EXPERIMENTAL STUDIES ON THE STRENGTH AND COMPRESSION CHARACTERISTICS OF SAND MIXED MONTMORILLONITIC CLAY

A PROJECT REPORT

Submitted in partial fulfilment of the requirements for the award of the degrees

of

BACHELOR OF TECHNOLOGY

in

CIVIL ENGINEERING

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CANDIDATE'S DECLARATION

We hereby declare that the project entitled **"EXPERIMENTAL STUDIES ON THE STRENGTH AND COMPRESSION CHARACTERISTICS OF SAND MIXED MONTMORILLONITIC CLAY"** submitted in partial fulfilment for the award of the degree of Bachelor of Technology in 'Civil Engineering' completed under the supervision of **Dr. Lalit Borana**, Assistant Professor, IIT Indore is an authentic work.

Further, we declare that we have not submitted this work for the award of any other degree elsewhere.

Date-

Satyajeet Meena **Lokesh Meena**

Signature and name of the student(s) with date

CERTIFICATE by BTP Guide

It is certified that the above statement made by the students is correct to the best of my/our knowledge.

Signature of BTP Guide with dates and their designation

Preface

This report on "**EXPERIMENTAL STUDIES ON THE STRENGTH AND COMPRESSION CHARACTERISTICS OF SAND MIXED MONTMORILLONITIC CLAY"** is prepared under the guidance of **Dr. Lalit Borana.**

"*In this report, we have tried to the best of our abilities and knowledge to Perform rigorous analysis of the experimental data from Oedometric test and Direct Shear Test. These analyses demonstrate the variation of soil behavior with the addition of admixture. In this study, Sand is taken as a reconstituting material to enhance strength and compression characteristics. Every single step in the experimental and analytical approach is explained and justified with proper philosophy and motive behind it. We have covered the relevant theories and concepts required to the analysis of test data. Illustrative graphs, figures, flowchart and user manual, have been added for better understanding of the reader.*"

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Abstract

With the increase of population and rising infrastructural needs of a rapidly developing country, like India, focused on the use of barren and unused lands. Among these, Black Cotton Soil is one that possesses many problems from a geotechnical and constructional point of view. To reduce these problems, many admixtures were used to enhance the geotechnical properties for constructional purposes. In central India, clayey soil type is one of the most abundantly available materials and covers approximately 20% of the geographical area. It is rich in the expansive minerals like montmorillonite and it undergoes both compressions as well as expansion. Because of its expansion as well as shrinkage properties, it possesses different strengths with the variation of water content, which is seasonally varied. In this study, Narmada River sand is used as a reconstituting material, since sand is naturally occurring, widely available and Eco-friendly. Which possesses pore grain sizes, which shorten the time required for expulsion of pore water. Many researchers have found that sand improves the properties of black cotton soil. A series of tests were performed to investigate the behavior of the blended soil with the addition of the sand at different percentages. The shear strength and compressibility characteristics were studied using direct shear and odometer test. From these experiment, it is known that with the addition of the sand the long term settlement of the blended soil decrease but the shear strength remains almost constant. The odometric results were compared using linear and non-linear analytical models to predict the creep settlement. It is found that the non-linear analytical model is suited to predict the creep.

Keywords: Black cotton soil, clayey soil, settlement, sand, Nonlinear creep, swelling behavior, shear strength consolidation, Elastic-Visco-plastic model (EVPS)

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1. Introduction

Black cotton soil is considered highly expansive soil because it has high shrink-swell properties due to the dominant presence of montmorillonite mineral. High montmorillonite is responsible for a significant volume increase and decrease in respectively wet and dry conditions. This Swelling and shrinkage of black cotton soil cause differential settlements, which is a reason for damage to the foundations, retaining structures, buildings, roads, etc. Alone in the United States, the damage from expansive soils to buildings, roads, airports, pipelines and other structures is more than twice the combined damage from earthquakes, floods, tornados and hurricanes (Jones and Holtz, 1973; Jones and Jones, 1987).

India has a large area of expansive soil in the form of Black Cotton soil, covering an area of 0.8 million square kilometers, which is about 20% of the total land area of India. The major areas of their occurrence are states of Maharashtra, Gujarat, southern parts of Uttar Pradesh, eastern parts of Madhya Pradesh, parts of Andhra Pradesh and Karnataka (Mehta, Sonecha, Daxini, Ratanpara, & Gaikwad, 2014). This type of soil is available to a depth of 3.7 meters on an average. Day by day requirement of land for structure building is increasing so, the construction of the foundations for structures has to be made on black cotton soils. It is a challenge to Civil Engineers.

Figure 1. Cracks in black cotton [20]

Figure 2. Differential settlement [21]

Soft and weak soils can only be used for the construction of structures after stabilization. There are few stabilization techniques (Microbial induced calcite precipitation (MICP), chemical stabilization, ground freezing, etc.) to improve the black cotton soil. But microbial induced precipitation is not feasible in most cases as the clay particles possess small grain. Also, admixtures like cement, lime, sand, fly ash and other chemical materials are used. Mixing chemicals is somehow hazardous for environment and soil fertilization and it also contaminates groundwater. Sand is naturally available inexpensive, eco-friendly and non-plastic soil and can be used as admixture because it reduces plasticity and swelling properties of black cotton soil. In this study, the Narmada River sand is used and the contents mixed in black cotton soil are 5%, 15% and 25% in terms of the dry mass of sand over the dry mass of the black cotton soil. Black cotton soil is of elastic visco-plastic nature, so it exhibits time-dependent stress-strain behavior. Several researchers have focused on the time-dependency of soft clayey soils by performing one-dimensional tests.

Engineering properties (Particle size distribution, Optimum moisture content and dry density, Specific gravity test, Atterberg limits and Free swell index) are examined for the pure condition of black cotton soil as well as for mixtures with different sand percentages and are compared to check how sand increment improves soil properties.

Also, Shear strength of clayey soil is studied by conducting tests based on the direct shear test. Because direct shear test is more appropriate to analyze rather than the triaxial shear test. In in-situ conditions, there is shear in soil layer with different rates depends upon the event happening on the layer it may be very high (in the case of an earthquake) or may be very low (in case of differential settlement). Direct shear test is conducted at different rates (0.025, 0.25, 0.625 mm/min) and different normal loading (0.5, 1.0, 1.5 kg/cm^2). At a slow rate like 0.025mm/min, it takes an average time of 8 hrs to perform one test, so it is not possible to continuously operate direct shear machine. A time gap of about 30 min is considered based on the shear apparatus working capacity. During this time slot, there are inter molecule arrangements due to energy relaxation and in this process, there is a change in shear strength. To identify this difference, the test is performed on one type of sample with multistage and single-stage process, keeping the rate fix.

The oedometer tests are used in practice to achieve data for obtaining parameters that are required to calculate final settlements, creep settlements, the rate of consolidation, and other perimeters to define a settlement process like Compression index (C_c), Rebounding index (C_r), creep coefficient (C_α), swelling coefficient (C_s). When a soil sample is loaded under a constant total vertical load in a 1D oedometer condition, the soil sample continues to settle after the expulsion of the excess pore water pressures, which is known as creep. Initially, total settlement is divided into two parts primary consolidation and secondary consolidation (creep). Creep settlement is usually defined by the creep coefficient or secondary consolidation coefficient (Cα). Now it is clear that creep occurs during the primary consolidation phase also, so there is no need to separate settlement in two different phases.

One-dimensional consolidation tests in the oedometer condition are performed with three different loading-unloading-reloading patterns. Each of these patterns has a different objective. Loading pattern T1 is to find long term creep, that's why in this pattern each load is kept for 7 days, so that there is sufficient time to creep. Loading pattern T2 is to examine the influence of the applied stress (loading history) on swelling behavior and here 10 kPa is taken as reference stress along which strain is to examine. Loading pattern T3 is with many loading-unloading cycles and used only for pure black cotton soil samples to investigate the influence of unloading reloading number of cycles on "swelling behavior while unloading" and "creep behavior while reloading."

Many researchers had proposed different models to predict the creep behavior of the soil. Among these EVPS model is defined as the best model for creep prediction. This model considers all the behavior of the soil viz. elastic behavior, plastic and viscous behavior again this model os suit for the swelling behavior also. Many researchers also cited this model is the best model. (O'Loughlin & Lehane, 2001; Sun, 1999).

2. Literature review

2.1. Consolidation

Consolidation is a settlement process by which soil changes its volume gradually in response to a change in loading. Consolidation is mainly divided into two parts primary and secondary consolidation. Primary consolidation is a short period settlement, but large deformation happens due to the expulsion of pore water under the applied load. Secondary consolidation takes long time, but there is lesser deformation because it is due to the plastic adjustment of soil particles, which happens less. During unloading, there is stress relaxation and water enter the soil sample, this phenomenon is called swell.

Many researchers also suggested many factors that are responsible for both primary and secondary consolidation. The applied effective stress is a major factor that is responsible for the primary consolidation. The viscous behavior of the clay is due to (1) viscous movement of the adsorbed water in the double layers around the clay particles, (Yin 2013, 2015) (2) deformation and viscous rearrangement of the clay particles.

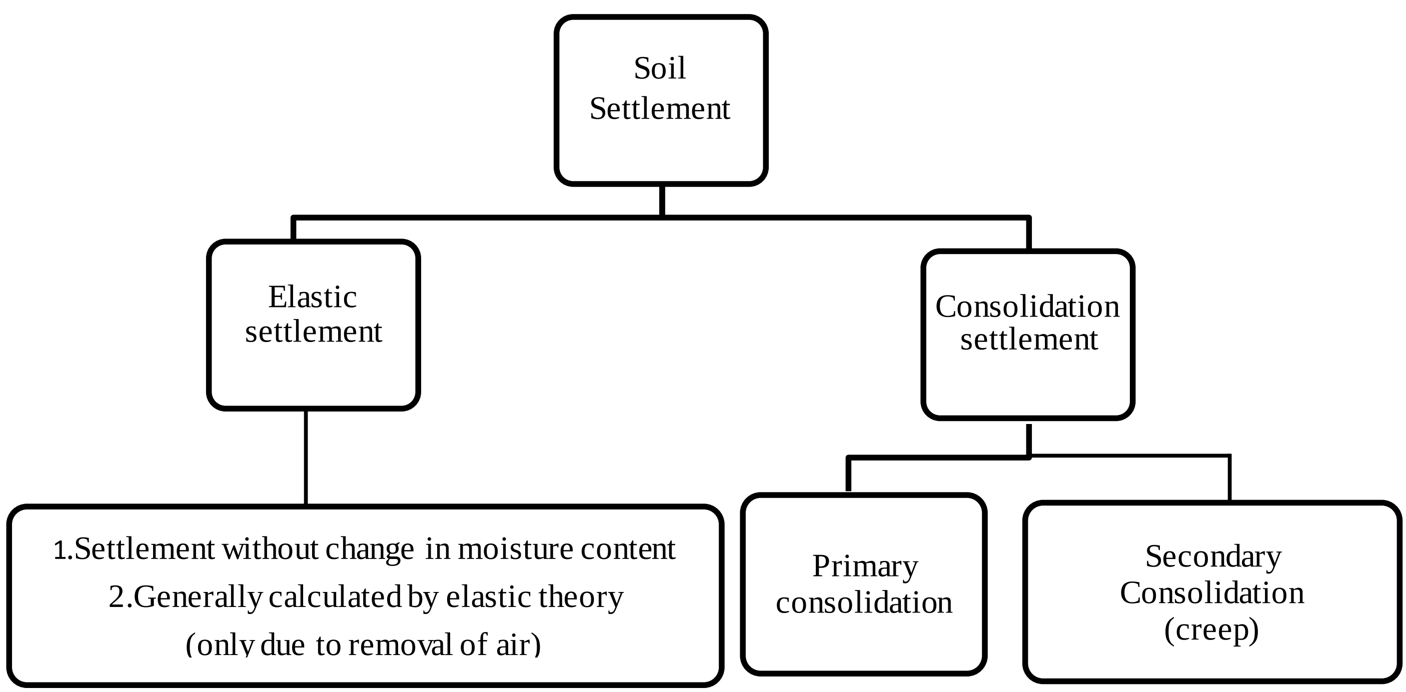


Figure 3. process of settlement

2.2. Shear strength

Shear strength of a soil is the resistance of soil against the shear stress. The shear strength of the soil is a result of friction and interlocking between soil particles. After compressive strength of soil, shear strength is an important factor. So it is necessary to check the shear strength of sand blended black cotton samples. Some studies had been done on the shear strength of black cotton mixed with sand. (Kim, Nam, & Youn, 2018).

The shear strength of black cotton soil is higher in a dry state, but after absorbing moisture, it's shear strength reduces a lot. The direct shear test using shear box is usually recommended by practicing geotechnical to find shear strength. Direct shear test on different sand content (pure, 5%, and 25%) is performed to check shear strength. The direct shear test is less time consuming and an easy way to determine undrained shear strength of disturbed clayey soil and sand mixtures.

In in-situ conditions, there is shear in soil layer with different rates depends upon the event happening on the layer it may be very high (in the case of an earthquake) or may be very low (in case of differential settlement). Direct shear test is conducted at different rates (0.025, 0.25, 0.625 mm/min) and different normal loading (0.5, 1.0, 1.5 kg/cm^2). At a slow rate like 0.025mm/min, it takes an average time of 8 hrs to perform one test, so it is not possible to continuously operate direct shear machine. A time gap of about 30 min is considered based on the shear apparatus working capacity. During this time slot, there are inter molecule arrangements due to energy relaxation and in this process, there is a change in shear strength. To identify this difference, the test is performed on one type of sample with multistage and single-stage process, keeping the rate fix.

2.3. Compression parameter and rebounding parameter

To understand the oedometer result, compression parameter (λ/v) and rebounding parameter (k/V) are defined as the slope of vertical strain v/s vertical stress in normal loading conditions and over consolidation loading condition, respectively (Juárez-Badillo, 2012).

$$\frac{\lambda}{v} = \frac{\Delta\epsilon_z}{\Delta \ln(\sigma_z')} \qquad (1)$$

The Compression index (Cc/V) and rebounding index (Cr/V) are also used to analyze the oedometer results. There are relationships between compression parameter and compression index, rebounding parameter and rebounding index:

$$\frac{\mathrm{Cc}}{v} = \ln(10)\frac{\lambda}{v} = 2.3\frac{\lambda}{v} \qquad (2)$$

$$\frac{\mathrm{Cr}}{v} = \ln(10)\frac{k}{v} = 2.3\frac{k}{v} \qquad (3)$$

2.4. Elastic-visco-plastic model (EVP model)

There are many models to analyze oedometer data but from all of them, the EVPS model is best suitable for clay soil analysis because it considers all the properties of clay that are elasticity, viscosity and plastic-behavior.

To analyze oedometer data EVP model is used that was first introduced by Yin and Graham in 1989 and modified in 1992,1993. Vertical strains in the oedometer are divided into three components. (Yin & Graham, 1989)

$$\varepsilon_z = \varepsilon_z^{e} + \varepsilon_z^{sp} + \varepsilon_z^{tp} \qquad (4)$$

Here

ε_z^e, is recoverable elastic strain

ε_z^{sp}, is time-independent plastic strain

ε_z^{tp}, is viscous strain, that is, time-dependent plastic strain

In 1992 yin and graham give concept of equivalent time lines and produce a rational EVP model that can predict the behaviour of viscus soil in 1D compression loading. (Jian Hua Yin, 1993)

2.5. Creep parameter($\frac{\psi^c}{v}$) and swell parameter($\frac{\psi^s}{v}$)

From EVP model the creep coefficient ($\frac{\psi^c}{v}$) is usually defined to describe a linear relationship of vertical strain and time (J. H. Yin & Graham, 1989; Jian Hua Yin, 1993):

$$\varepsilon_z = \varepsilon_{z_0} + \frac{\Psi^c}{v} \ln\left(\frac{t_0^C + t_e^C}{t_0^C}\right) \qquad (5)$$

Where

t_e^C, is equivalent time related to creep; t_0^C, end of primary consolidation

ε_{z_0}, is the strain at the time t = t_0^C; $\frac{\Psi^c}{v}$, creep coefficient (constant)

And

Creep coefficient is related to secondary consolidation coefficient (Cα) as likewise swelling coefficient also:

$$2.3\frac{C\alpha}{v} = \frac{\Psi^c}{v}$$

$$2.3\frac{Cs}{v} = \frac{\Psi^s}{v}$$

Where

$C\alpha$, is secondary consolidation coefficient

Cs, is swelling coefficient

v, is unitary volume i.e. $1+e_0$

Regarding the swelling behaviour , the swelling coefficient ($\frac{\psi^s}{v}$) is also defined as the rate of vertical strain and time:

$$\varepsilon_z = \varepsilon_{z_0} - \frac{\Psi^s}{v} \ln\left(\frac{t_0^s + t_e^s}{t_0^s}\right) \qquad (6)$$

2.6. Equivalent time (t_e)

Is the time needed to creep from a reference time line to current value of ε_z and σ_z under constant effective stress. Illustration of equivalent time line is shown in figure 4. (Jian Hua Yin, 1993).

