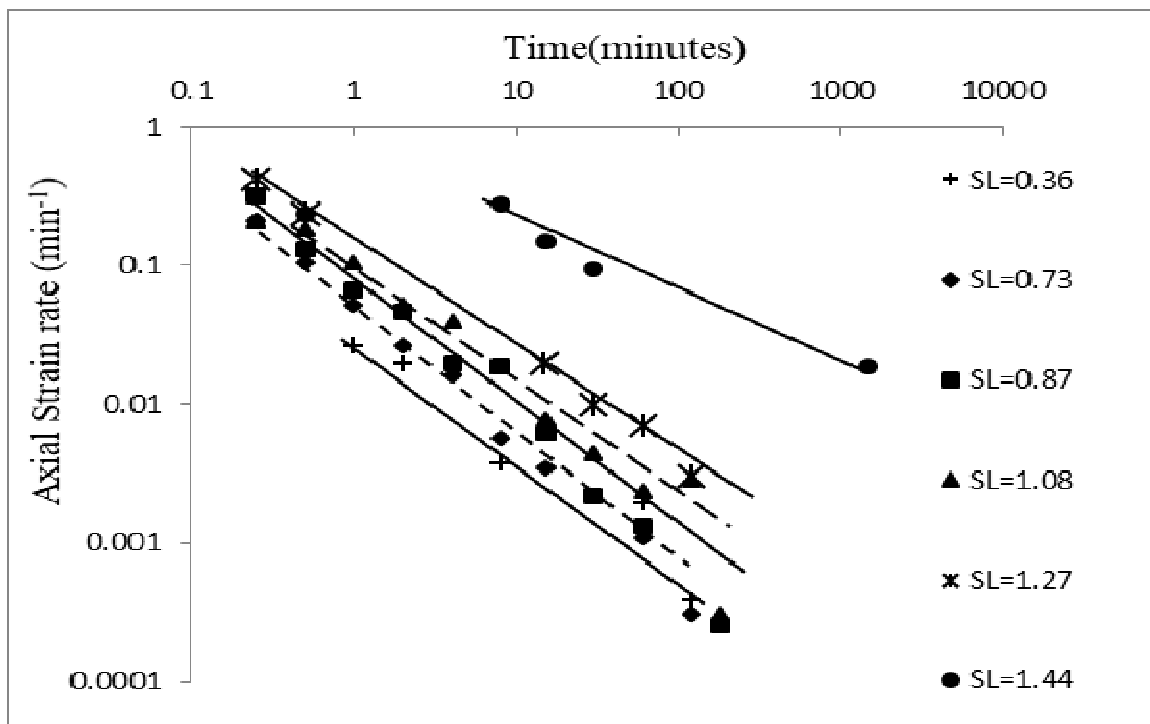


Fig. 6.10 c: Strain rate vs. time from undrained creep tests Organic clay-100kPa

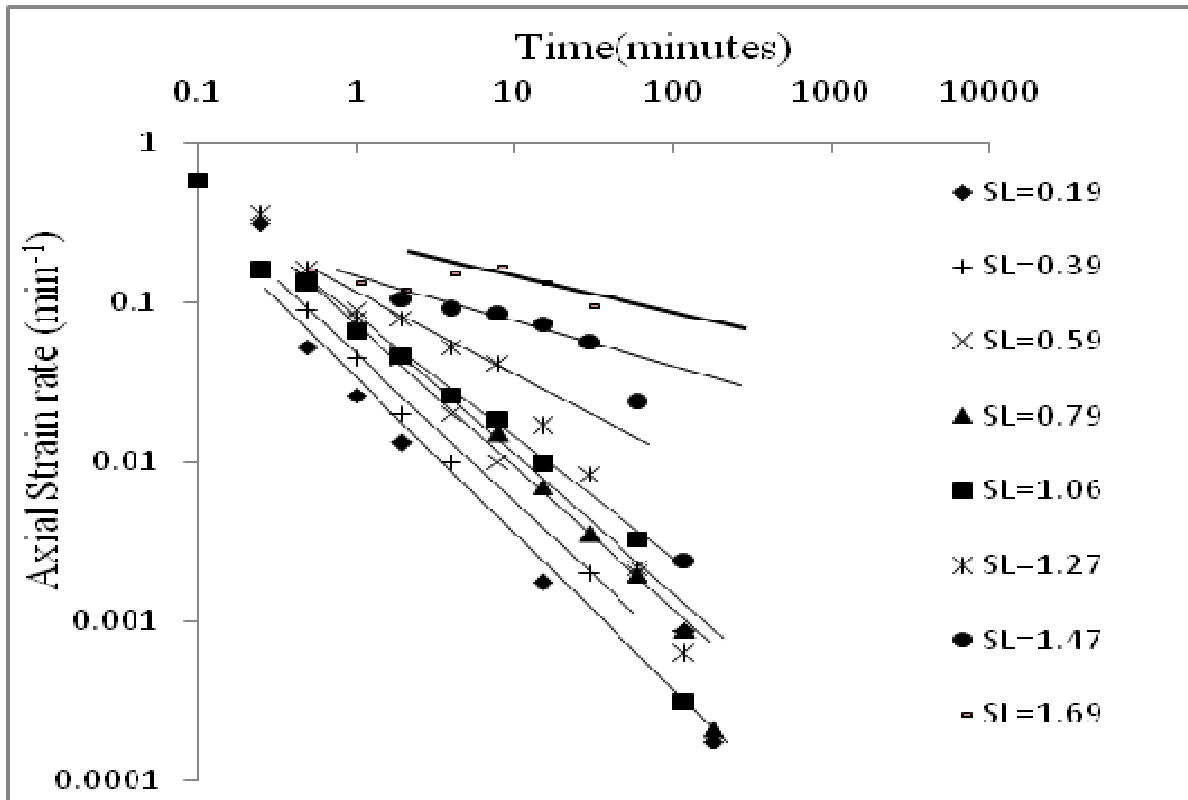


Fig. 6.10 d: Strain rate vs. time from undrained creep tests Organic clay-200kPa

Table 6.7 Creep potential parameters – Inorganic soil -100kPa

Deviator stress(kPa)	38	58	68	79	90	105	117	142	153
SL	0.48	0.73	0.86	1.00	1.15	1.3	1.48	1.79	1.96
$\sigma_d/p_o$	0.38	0.58	0.68	0.79	0.90	1.05	1.17	1.42	1.53
m	1.19	0.84	0.72	0.72	0.68	0.68	0.51	0.36	0.31

Table 6.8 Creep potential parameters – Inorganic soil -200kPa

Deviator stress(kPa)	50	78	109	145	183	223	256	289
SL	0.28	0.44	0.61	0.81	1.03	1.24	1.44	1.62
$\sigma_d/p_o$	0.25	0.39	0.55	0.73	0.92	1.12	1.28	1.45
m	1.86	1.55	1.5	1.485	1.32	1.15	1.05	1.02

Table 6.9 Creep potential parameters – organic soil -100kPa

Deviator stress(kPa)	38	78	93	115	135	153
SL	0.35	0.73	0.87	1.08	1.27	1.44
$\sigma_d/p_o$	0.38	0.78	0.93	1.15	1.35	1.53
m	0.95	0.93	0.77	0.77	0.72	0.62

Table 6.10 Creep potential parameters – Organic soil -200kPa

Deviator stress(kPa)	39	79	117	155	209	248	289	309
SL	0.19	0.39	0.59	0.79	1.06	1.27	1.47	1.56
$\sigma_d/p_o$	0.19	0.39	0.59	0.78	1.04	1.26	1.45	1.545
m	0.99	0.91	0.80	0.79	0.78	0.47	0.30	0.28

The variation of parameter m with stress level is plotted in Fig. 6.11 a and 6.11b for inorganic and organic clays respectively.

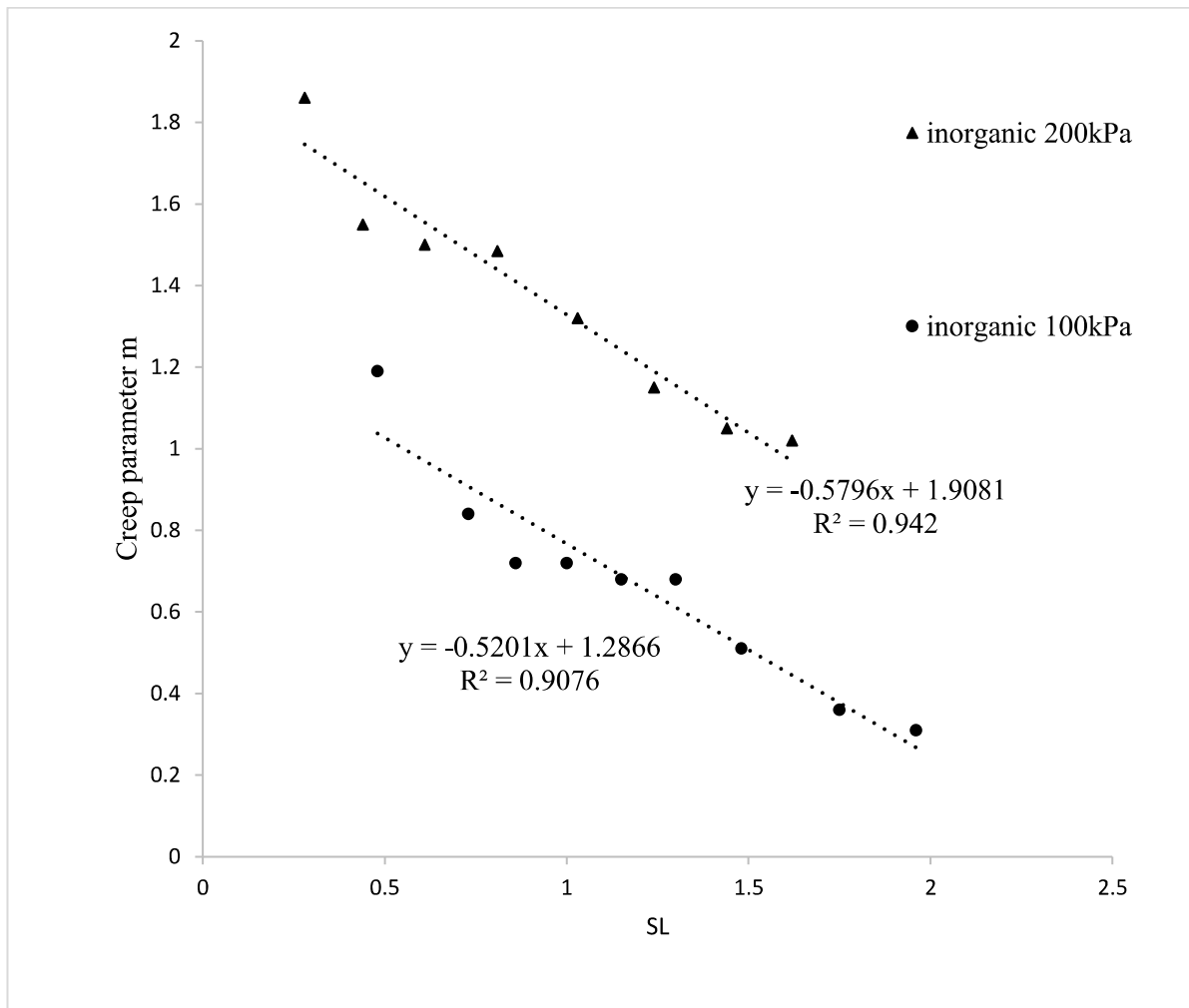


Fig 6.11 a : Variation of parameter m with stress level for Inorganic clays

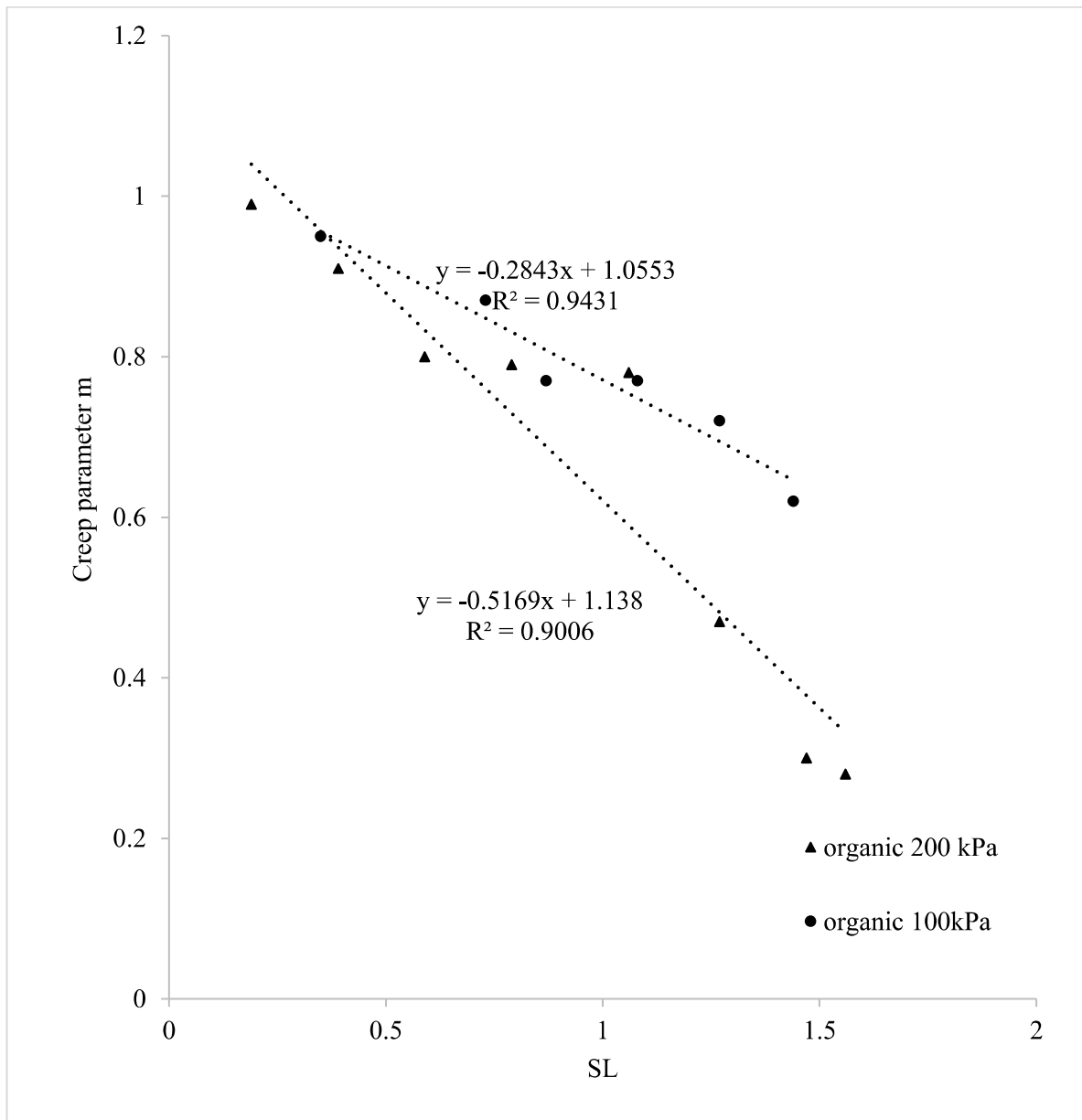


Fig 6.11 b : Variation of parameter m with stress level for Organic clays

In case of organic soil under undrained conditions the creep susceptibility is high as the value of 'm' is less than 1 even at low stress levels, i.e., at working load corresponding to stress level in the order of 0.40 (equivalent to a factor of safety =  $1/0.40 = 2.5$ ). However, in case of inorganic soil it falls below 1 at higher stress levels (greater than 0.58 which is corresponding to a factor of safety =  $1/0.58 = 1.72$ ) for confining pressure of 100kPa. However, at higher confining pressure, which corresponds to greater depths of soil, the creep susceptibility is

negligible. Usually a factor of safety of 1.1 to 1.5 is adopted for slopes and embankments whereas in case of buildings the factor of safety adopted is 2 to 2.5, for railway and roadway bridges foundation the factor of safety adopted is higher (Li et al. 2017). Thus cases of creep reported, as highlighted earlier, are mostly experienced for structures such as embankments and slopes where factor of safety is relatively less. Here, for Normal Kolkata deposit the long term deformation behaviour is undoubtedly affected by the presence of organic matter. For organic Kolkata clay not only slopes and embankments having less factor of safety (in the range of 1.5) but also buildings with factor of safety near about 2.5 may undergo creep settlement. The value of ‘m’ for both inorganic and organic soils corresponding to different magnitude of factor of safety is summarised in Table 6.11.

Table 6.11 ‘m’ values corresponding to F.O.S. adopted

Type of soil	Factor of safety	m	Remarks
Inorganic Kolkata clay	> 1.5	>1	No effect of creep
Organic Kolkata clay	≤ 2.5	<1	Susceptible to creep, secondary settlement should be considered during design of foundation.

\*Here stress levels 1 corresponds to the load carrying capacity of soil from conventional monotonic triaxial tests. Therefore a SL of 0.40 and 0.6 would correspond to factor of safety (F.O.S) of 2.5 and 1.5 respectively.

### 6.3 Test results and discussion – drained analysis

The axial deformation has been normalised in order to represent the accumulation of deformation under the action of applied loads corresponding to specific stress levels. The stress

level is defined as the ratio of load applied axially to the ultimate load obtained from conventional monotonic triaxial tests under similar conditions as given in Fig.6.12 and the volume change response has been presented in Fig 6.13. The results have been summarised in Table 6.12. Further,  $\sigma_d/p_o$  is the ratio of deviator stress to the effective confining pressure.

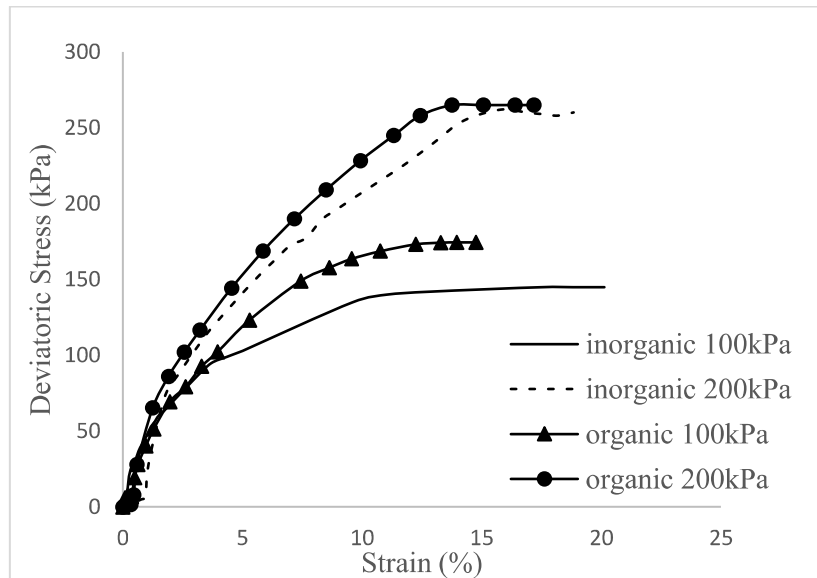


Fig. 6.12: Stress-strain response from conventional triaxial drained tests

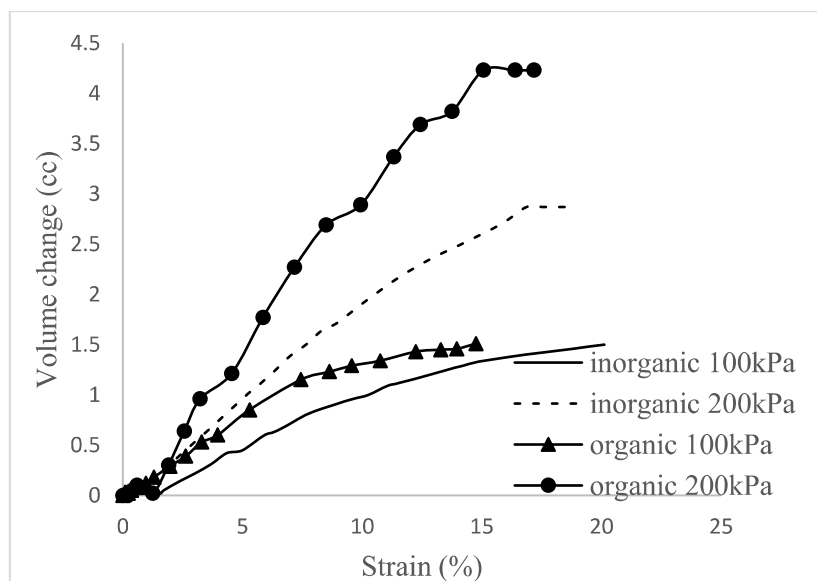


Fig. 6.13 : Volume changes recorded from conventional triaxial drained tests.

The results have been summarized in following Table.

Table 6.12 Maximum deviator stress and volume change response

Confining pressure (kPa)	Back pressure (kPa)	Inorganic Soil		Organic Soil	
		Maximum Deviator stress	Maximum volume change recorded (cc)	Maximum Deviator stress	Maximum volume change recorded (cc)
200	100	145	1.42	174	1.51
300	100	265	2.87	265	4.23

### 6.3.1 Deformation and strain variations with vs. time

Time vs. deformation diagram for different load increments for both inorganic and organic soils tested under confining pressure of 100 and 200 kPa are presented in Fig. 6.14. With the increase in stress levels the deformation and the rate of deformation for a particular load increment increased. In case of inorganic clays failure occurred at stress levels of 1.02 and 0.96 for 100kPa and 200kPa respectively. The corresponding SL values for organic clays were 1.06 and 0.96. This highlights that the deformation pattern for creep drained tests is similar to the conventional triaxial drained tests. The normalized axial deformation variation with time variation is presented in Fig 6.15.