- Time line - corresponding to constant duration of loading
- example- 1-day time line

- Instant time line – corresponding to
- strain due to pore water dissipation

- Reference time line – along which equivalent time has zero value

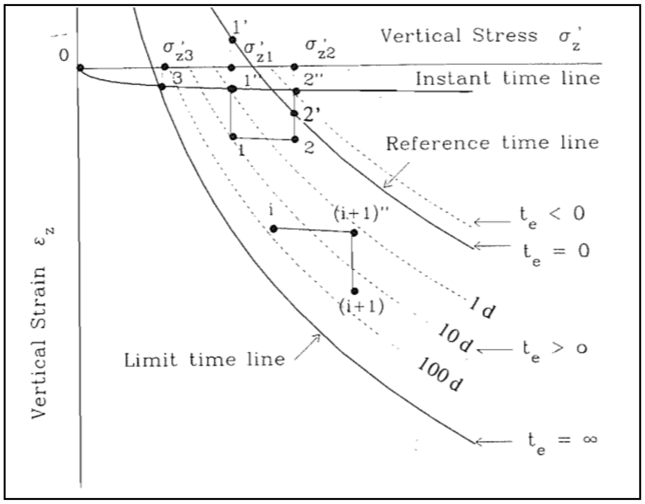


Figure 4. Illustration of time lines

At instant time line load duration is zero
So at 1" load duration is zero but t_e is 1 day
Likewise at 1 load duration is 9 day but t_e is 10 day

So in general $t_e = t - t_o$ is considered (Yin and Tong, 2011)

2.7. Nonlinear function

Yin proposed a nonlinear function to describe the long-term creep considering the creep limit when time is infinite(J. H. Yin, 1999):

$$\epsilon_z^c = \boldsymbol{\varepsilon_{z_0}} + \frac{\frac{\psi_0^c}{v}}{1 + \frac{\psi_0^c}{\varepsilon_z^{cl} v}\ln(\frac{t_0^c + t_0^e}{t_0^c})}\ln\left(\frac{t_0^c + t_0^e}{t_0^c}\right) \qquad (7)$$

Here ϵ_z^{cl} is creep strain limit

Similarly, there is nonlinear function for the nonlinear swelling behaviour:

$$\epsilon_z^s = \boldsymbol{\varepsilon_{z_0}} - \frac{\frac{\psi_0^s}{v}}{1 - \frac{\psi_0^s}{\varepsilon_z^{sl} v}\ln\left(\frac{t_0^s + t_0^e}{t_0^s}\right)}\ln\left(\frac{t_0^s + t_0^e}{t_0^s}\right) \qquad (8)$$

To find ϵ_z^c there is requirement to find $\frac{\psi_0^c}{v}$ and ε_z^{cl}

2.8. Calculation $\boldsymbol{\frac{\psi_0^c}{v}}$ and $\boldsymbol{\varepsilon_z^{cl}}$

$\frac{\psi_0^c}{v}$ and ε_z^{cl} are obtained from the experimental swelling strain using equation (8):

$$\frac{1}{\varepsilon_z^c}\ln\left(\frac{t_0^c + t_0^e}{t_0^c}\right) = \frac{v}{\psi_0^c} + \frac{1}{\varepsilon_z^{cl}}\ln\left(\frac{t_0^c + t_0^e}{t_0^c}\right) \qquad (9)$$

If $\ln\left(\frac{t_0^c+t_0^e}{t_0^c}\right)$ is considered as one variable, and $\frac{1}{\varepsilon_z^c}\ln\left(\frac{t_0^c+t_0^e}{t_0^c}\right)$ is taken as another variable. Than Equation (9) is a straight line of the format y=mx+c. where m = $\frac{1}{\varepsilon_z^{cl}}$ and c = $\frac{v}{\psi_0^c}$. by fitting the experimental data ε_z^{cl} and $\frac{\psi_0^c}{v}$ can be calculated.

Comparison of experimental data with predicted strain from linear equation (5), (6) and with predicted strain from nonlinear equation (7), (8) will be done and will see that which prediction is more suitable and fit to experimental data.

3. Experimentation and test detail

3.1. Particle size distribution

For particle size distribution, it is needed to determine the internal particle size present in the black cotton soil using sieve analysis and hydrometer test. For sieve analysis, a mechanical sieve shaker is required, which includes sieves of different sizes i.e., 4.75, 2, 1.18, 0.6, 0.3, 0.15, 0.075 mm. The test is performed on 500g black cotton soil, which is 24 hours heated in an oven and cooled in normal conditions. Then soil sample is placed in the 4.75mm sieve with a cover on it, which is on the top of all other sieves placed in the decreasing order by sieve size. Sieve shaker is started and in 5 minutes, all the sieves have individual retained soil mass.

In the hydrometer test, grain size distribution of the soil mass passing the 0.075 mm sieve and retaining on the pan is determined. Sodium hexa-meta-phosphate is used as a dispersing agent and the test is performed. The combined results of Sieve analysis and hydrometer test are plotted as log (sieve size in mm) vs % finer, which is known as particle size distribution curve or S-curve. From this curve, one can identify the diameters of the particles present in the soil mass.

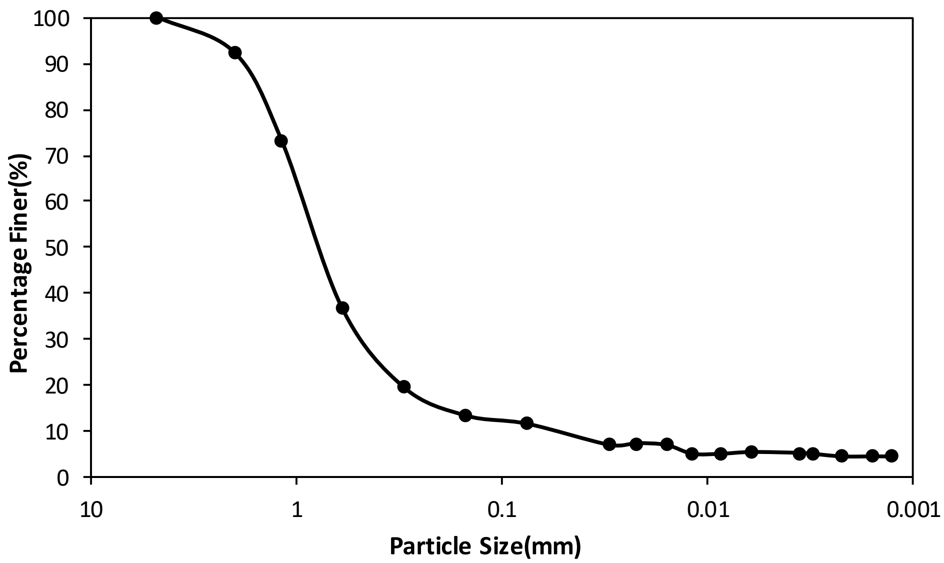


Figure 5.Particle size distribution curve of black cotton soil

3.2. Optimum moisture content and dry density

The optimum moisture content (%) and dry density (g/cm^3) of black cotton soil and its mixtures with sand is determined using standard proctor test. This test consists of a mould and rammer of standard size and weight. The rammer is used to compact the soil sample in the mould in three layers. This test is performed 4-5 times for a soil at different water content. After compaction, the soil mass is weighted and some mass is placed in the oven for 24 hours to determine the moisture content. The dry density is the ratio of the wet density (weight of compacted soil/volume) to (1+moisture content). The plot of dry density v/s % moisture content is shown in the figures below and optimum moisture content is the moisture content corresponding to the maximum dry density.

Table 1. value of OMC (%) and MDD (g/cc) for different samples

Property	Pure BC soil	BC+5% sand	BC+15% sand	BC+25% sand
OMC(%)	28.025	26.214	25.821	23.816
γ_d(g/cc)	1.406	1.410	1.448	1.504

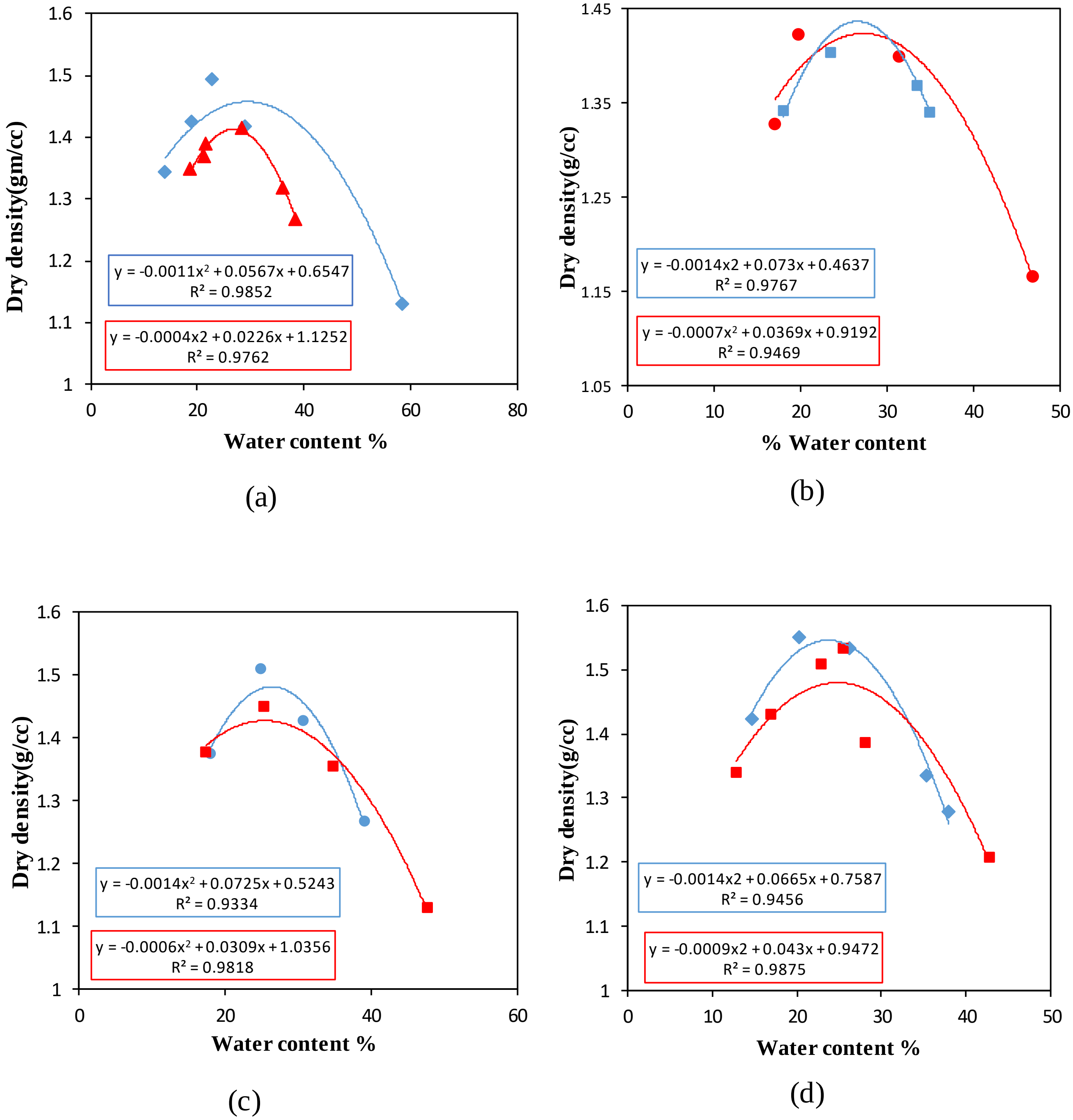


Figure 6. Dry density (gm/cc) Vs Moisture content (%) for (a) B.C. (b) B.C. + 5% sand (C) B.C.+ 15% sand (d) B.C. + 25% sand

3.3. Specific gravity test:

Specific gravity (G) of black cotton soil and its mixtures with sand is determined using density bottle test. Initially, cleaned and properly washed density bottle is weighted (w1). About 20 g of oven-dried soil is filled in the density bottle and weighted (w2). Now, about 10ml of distilled water is filled in the density bottle and mixed properly and placed for some time so that the soil soaks the water completely and weighted (w3). Again, the bottle is cleaned and completely filled with distilled water and weighted (w4). Also, field density test is performed to estimate the field density of pure sand. Specific gravity is the ratio of dry mass of soil to the equal mass of distilled water. It is calculated as: G = [w2-w1]/[(w4-w1)-(w3-w2)]

Table 2. Specific gravity of black cotton soil and black cotton soil mixed with 5%, 15% and 25% sand.

Specimen	Pure BC soil	BC+5% sand	BC+15% sand	BC+25% sand	Sand
S.G.	2.41	2.43	2.47	2.48	2.67

3.4. Atterberg limits:

Liquid limit and plastic limit are two important atterberg limits and determined using Casagrede's apparatus. Liquid limit (LL) is the moisture content (%) at which the soil begins to behave as a liquid material i.e. the moisture content at which the soil changes its state from plastic to liquid. The Plastic limit (PL) is the moisture content at which soil changes its state from semi-solid to plastic stage.

Liquid limit test results are presented as a curve between % moisture content and number of blows, which is called flow curve. In the flow curve, the liquid is the moisture content corresponding to the 25 number of blows.

The plastic limit test is performed using plastic limit set by mixing different water contents in the soil and the moisture content at which the soil will start to crumble when rolled into 3.2mm threads is plastic limit.

The measure of plasticity of soil is called Plasticity Index and is the difference between Liquid limit and Plastic limit.Results of Liquid Limit, Plastic Limit and Plasticity Index values for all the soil combinations are shown below in figure 8:

Table 3. Atterberg limits of black cotton soil and black cotton soil mixed with 5%, 15% and 25% sand.

Specimen	Pure BC soil	BC+5% sand	BC+15% sand	BC+25% sand
LL(%)	69.29	65.06	59.53	55.41
PL(%)	34.12	33.27	31.47	29.05
PI(%)	35.17	31.79	28.06	26.36

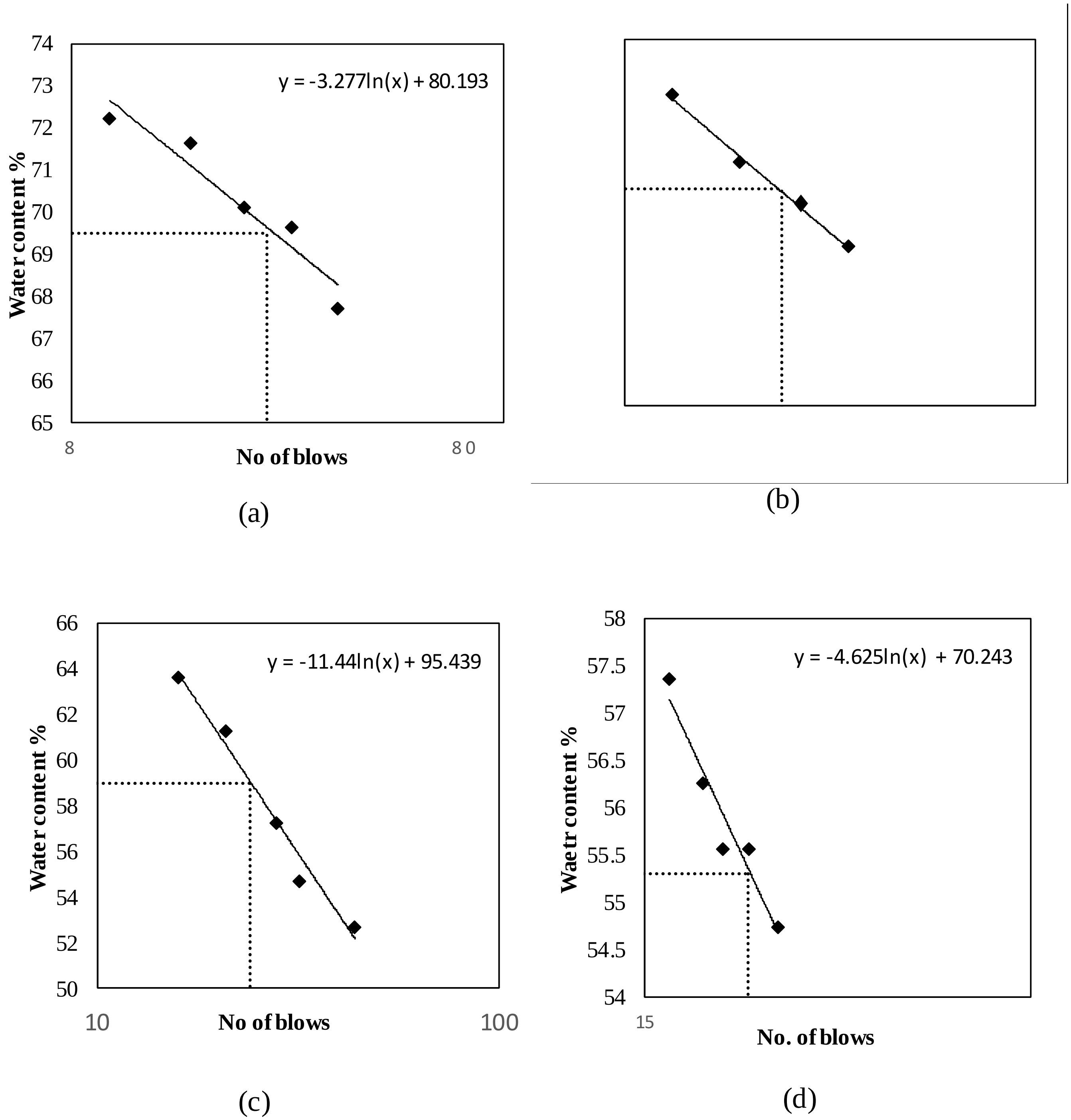


Figure 7. Moisture Content (%) Vs No. of blow for (a) Black cotton soil (b) Black cotton soil mixed with 5% sand, (c) Black cotton soil mixed with 15%sand, (d) Black cotton soil mixed with 25% sand

3.5. Free swell index:

When the clayey soil is submerged in the water due to the presence of montmorillonite mineral, it shows a significant increase in volume which is known as free swell index.

Using free swell index swelling properties of a soil is found. That is the measure of volume change.

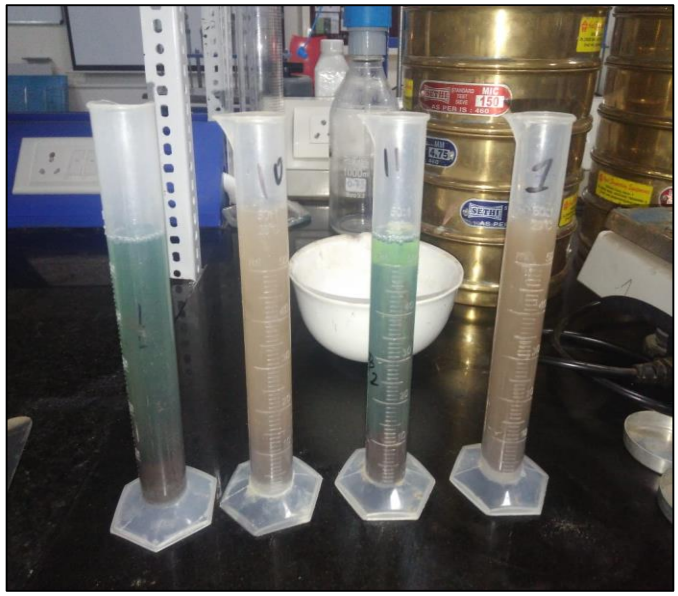


Figure 8.Ongoing free swell index test

The oven dried black cotton soil and its mixtures with sand are filled in the graduated cylinders of 50ml.For a particular type of soil, in one-cylinder kerosene and in another water is filled and then the soil is mixed properly by shaking gently to remove the trapped air in the soil voids. Now samples are placed for 24 hrs at a horizontal place. At this equilibrium state the final heights are noted. Since it is known the diameter of the graduated cylinders so the initial and final volumes can be calculated.

Free swell index (%) is the ratio of the (difference of volumes between solution with distilled water and with kerosene) to (volume of solution with kerosene). The values are shown below:

Table 4.Values of free swell index

Specimen	Pure BC	BC+5% sand	BC+15% sand	BC+25% sand
Free swell index	60.86	59.09	52.17	45.45

3.6. Oedometer test:

One dimensional oedometer tests are performed to study the consolidation behaviour of clayey soils. Oedometer tests is performed on black cotton soil and its mixtures with sand in completely saturated conditions.

Figure 9. Oedometer cell

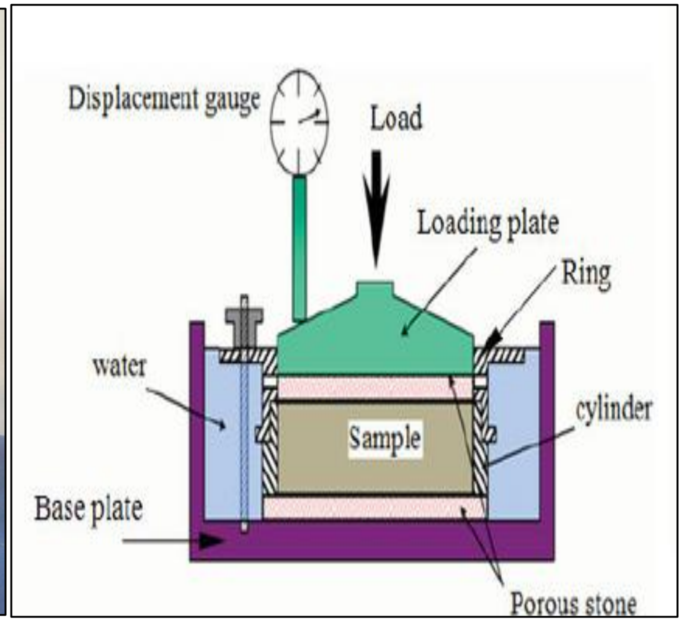


Figure 10. Setup of Oedometer cell

Sample preparation and test details: - Oedometer available in lab has three vertical displacement gauges, oedometer cells, loading setup with all essential things, water bottles to keep the soil sample saturated. Various loads were available for vertical loading. Samples for oedometer test were prepared by computing the volume of the mould (diameter 60mm and height 20mm) and 80 % of maximum dry density of soil. The samples were placed into the steel ring, and internal surface of the ring was applied with the lubricant to minimize the possible friction. Filter papers were used on top and bottom of the samples to avoid the entry of soil particles into the porous stones. Then steel ring was placed into the oedometer cell and connected to the water bottle for saturation. Initially 5 kPa load was applied for 24 hrs to settle the specimens. Then the loading plan was followed in order to load and unload the sample.
Oedometer test results are used to study compressibility characteristics, creep and swelling behaviours. During oedometer test three loading plans are used, each have different objective. The plans are shown in table below.

Table 5. loading pattern T1, T2 and T3.

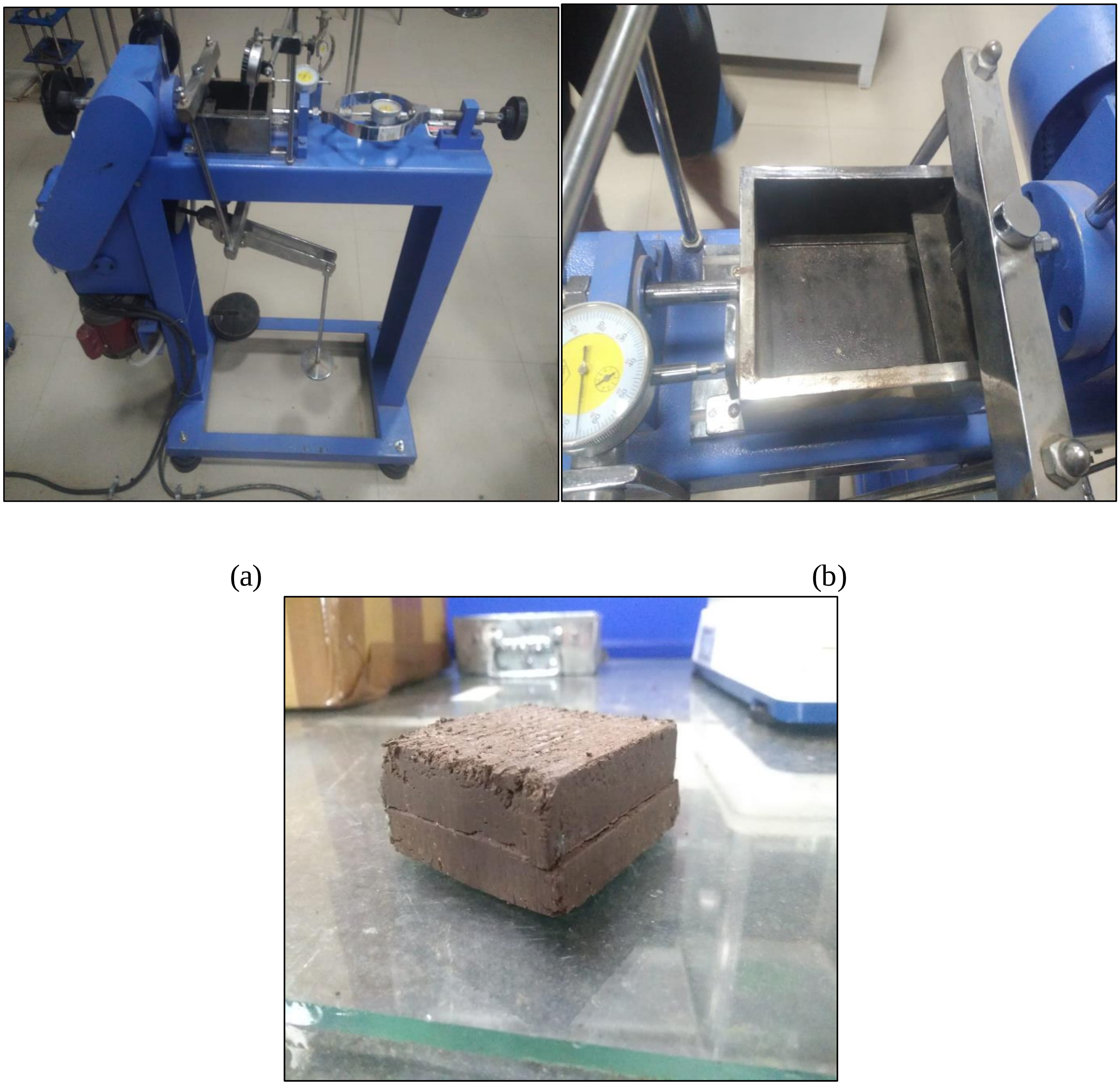
Aim: To check Swelling Behaviour		
Order	Loading (kPa)	Duration of Loading (days)
1	5	1
2	10	**7**
3	25	1
4	50	1
5	100	**7**
6	50	1
7	25	1
8	10	7
9	25	1
10	50	1
11	100	1
12	500	1
13	1000	7
14	500	1
15	250	1
16	50	1
17	10	7
18	50	1
19	250	1
20	500	1
21	1000	1
22	1250	7
23	1000	1
24	500	1
25	250	1
26	100	1
27	10	7
28	100	1
29	250	1
30	500	1
31	1000	1
32	1250	7
33	1000	1
34	500	1
35	250	1
36	100	1
37	50	1
38	10	7
Total Days		**92**

Aim: To check the creep behaviour		
Order	Loading (kPa)	Duration of Loading (days)
1	5	7
2	10	7
3	25	7
4	50	7
5	100	7
6	250	7
7	500	7
8	250	7
9	100	7
10	50	7
11	10	7
12	50	7
13	100	7
14	250	7
15	500	7
16	1000	7
17	1250	7
Total Days		**119**

Aim: To check the influence of loading on Swelling Behaviour with the same amplitude but different stresses		
Order	Loading (kPa)	Duration of Loading (days)
1	5	7
2	10	1
3	25	1
4	50	7
5	25	1
6	10	1
7	5	7
8	10	1
9	25	1
10	50	7
11	100	1
12	250	1
13	500	7
14	250	1
15	100	1
16	50	7
17	500	1
18	1000	1
20	1250	7
22	1000	1
23	500	7
24	1000	1
25	1250	7
26	1000	1
27	500	7
28	1000	1
29	100	1
30	50	1
31	10	7
Total Days		**95**

3.7. Direct shear test

To determine the shear strength of black cotton soil and its mixtures with sand in different proportions by direct shear apparatus.



(a) (b)

(C)

Figure 11. (a) Direct shear test machine, (b) setup, (c) Failed sample of BC+25%sand at 0.025 mm/min rate and 100 kpa loading

Direct shear test is less time consuming and an easy way to determine undrained shear strength of disturbed clayey soil and sand mixtures. It has a shear box with a loading pad to place the soil sample, loading mechanism which works by an electric motor, a proving ring, dial gauges to measure shear displacement and vertical displacement. Sample is prepared in a square ring of a known volume and by 80% maximum dry density of soil and optimum moisture content. The sample is placed in the shear box having two steel plates one upper and one below for undrained conditions and two porous stones. Normal loads were 0.5,1.0 and 1.5 kg/cm^2 and shear strain rates were 0.25,0.625 and 0.025 mm/min in both ways single stage and multistage.

Results of direct shear test are plotted as shear stress v/s shear strain curve, from which failure stress is estimated. For different normal loads there are different failure stresses. A curve between shear stress and normal stress is a straight line. In this curve the intercept and slope of the straight line are respectively cohesion(c) and angle of friction (Φ).

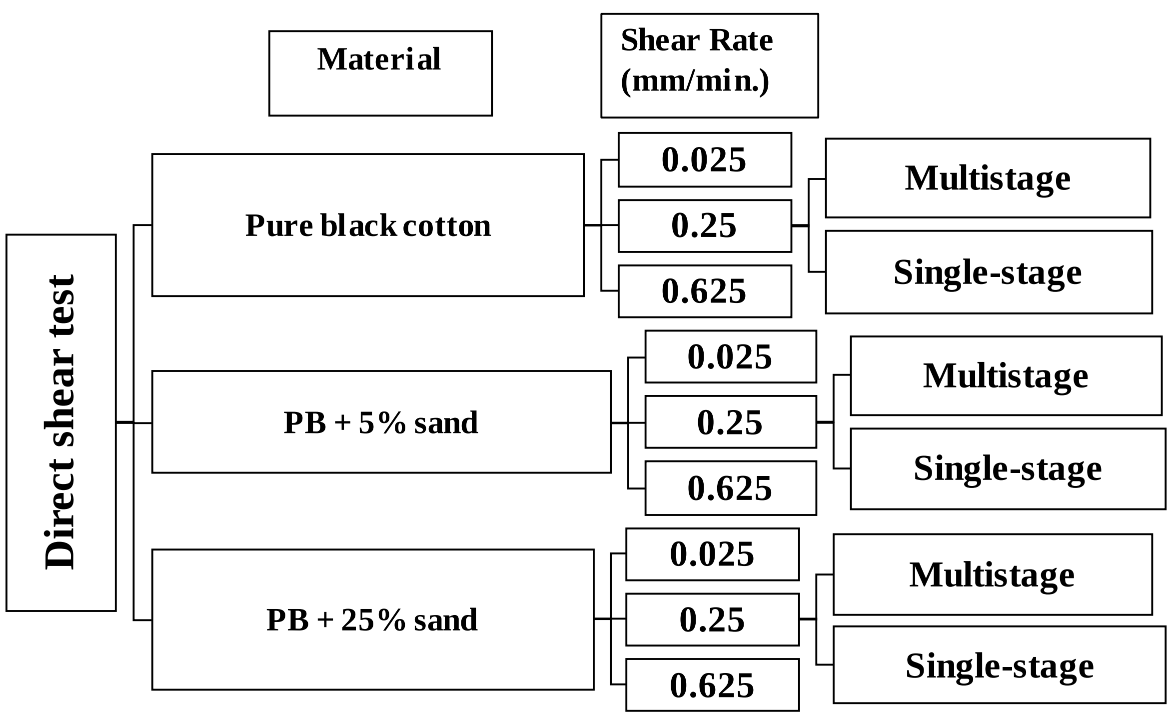


Figure 12. Direct shear test plan

3.8. Results of index properties

Based on the engineering property experimentation and results following key observations are shown below.