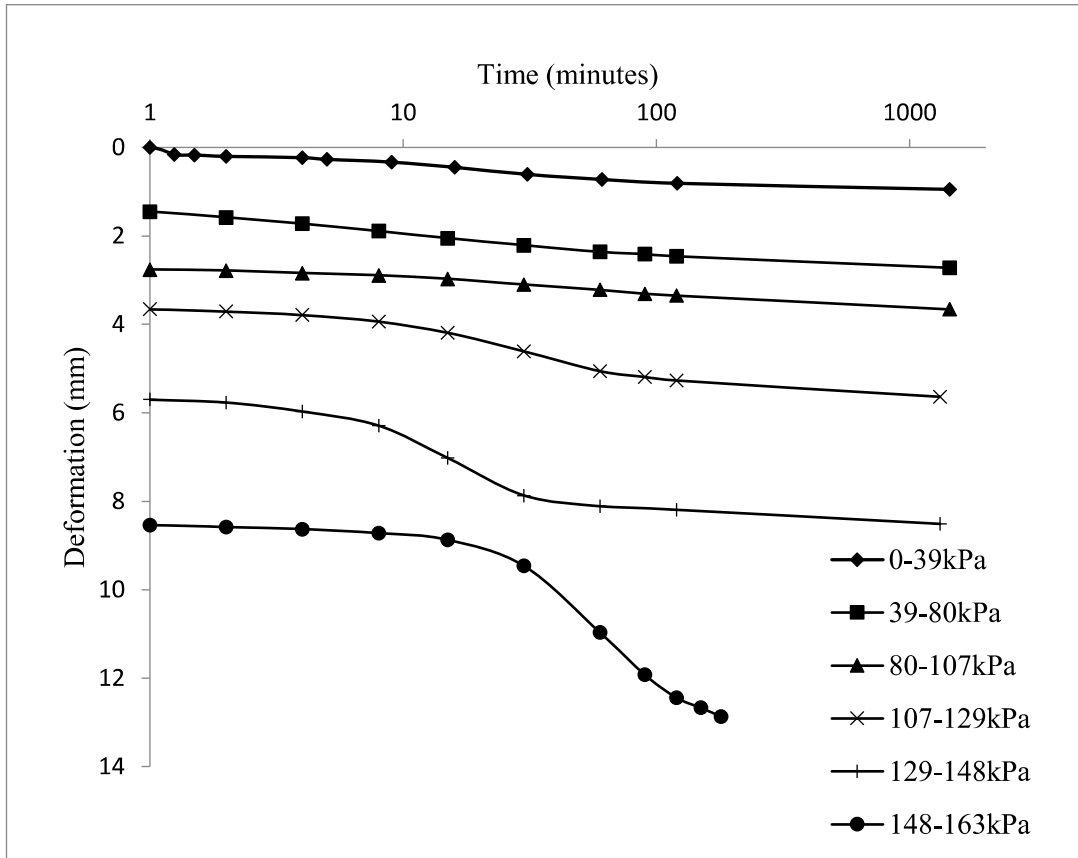


Fig.6.14.a Time vs. deformation diagram for inorganic clay at 100kPa

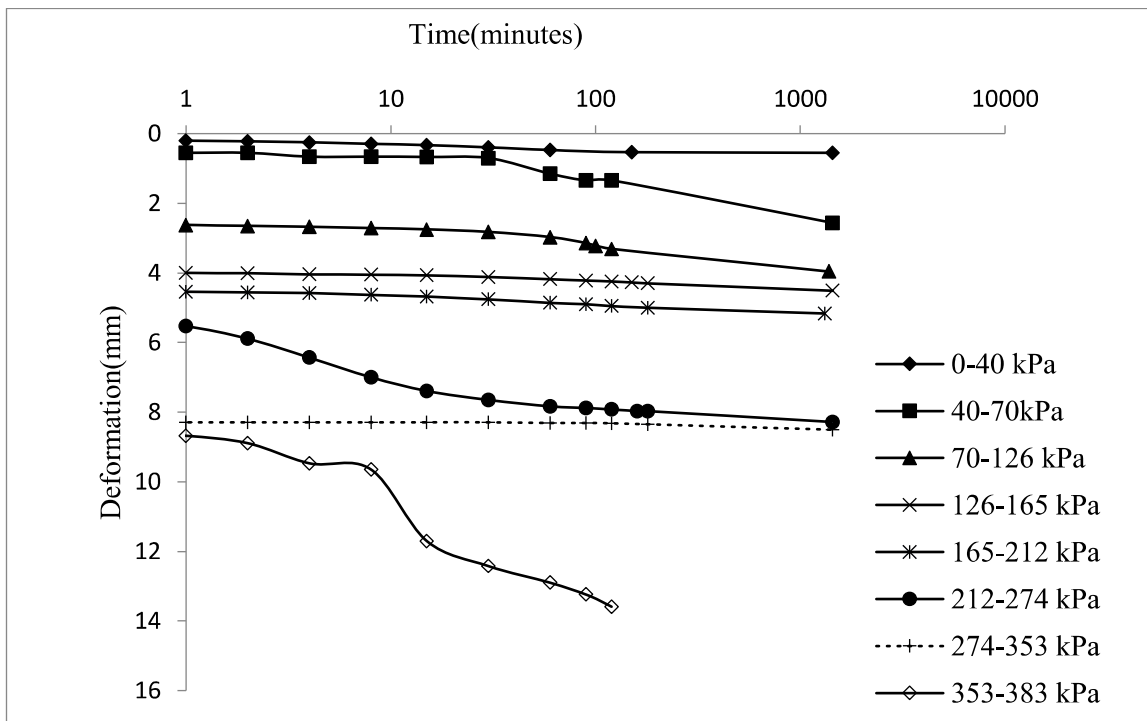


Fig.6.14.b Time vs. deformation diagram for inorganic clay at 200kPa

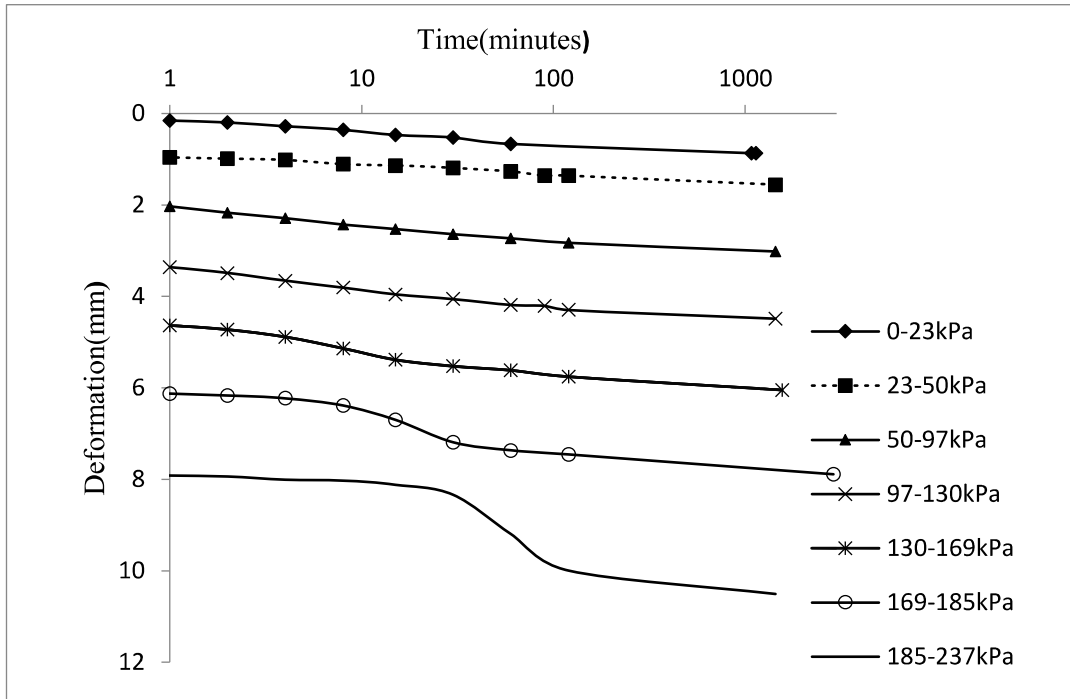


Fig.6.14.c Time vs. deformation diagram for organic clay at 100kPa

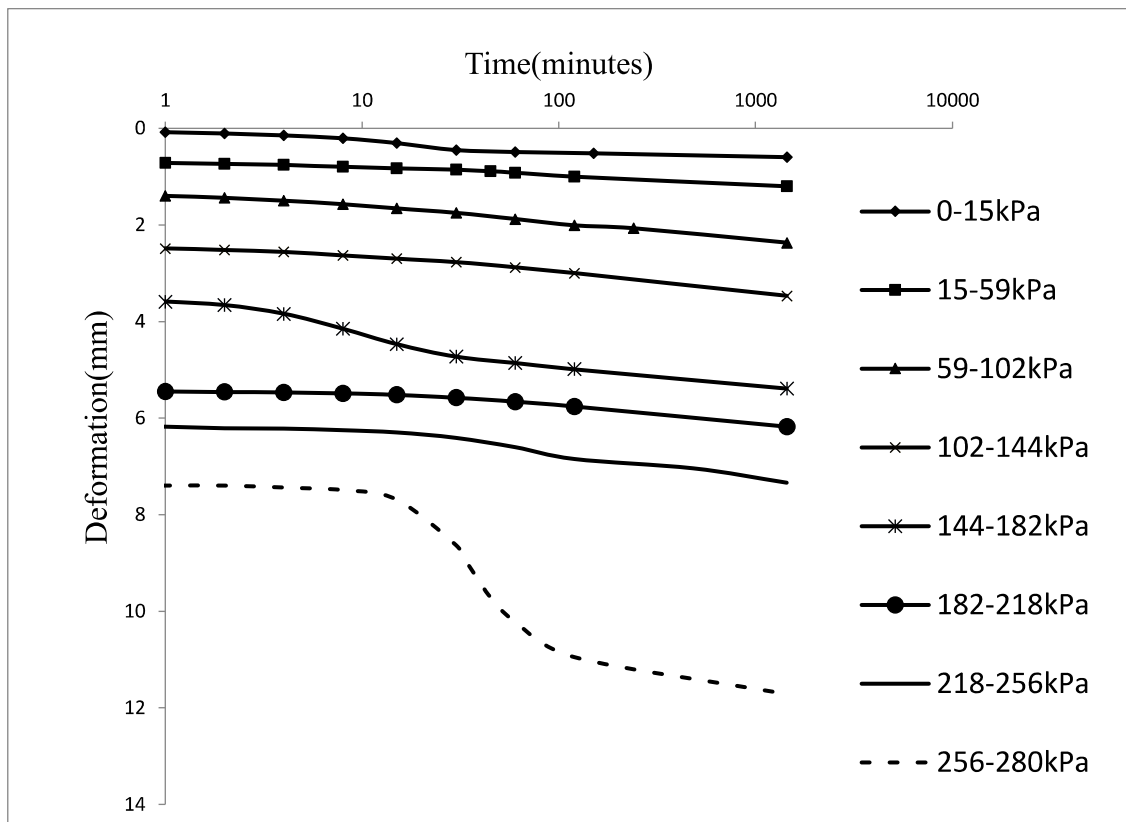


Fig.6.14.d Time vs. deformation diagram for organic clay at 200kPa.

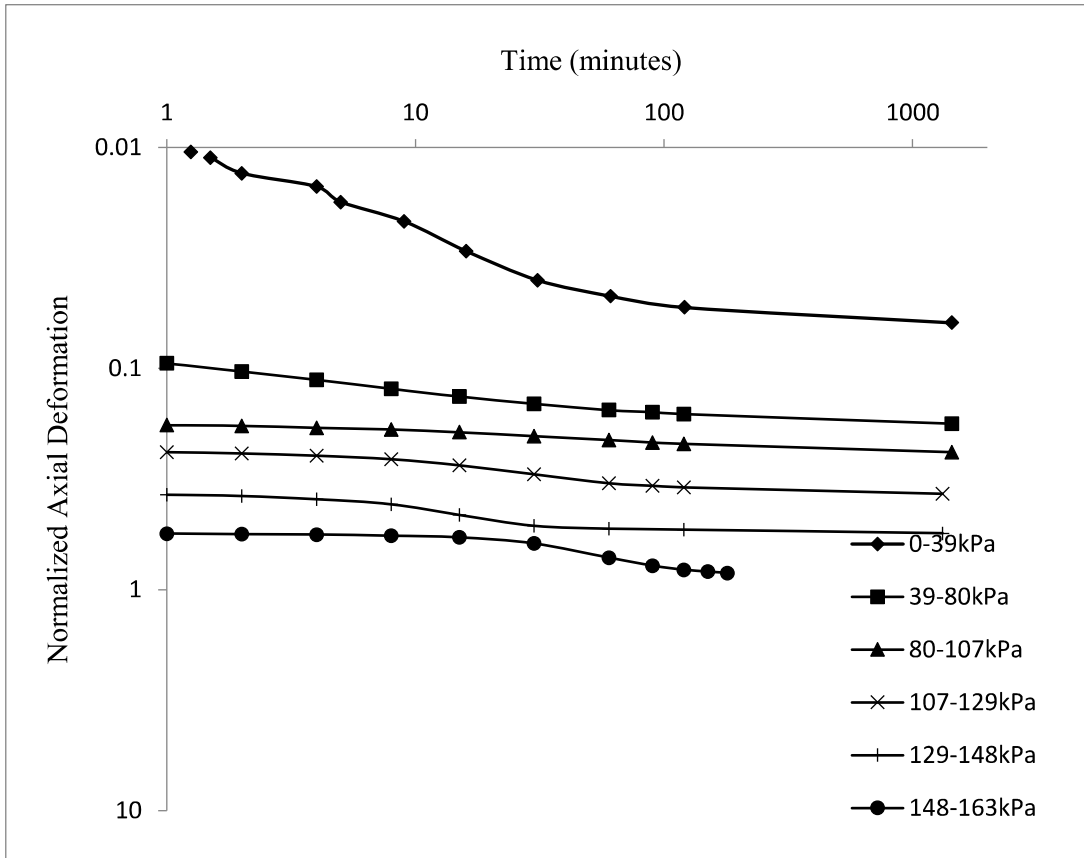


Fig.6.15(a) Normalized Axial deformation vs. time inorganic clay-Drained creep -100kPa

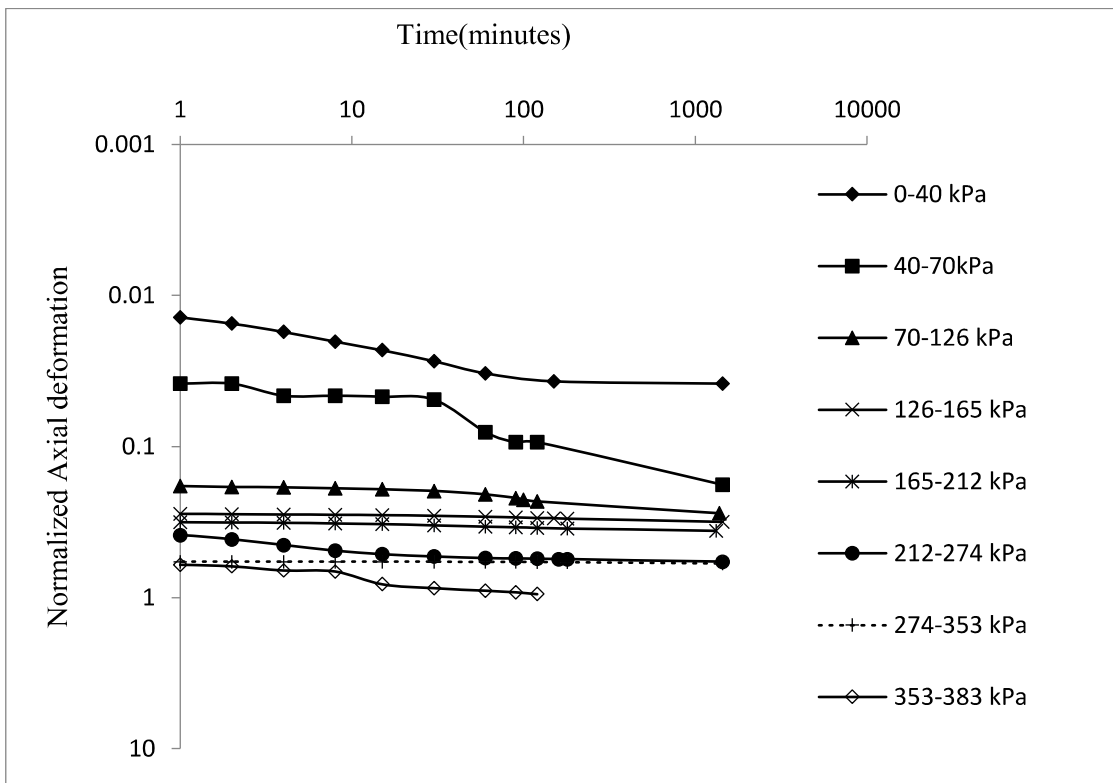


Fig.6.15(b) Normalized Axial deformation vs. time inorganic clay-Drained creep -200kPa

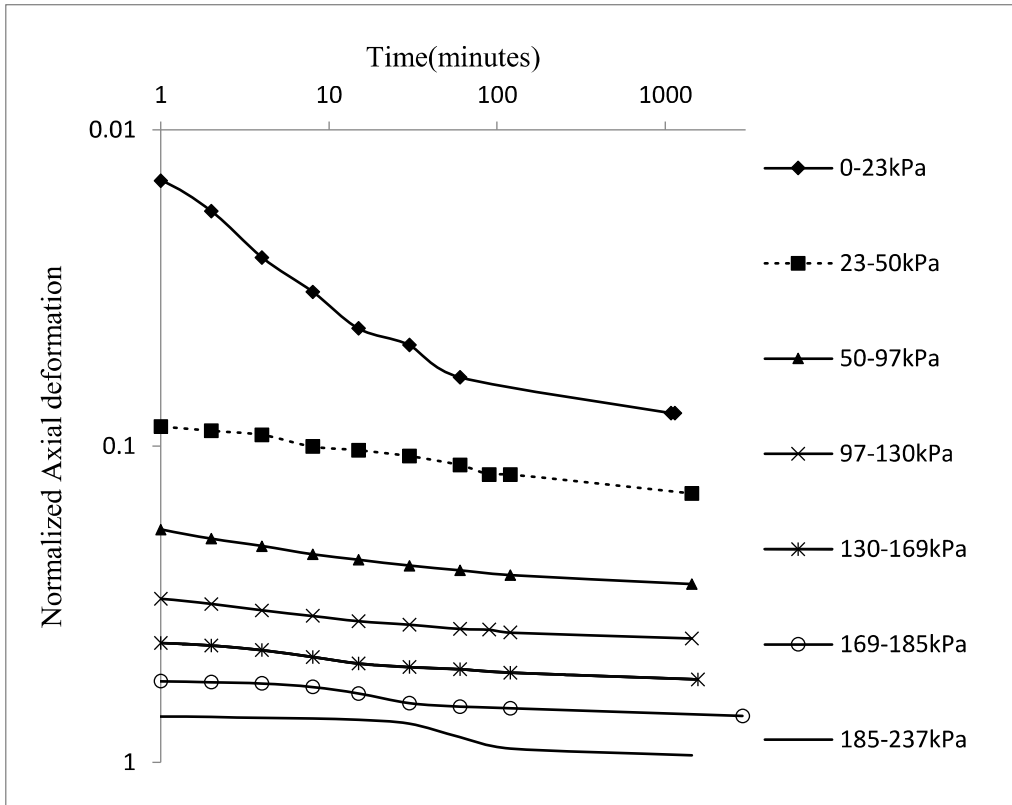


Fig.6.15(c) Normalized Axial deformation vs. time organic clay-Drained creep -100kPa

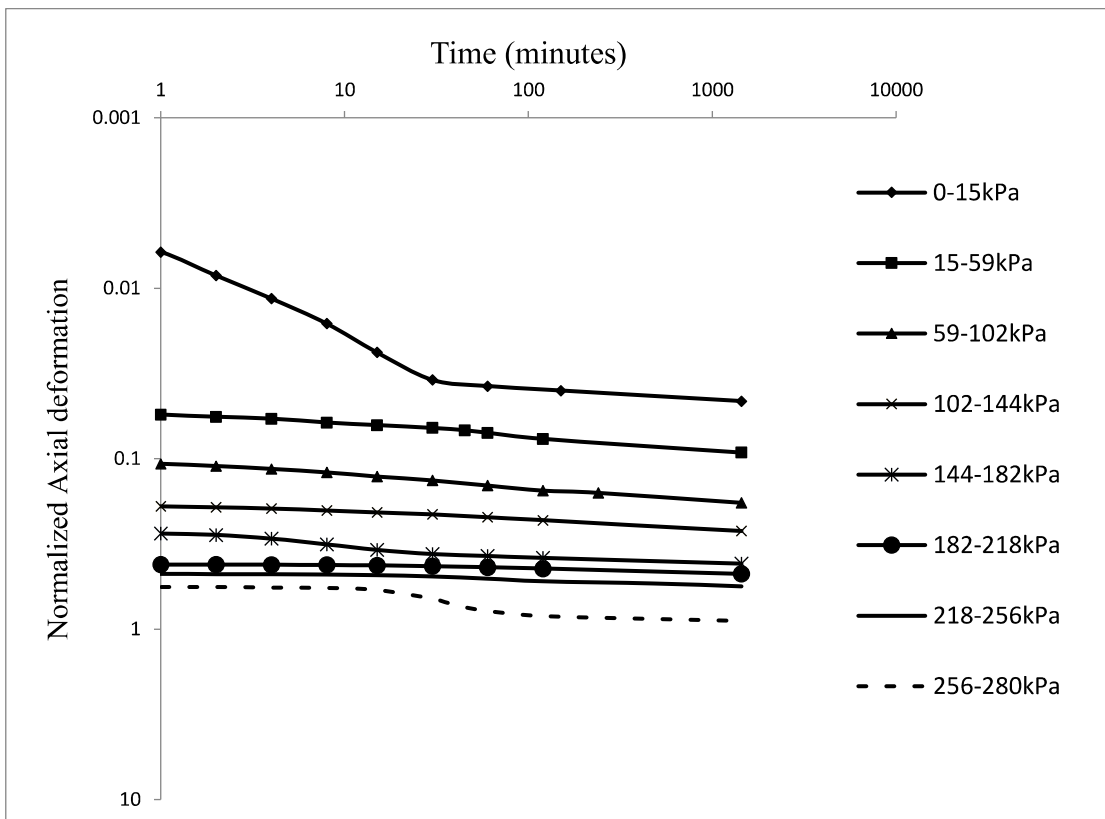


Fig.6.15(d) Normalized Axial deformation vs. time organic clay-Drained creep -200kPa

### **6.3.2 Volume change vs. time**

Variation of volume change with time under the action of multistage for both inorganic and organic soils at 100kPa and 200kPa confining pressures are presented in Fig.6.16. The volume change of organic clays were significantly greater than inorganic clays. This may be attributed to the higher initial moisture content of the organic soil . For inorganic clays the volume change was 1.2 cc and 3.2cc for 100kPa and 200kPa respectively whereas for organic clays the corresponding values were 5cc and 10cc.

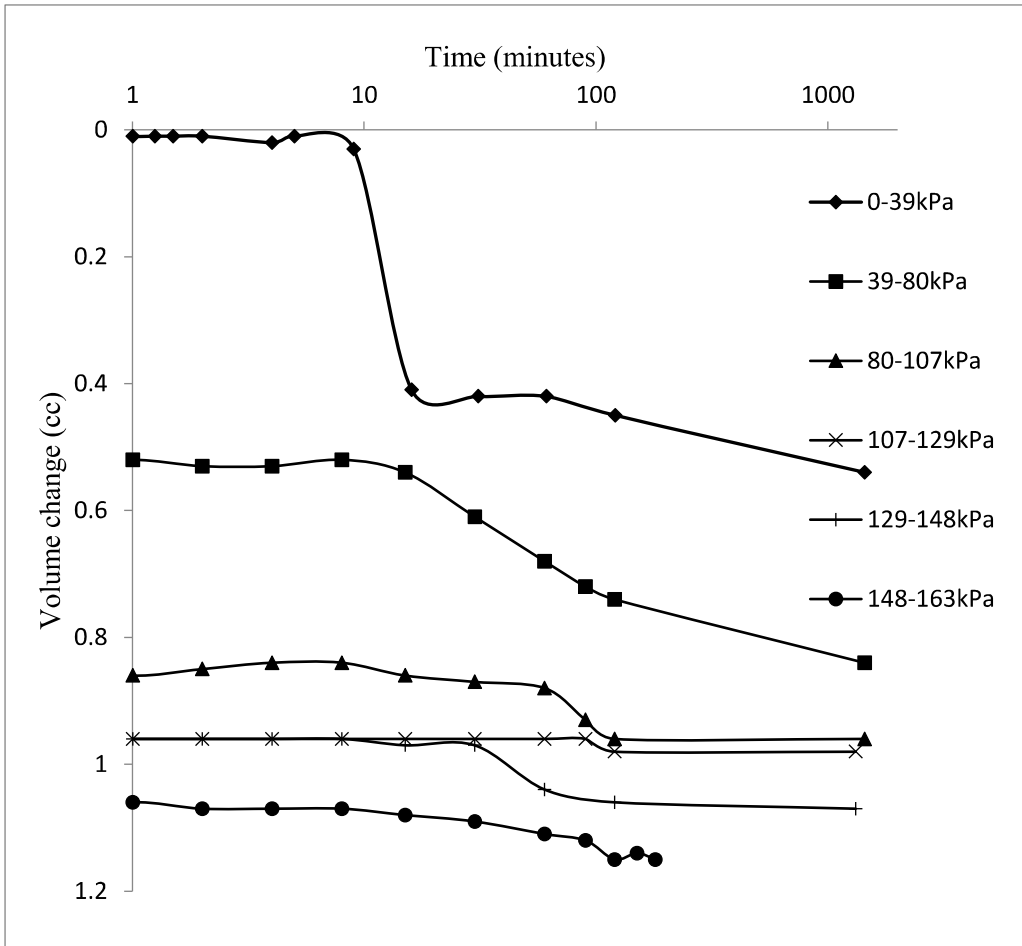


Fig. 6.16(b) Volume change vs. time curve for inorganic clay-100kPa

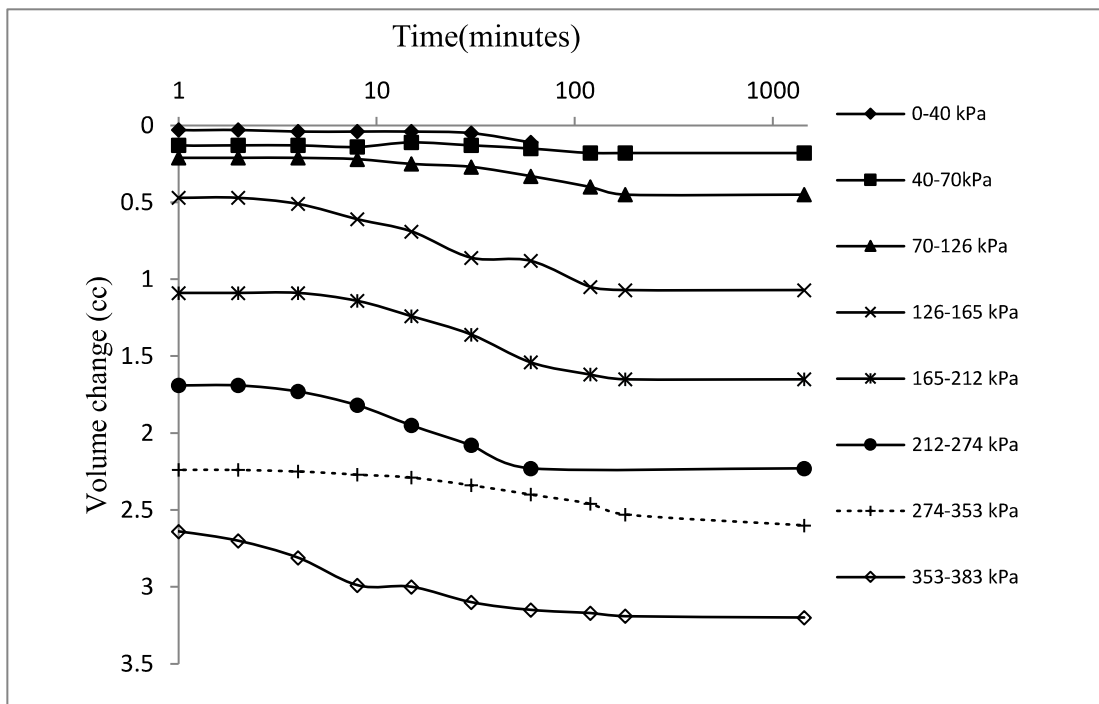


Fig. 6.16(c): Volume change vs. time curve for inorganic clay-200kPa

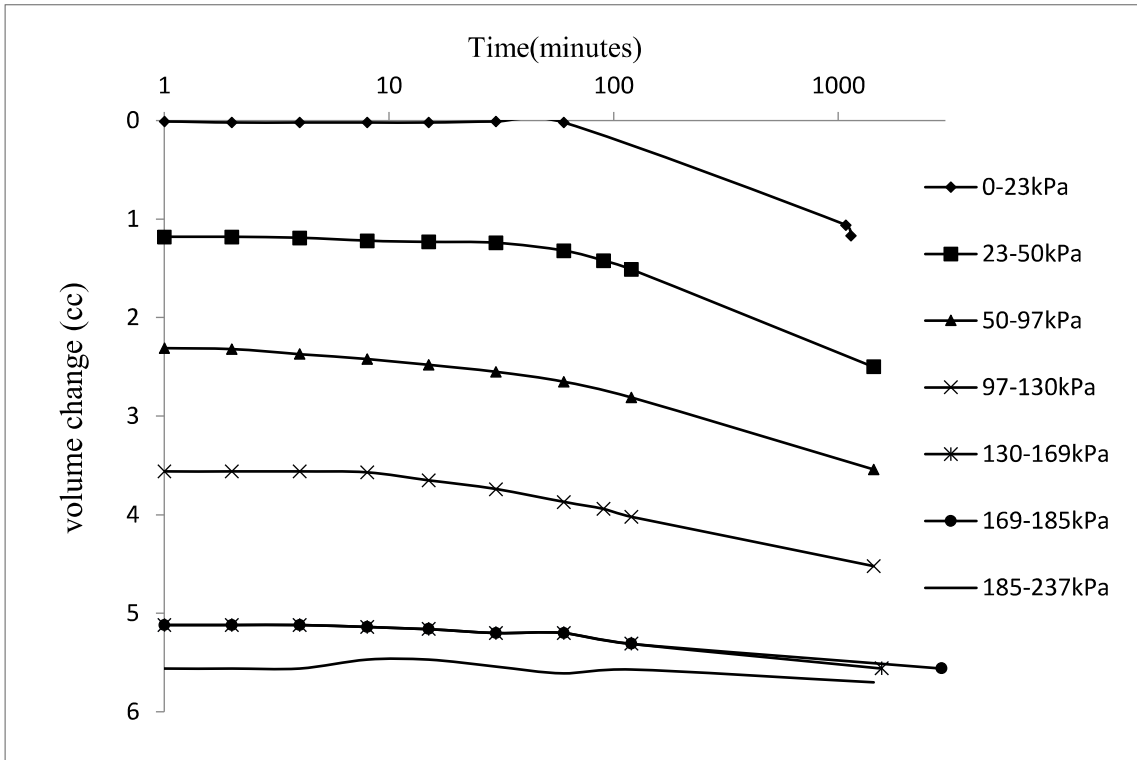


Fig. 6.16 (a) Volume change vs. time curve for organic clay-100kPa

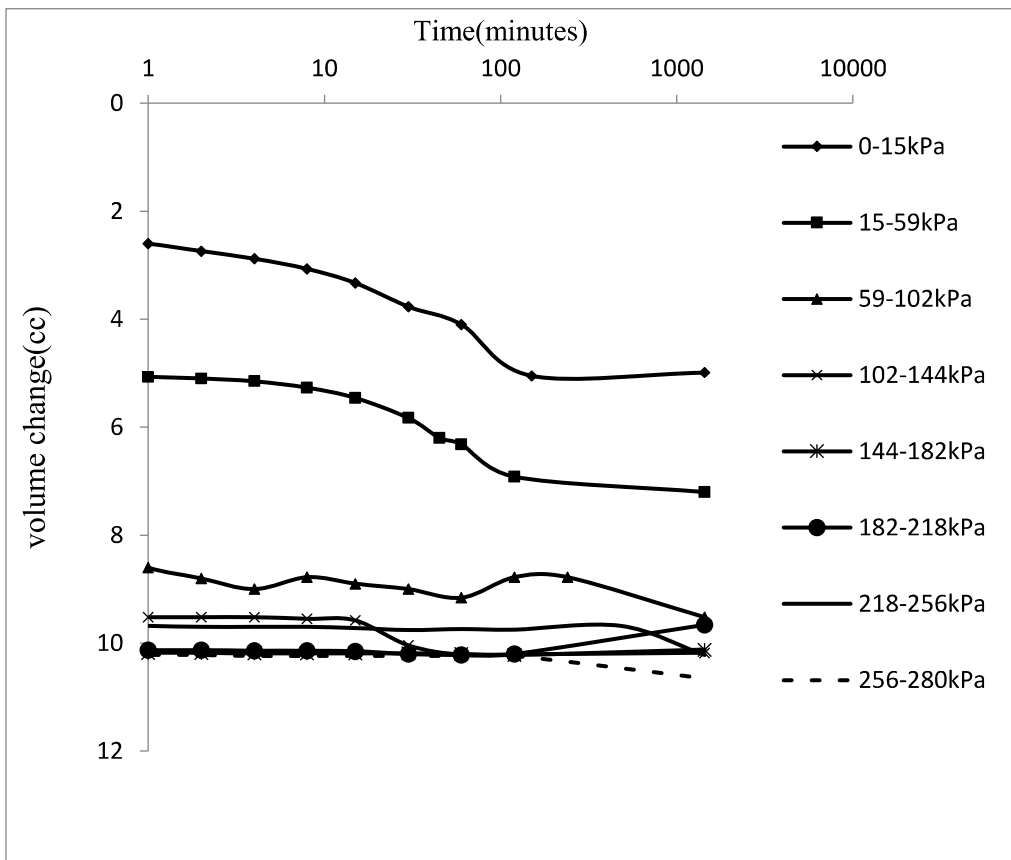


Fig. 6.16 (d) Volumetric strain in % vs. time curve for organic clay-200kPa.

#### 6.4 Drained creep parametric analysis

In this section an attempt to estimate the Singh and Mitchell (1969) creep parameters for drained creep results has been made. The three parameter phenomenological general function for soils expressing strain rate as a function of time after application of sustained loading was proposed by Singh and Mitchell (1969) and has already been discussed in detail in section 6.3. The values of parameter  $m$  have been estimated using Fig 6.17 a – Fig 6.17d. The parameter  $\alpha$  has been estimated using Fig 6.18a – 6.18d. The results of the parametric study of drained creep tests are presented Table 6.13 to Table 6.16.

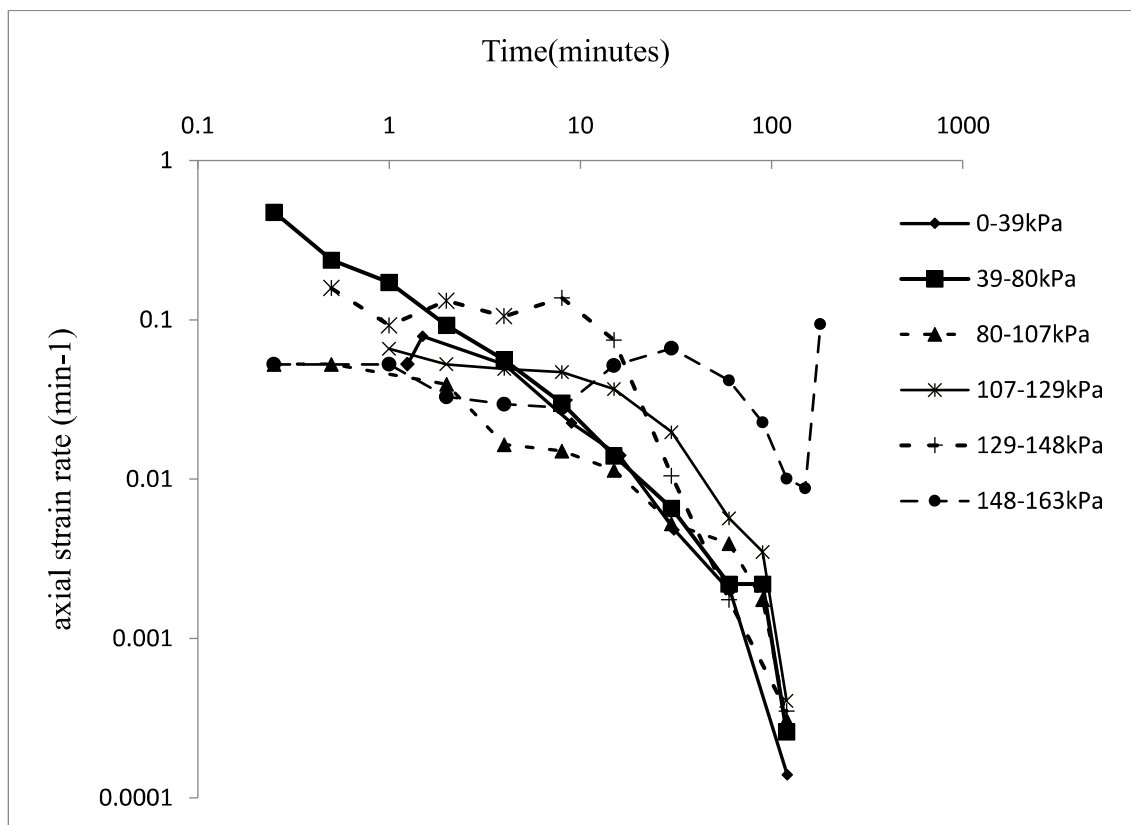


Fig.6.17(a) Axial strain rate vs. time for inorganic clay-Drained creep -100kPa

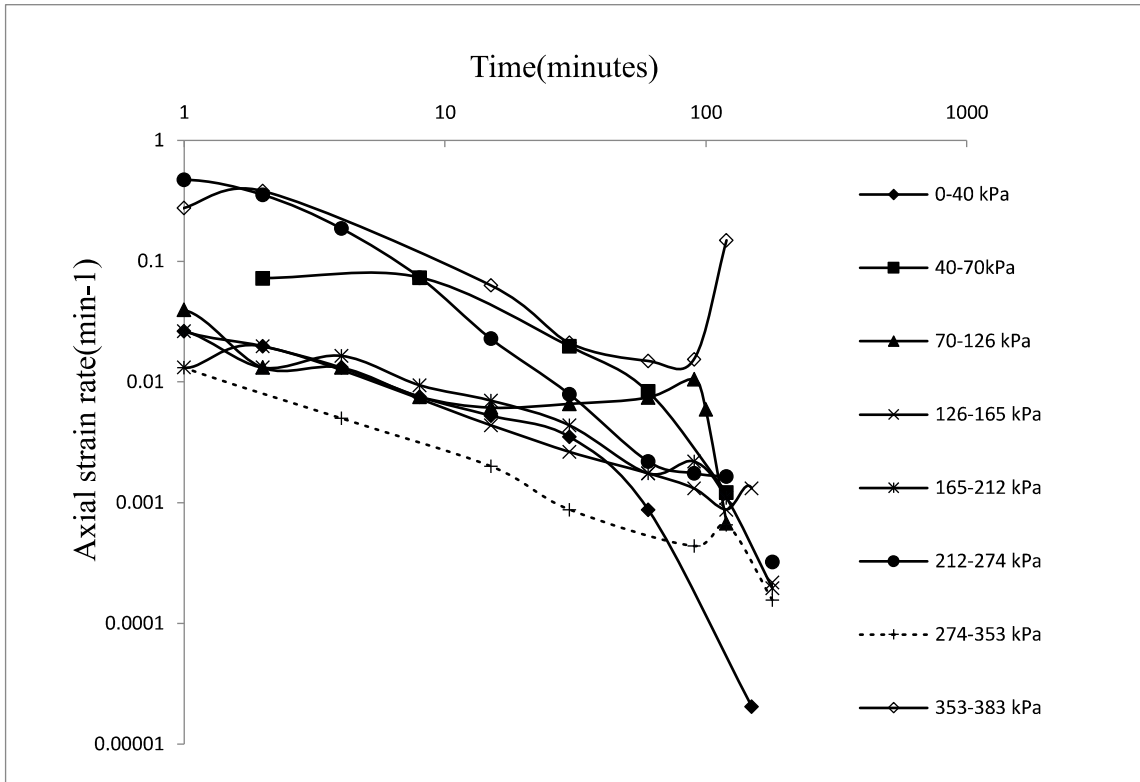


Fig.6.17(b) Axial strain rate vs. time for inorganic clay-Drained creep -200kPa

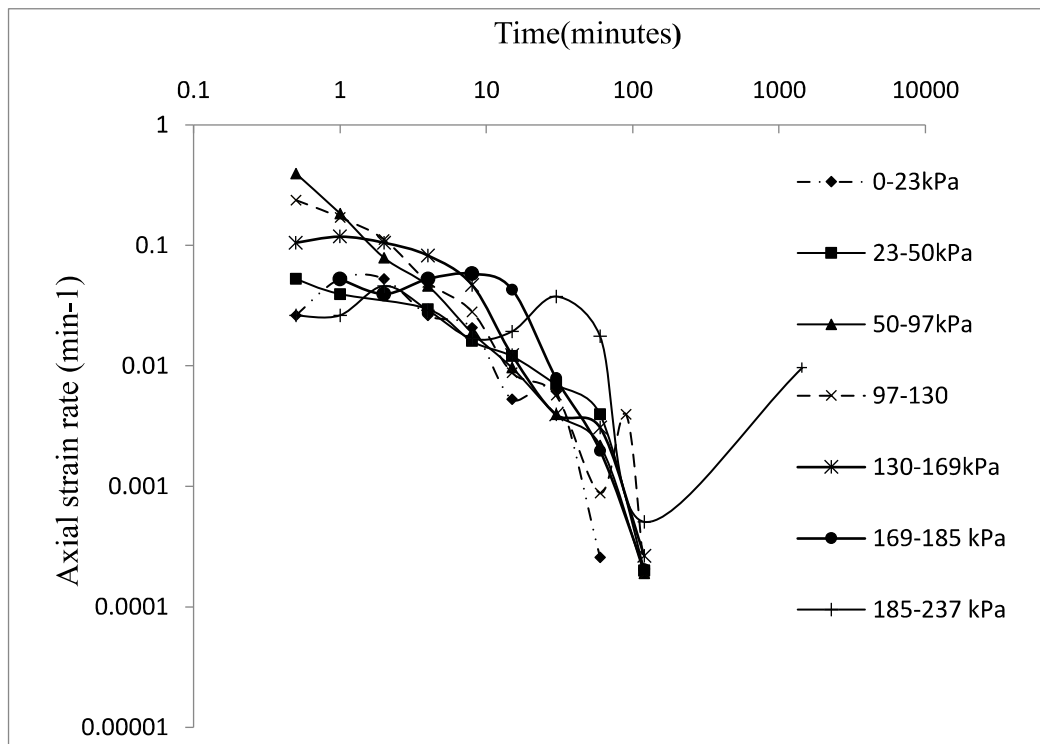


Fig.6.17(c) Axial strain rate vs. time for organic clay-Drained creep -100kPa

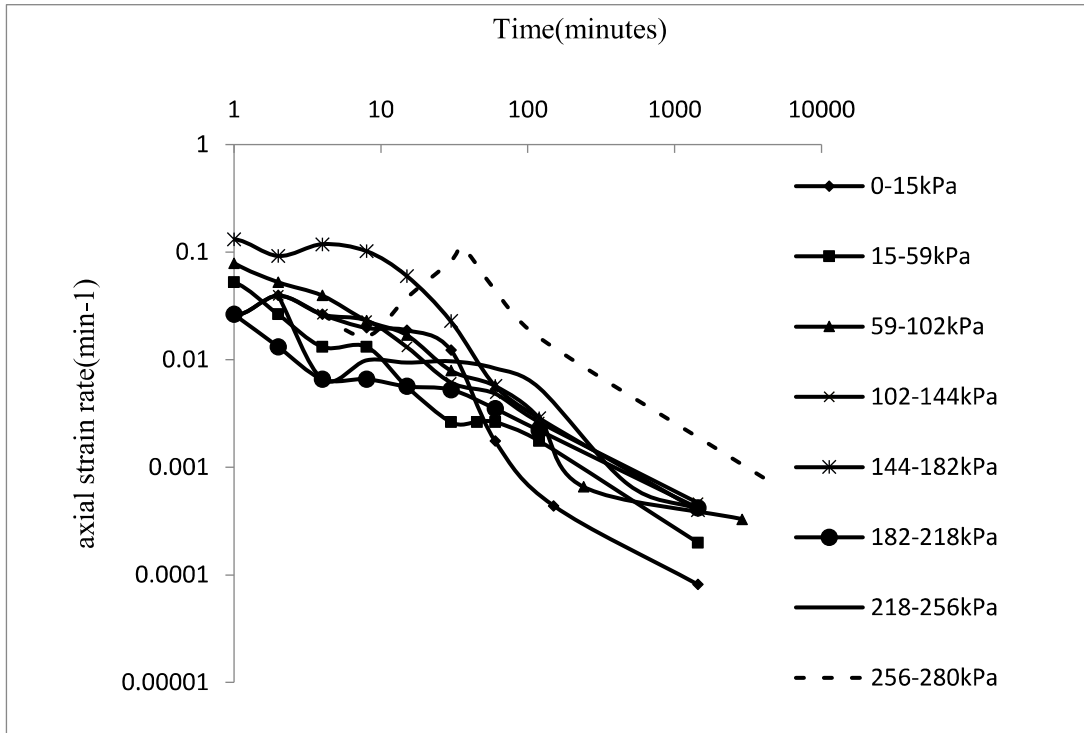


Fig.6.17(d) Axial strain rate vs. time for organic clay-Drained creep -200kPa

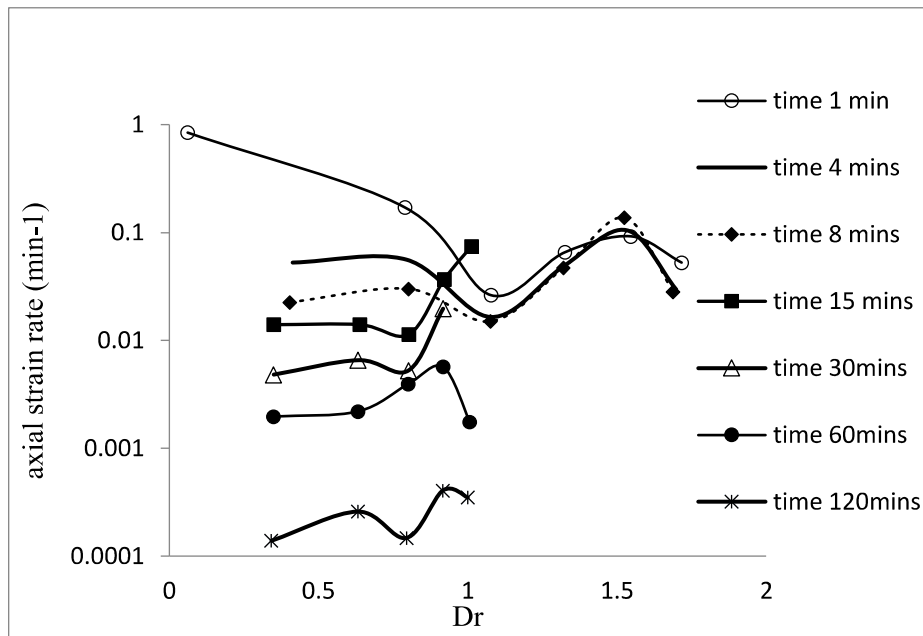


Fig.6.18(a) Axial strain rate vs. stress ratio inorganic clay-Drained creep -100kPa