- From the above observations, it is observed that mixing of Sand in black cotton soil reduces the swelling properties of this soil.

- Value of free swell index reduced from 60% to 45%, which makes a large difference in soil properties.

- The index properties of expansive soil have been improved due to the replacement of expansive soil by non-expansive sand. A mixing of sand reduced index parameter significantly.

- such as liquid limit reduced about 11%, plastic limit reduced about 6%, plasticity index reduced as 8%. In the case of OMC and MDD, Optimum moisture content decreased by about 4%. while maximum dry density increased from 1.4 to 1.5.

- Thus if the sand is admixed in expansive soils, maximum benefit in terms of volume stability can be achieved.

Table 5. Basic engineering properties of BC soil mixed with different sand content

Specimen	Pure BC soil	BC+5%sand	BC+15%sand	BC+25%sand
OMC(%)	28.025	26.214	25.821	23.816
γ_d(g/cc)	1.406	1.410	1.448	1.504
S.G.	2.41	2.43	2.47	2.48
LL(%)	66.290	65.063	59.534	55.410
PL(%)	34.12	33.27	31.47	29.05
PI(%)	32.17	31.79	28.06	26.36
Free swell index	60.87	59.09	52.17	45.45

4. Oedometer Test Results and Interpretation

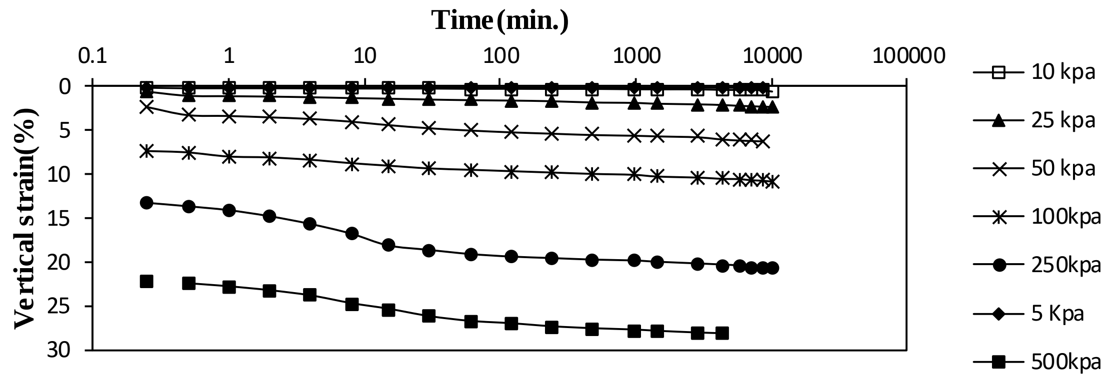
Long term oedometer tests on the black cotton-sand mixture have been done. In this study, it is clearly shown that there is a large contribution of creep to the total settlement during a fix loading time period, likewise swelling strain to the overall rebounding effect. The primary consolidation mostly finishes within the time period of one day. Beyond one day, it could be seen that the compression rate of sand mixed black cotton is very low. For whenever there is change from loading to unloading or vice versa, the loading time is of 7 days beyond after 7 days, there is no significant change in strain.

The multi-staged oedometer test results in vertical strain vs. time (log scale) are shown here

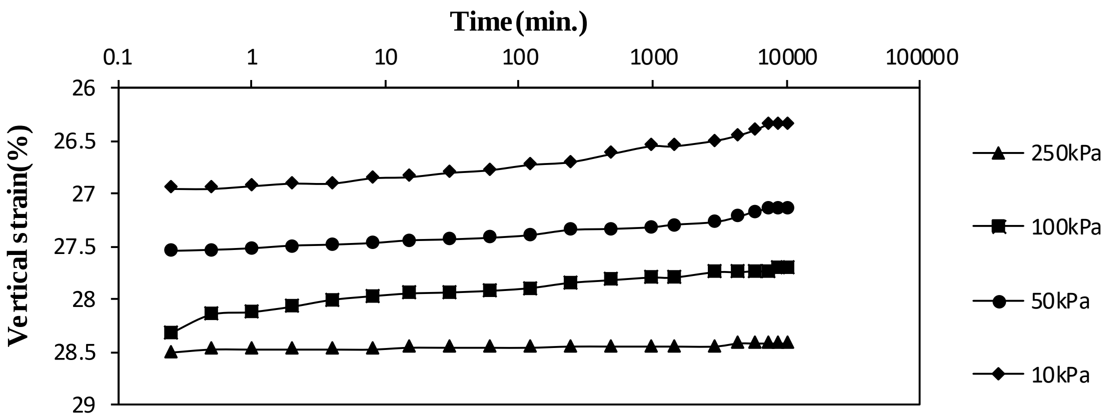
4.1 Loading pattern T1 to check the creep of the given Soil

In this pattern every load is applied for a time duration of 7 days so that the sample undergoes complete creep or swelling during loading or unloading pattern respectively. Three samples (black cotton in pure state, mixed with 5 % sand, 25% sand) are tested under oedometric condition. From this test results one can check that at the end of long term test which one of these sample is more settled and also how swelling happening during unloading. So from test on this pattern, one can explain that how sand content is effecting settlement and swelling of black cotton soil.

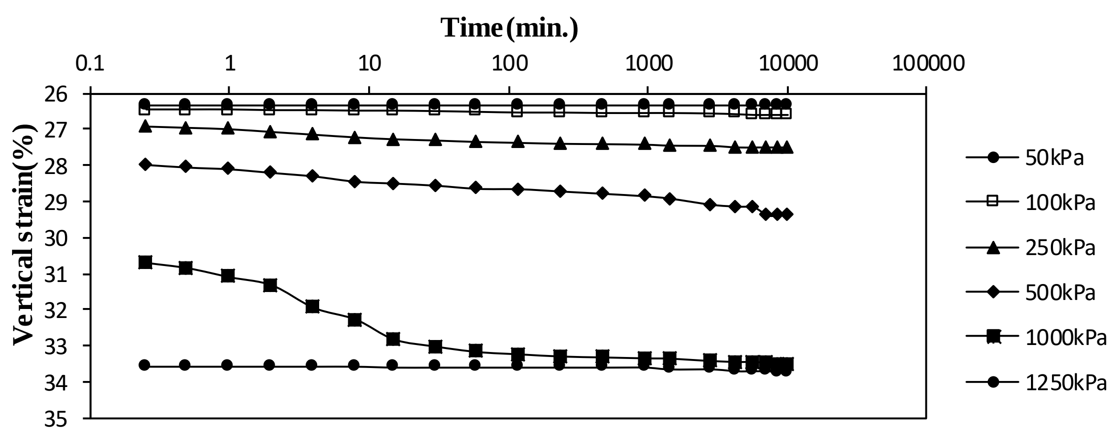
Experiment done on loading pattern T1 and graph between vertical strain (%) and Log(t) is plotted and are shown below in figure. 13, 14 and 15. To get compression parameters vertical strain (%) vs effective stress (kPa) are plotted in figure 16.



(a)

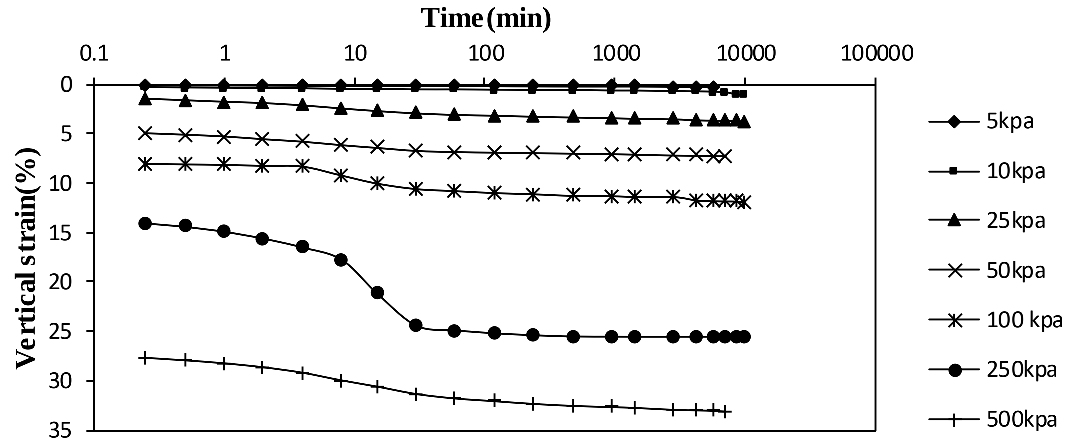


(b)

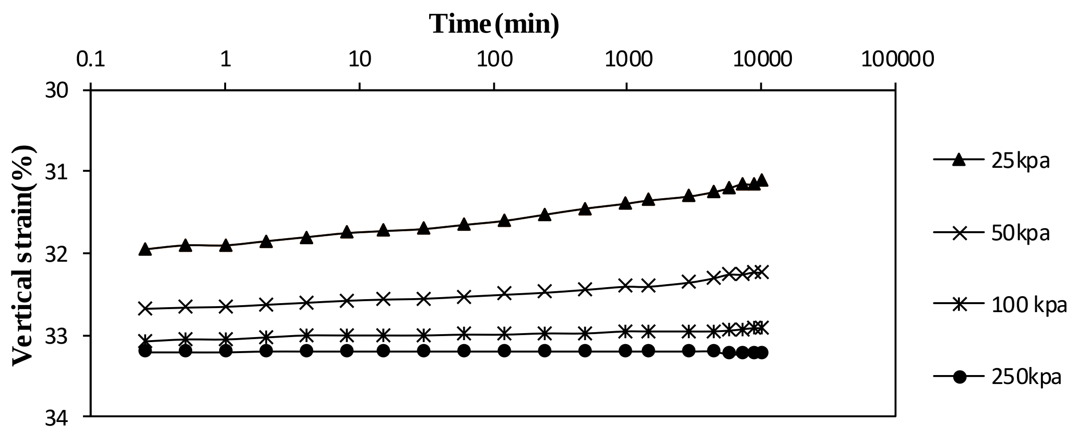


(c)

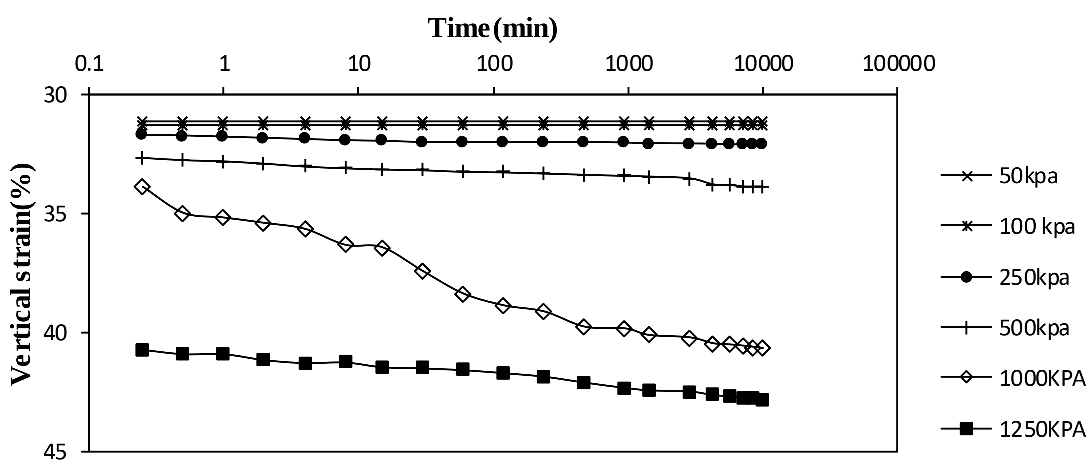
Figure 13. Vertical strain against time in log scale for pure black cotton soil for stage (a) Loading1 (b) Unloading 1 (c) Reloading 1



(a)



(b)



(c)

Figure 14. Vertical strain against time in log scale for black cotton mixed with 5% sand of L1, U1 and R1

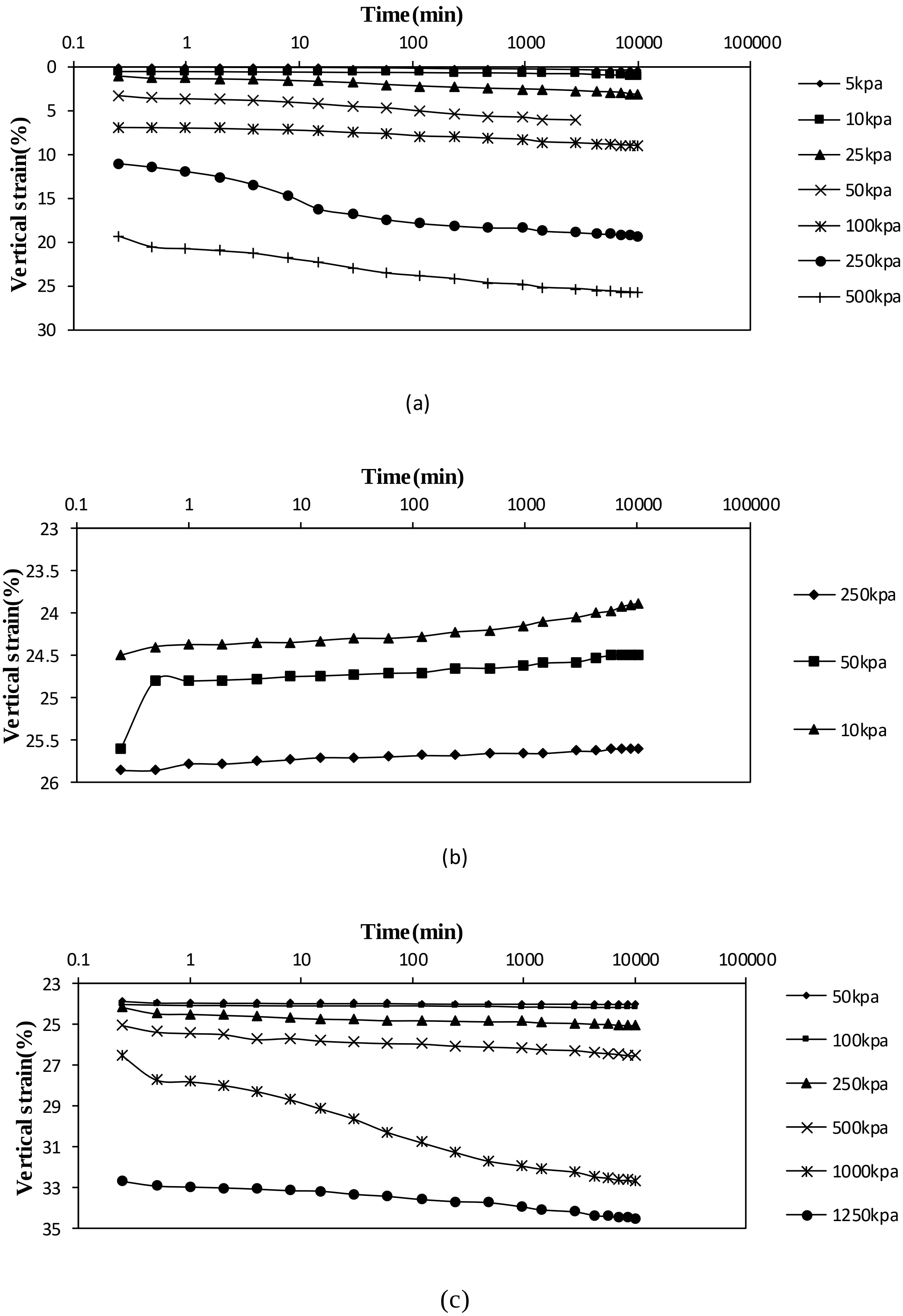


Figure 15. Vertical strain against time in log scale for black cotton mixed with 25% sand of L1, U1 and R1