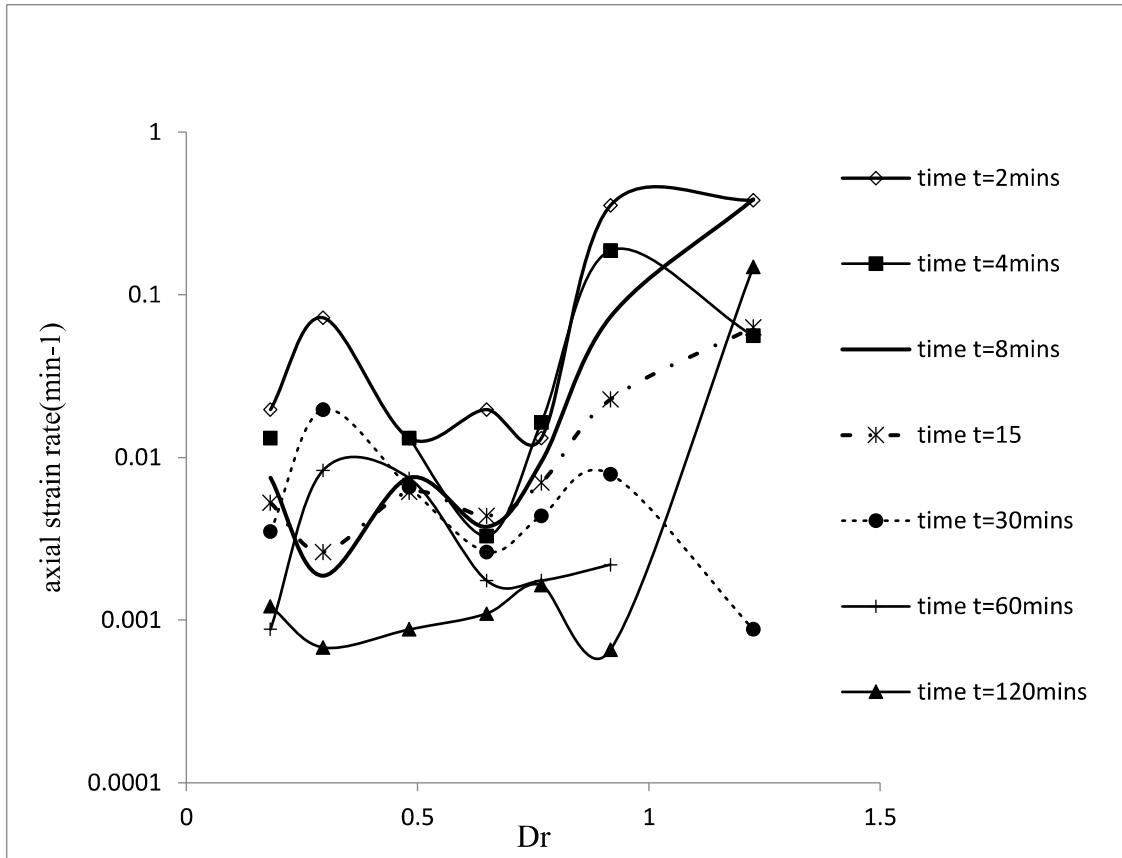


Fig.6.18(b) Axial strain rate vs. stress ratio for inorganic clay-Drained creep -200kPa

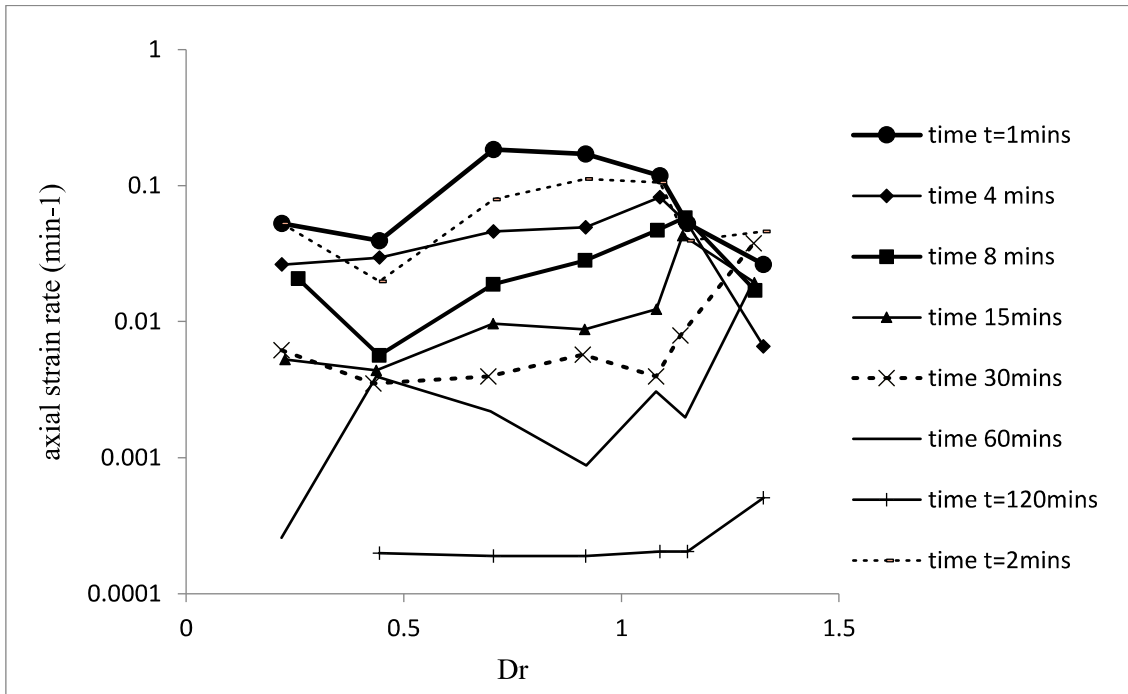


Fig.6.18(c) Axial strain rate vs. stress ratio for organic clay-Drained creep -100kPa

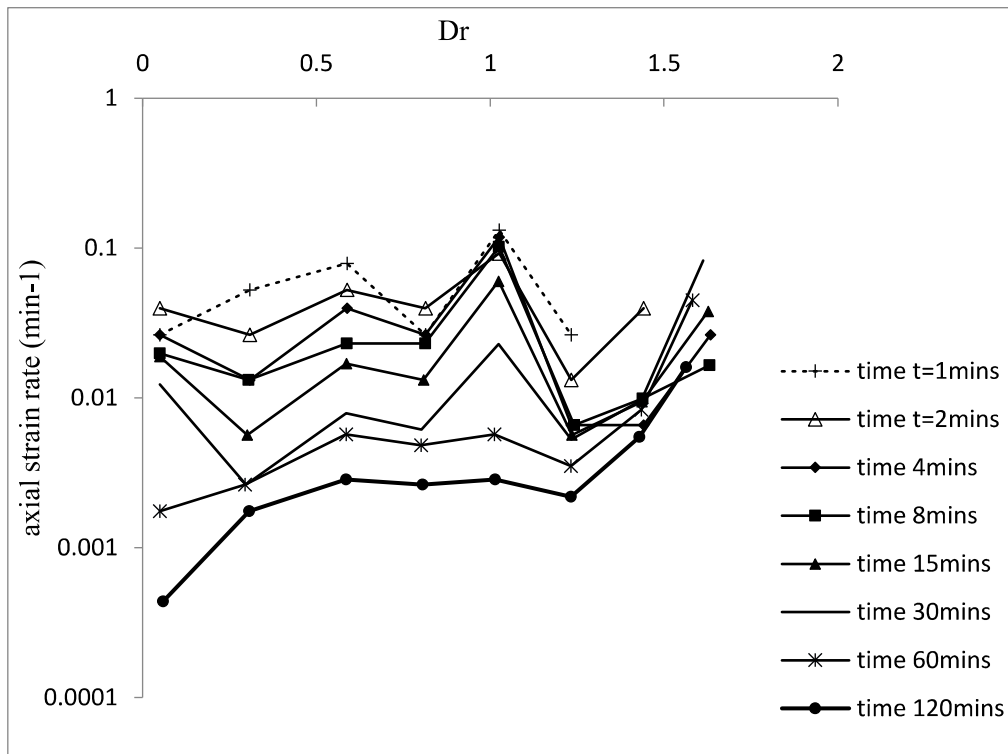


Fig.6.18(d) Axial strain rate vs. stress ratio for organic clay-Drained creep -200kPa

Table 6.13 Strain rate parameters – Drained - Inorganic soil -100kPa

Deviator stress(kPa)	0-39(20)	39-80(60)	80-107(94)	107-129(118)	129-148(139)
SL	0.19	0.40	0.54	0.645	0.74
$\sigma_d/p_o$	0.39	0.80	1.07	1.29	1.48
m	1.27	1.27	1.12	1.48	1.40

Table 6.14 Strain rate parameters – Drained - Inorganic soil -200kPa

Deviator stress(kPa)	0-41(21)	41-70(55)	71-126(98)	89-165(127)	111-160(136)	135-275(205)
SL	0.16	0.27	0.48	0.63	0.61	1.05
$\sigma_d/p_o$	0.205	0.35	0.63	0.825	0.80	1.375
m	1.1	1.20	1.1	1.1	1	1.30

Table 6.15 Strain rate parameters – Drained -Organic soil -100kPa

Deviator stress(kPa)	0-24(12)	24-50(37)	50-91(70)	91-130(110)	130-169(150)
SL	0.13	0.28	0.52	0.74	0.96
$\sigma_d/p_o$	0.24	0.50	0.91	1.30	1.69
m	1	1.196	1.21	1.47	1.36

Table 6.16 Strain rate parameters – Drained -Organic soil -200kPa

Deviator stress(kPa)	18-72(45)	72-128(100)	128-171(150)	171-218(195)	265-302(284)
SL	0.24	0.42	0.57	0.72	1
$\sigma_d/p_o$	0.36	0.64	0.85	1.09	1.51
m	0.95	1.06	1	1	0.90

In the Fig 6.17 and Fig 6.18 the creep potential parameters for inorganic and organic clays of Kolkata deposit under drained conditions have been plotted. It is observed that the values of  $\alpha$  for drained cannot be determined using the method suggested by Singh and Mitchell (1969). There is a non-linear dependency of the slope of stress level and axial strain rate hence Singh-Mitchell equation cannot be used for direct fitting of test data. Similar results were reported by Tian et. al. (1994), Zhu et. al. (1999) amongst others who highlighted that the Singh and Mitchell 1969 parameters are more applicable under undrained condition. The average value of m estimated from Fig 6.17 shows that for almost all load increments the is greater than 1. The higher the value of m, lesser is the effect of viscosity in the soil. This indicates that the effect of viscosity/ creep effect is not very significant for inorganic and organic clays of Kolkata deposit under drained conditions.

To understand and analyse the test results obtained from drained creep triaxial tests the Singh and Mitchell (1969) equation was modified by Zhao et al (2018).

$$\epsilon = B e^{\beta D_r} (t/t_r)^{\lambda} \quad (6.6)$$

where  $B = A_r t_r / (1 - m)$ ;

$$\beta = \alpha;$$

$$\lambda = 1 - m.$$

The three parameters that the model needs to determine are  $B$ ,  $\beta$  and  $\lambda$ . When  $t = t_r$ , above formula can be written as:

$$\epsilon = B e^{\beta D_r} \quad (6.7)$$

$$\ln \epsilon = \beta D_r + \ln B \quad (6.8)$$

Thus  $\beta$  and  $B$  can be obtained directly from the slope and intercept in the relation curve of  $\ln \epsilon$  - stress level when reference time is 1 minute.  $\lambda$  is determined by the slope on the curve of  $\ln \dot{\epsilon} - \ln t$ .

The model parameters are estimated using Fig 6.19 and are presented in Table 6.17.

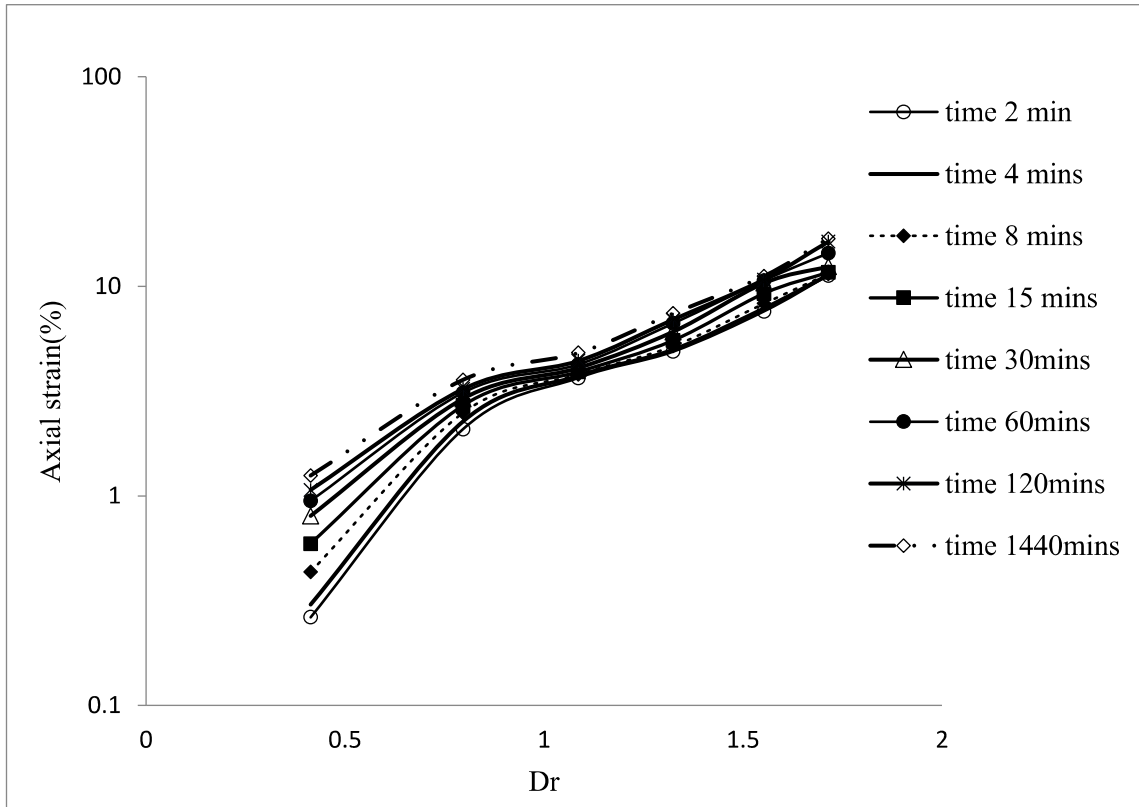


Fig.6.19(a) Axial strain vs. stress ratio inorganic clay-Drained creep -100kPa

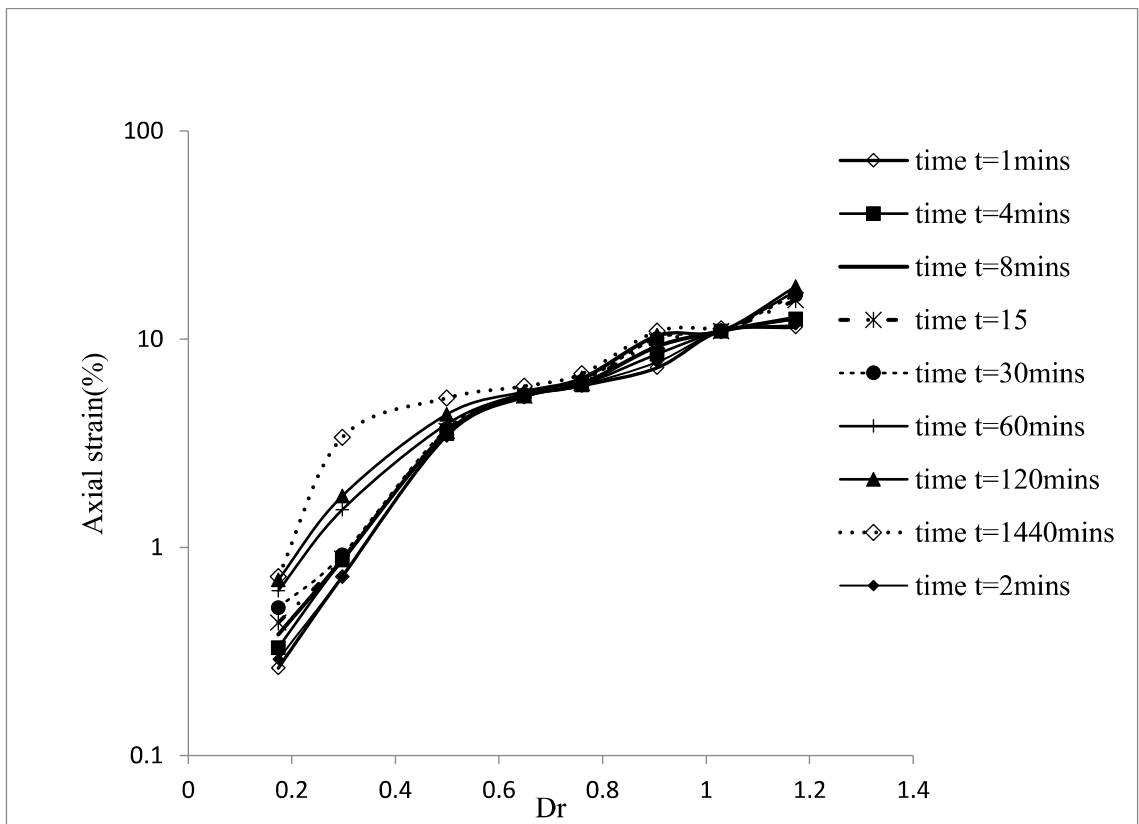


Fig.6.19(b) Axial strain vs. stress ratio inorganic clay-Drained creep -200kPa

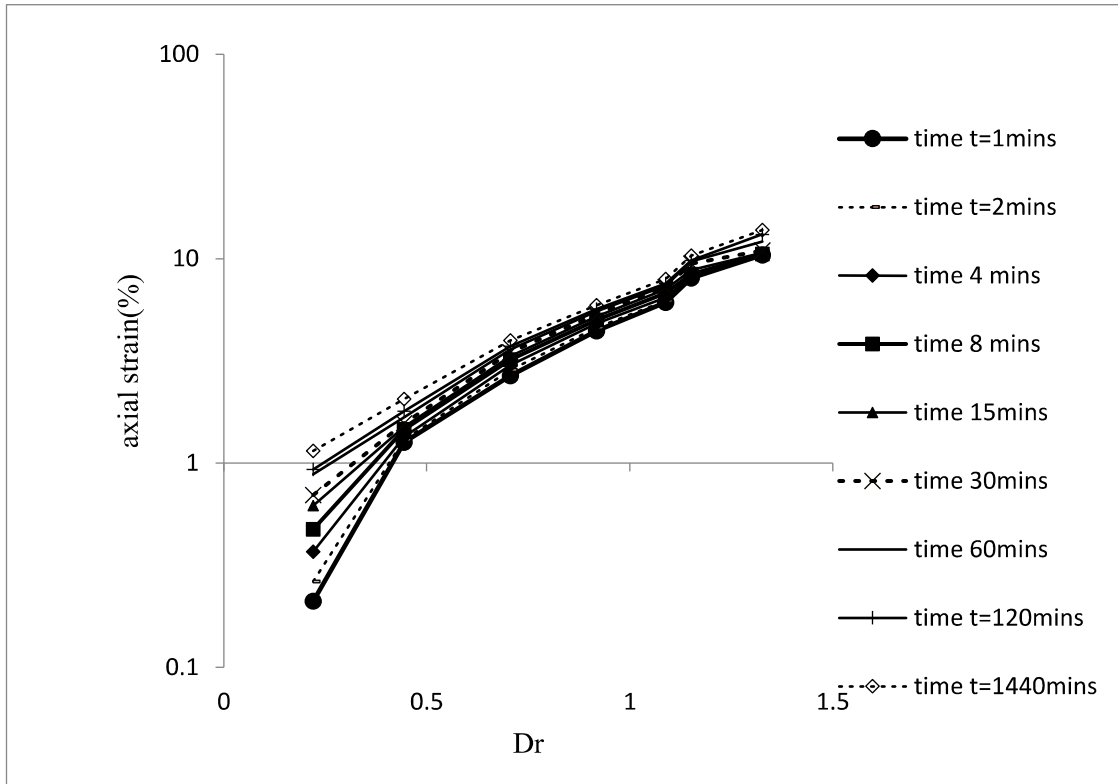


Fig.6.19(c) Axial strain vs. stress ratio organic clay-Drained creep -100kPa

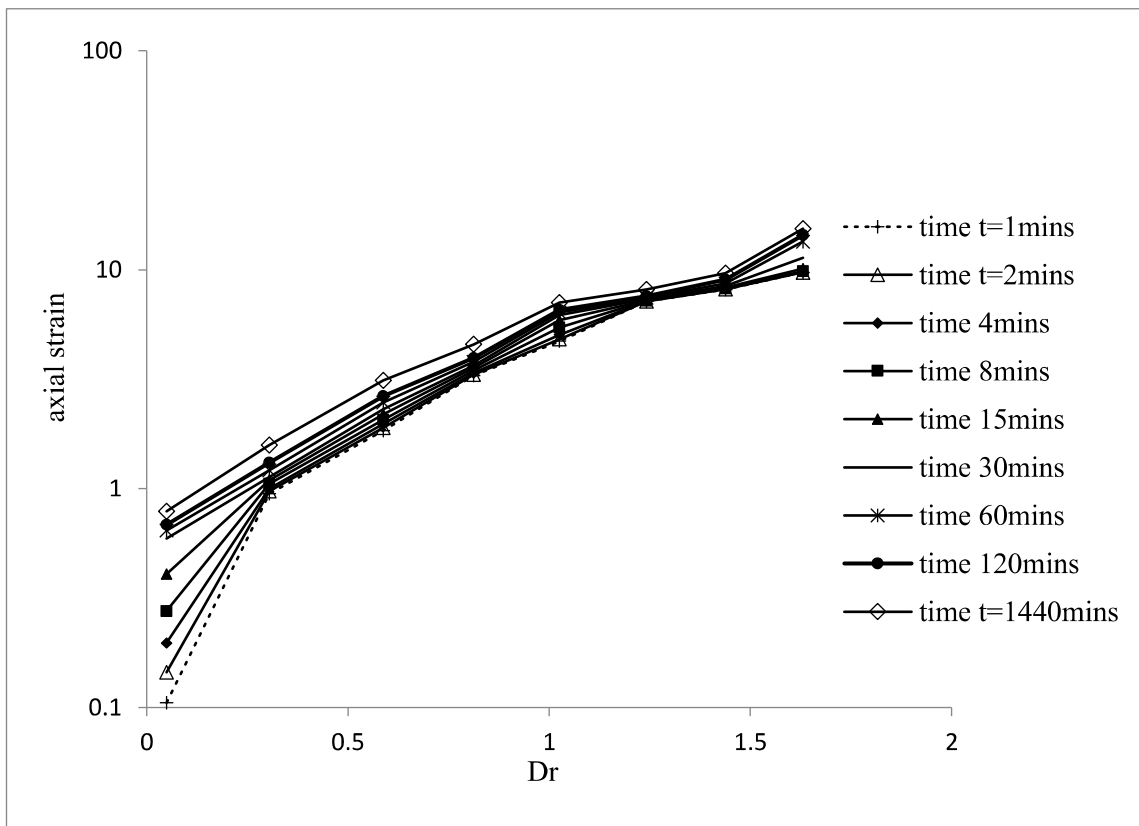


Fig.6.19(d) Axial strain vs. stress ratio organic clay-Drained creep -200kPa

The values of parameter B for drained creep tests for inorganic clays under 100kPa and 200kPa confining pressure are 0.4 and 0.5 respectively. For organic clays subjected to 100kPa and 200kPa confining pressures the values were 0.5 and 0.5 respectively.

Table 6.17: The values of  $\beta$

Time (mins) Soil type	1	2	4	8	15	30	60	120	1440
inorganic clay -100kPa	1.21	1.21	1.21	1.30	1.45	1.67	1.90	1.90	1.82
inorganic clay -200kPa	1.36	1.76	2.34	2.85	3.15	3.27	3.29	3.24	3.25
organic clay - 100kPa	2.44	2.43	2.43	2.38	2.41	2.38	2.31	2.24	2.10
organic clay - 200kPa	2.23	2.22	2.25	2.28	2.37	2.37	2.31	2.23	2.08

## 6.5 Calcutta High Court

In an attempt to highlight the effect of time-dependent deformation of typical soft clay deposit in Kolkata the settlement behaviour of a typical foundation of Kolkata High court building constructed in the nineteenth century has been discussed. The Calcutta High Court, the oldest High Court in India was constructed in neo-Gothic style between 1864 and 1868. It is a 150 years old brick structure constructed close to the eastern bank of The Hooghly River. The dead load on the structure has increased over the years, probably attributed to the necessity to maintain a record of documents. The structure experienced excessive settlement which led to the development of

distress cracks of certain structural members (Mittal and Babu 2018).

In order to study and review the settlement the soil investigation work around the Main Building of the High Court was conducted. The settlement analysis for a typical foundation of the High court building has been presented in order to examine time dependent behavior of normal Kolkata deposit.

### **6.5.1 Soil sampling of the Calcutta High Court**

6 number of boreholes were sunk at the locations, conducting standard penetration tests and collection of disturbed and undisturbed samples from different depths. Depth of the boreholes ranges from 15.45 – 25.60 m existing ground level. Auger boring was adopted up to a depth of about 3.0 m below the Existing Ground Level (E.G.L.) followed by Rotary Wash boring technique to advance the 150 mm dia. boreholes up to termination depths. Casing was used in the top 2 – 3m of the boreholes and, thereafter, bentonite solution of adequate density was used for stabilization of boreholes. S.P.T. was conducted at the boring points at suitable intervals. The number of blows required for 30.0 cm penetration of split spoon sampler, after first 15 cm penetration, driven by a 63.5 kg hammer falling freely through a height of 75 cm was recorded as ‘N’ value. The sample from split spoon sampler was collected after each test and was properly labelled and placed in air-tight polythene bags before sending it to the laboratory. The test procedure conformed to IS: 2131-1981. Undisturbed samples were collected by means of a two tier 100 mm I.D. open drive sampling assembly. Before sampling, the borehole was thoroughly cleaned. The sampling assembly was driven to the required depth manually with the help of a jarring link. Sample collected in the lower tube was retained, labelled and waxed at both ends before sending to the laboratory.

Representative soil samples were collected frequently from auger, split spoon sampler of standard penetrometer and cutting shoe of undisturbed sampler to maintain a continuous record for strata encountered.

Laboratory testing on selected soil samples for determination of their index properties, shear strength and consolidation parameters. The following laboratory tests were conducted on soil samples namely determination of Natural Moisture Content and Bulk density, determination of Liquid Limit and Plastic Limit, Sieve and Hydrometer Analysis, determination of Specific Gravity, Unconfined Compression / UU Triaxial Tests and one-dimensional Consolidation Tests.

#### **6.5.2 Sub-soil stratification of the Calcutta High Court**

A study of the borehole logs, laboratory test results and field standard penetration test results indicates the following stratification of the sub-soil deposit. The deposits in all the boreholes are more or less of similar nature. The stratification is presented in Table 6.18.

Table-6.18 : SUB-SOIL STRATIFICATION

STRATUM	DESCRIPTION OF STRATUM	THICKNESS (m)	Properties
<b>Top fill</b>	The deposit in this stratum consists of heterogeneous filling with rubbish and brickbats having no definite engineering properties with average thickness of 3.00 m (range 1.50 – 5.00m). For calculation of overburden pressure,	1.50 – 5.00 (av. 3.00)	bulk density of the deposit may be taken as 1.6 t/m <sup>3</sup> .
<b>I</b>	Soft to firm brownish grey silty clay / clayey silt. The soil in this stratum is basically cohesive in nature with Field 'N' value of this layer is 2 - 4 blows / 30 cm and average thickness of 3.00m. As per IS, the soil may be classified as CI. The following values of relevant soil parameters may be considered in design (Based on field 'N' values, and laboratory tests ):	2.00 – 4.50 (av. 3.00)	Bulk Unit Weight : 1.85t/m <sup>3</sup> LL = 46 %, PL = 22 % Moisture Content : 32% Cohesion Value, C <sub>u</sub> = 3.6 t/m <sup>2</sup> m <sub>v</sub> : 0.040 cm <sup>2</sup> /kg (pressure range 0.5 – 1.0 kg/cm <sup>2</sup> )
<b>II</b>	The soil in this stratum is Soft dark grey silty clay / clayey silt with organic matter and decomposed wood. This is basically cohesive in nature with high plasticity in general. Field 'N' values vary from 2 to 3 extending down to an average depth of 15.00 m below E.G.L. Consistency of soil in this layer is soft. As per IS, the soil may be classified as CH/OH. The following values of relevant soil parameters may be considered in design (Based on field 'N' values, laboratory tests )	7.00 -10.00 (av. 9.00)	Bulk Unit Weight : 1.70 t/m <sup>3</sup> LL = 62 %, PL = 25% Moisture Content : 45% Cohesion Value, C <sub>u</sub> = 2.0 t/m <sup>2</sup> m <sub>v</sub> : 0.060 cm <sup>2</sup> /kg (pressure range 0.5 – 1.0 kg/cm <sup>2</sup> )

<p><b>III</b></p>	<p>The soil in this stratum is medium / stiff grey to light bluish grey silty clay/clayey silt with kankars. Field ‘N’ value is 9 - 12 blows / 30cm. extending down to an average depth of 20.00 m below E.G.L. Consistency of soil is medium / stiff. The following values of relevant soil parameters may be considered in design.</p>	<p>4.50 - 6.00 (av. 5.00)</p>	<p>Bulk unit weight: 1.87 t/m<sup>3</sup> Cohesion Value, C<sub>u</sub> = 5.5 t/m<sup>2</sup> Angle of Internal Friction, φ = 0<sup>0</sup> m<sub>v</sub> : 0.025 cm<sup>2</sup>/kg (pressure range 0.5 – 1.0 kg/cm<sup>2</sup>)</p>
<p><b>IV</b></p>	<p>The soil in this stratum is stiff mottled brown silty clay / clayey silt extending down to a maximum depth of 23.00 m below E.G.L. Field ‘N’ value is 11 - 25 blows / 30cm. The following values of relevant soil parameters may be considered in design (Based on field &amp; corrected ‘N’ values )</p>	<p>2.50 – 3.50 (av. 3.00)</p>	<p>Bulk Unit Weight : 1.90 t/m<sup>3</sup> Cohesion Value, C<sub>u</sub> = 6.5 t/m<sup>2</sup> Angle of Internal Friction, φ = 0<sup>0</sup></p>
<p><b>V</b></p>	<p>The soil in this stratum consists of medium /dense brownish grey silty fine sand / sandy silt with mica and extends down to a maximum depth of <b>25.60m</b> below E.G.L. (termination level). The soil in this stratum is basically non-cohesive in nature with no plasticity. Field ‘N’ value is 35 - 52 blows / 30 cm . Compactness of soil is medium / dense. The following values of relevant soil parameters may be considered in design (Based on field &amp; corrected ‘N’ values) :</p>	<p>Till termination depth</p>	<p>Bulk Unit Weight : 2.00 t/m<sup>3</sup>  Angle of Internal Friction, φ = 35<sup>0</sup></p>

**STANDING WATER LEVEL :**

Water levels were observed over a period of twenty four hours after the post completion of boring operations. The final water readings were recorded in the field. The average level of water was found approximately at 3.00m below E.G.L.

### 6.5.3 Settlement analysis of the Calcutta High Court

The average soil profile along with the average properties at the site is given in Fig.6.20. Subsoil at the site comprises of a top fill of about 3m followed by a thick compressible soft clay deposit of brownish grey/dark grey silty clay/clayey silt down to a depth of 15.0m below ground level .

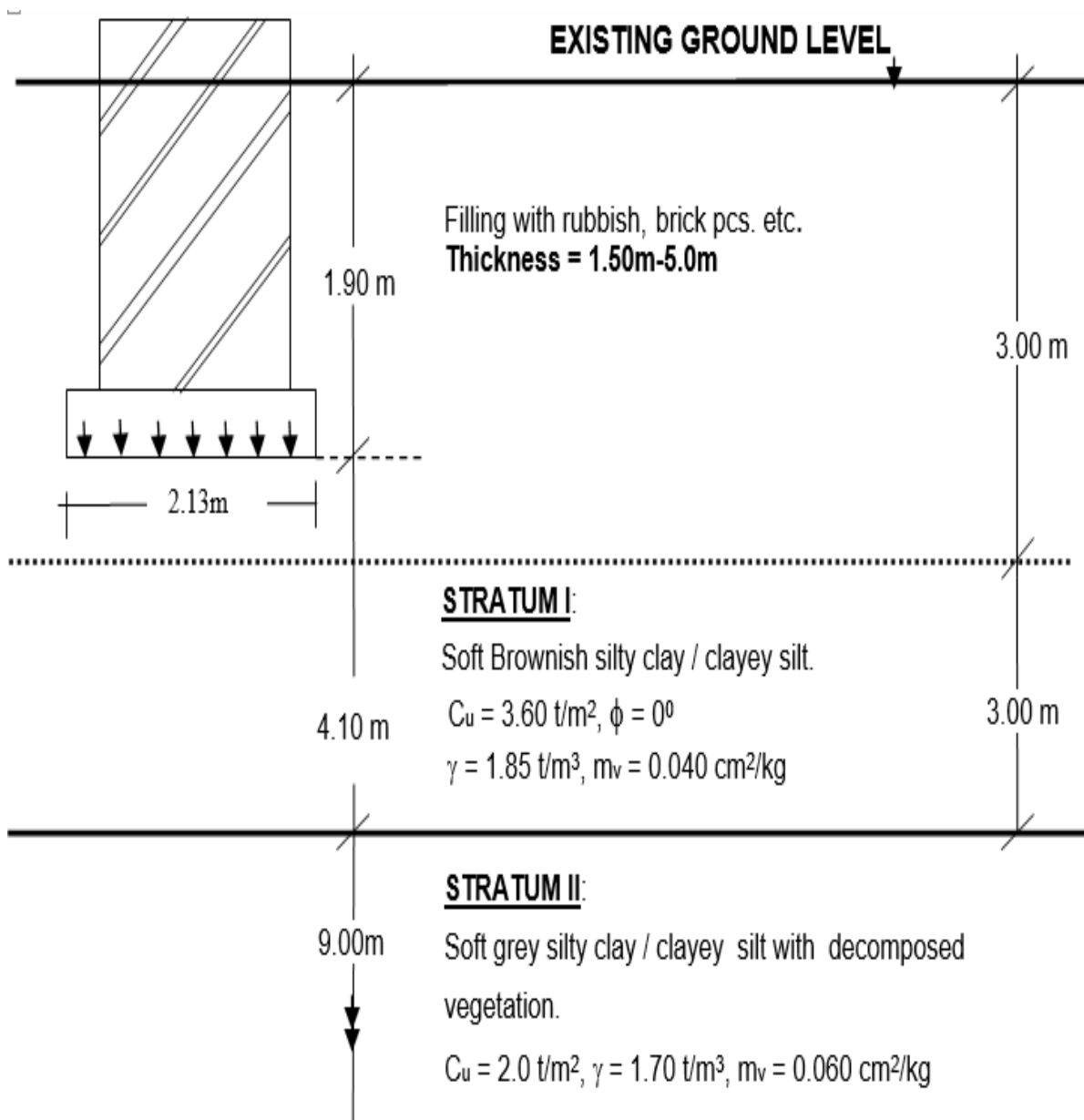


Fig. 6.20: Sub-soil Profile for Settlement calculations

### ***Settlement analysis of a typical foundation of the High Court building***

Considering the subsoil conditions and soil parameters as obtained from the soil investigation at the site settlement analysis of a typical strip foundation of width 2.13 m resting at a depth of 1.90 m (see Fig. 6.20) below existing ground level (reported to be found from excavation at a typical location of the building) has been done considering an average foundation pressure of 20 t/m<sup>2</sup>.

$$\text{Immediate settlement} = \frac{q B (1 - \gamma^2) I_\sigma}{E} \quad (6.9)$$

Where  $q$  = average foundation pressure = 20 t/m<sup>2</sup>

$B$  = width of foundation = 2.13 m (Strip)

$\gamma$  = Poisson's ratio = 0.5

$I_\sigma$  = influence factor = 2.0

$E$  = young's modulus = 500 x 3.6 = 1800 t/m<sup>2</sup>

So, Immediate settlement = 36 mm

$$\text{Consolidation settlement} = m_v \Delta P H \quad (6.10)$$

where  $m_v$  = coefficient of volume change = 0.004 m<sup>2</sup> / ton

$\Delta P$  = increase in overburden pressure at the centre of influence zone = 10t/m<sup>2</sup>

$H$  = thickness of influence zone = 4.26 m

So, consolidation settlement = 170 mm

In addition to this there was some secondary settlement as the High court building was constructed about 150 years back and its magnitude was estimated considering the ratio of  $c_\alpha/c_c$  = 0.04 (Koti Reddy et al. 2014) and the value of  $c_c$  obtained from laboratory testing was 0.25.

$$\Delta H = H c_\alpha \log\left(\frac{t_2}{t_1}\right) \quad (6.11)$$

Where  $c_\alpha$  = coefficient of secondary compression = 0.01

H = thickness of influence zone=4.26

t<sub>2</sub> = time period after end of construction=150years

t<sub>1</sub> = time for 95% consolidation in years = 3 years

So, secondary consolidation settlement = 72 mm

Thus, the total predicted settlement is about 278 mm and thus the secondary settlement was about 25% of the total settlement.

Again, using equation (1) and the relevant creep parameters, settlement due to creep has been estimated. The parameters considered are A=0.00052/min, reference time t<sub>1</sub>=1 min, m=0.70, the value of stress ratio D<sub>r</sub> has been considered as 3 as the average pressure acting on soil was 20 t/m<sup>2</sup> and the value of α has been taken as 0.001. Further, it may be noted that α reduces with time and thus the term e<sup>(αD)</sup> is equal to 1 and has less impact in settlement calculation as compared to A and m.

$$\dot{\epsilon}_a t = A t_1^m e^{(\alpha D_r)} t^{(1-m)}$$

$$\text{Strain } (\epsilon) = \dot{\epsilon}_a \times t = 0.00052 \times 1^{(0.70)} \times e^{(0.001 \times 3)} \times (150 \times 365 \times 24 \times 60)^{(1-0.70)} = 0.122$$

Therefore, settlement due to creep = ε x H

$$= 0.122 \times 4.26 \text{ m}$$

$$= 520 \text{ mm}$$

This settlement is about twice of the total settlement predicted based on primary and secondary consolidation and appears to be practically possible as the structure developed cracks in certain structural members over the years.

Using the relevant drained creep parameters the settlement due to creep may be estimated.

$$\epsilon = B e^{\beta D_r} (t/tr)^{(1-m)}$$

$$\epsilon = 0.4 \times e^{(1.82 \times 3)} \times (150 \times 365 \times 24 \times 60)^{(1-1.40)}$$

$$\epsilon = 0.065$$

Therefore, settlement due to creep =  $\epsilon \times H$

$$= 0.065 \times 4.26 \text{ m}$$

$$= 277 \text{ mm}$$

This settlement is close to value calculated considering immediate and both primary and secondary consolidation.

## 6.6 Inference

On the basis of above study following conclusions may be drawn:

1. For both inorganic and organic soils the initial increase in deformation with load increment is much less, within 2%, initially up to load increment corresponding to stress level of 0.8 – 0.9 and  $\sigma_d/p_o$  of 0.7 – 0.8, a part of which is attributed to the creep of soil.
2. In case of creep tests the average pore water pressure parameter, 'A'-value is 0.33 and 0.48 for inorganic soils and 0.21 and 0.35 for organic soils at stress level nearly 1.0 for effective confining pressure of 100kPa and 200kPa respectively. These are much less than the corresponding values of 0.82 and 0.67 for inorganic soils and 0.52 and 0.66 for organic soils obtained from conventional tests. This may be due to growth in bonding of the soils under sustained loading in spite of their creep deformation.
3. The three parameter phenomenological general function for soils expressing strain rate as a function of time after application of sustained loading was proposed by Singh and Mitchell (1969) was found to be more effectively applicable under undrained conditions
4. The creep potential parameters for inorganic and organic soils of Normal Kolkata Deposits under undrained conditions have been determined and the parameter 'm' suggested by Singh and Mitchell (1969) used to quantify creep potential is dependent on axial stress acting on the soil and its magnitude reduces with the increase in stress level due to application of load during construction.

5. For inorganic soils of Normal Kolkata deposit, geotechnical constructions for shallow foundations placed with a factor of safety less than 1.5 are susceptible to creep settlement. The creep susceptibility ceases for soils at a confining pressure of 200kPa, which corresponds to a depth close to 20m.
6. For organic soils of Normal Kolkata deposit, under undrained conditions the creep susceptibility is significant. During construction over such soft soils the effect of creep settlement should be considered for construction of foundations and embankments.
7. In case of drained creep test results a three parameter modified version of Singh and Mitchell equation has to be used. The parameter  $m$  was found to be greater than 1 in case of drained creep tests which suggests the possibility of creep is less.
8. The applicability of the creep parameters obtained from these tests were assessed with the help of a design example for a typical foundation of the High Court building resting over soft Normal Kolkata deposit.



**SINGLE LOADING UNDRAINED CREEP TRIAXIAL TESTS AND COMPARISON OF CONVENTIONAL, MULTISTAGE AND SINGLE STAGE LOADING**

**7.0 General**

The results of multistage triaxial creep testing presented in the previous chapter highlight that the effect of creep on clays of Kolkata region are more pronounced under undrained conditions. To further investigate the creep behaviour of the soil single loading triaxial creep tests were conducted on inorganic and organic soft clays of Kolkata region under undrained conditions and the results were compared with those obtained from multistage tests as well conventional triaxial test results.

**7.1 Test Results- single loading undrained tests**

The single stage loading tests were carried out under effective pressures of 50kPa and 100 kPa for both inorganic and organic clays. Further the magnitude of load applied on the soil specimen was varied with respect to the ultimate load capacity of the soil specimens. The ultimate capacity of the samples were estimated based on the confining pressure that the samples were subjected to. A load corresponding to 40%, 60%, 100% and greater than 100% was applied on the specimens and the responses were recorded. These loads correspond to a factor of safety of 2.5, 1.5, 1 and less than 1. The deformation responses are presented in Fig 7.1. The excess pore pressures generated post the application of load has been presented in Fig 7.2.

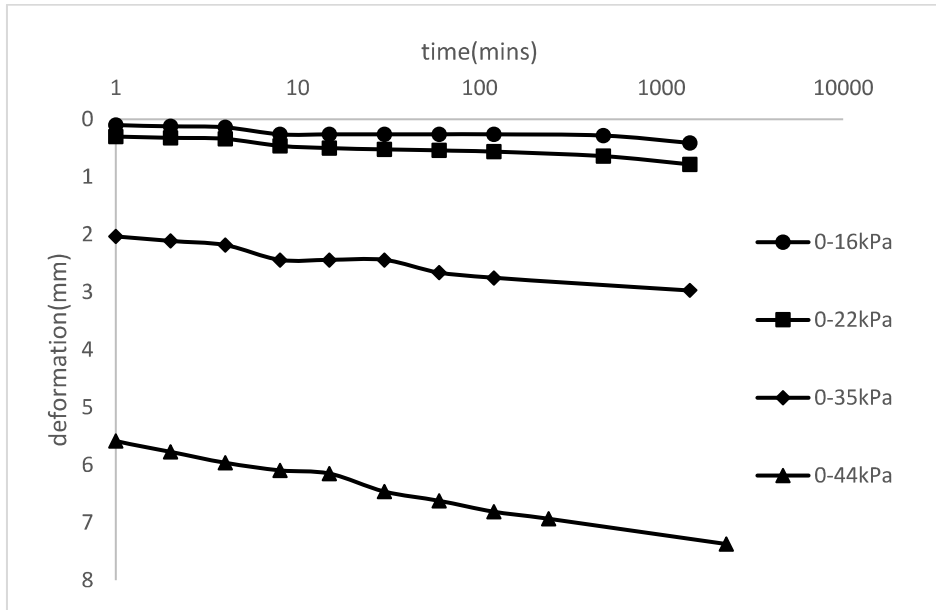


Fig 7.1a : Axial deformation vs time responses for Inorganic clay at effective confining pressures of 50kPa

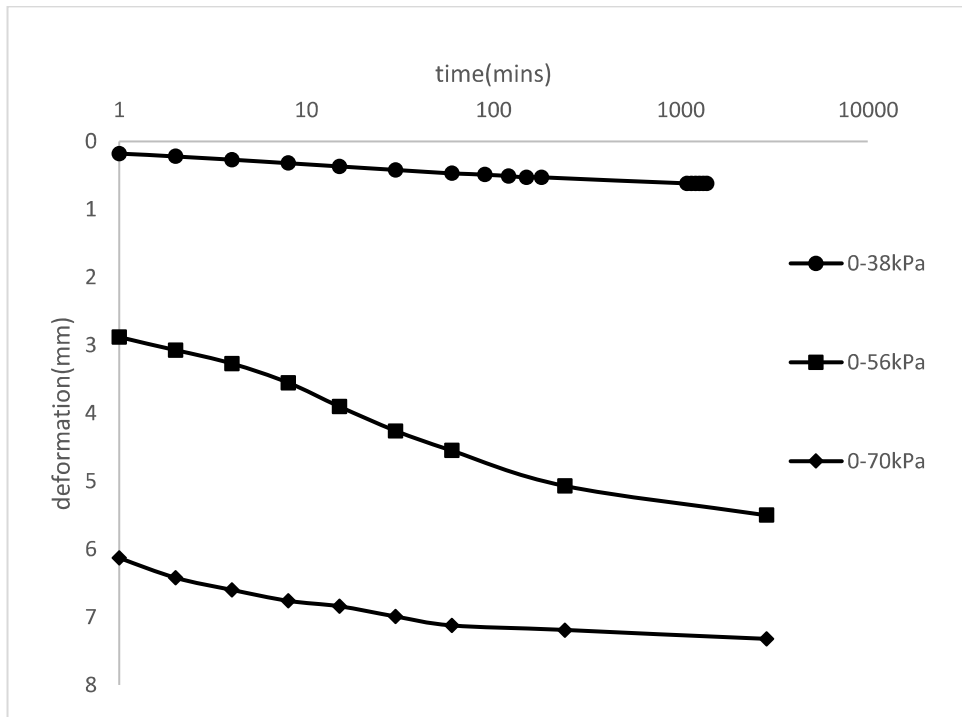


Fig 7.1b : Axial deformation vs time responses for inorganic clay at effective confining pressures of 100kPa

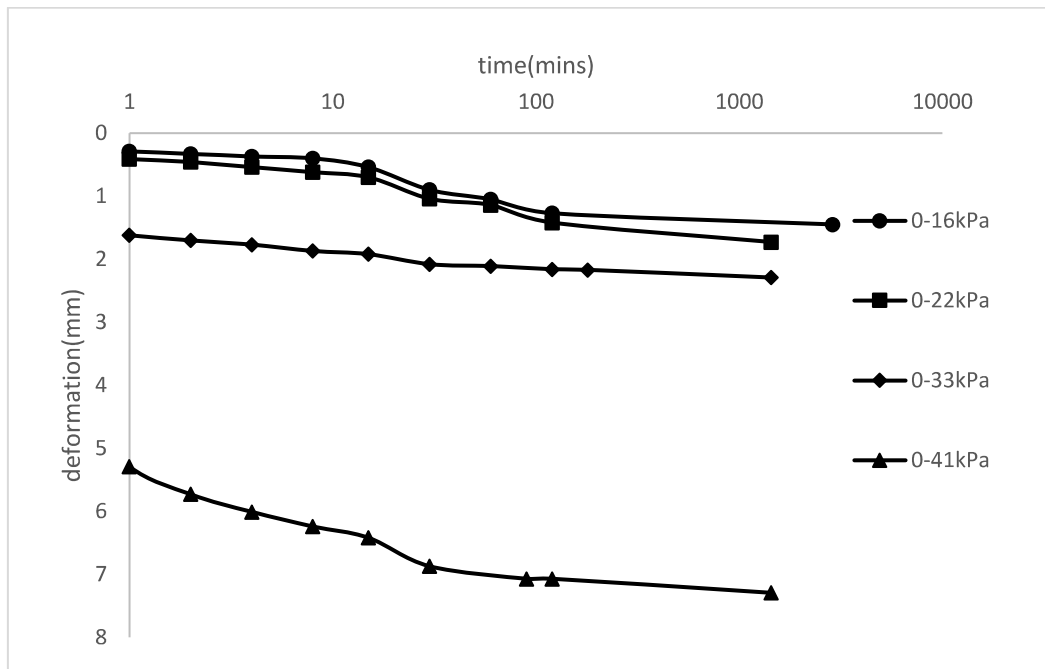


Fig 7.1c : Axial deformation vs time responses. Organic clay at effective confining pressures of 50kPa