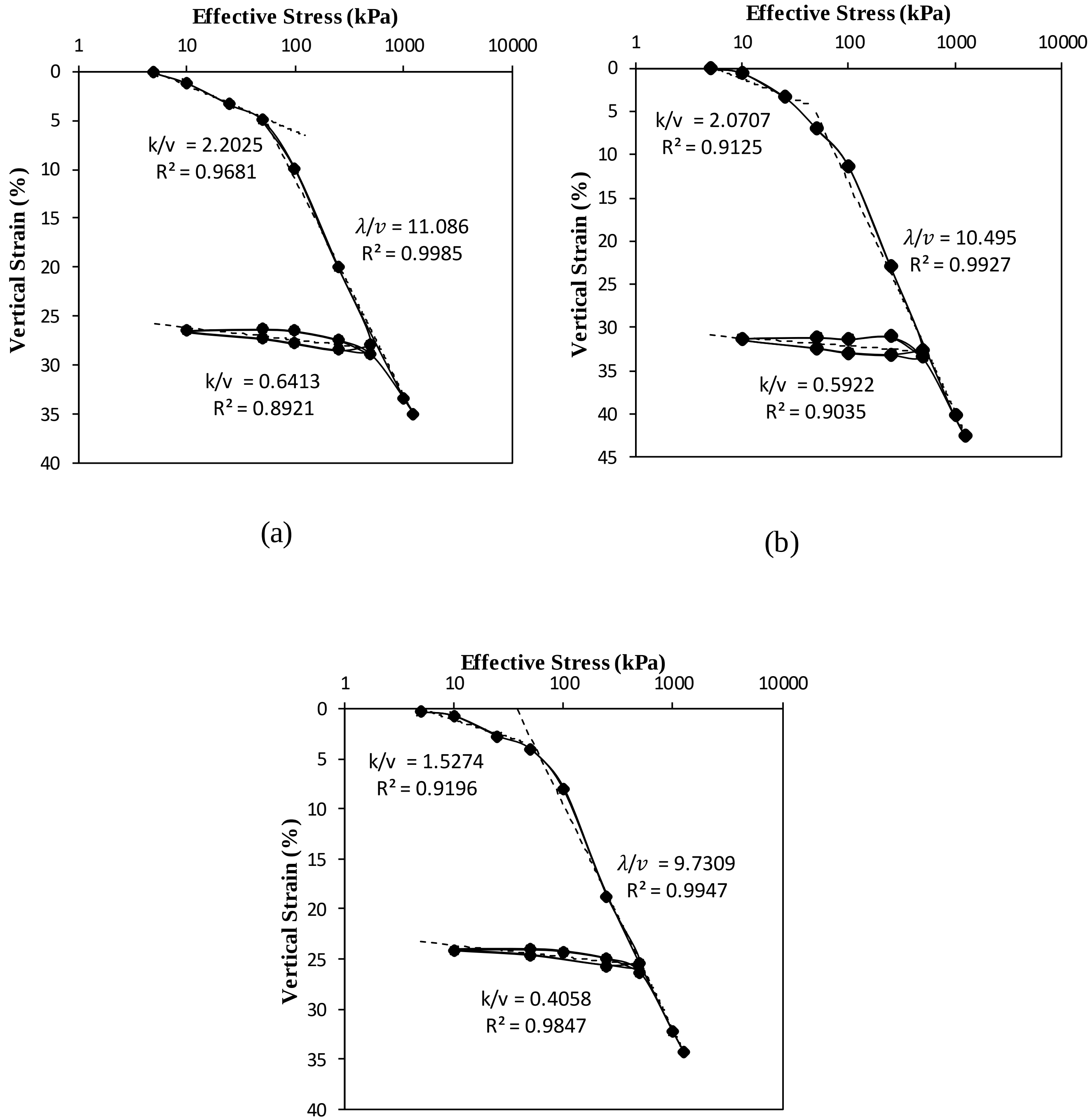


Figure 16. Vertical strain against effective vertical stress in log scale: (a) pure BC, (b) 5% sand content, and (c) 25% sand content.

Table 6. Cc and Cr values for loading pattern T1

soil sample	Cc	Cr	Cr1
pure soil	0.479	0.095	0.0277
5%sand	0.454	0.084	0.025
25%sand	0.421	0.065	0.017

Table 7. Total strain at every loading unloading strain in percentage

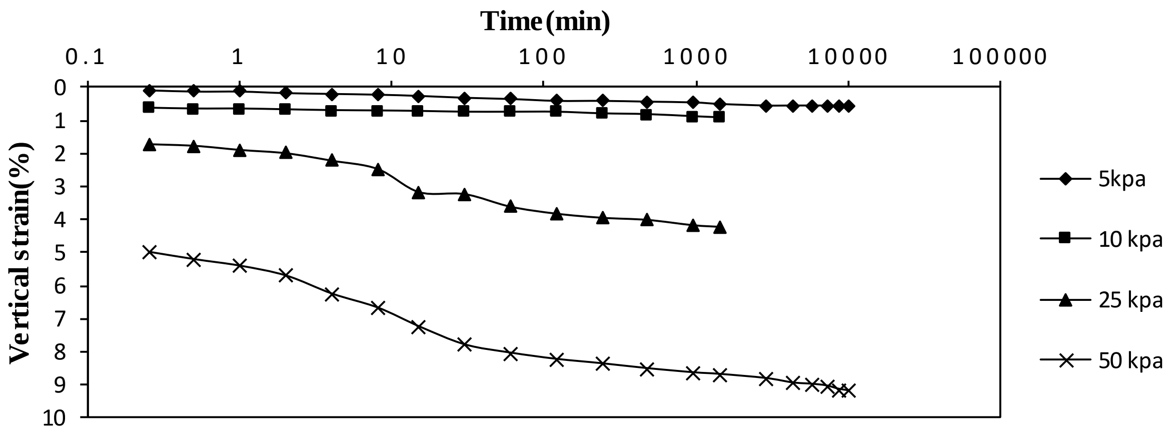
Soil sample	Loading(till 500kPa)	Unloading(till 10kPa)	Reloading(till 1250kPa)
pure	28.5	26.35	33.72
5% sand	33.15	31.1	42.85
25% sand	25.85	23.87	32.6

These result clearly shows that mixing 25% sand have improvement in creep and swell, swell and creep are decreasing.

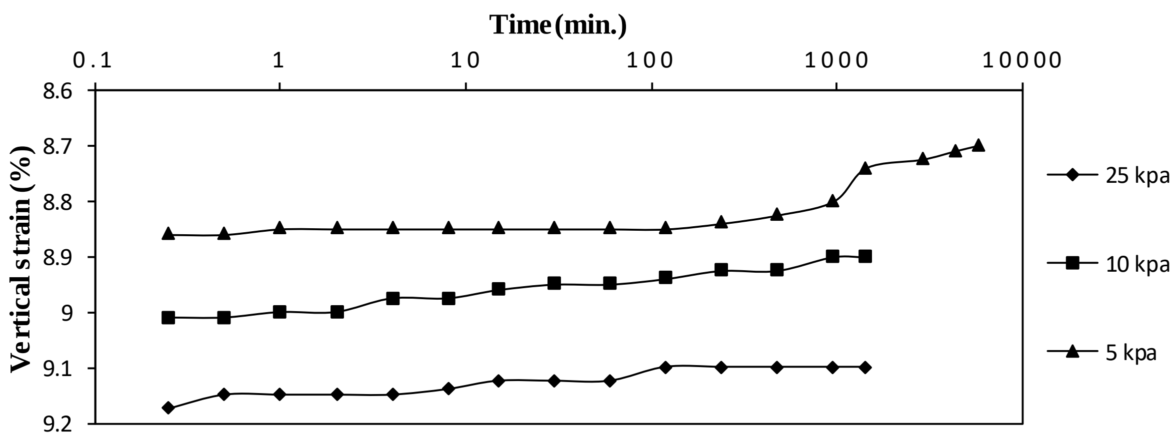
4.2. Loading pattern T3 to check influence of loading on Swelling Behaviour

This loading pattern is to examine the influence of unloading reloading number of cycle on swelling behavior while unloading and creep behavior while reloading. That's why there is so many loading-unloading cycles.

At this loading pattern experiment is done on pure black cotton sample only. Graphs between vertical strain (%) and Log(t) is plotted and are shown below in figure. 17.

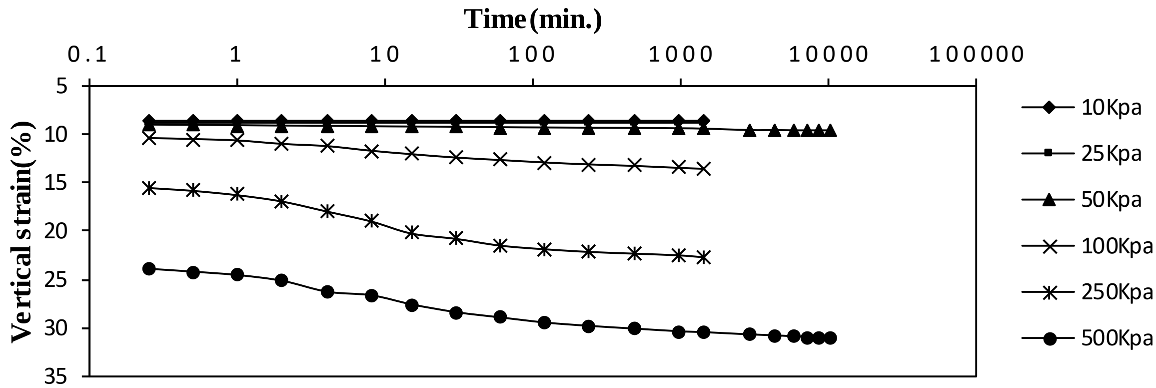


(a)

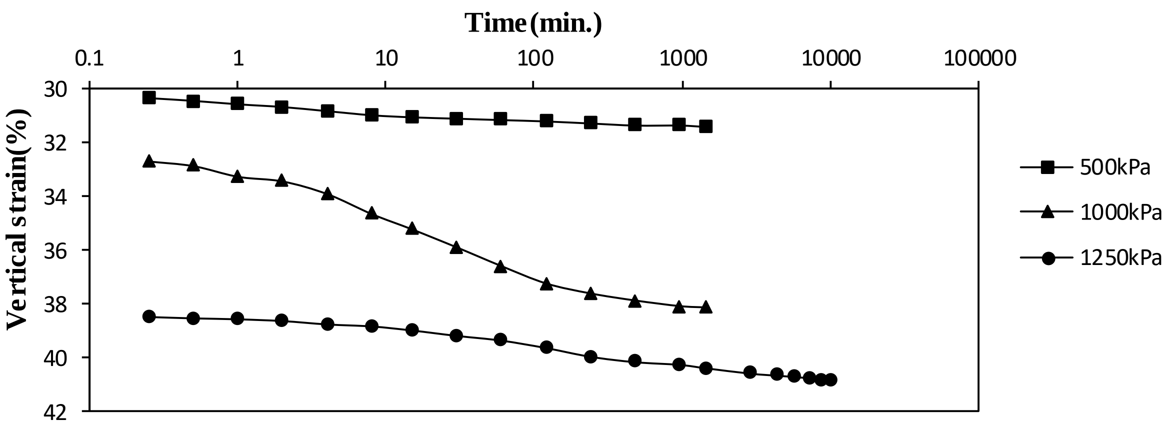


(b)

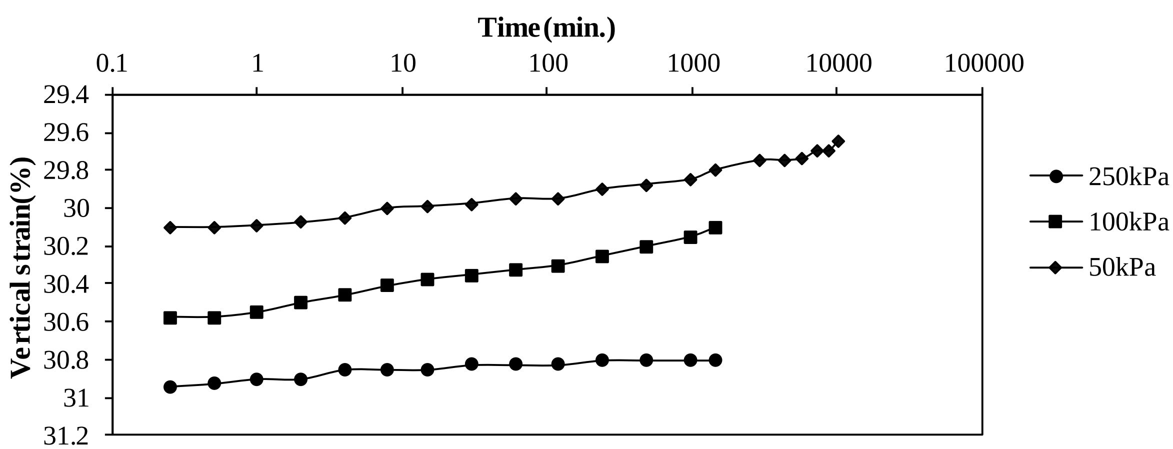
Figure 17. Vertical strain against time in log scale for pure black cotton soil at L1 and U1 stages



(a)

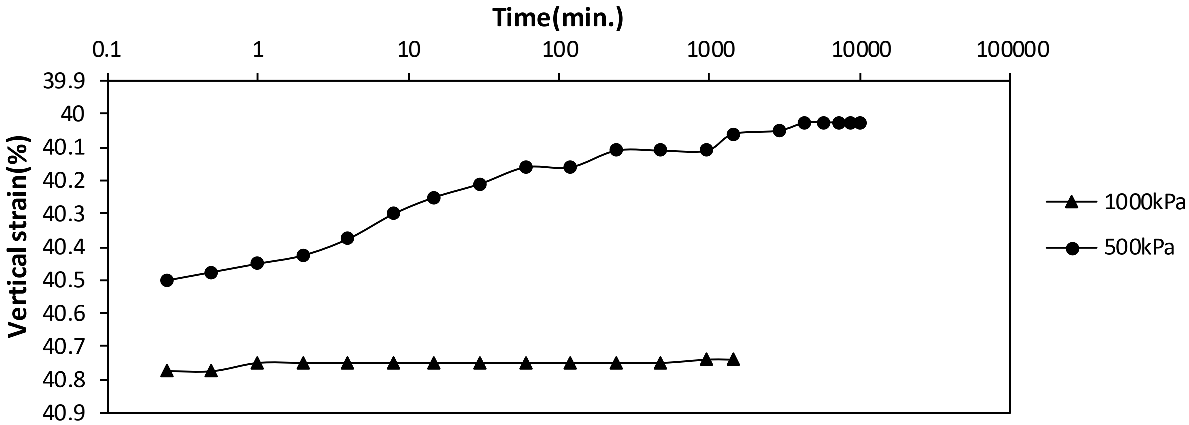


(b)

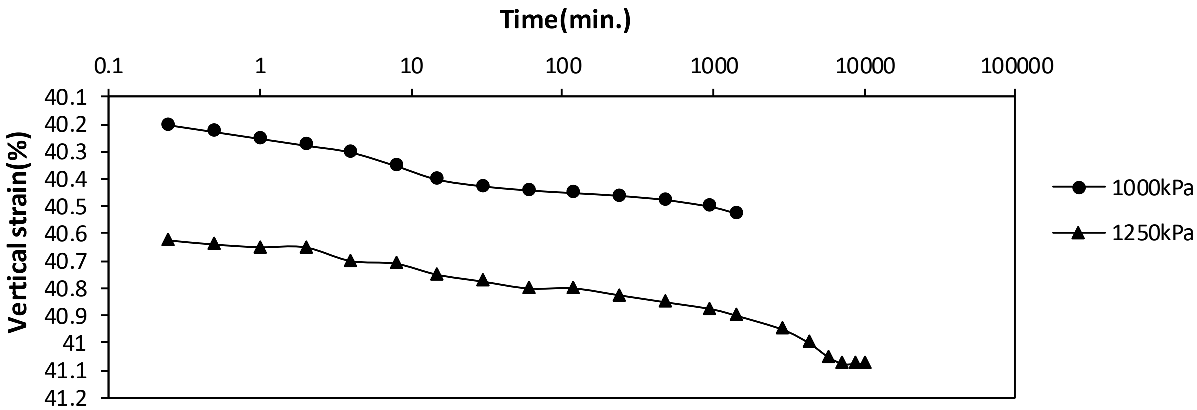


(c)