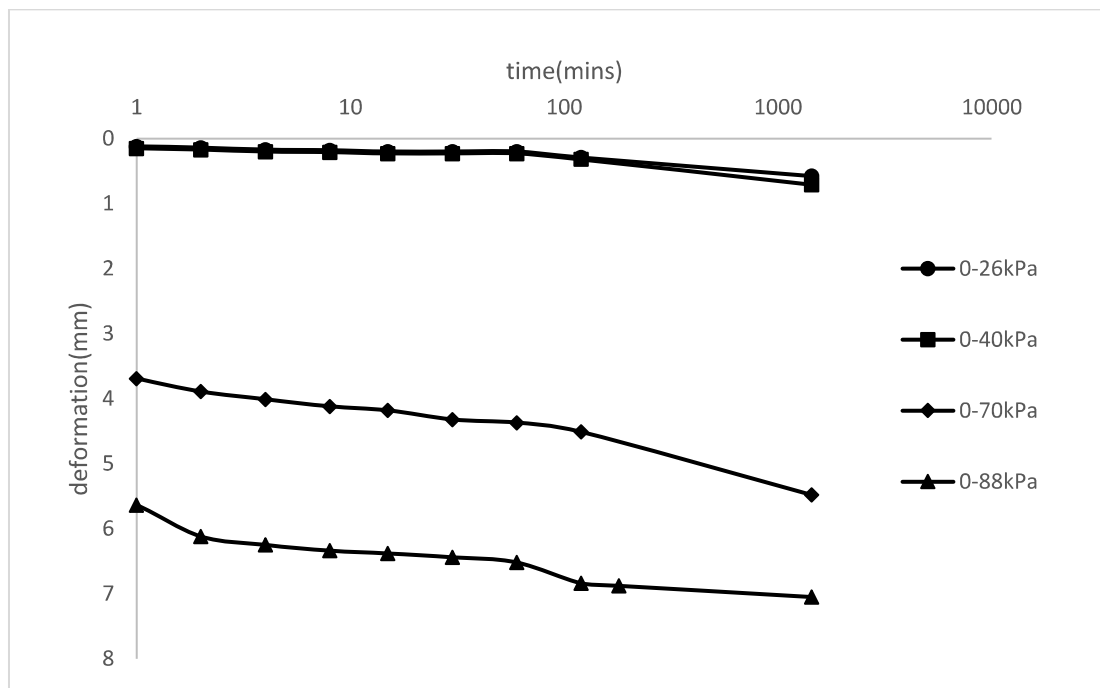


Fig 7.1d : Axial deformation vs time responses. Organic clay at effective confining pressures of 100kPa

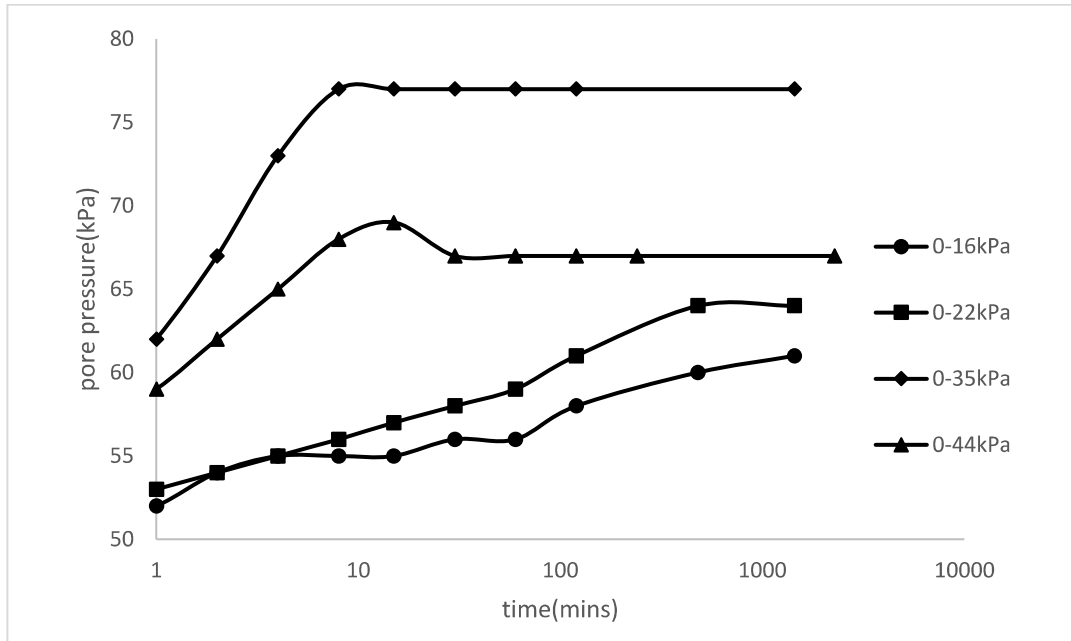


Fig 7.2 a : Excess pore pressure vs time responses Inorganic clay at effective confining pressures of 50kPa

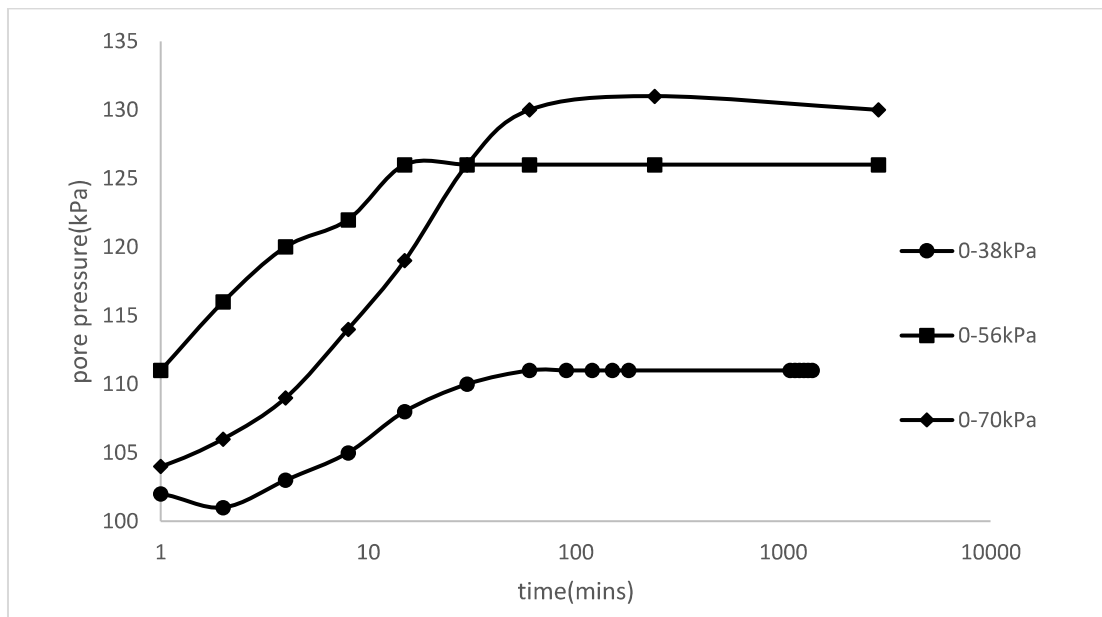


Fig 7.2 b : Excess pore pressure vs time responses Inorganic clay at effective confining pressures of 100kPa

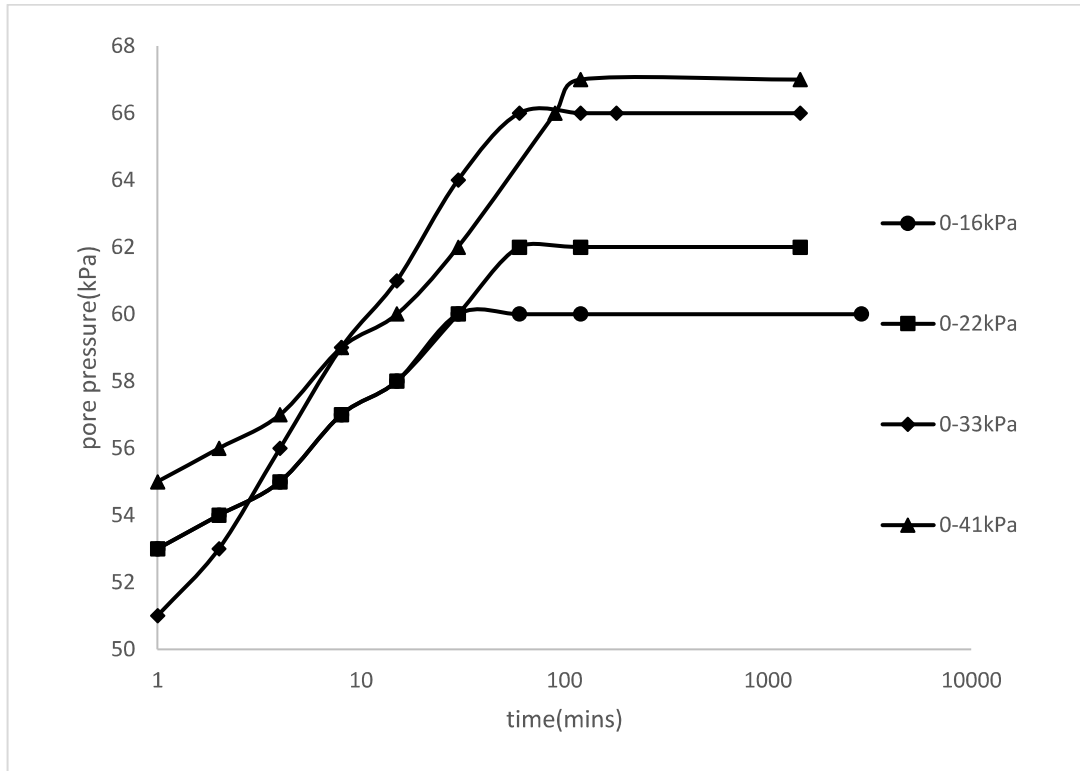


Fig 7.2 c : Excess pore pressure vs time responses Organic clay at effective confining pressures of 50kPa

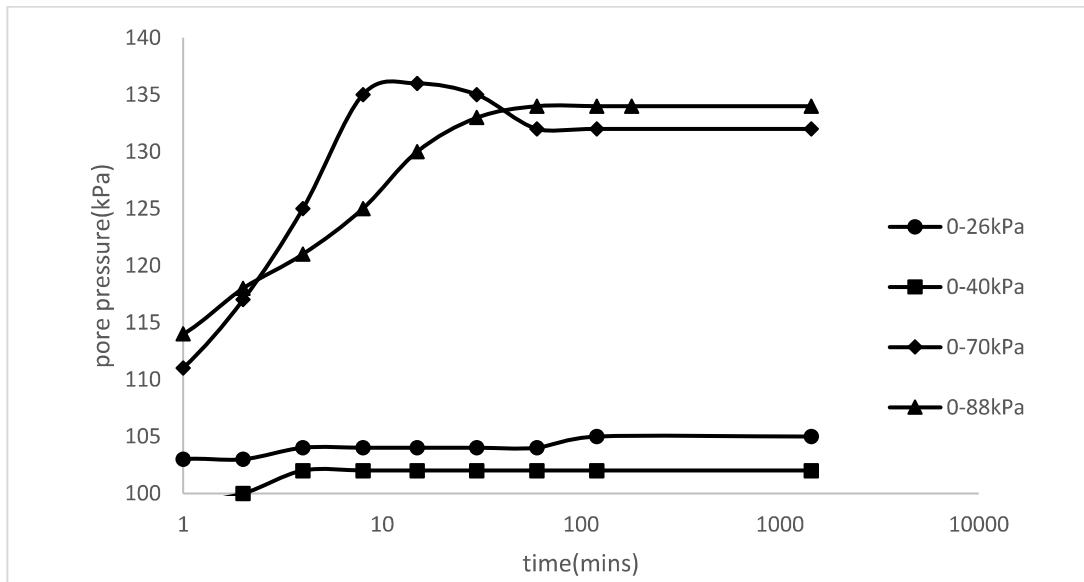


Fig 7.2 d : Excess pore pressure vs time responses Organic clay at effective confining pressures of 100kPa

The stress-strains response of the inorganic and organic clays are presented in Fig 7.3 by considering the peak stress-strain response obtained from the individual loading results.

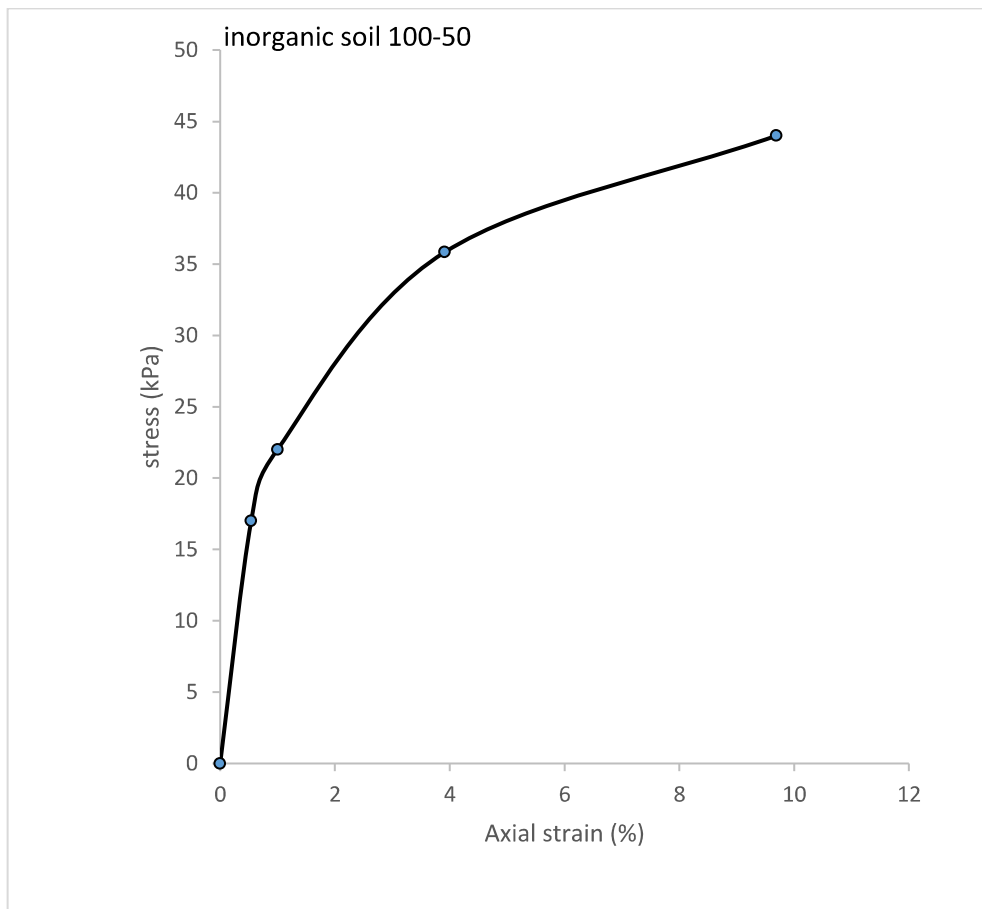


Fig. 7.3 a : Stress-strain response of Kolkata clay under single stage creep loading  
Inorganic clay at effective confining pressures of 50kPa

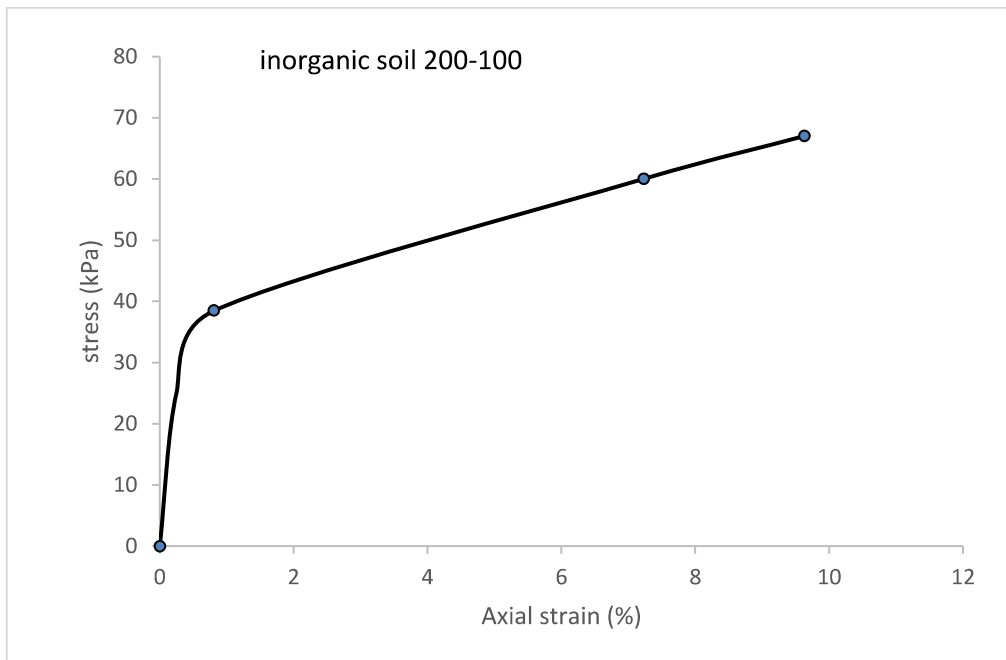


Fig. 7.3 b : Stress-strain response of Kolkata clay under single stage creep loading  
Inorganic clay at effective confining pressures of 100kPa

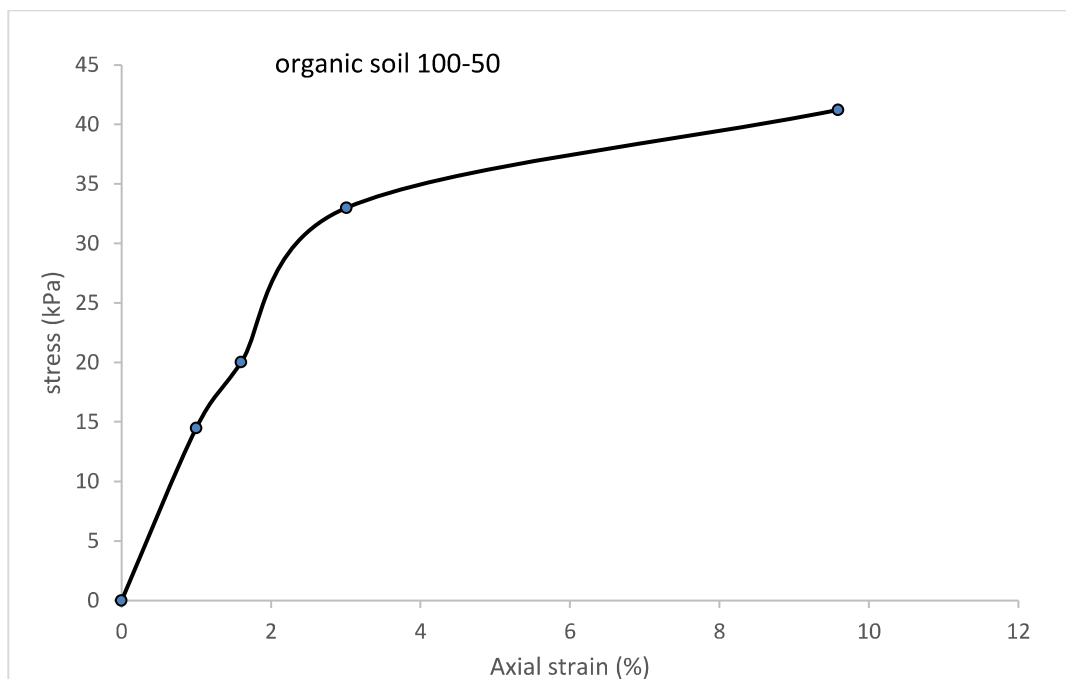


Fig. 7.3 c : Stress-strain response of Kolkata clay under single stage creep loading Organic  
clay at effective confining pressures of 50kPa

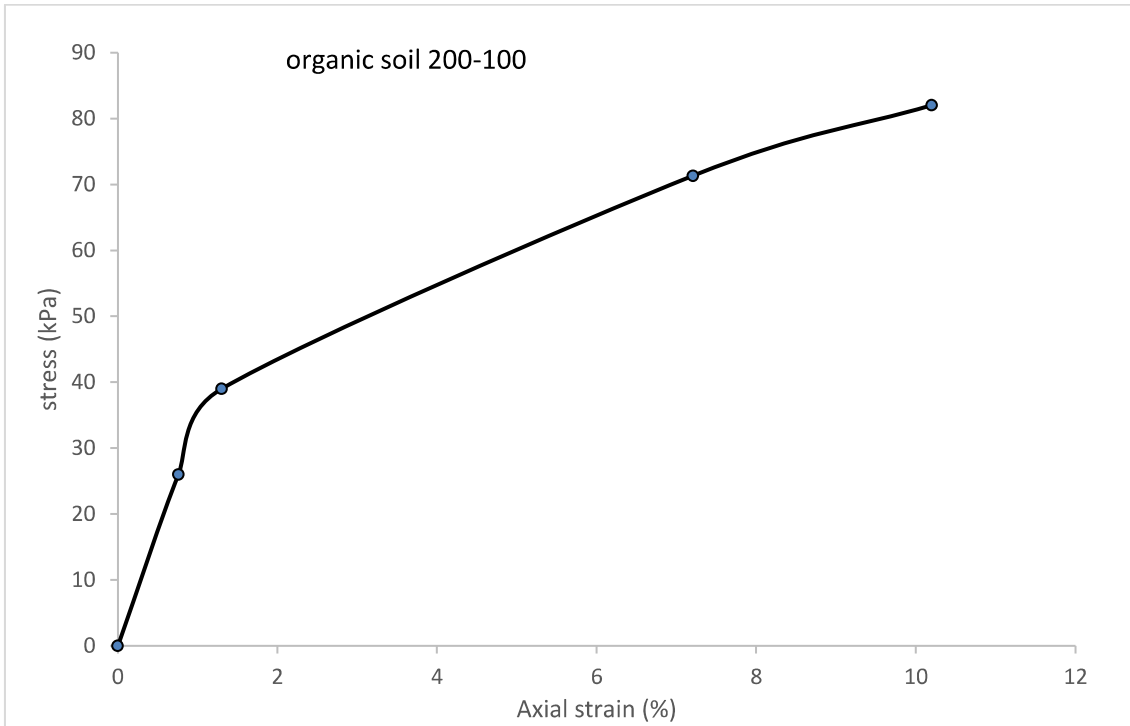


Fig. 7.3 d : Stress-strain response of Kolkata clay under single stage creep loading Organic clay at effective confining pressures of 100kPa

The parameter  $m$  defined by Singh and Mitchell (1969) is calculated using the axial strain vs time response presented in Fig 7.4. The results are tabulated in Table 7.1

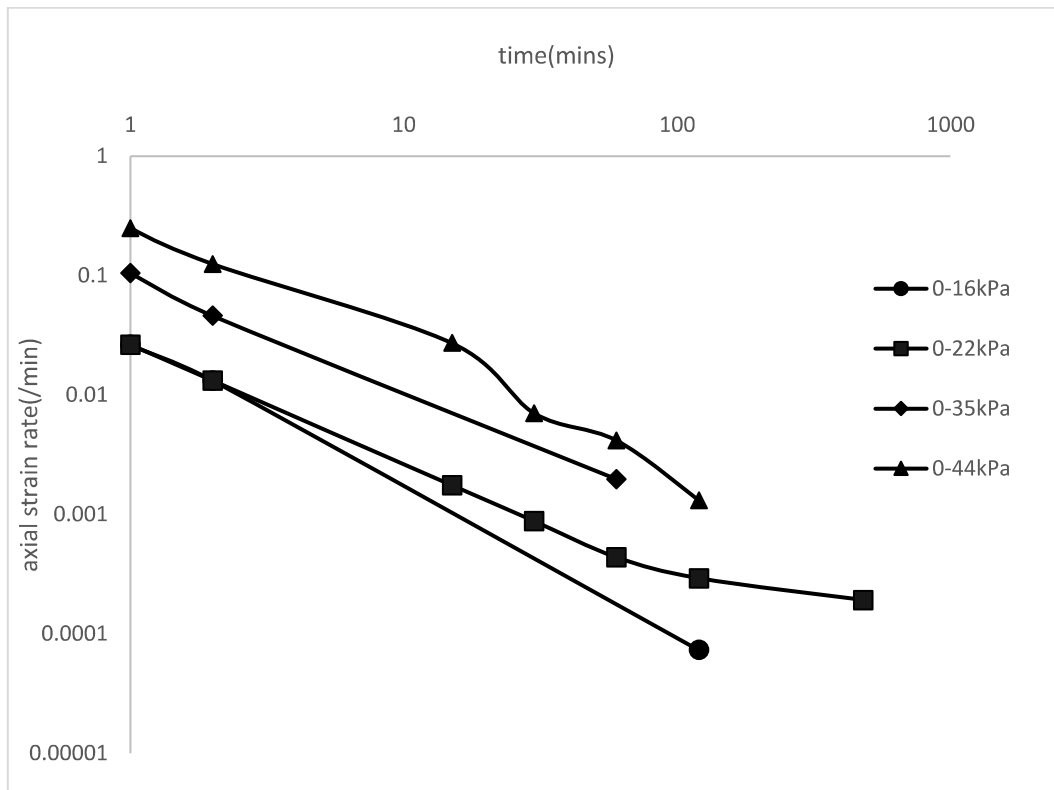


Fig 7.4 a: Axial strain vs Time response Inorganic clay at effective confining pressures of 50kPa

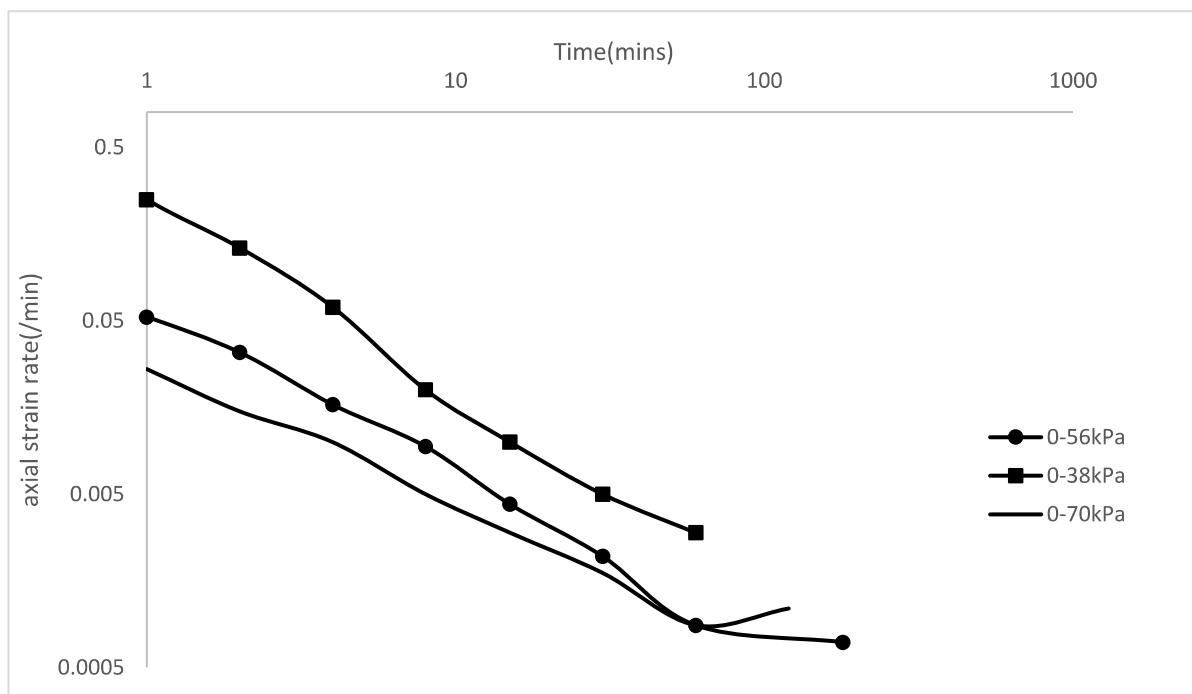


Fig 7.4 b: Axial strain vs Time response Inorganic clay at effective confining pressures of 100kPa

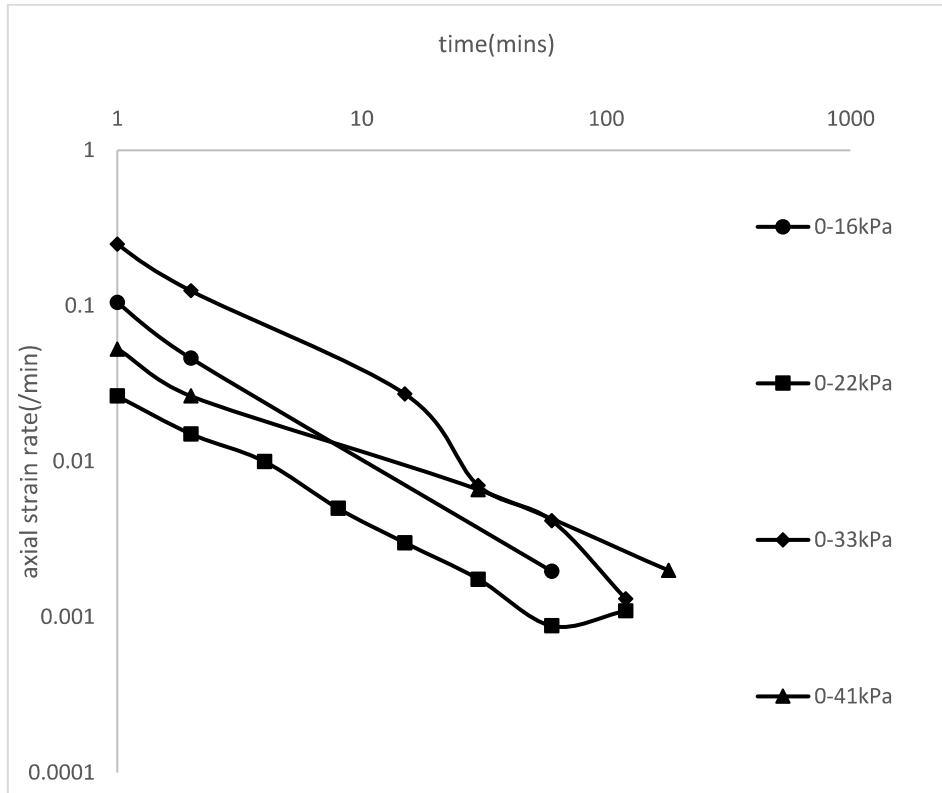


Fig 7.4 c: Axial strain vs Time response Organic clay at effective confining pressures of 50kPa

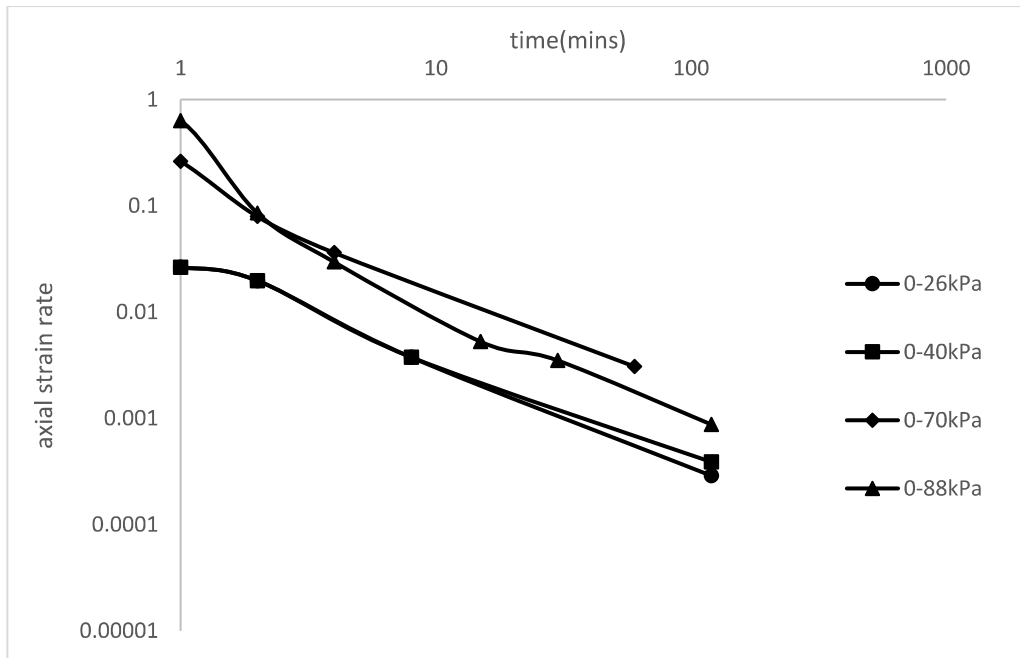


Fig 7.4 d: Axial strain vs Time response Organic clay at effective confining pressures of 100kPa

Table 7.1 Creep potential parameter m

Soil	Confining Pressure (kPa)	Load Applied (%)	m
Inorganic clay	50	40	1.23
		60	0.99
		100	0.97
		150	0.82
Inorganic clay	100	40	1
		60	0.83
		100	0.81
Organic clay	50	40	0.92
		60	0.83
		100	0.76
		150	0.51
Organic clay	100	40	1.11
		60	1.01
		100	0.9
		150	0.83

It is observed that the values of m in case of single loading undrained tests are greater than the corresponding multistage loading tests. The parameter m is a measure of creep potential of the soil, the conditions under which the values of m are close to or less than 1 is an indication of possibility of creep occurring under those conditions. This indicates that the possibility of creep under single loading undrained tests is lesser as compared to multistage loading tests. This may

be owing to the fact the under one-time loading especially when the samples are subjected to higher stress levels the failure occurs far quicker than the time required under multistage loading thereby preventing the possibility of particle rearrangement. Therefore the phenomenon of creep does not develop under this process of testing.

To further understand how the creep responses are generating, the comparison of behaviour of the soil under and conventional, single and Multistage Loading Triaxial tests are presented in the following section.

## **7.2 Comparison between conventional, Single and Multistage Loading Triaxial tests**

### **7.2.1 Strength Characteristics**

The following figures Fig 7.5 to Fig 7.8 represents a comparative plot of conventional consolidated undrained (CU) triaxial test, single stage creep triaxial test and multistage creep triaxial test. The stress-strain response of both inorganic and organic clays of Kolkata soil are similar under conventional CU test and single load creep tests taking the peak value of individual tests as presented in Fig 7.3. There is a gain in strength in case of multistage creep triaxial tests. The value of peak deviator stress as obtained from CU tests, single stage and multistage loading tests for inorganic clays under effective confining pressure of 100kPa are 77kPa, 70kPa and 134kPa respectively. Thus the effect of gain in soil strength is only evident in case of multistage creep triaxial tests. The same phenomenon is evident from results of organic clays where the peak deviator stress recorded were 106 kPa, 90kPa and 209kPa respectively for CU tests, single stage and multistage loading tests. The strength gain in case of multistage loading is significant.

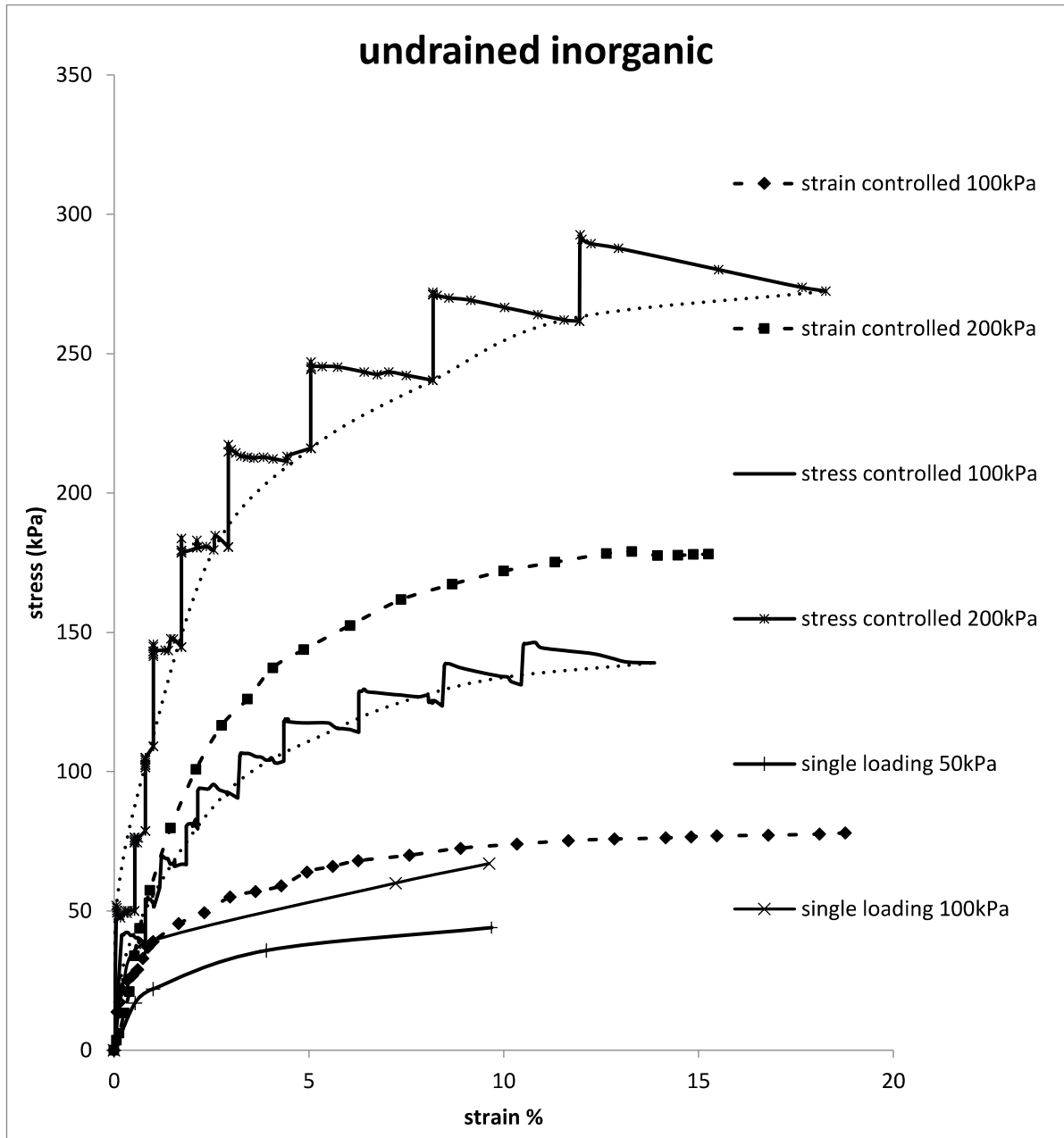


Fig 7.5 : Axial stress-strain response comparison for inorganic clays

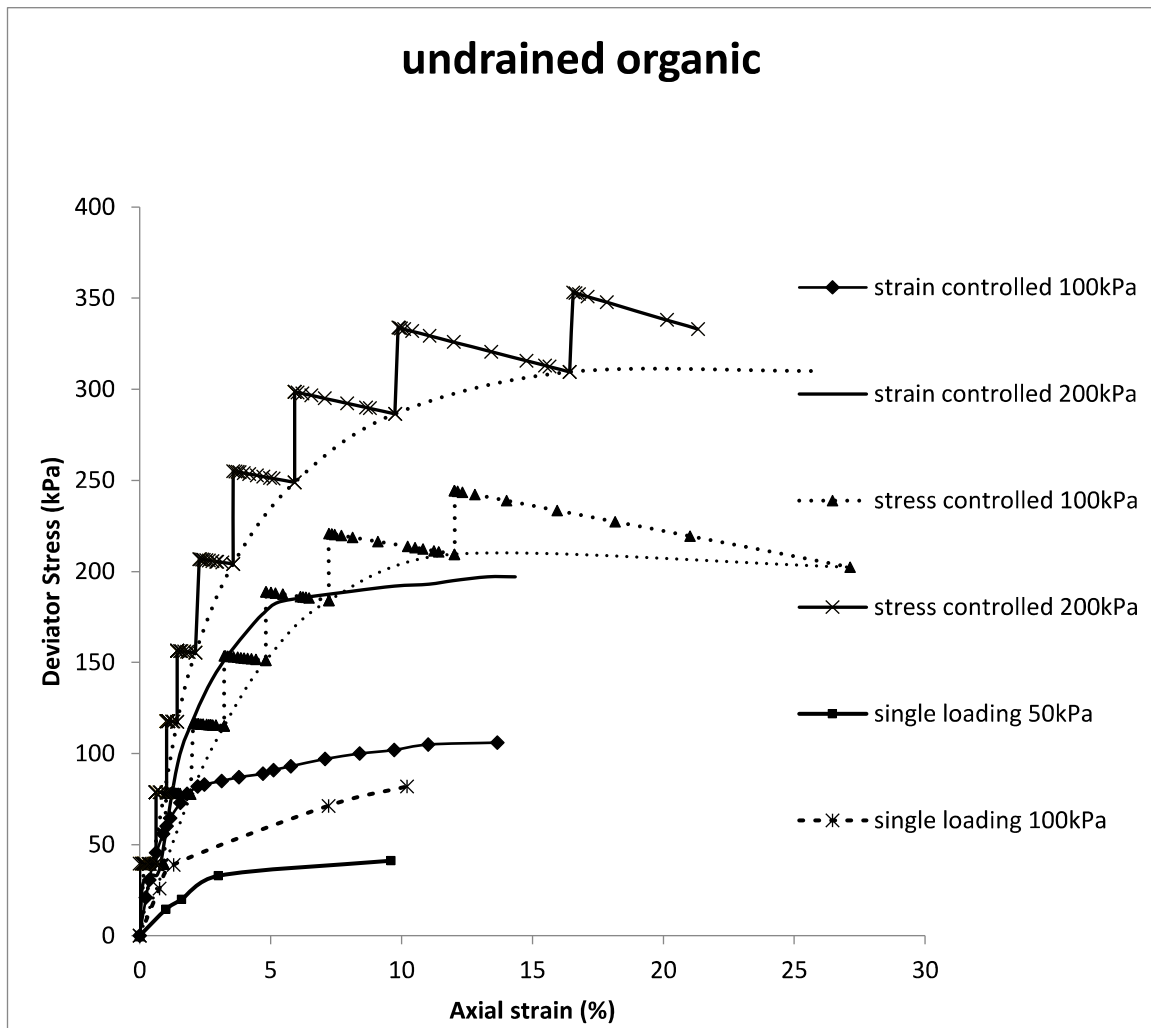


Fig 7.6 : Axial stress-strain response comparison for organic clays

The pore pressure responses are presented in Fig 7.7 and Fig 7.8. The pore pressure generated in case of single stage loading tests is lower than corresponding multistage and CU test responses. The failure takes place earlier in case of one-time loading and is a reason why pore pressure does not generate in the order and magnitude evidenced during multistage loading.