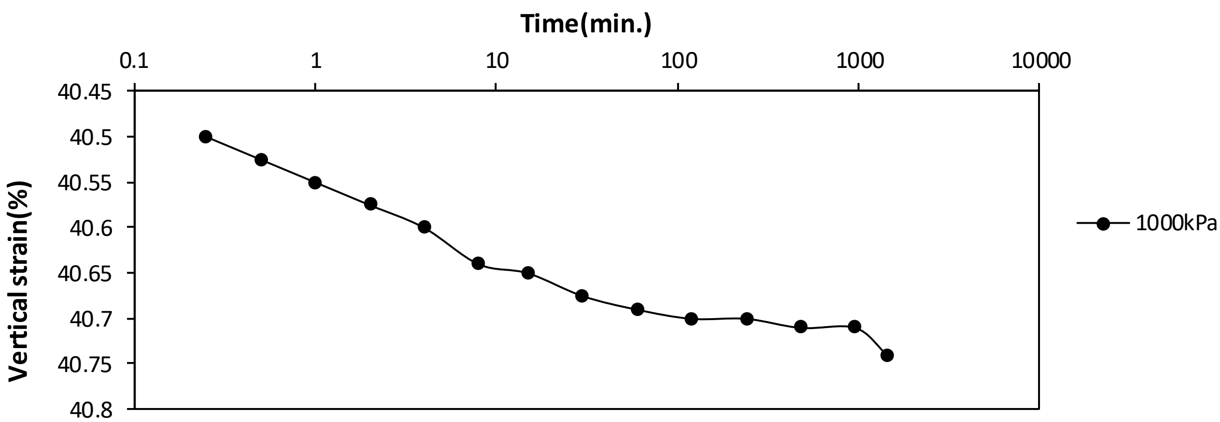
Figure 18.Vertical strain against time in log scale for pure black cotton soil at R1, R2 and U2 stages



(a)

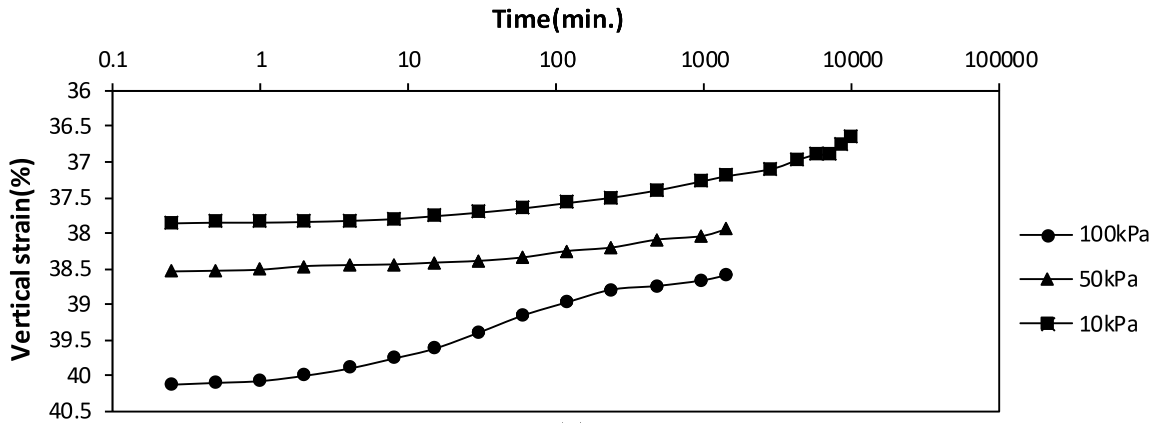


(b)

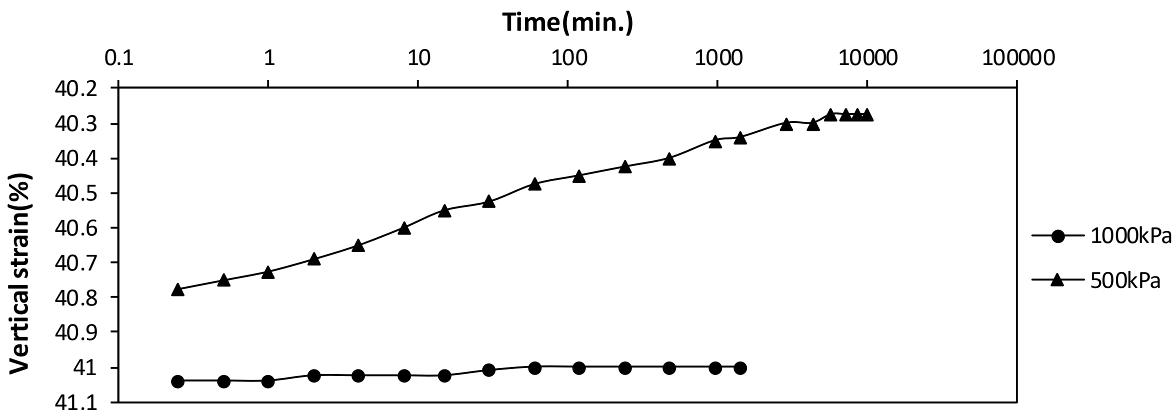


(c)

Figure 19. Vertical strain against time in log scale for pure black cotton soil at U3, R3 and R4 stages



(a)



(b)

Figure 20. Vertical strain against time in log scale for pure black cotton soil at U5 and U4 stages

Table 7. strain at the end of every stage of loading-unloading

L,U	L1	U1	L2	U2	L3	U3	L4	U4
Strain(%)	9.2	8.7	31.07	29.65	40.82	40.025	41.075	36.65

U- unloading, L- loading, R- Reloading

As number of loading-unloading cycles are increasing strain change is decreasing in both creep and swell. This means that after certain loading-unloading cycles, there is no significant difference in strain, in table 8 strain at L3 and L4 are very close to each other rather than strain at L1 and L2. Likewise, strain at U3 and U4 are close to strain at U1 and U2.

4.3. Loading pattern T2 to compare settlement parameters and analyse oedometer data

This loading pattern is used to calculate Creep and swelling indexes (Cc, Cr, C_{α}, and Cs) for different black cotton soil samples with different sand content And compare between these indexes. Again three samples (black cotton in pure state, mixed with 5 % sand, 25% sand) are tested under oedometric condition. Another aim is to examine the influence of the applied stress (loading history) on the swelling behaviour. Here 10 kPa is taken as reference stress along which strain is to examine.

Experiment done on loading pattern T2 and graph between vertical strain (%) and Log(t) is plotted and are shown below in figure 18 (and Appendix A). To understand compression and swelling parameters vertical strain (%) vs effective stress(kPa) are plotted in figure 24.

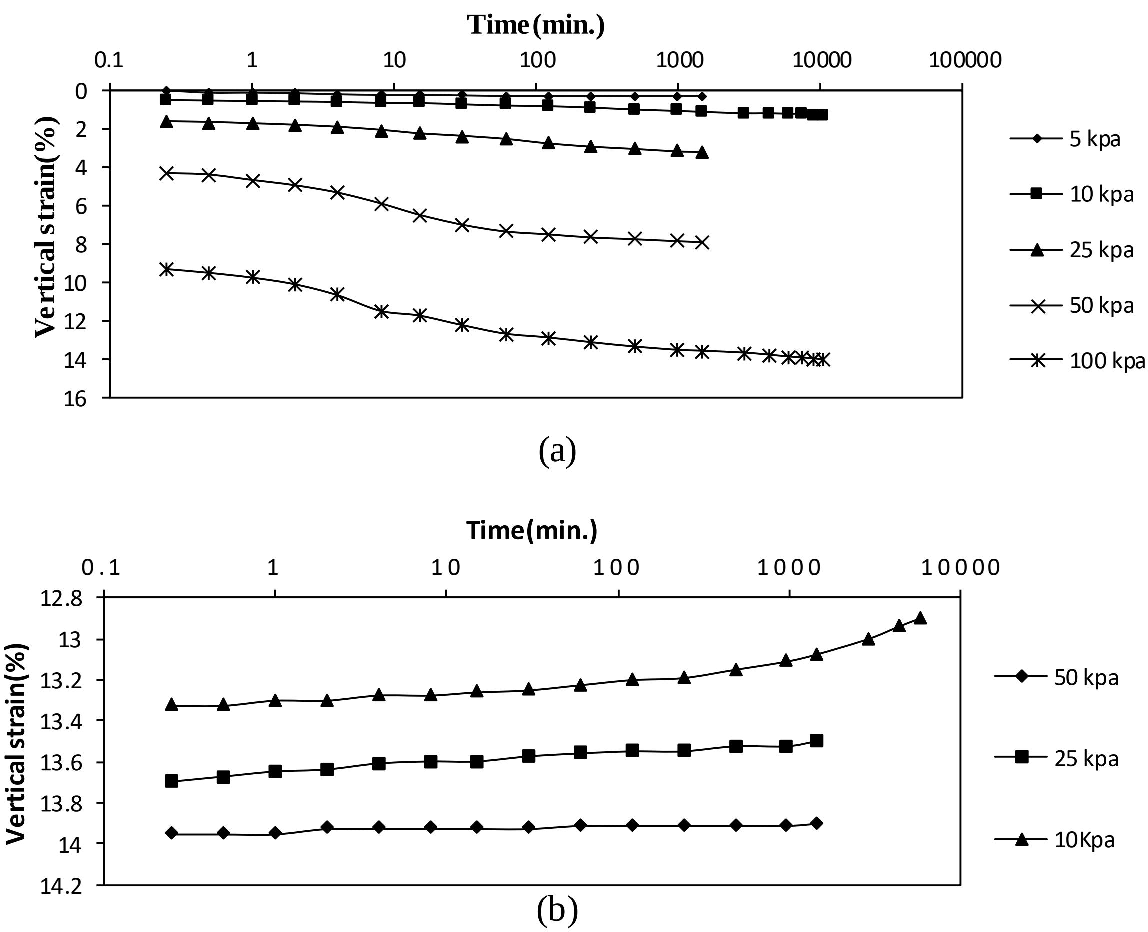
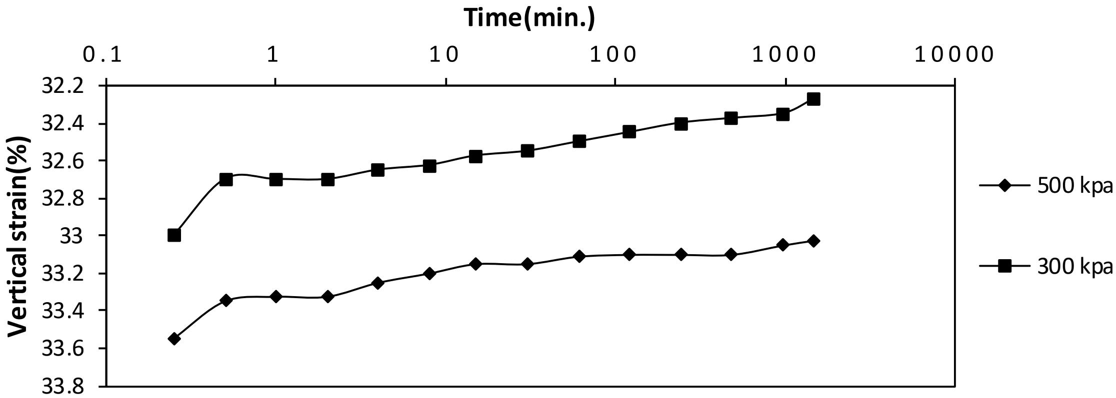
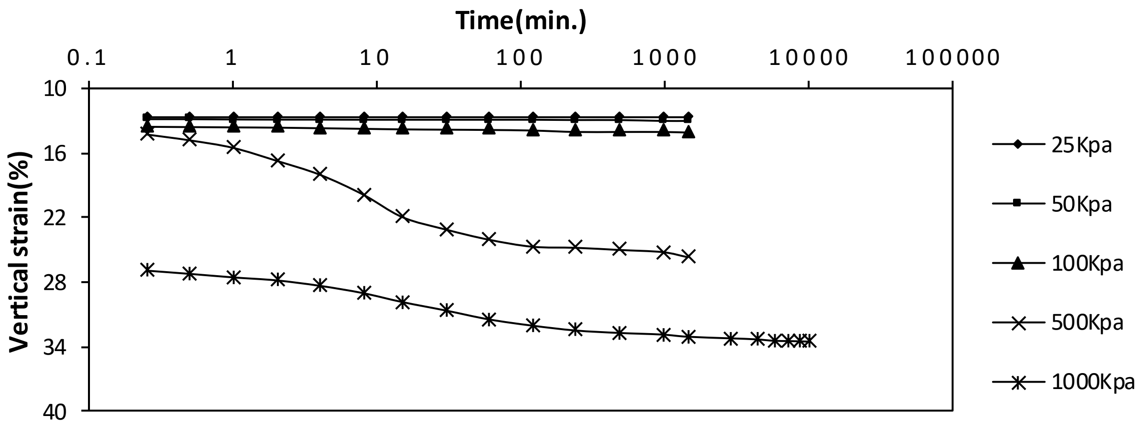


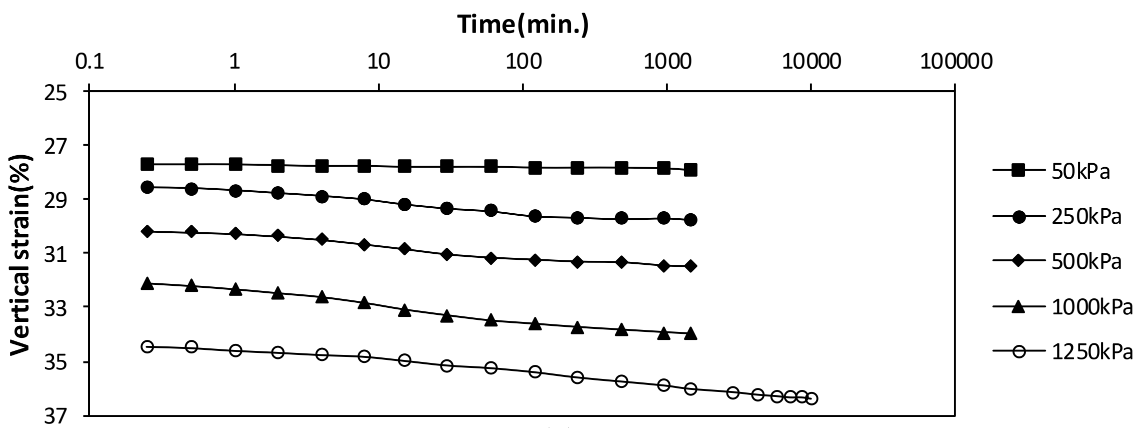
Figure 21. . Vertical strain against time in log scale for pure black cotton soil for L1 and U1 in loading pattern T2



(a)



(b)



(c)

Figure 22. Vertical strain against time in log scale for pure black cotton soil for U2, R1 and R2 loading pattern T2

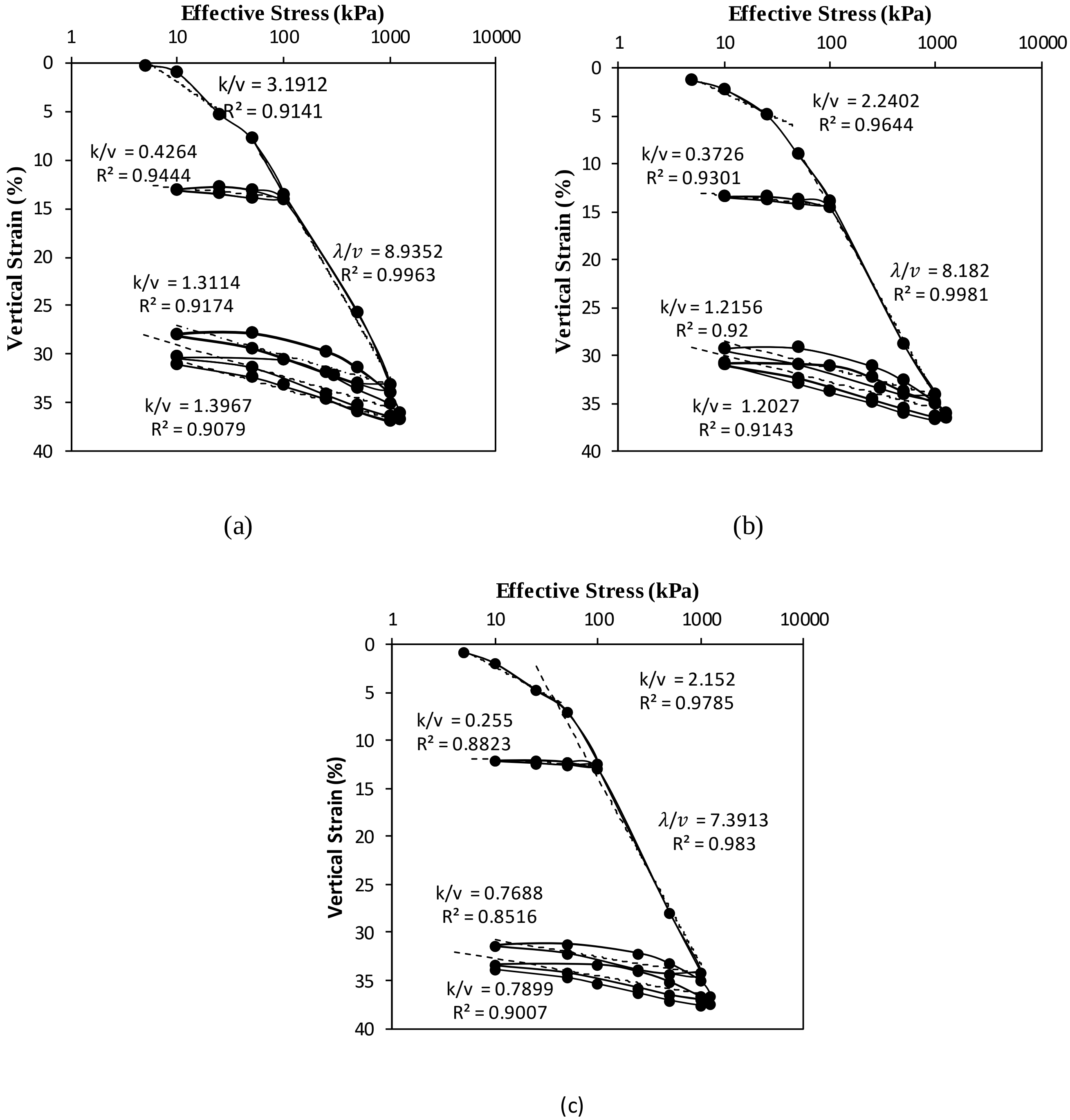


Figure 23. Vertical strain against effective vertical stress in log scale: (a) pure BC, (b) 5% sand content, and (c) 25% sand content.

4.4. Calculation and discussion on Cc, Cr

As discussed in literature review Cc/v and Cr/v have relation with λ/v and k/v respectively and λ/v and k/v can be calculated using graphs in figure 24. Using equation (2) and (3) Cc/v and Cr/v are calculated. And by multiplying it with 1+eo, value of Cc and Cr are calculated as follows for different sand content.

Table 8. Cc and Cr values

Soil sample	Cc	Cr	Cr1	Cr2	Cr3
pure soil	0.386	0.138	0.01818	0.056705	0.060168
5%sand	0.354	0.096	0.016	0.05335	0.052909
25%sand	0.312984	0.017	0.011	0.033037	0.033907

Table 9. clearly shows that increasing sand content decreases Cc and Cr values. And with the increasing number of loading-unloading cycles, the value of Cr is increasing. The compression index Cc is obviously more prominent than the rebounding index Cr1 for all oedometer tests. The rebounding index decreases with higher sand proportion. It means that the sample with a higher mixing proportion of sand is easy to rebound, Because of the high swelling behavior of black cotton soil.

4.5. Calculation and discussion on Cα and Cs

As discussed in literature review Cα/v and Cs/v have relation with $\frac{\boldsymbol{\psi^c}}{\boldsymbol{v}}$ and $\frac{\boldsymbol{\psi^s}}{\boldsymbol{v}}$ respectively. And also discussed that how $\frac{\boldsymbol{\psi^c}}{\boldsymbol{v}}$ can be calculated using equation (9). In equation there are a term $\boldsymbol{t_0^C}$ that is time of end of primary consolidation ($\boldsymbol{T_{EOP}}$) a sample graph to find $\boldsymbol{T_{EOP}}$ is demonstrated in figure 25. In figure where the trend line cut each other is the point of EOP. In figure $\boldsymbol{T_{EOP}}$ is 118.621min. EOP point for T2 loading pattern is shown in table 10.

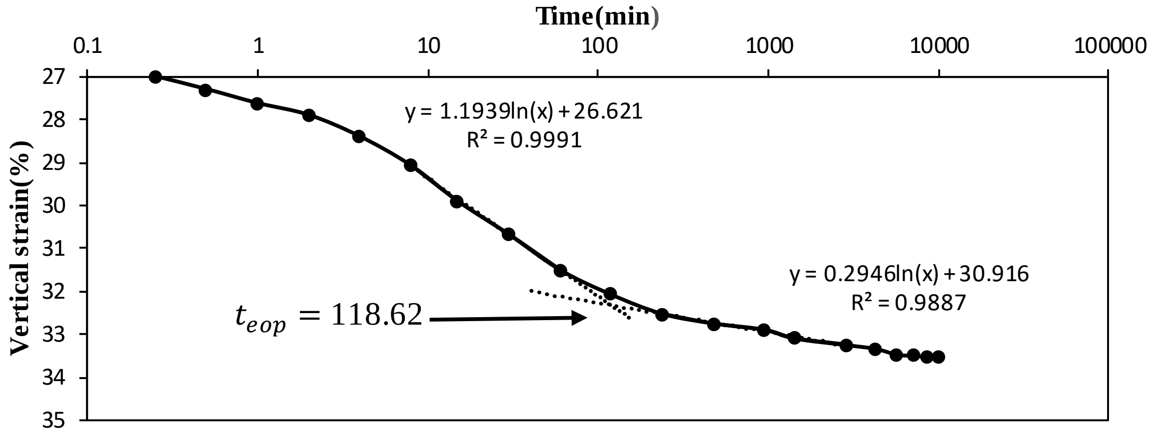


Figure 24. Calculation of EOP point

Using $\boldsymbol{T_{EOP}}$ and strain at EOP and equation (9) a linear curve is drown and value of $\frac{\boldsymbol{\Psi^c}}{\boldsymbol{v}}$ and $\frac{\boldsymbol{\Psi^s}}{\boldsymbol{v}}$ are calculated in creep and swelling process respectively. For some loading graphs are shown in figure 26.

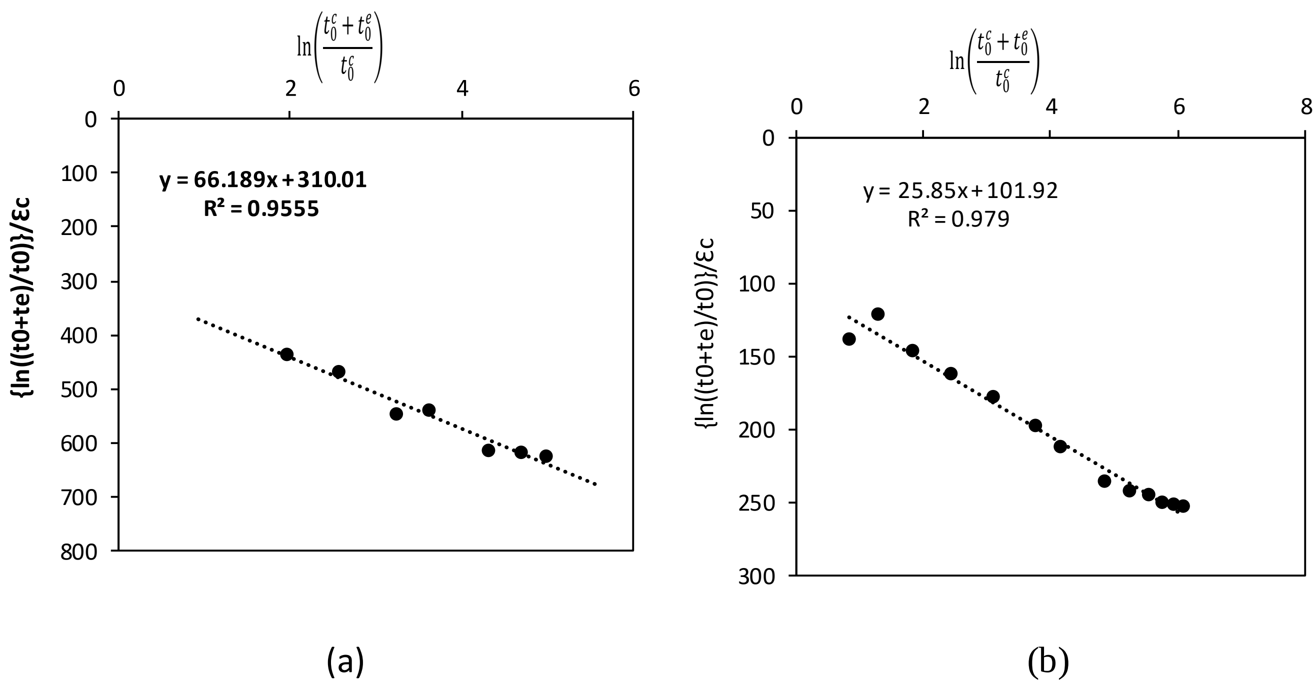


Figure 25. Graph for calculation of (a) $\frac{\boldsymbol{\Psi^c}}{\boldsymbol{v}}$ for 10 kPa unloading (b) $\frac{\boldsymbol{\Psi^s}}{\boldsymbol{v}}$ for 100 kPa loading for pure BC soil.

Using $\frac{\boldsymbol{\Psi^c}}{\boldsymbol{v}}$ and $\frac{\boldsymbol{\Psi^s}}{\boldsymbol{v}}$ and (1+eo) calculation of Cα and Cs is done are shown in table 11.

Table 9. EOP of different loading-unloading case in loading pattern T2

	Sand proportion	pure	5%	25%
100kPa load	EOP(min)	23.14	25.56	43.06
	Strain at EOP(%)	12.48	12.87	11.31
10kPa U1	EOP(min)	35.25	19.26	4.482
	Strain at EOP(%)	13.2	12.2	12.902
1000kPa R1	EOP(min)	118.621	75.33	72.98
	Strain at EOP(%)	32.32	31.19	33.35
10kPa U2	EOP(min)	35.51	16.6	13.05
	Strain at EOP(%)	31.05	31.09	32.45
1250kPa R2	EOP(min)	218.21	302.36	304.49
	Strain at EOP(%)	35.03	35.82	35.58
10kPa U3	EOP(min)	47.7	25.7	24.1
	Strain at EOP(%)	31.1	31.9	34.01
1250kPa R3	EOP(min)	86.98	115.81	53.27
	Strain at EOP(%)	36.4	36.22	37
10kPa U4	EOP(min)	37.4	44.8	17
	Strain at EOP(%)	31.8	31.6	34.5

Table 10. Creep and swelling coefficient for different soil sample in loading pattern T2

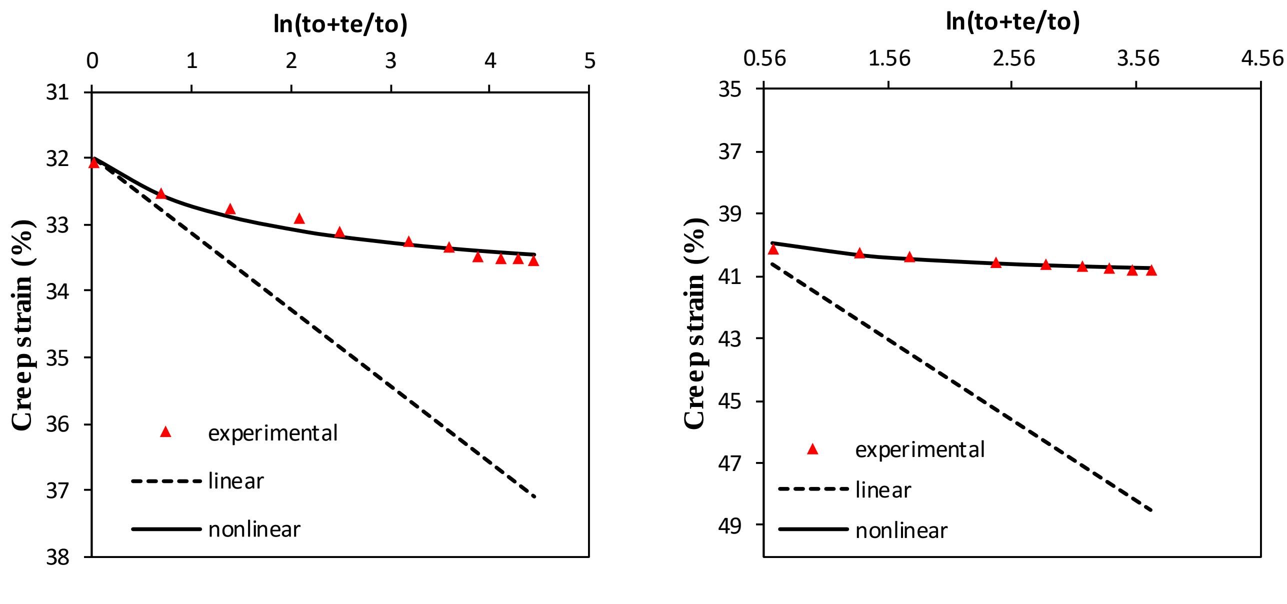
Sand proportion		Pure	5%	25%
100kPa loading	$C_{\alpha1}$	0.005574	0.006019	0.005364
10kPa U1	C_{s1}	0.002215	0.002344	0.00149
1000kPa R1	$C_{\alpha2}$	0.003658	0.006019	0.003129
10kPa U2	C_{s2}	0.005868	0.006811	0.003427
1250kPa R2	$C_{\alpha3}$	0.002494	0.002851	0.001788
10kPa U3	C_{s3}	0.003961	0.004119	0.002235
1250kPa R3	$C_{\alpha4}$	0.006308	0.004594	0.002235
10kPa U4	C_{s4}	0.004547	0.00396	0.003874

Value of Cα and Cs is less in sample of 25% sand then in sample of pure black cotton. the creep behavior coefficient Cα is apparently much smaller than the rebounding coefficient. As loading is increasing Cα is decreasing this is why because During creep, the specimens re-arrange its particles to a denser structure than previous that is more difficult in compression.

4.6. Comparison between the predicted results from EVP model and experimental data

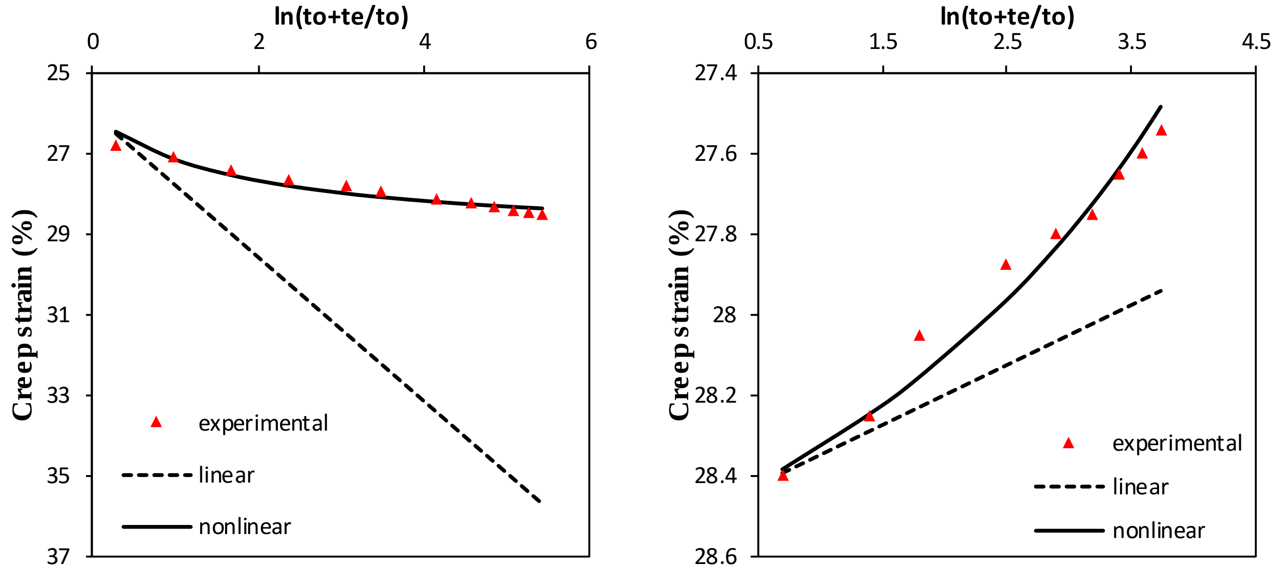
Comparison of experimental data with predicted strain from linear equation (5), (6) and with predicted strain from nonlinear equation (7), (8) will be done and will see that which prediction is more suitable and fit to experimental data.