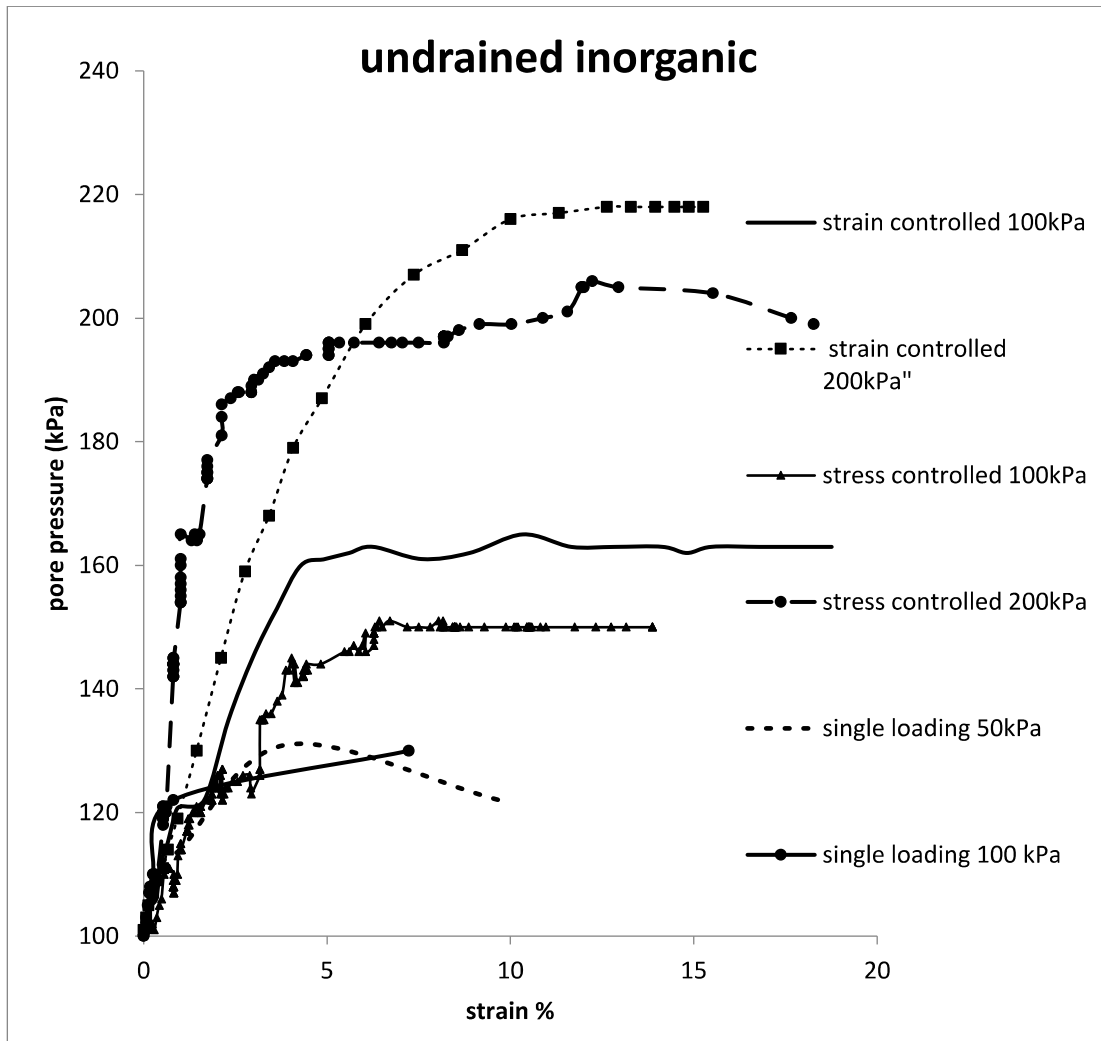


Fig 7.7 : Pore pressure response comparison for inorganic clays

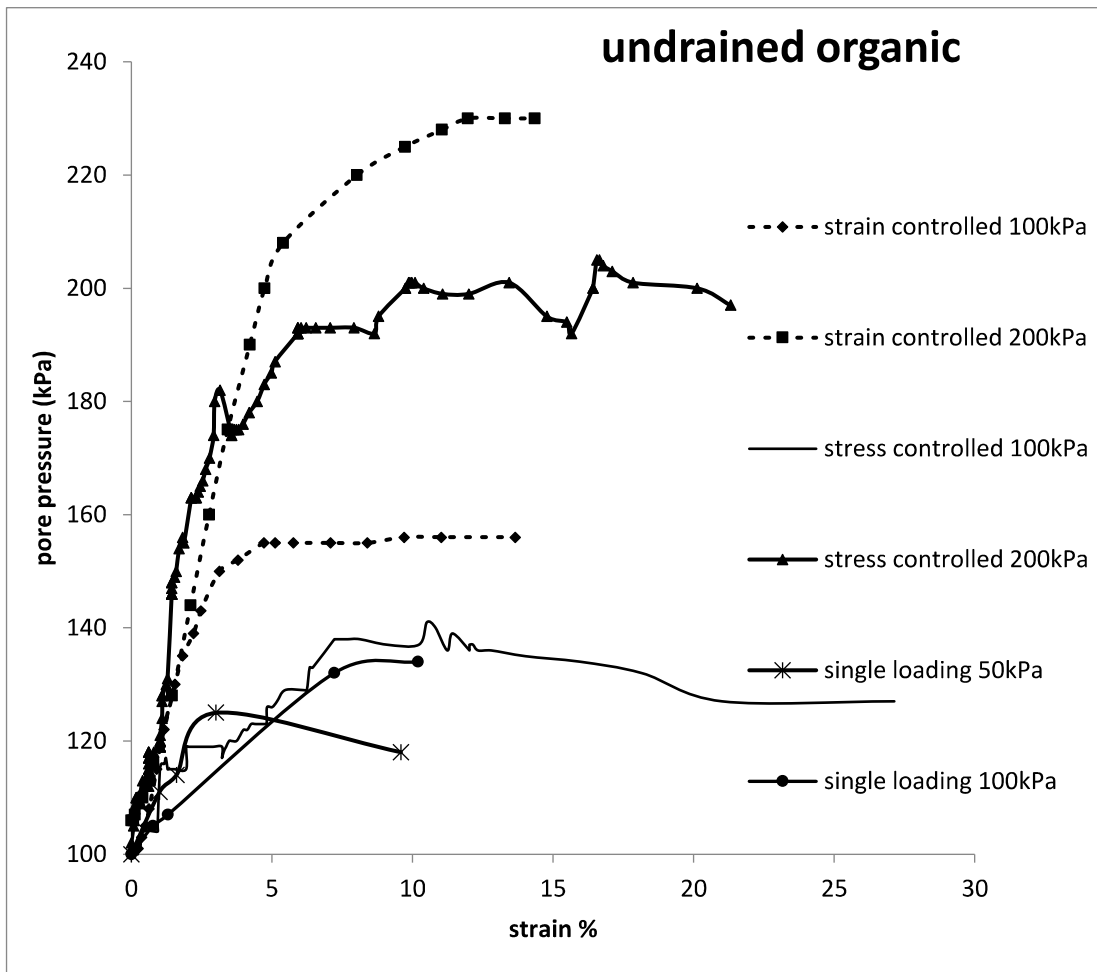


Fig 7.8 : Pore pressure response comparison for organic clays

### 7.2.2 Deformation Characteristics

The comparison of deformation behaviour of inorganic and organic clays under different testing conditions has been presented in Fig. 7.9 and Fig. 7.10 respectively. From the experimental results it is observed that the deformation in case of single stage creep tests occurs at a rapid rate post loading as compared to equivalent load application during multistage loading. The ultimate deformation in case of multistage loading is the maximum whereas for single stage loading it is the least amongst the three tests. In single stage loading more than 90% of the ultimate settlement is obtained within two hours of loading. The pore pressures developed is the least which is an indication of the fact that in case of single loading the pore pressure dissipates very quickly and in-turn causes significant settlement in a short span of

time. In case of multistage loading the time-dependent phenomenon is dominant and settlement magnitude post loading is significantly slower than single loading tests.

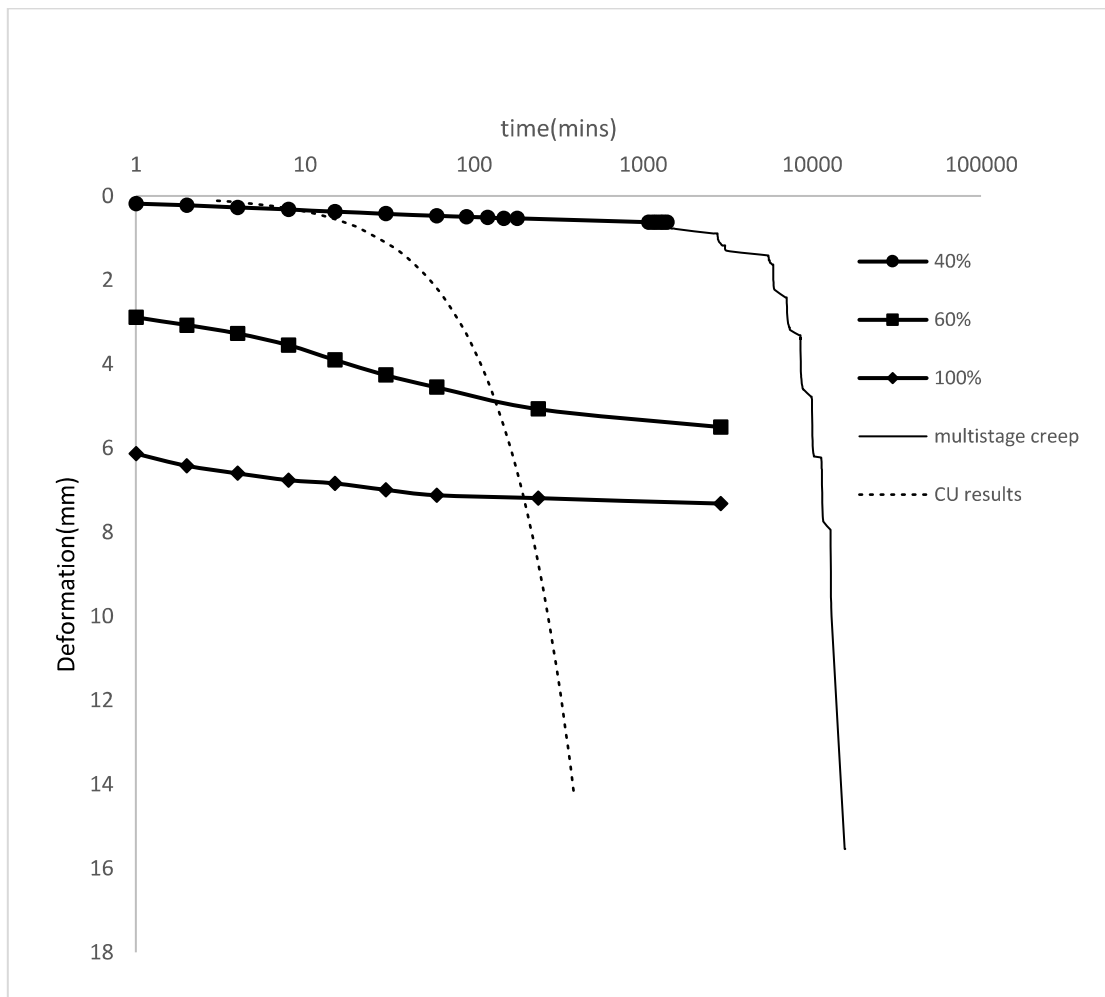


Fig 7.9 : Deformation response comparison for inorganic clays

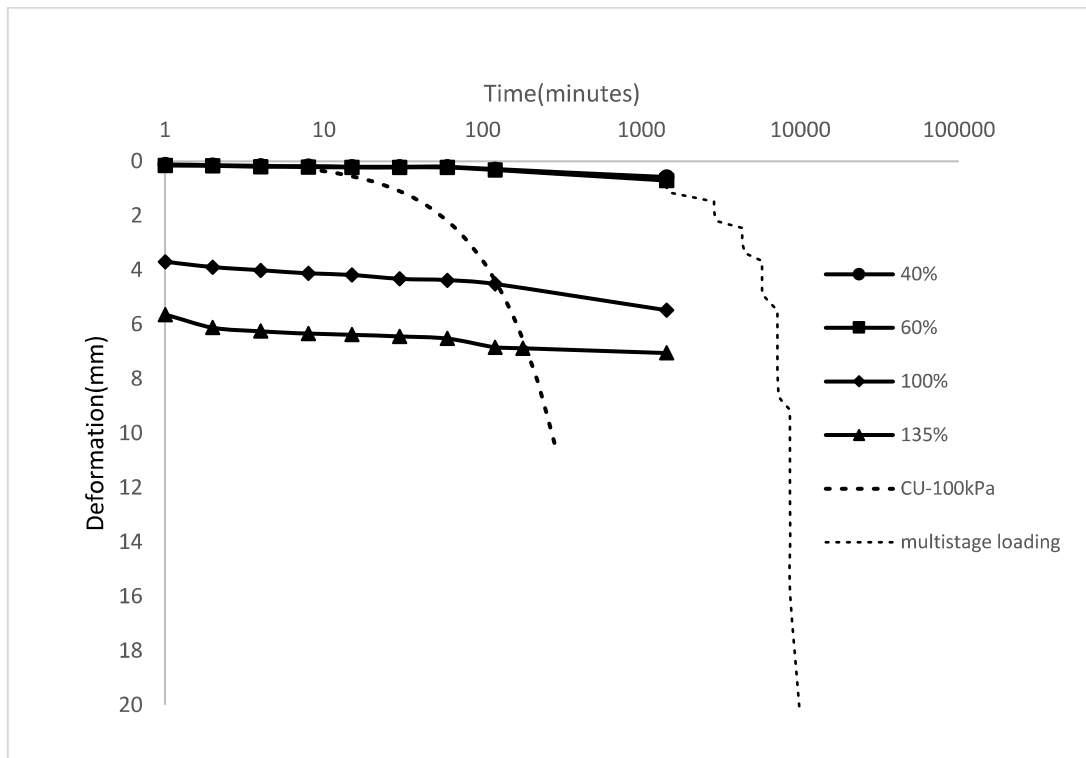


Fig 7.10 : Deformation response comparison for organic clays

### 7.3 Inference

1. For single loading undrained tests, post loading deformation occurs at a rate significantly higher than the settlement rate observed during multistage loading tests. Close to 90% of the ultimate deformation is achieved within 2 hours of loading and thus time-dependent deformation is negligible. For multistage loading time-dependent deformation is pronounced but the overall settlement does not exceed the settlement recorded during single loading creep tests.
2. In case of undrained multistage loading tests the soil gains significant strength but in case of single loading tests the results are similar to strain controlled tests and additional soil resistance does not develop. Thus application of load in stages can result in significantly increased capacity of soil mass.
3. The pore pressure generated during single stage loading are lesser compared to corresponding multistage loading and conventional triaxial tests.

**SUMMARY AND CONCLUSIONS****8.0 Summary**

The safety of any construction depends significantly on the type of foundation/substructure it rests on. The nature of existing sub-soil conditions at a particular construction site plays a crucial role in the choice of foundation. One of the most important properties of soils is its shear strength. There is lack of published data investigating the shear strength and creep behaviour of soft soils of this region despite the presence such detailed investigations on other clays across the globe such as London clay, Hong Kong marine deposits, Boston Blue clay, Pancone clay of Pisa, Norwegian clay. In this study a laboratory investigation to evaluate creep characteristics and shear strength parameters ( $c$ ,  $\phi$ ) of remoulded (artificially consolidated) inorganic and organic clay using triaxial compression and multistage triaxial creep tests was conducted. An attempt has been made to investigate the variations in magnitude of shear strength parameters ( $c$ ,  $\phi$ ) due to difference in testing procedures and drainage conditions in order to predict the realistic and most appropriate ( $c, \phi$ ) values which should be used for evaluating soil strength for construction. An estimation of the creep potential, creep parameters of soft clays of Normal Kolkata Deposit was carried out.

**8.1 Conclusions**

1. The effect of soil structure or soil fabric on the mechanical behaviour of clays of Normal Kolkata region were examined. It was inferred that for natural and remoulded clays of Kolkata region the compression behaviour and parameters governing compression behaviour are similar and that the effect of soil fabric in case Kolkata clays is not prominent at all. This was the basis for selecting remoulded soil for examination of

shear strength parameters using different loading mechanisms as it gave greater control and uniformity in sample preparations.

2. The shear strength determination of a soil is dependent on the method of application of loading during testing. In case of strain-controlled tests, maximum deviator stress obtained from drained tests are higher than that from undrained tests for both inorganic and organic clays. However, friction angle obtained from undrained tests are quite higher than that obtained from drained tests,  $4-6^\circ$  and  $5-8^\circ$  for inorganic and organic clays respectively. The maximum deviator stress and also effective friction angle obtained from stress-controlled undrained tests are higher than those obtained from strain-controlled undrained tests for both inorganic and organic clays. For drained tests no significant difference in the values of effective friction angle is observed. The shear strength parameters adopted for projects is determined using strain-controlled triaxial laboratory tests whereas the actual reality at site conditions is that the load application mechanism is stress-controlled. Thus according to the findings of this study an additional factor of safety is induced automatically in selection of strength parameters.
3. Investigations regarding the creep characteristics of Normal Kolkata Deposit revealed that under undrained conditions the effect of time dependent evolution of behaviour was evident for inorganic and organic clays. The creep potential calculated using Singh and Mitchell 1969 parameters is indicative of the same. Under drained conditions the effect of creep is not significant. In practical scenarios clayey soil subjected to constant loading over a significant period time will eventually allow drainage, even if it is at a very slow rate. Thus under such circumstances the effect of creep will not be a major factor in assessing the behaviour of Kolkata clays. However, lack of drainage may lead generation of pore pressure resulting in a scenario the effect of creep will play a significant role in determining the soil response to the applied loading.

## **8.2 Scope of further research**

1. Conducting further studies under lower confining pressures would bring further clarity to the time-dependent behaviour of Normal Kolkata Deposits.
2. There is a scope for further research using different soil samples of the region having varying quantity of organic content, specially organic content in the range of 5-6% and 15-16% to consolidate the findings.
3. More data would help generalize the overall behaviour and allow better and accurate predictions.
4. A numerical model specific to Kolkata soil behaviour can be developed by compiling larger sets of data.



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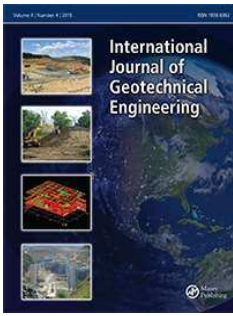
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## Strain- and stress-controlled shear strength behaviour of Kolkata clays

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To cite this article: Atriya Chowdhury & Ramendu Bikas Sahu (2021): Strain- and stress-controlled shear strength behaviour of Kolkata clays, International Journal of Geotechnical Engineering, DOI: [10.1080/19386362.2021.1902646](https://doi.org/10.1080/19386362.2021.1902646)

To link to this article: <https://doi.org/10.1080/19386362.2021.1902646>



Published online: 01 Apr 2021.



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## Strain- and stress-controlled shear strength behaviour of Kolkata clays

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### ABSTRACT

Shear strength of soil plays a crucial role in the estimation of bearing capacity of foundations and also for assessing the stability of slopes, dams and embankments, excavations, retaining structures and highway pavements. In this paper strain-controlled and stress-controlled consolidated undrained and drained triaxial tests were conducted on remoulded inorganic and organic clay available in Kolkata to examine and understand the variations in shear strength parameters due to testing procedures. The results show that the maximum deviator stress is higher for drained tests than that of undrained tests whereas the effective friction angle in case of undrained tests is greater than the drained tests by 4° to 6° for inorganic clays and 5° to 8° for organic clays. However, the effective friction angle in the case of stress-controlled tests are significantly higher than those obtained during strain-controlled tests for both inorganic and organic clays under-undrained conditions only.

### ARTICLE HISTORY

Received 26 November 2019  
Accepted 9 March 2021

### KEYWORDS

Strain-controlled tests; stress-controlled tests; Kolkata clay; consolidated drained triaxial tests; consolidated undrained triaxial tests; effective friction angle; shear strength

### Introduction

Shear strength of soil plays a significant role in the estimation of bearing capacity required for designing the foundation of structures, stability of slopes, embankments and dams, lateral earth pressure on retaining structures, and also design of excavations and highway pavements. It can be defined as the inherent capacity of the soil mass to resist failure under the action of external forces. Soil derives its shear strength primarily due to three aspects, viz., interlocking of particles, frictional resistance between individual soil grains and cohesion, that is, adhesion between soil particles. Over the year researchers have extensively studied the shear strength of soils. In 1776, Coulomb expressed the failure strength of soil in terms of two variables, namely the cohesion ( $c$ ) and angle of internal friction ( $\phi$ ). Otto Mohr, later in 1882, developed a graphical method for analysing stress known as Mohr's circle and using it proposed an early theory of strength based on shear stress. The Mohr-Coulomb failure criterion represents the linear envelope obtained from a two-dimensional plot of the shear strength of a material versus the applied normal stress. When the applied shear stress exceeds the shear strength of the soil, the soil is considered to have failed. Subsequently, several other researchers Terzaghi (1942), Henkel (1960), Bjerrum and Simons (1960), Bishop (1966) and others carried out notable researches investigating the strength of soil. The shear strength of partially saturated/unsaturated soils were investigated by Bishop (1959), Aitchinson (1960), Jennings (1960), Sridharan (1968), Fredlund and Morgenstem (1977). These authors suggested various parameters to modify the relationship between shear strength and effective stress for unsaturated/partially saturated soils.

The shear strength of soil can be determined using laboratory tests such as unconfined compression tests, direct shear

tests, and triaxial tests. Vane shear tests are also used to determine the shear strength of very soft/soft clays in both field and laboratory. Amongst these, triaxial tests provide close simulation of the stress conditions present in the ground and thus the shear strength parameters are more realistic. Consolidated drained (CD) and consolidated undrained (CU) triaxial tests were introduced to demonstrate the principle of effective stress and for determination of effective shear strength parameters (Skempton and Bishop 1950; Bishop 1950; Skempton 1960). The CU triaxial tests are generally used for behaviour of clays under-undrained conditions to assess the stability of slopes, foundations, retaining walls, and excavations. The CD test results highlight the long-term behaviour of clayey soils where drainage takes place. The CU test is a more popular triaxial test as the undrained shear strength in addition to the effective shear parameters can be calculated using these tests. For clayey soils, owing to the very low coefficient of permeability of clays, the samples need to be sheared very slowly to allow the pore pressure to develop more or less uniformly throughout the sample. In this context, it may be noted that the effect of the rate of strain on mechanical properties of soils can have significant implications in the design of various engineering systems as they generally experience strain rates in the range of  $10^{-2}$  and  $10^{-3}$ %/hr (Bjerrum 1972; Holtz, Chameau, and Prapaharan 1989).

The conventional strain-controlled undrained and drained triaxial tests are conducted in the laboratory to determine the shear strength parameters for analysis and design of foundations, embankments, dams, excavations, etc. Theoretically, the effective stress parameters obtained from CD and undrained tests should be identical, but practically differences in results have been reported by various researchers. Bjerrum and Simons (1960) reported



# Creep potential of soft clays of Normal Kolkata Deposit

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Received: 19 February 2021 / Accepted: 21 June 2021 / Published online: 8 July 2021  
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## Abstract

Creep behaviour of clayey soils across the globe has been extensively studied and documented indicating their detrimental effects on the stability of structures, embankments and dams, excavations, etc. using a large number of case studies since the middle of last century at various locations all over the world. A number of similar cases were also reported on distressing/failure of structures in and near Kolkata due to the presence of soft clay in the upper region. The creep/long-term behaviour of soft inorganic and organic clays of Normal Kolkata Deposits has been examined in this paper by conducting multistage creep triaxial tests on artificially consolidated soil specimens under undrained conditions. The creep parameters developed by Singh and Mitchell have been calculated to quantify the creep potential of soft clays of Kolkata. The results establish that the organic layer of the Normal Kolkata soil is vulnerable to creep. A design example has also been presented to estimate the overall settlement of an old building in Kolkata using these parameters.

**Keywords** Normal Kolkata Deposits · Creep potential · Creep triaxial tests · Soft clays

## Introduction

The time-dependent deformation of clays is an important phenomenon in the analysis and design of structure constructed with or over soil. Soft clays are found in many areas of Scandinavia, Mexico, Japan, China, Northern United States and Canada, India, and Southeast Asian countries. Soft clays are generally recent sediments laid down by sea (marine), river (alluvial), or lakes (lacustrine). Most river valleys of the world have abundant deposits of soft clays of alluvial origin. The Mississippi in the USA, the Nile in Egypt, Yangtze in China, Euphrates and Tigris in Iraq, Lancang River in China and Southeast Asian countries, and Ganges-Brahmaputra in India are examples of alluvial deposits. Marine clays are found along coastal plains of the world. Soft clays are fine-grained soils with moderate to high clay fraction (30%) having high compressibility and shear strength

less than 25 kPa (Som 1989). For structure resting over soft clayey soils, deformation consists of two components: consolidation which occurs due to expulsion of pore water and creep which occurs after or during primary consolidation due to reorientation of soil particles.

For completed structures and man-made or natural slopes, magnitude and rate of deformation due to creep depend upon a number of factors like mobilized shear stresses, properties of soil, and its variation with space and time. Over the years, there have been plenty of instances where creep of soil has led to instability or failure of a structure. Skempton (Skempton 1964) presented detailed discussions and analysis of landslides caused due to creep of soils which led to failure of retaining wall at Kensal Green in 1941, Sudbury Hill failure in 1949, and Northolt Station in 1955. Vyalov (Vyalov 1986) discussed the effect of soil creep on dams, retaining walls, intake structure of hydropower project, sheet pile walls, and a reinforced concrete arched viaduct of an abutment. These are cases which highlight the effect of horizontal load on structures induced due to creep which led to failure of the structure in certain instances and in others the structure was damaged. A classic example of creep settlement is the Leaning Tower of Pisa. The long-term settlements of San Jacinto Monument, Texas (Briaud et al. 2007), and plain earth dams of the Kakhovka hydropower station on the Dnieper resting on clayey silts (Karpyshev et al. 1972) are some examples of vertical load-induced creep settlements. Bjerrum (Bjerrum 1967)

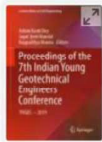
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Responsible Editor: Zeynal Abiddin Erguler

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## Long-Term Deformations of Soft Clays: A Case Study

[Atriya Chowdhury](#)Conference paper | [First Online: 17 March 2022](#)

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### Abstract

Any material subjected to constant loading will deform in due course of time. The magnitude of time-dependent deformation depends upon the properties of individual materials. Creep is the time-dependent deformation phenomenon experienced in soft soils depending upon their consistency and magnitude of superimposed load. Clayey soils exhibit many rheological properties amongst which creep is one of the most widely observed phenomenon. The creep behaviour of clay has been extensively studied by many researchers and there are several cases on record where the consequences of rheological process of creep of soils have been disastrous for various types of structures constructed over them. In this paper, two case studies based on creep-related rheological phenomenon of soil have been presented, namely The Leaning Tower of Pisa, Italy and The San Jacinto Monument, Texas, U.S.A. The Leaning Tower of Pisa, Italy is an engineering marvel with the tower being inclined at a degree of 5.5 due to presence of the soft soil under one of the sides of the tower. The San Jacinto Monument was constructed in 1936 comprising of a 176.5 m tower resting on a square mat of 37.8 m. The settlement of the structure as recorded after 70 years was 0.329 m. Subsequently, the settlement behaviour of a typical foundation of Kolkata High court building constructed in the nineteenth century has been discussed in an attempt to highlight the effect of time-dependent deformation of typical soft clay deposit in Kolkata.

### Keywords

**Creep**   **Rheological phenomenon**   **Kolkata****Time-dependent deformation**

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