To confirm, have to check at least two stages from one loading pattern. From those, one should swell and one creep. There are some graphs shown in figure 27.



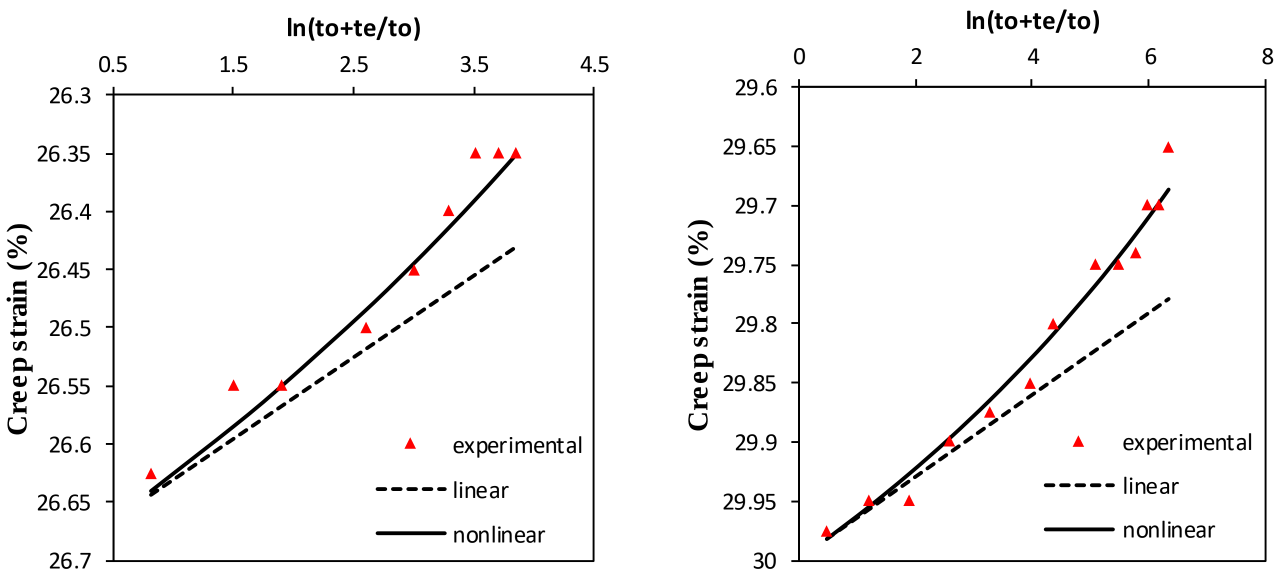
(a) (b)

Figure 26. comparison between linear prediction, nonlinear prediction strain and experimental data for (a) 1000kPa load from T2 loading pattern(R1) and (b) 1250kPa load from T3 loading pattern(R2)



(c) (d)

Figure 27. comparison between liner prediction, nonlinear prediction strain and experimental data for (c) 500kPa load from T1 loading pattern(L1) and (d) 10kPa load from T2 loading pattern (U2)



(e) (f)

Figure 28. comparison between liner prediction, nonlinear prediction strain and experimental data for (e) 10kPa load from T1 loading pattern (U1) and (f) 50kPa load from T3 loading pattern (U2)

From these graphs it is clear that The nonlinear function (7) and (8) is suitable for predicting the long term creep and swelling behaviours as there is a good fitting of nonlinear function with experimental results.

5. Shear test results and interpretation

Shear strength of clayey soil is studied by conducting tests by direct shear apparatus.

5.1. Comparison of different rates

Comparison between shear rates for different samples (pure state, 5% sand ,25% sand) at different normal loads is shown in figure 28 and 29.

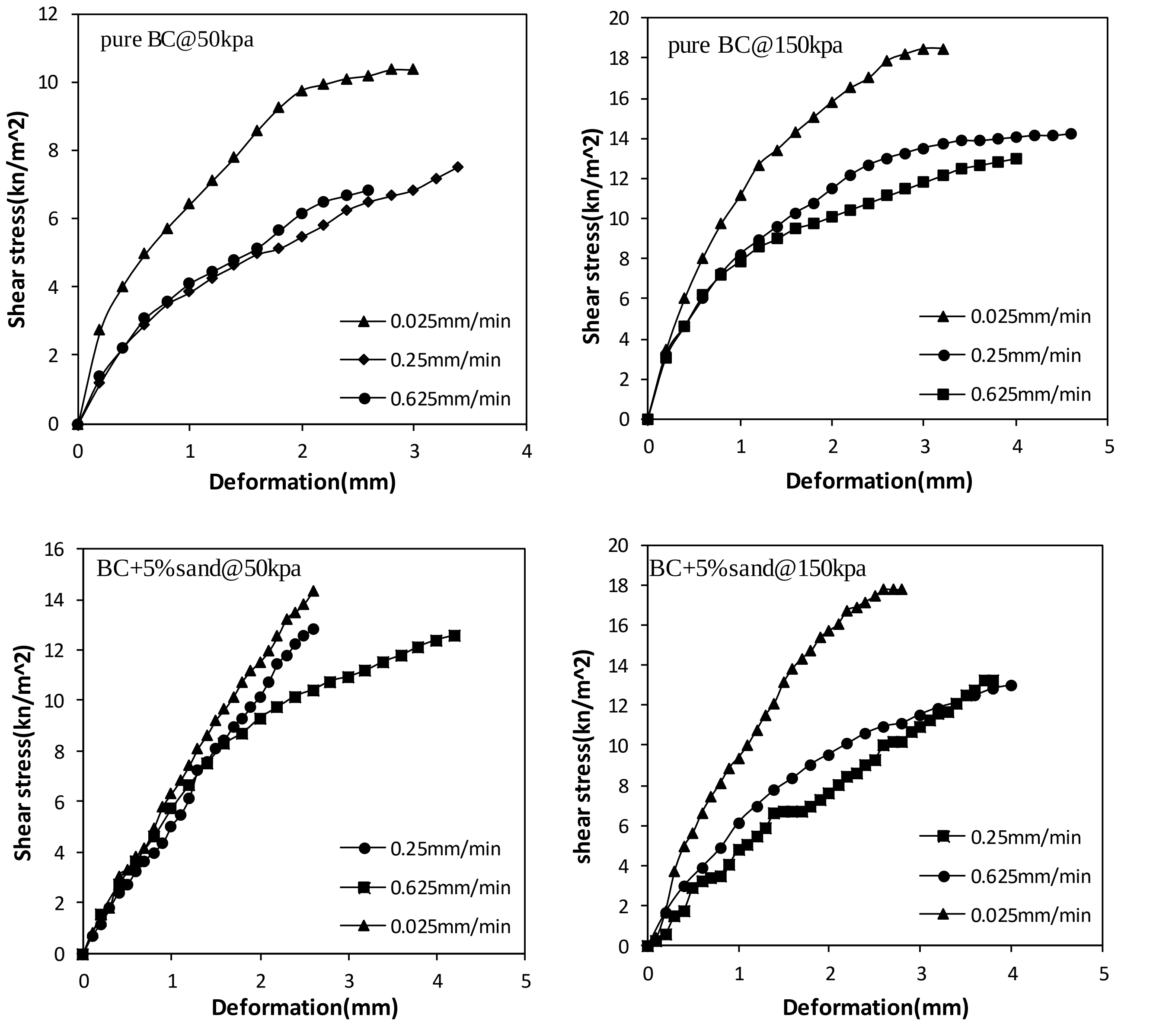


Figure 29. Comparison of different shear rates for (pure black cotton soil) and (BC+5%sand) both at normal loads of 50kPa and 150kPa

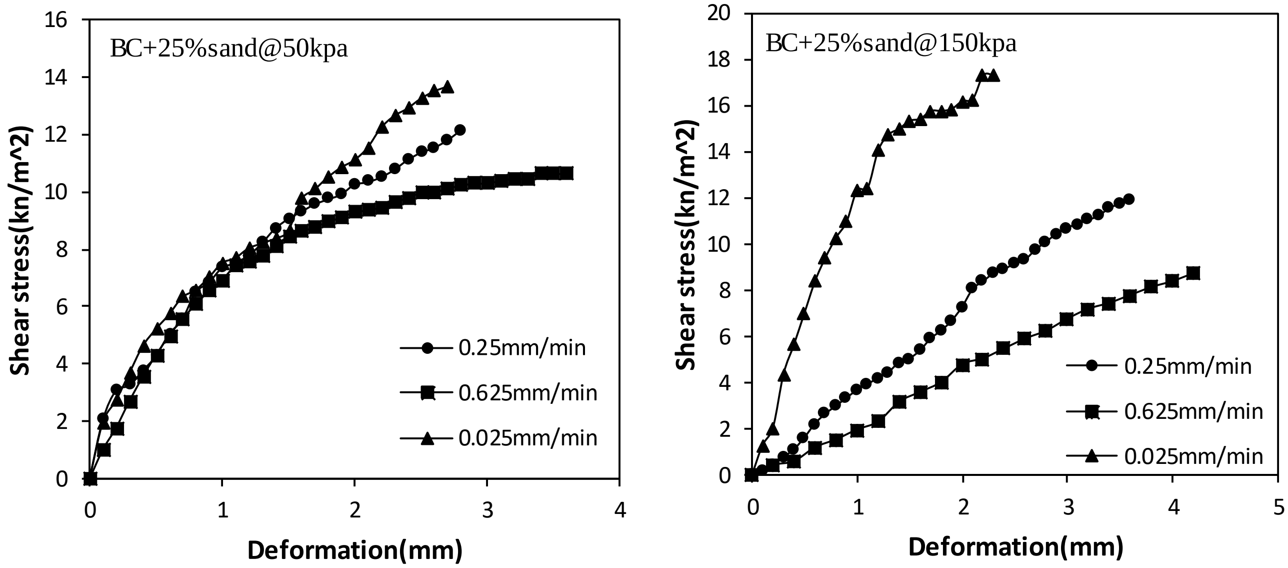
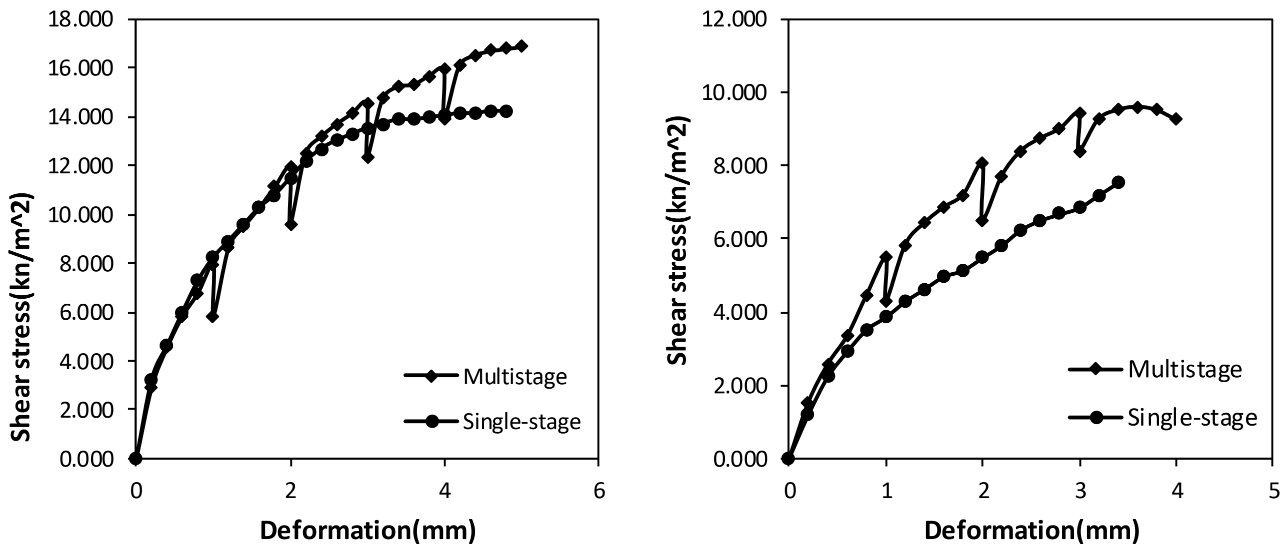


Figure 30. Comparison of different direct shear rates for BC+25% sand at 50 kPa and 150 kPa loading

From the above graph, it is clear that as the rate is decreasing shear strength of the sample is increasing because at a slow rate, there is enough time for particles to arrange in a stable manner. On the other hand, at a fast rate like 0.625mm/min there is less time for the sample to come in a stable position. Maximum shear stress is decreasing as there is an increment in shear rate.

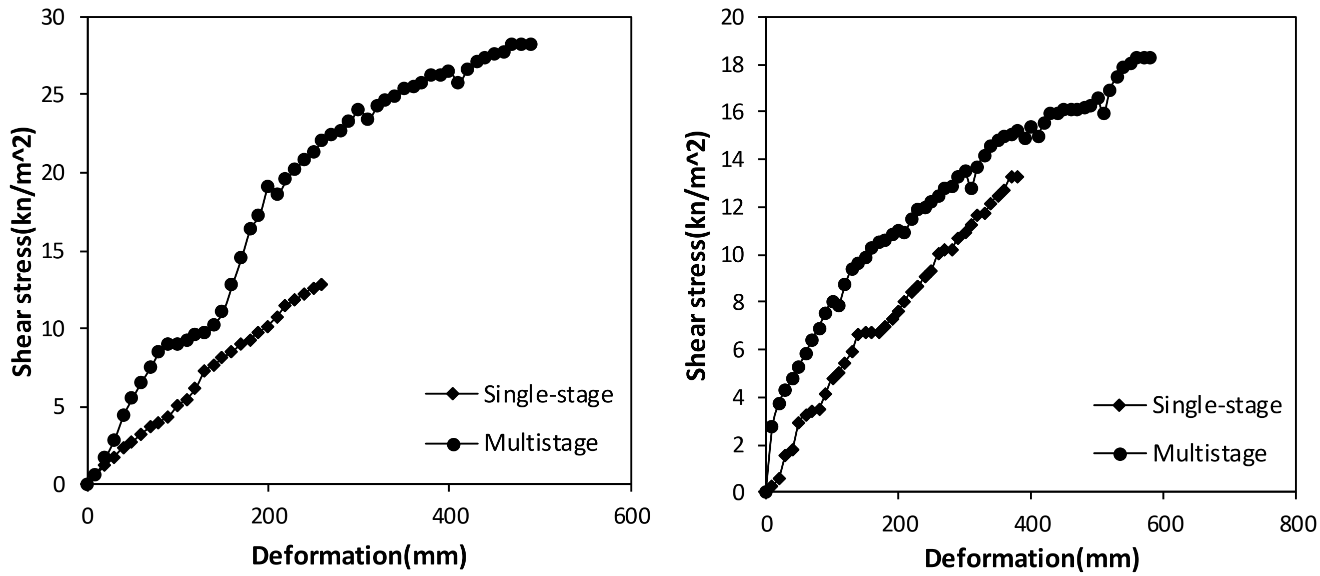
5.2. Comparison between multistage and single stage shear test

Because in slow rate like 0.025mm/min it takes average time of 8 hrs to do one test so it is not possible to continuously operate direct shear machine. There is some time gap (about 30 min), during this time slot there are inter molecule arrangement due to energy relaxation. In this process there are change in shear strength. To identify this difference on one type of sample, multistage and single stage tests are done keeping the rate fix. In this study 0.25 mm/min shear rate kept fix and results are shown below in figure 29.



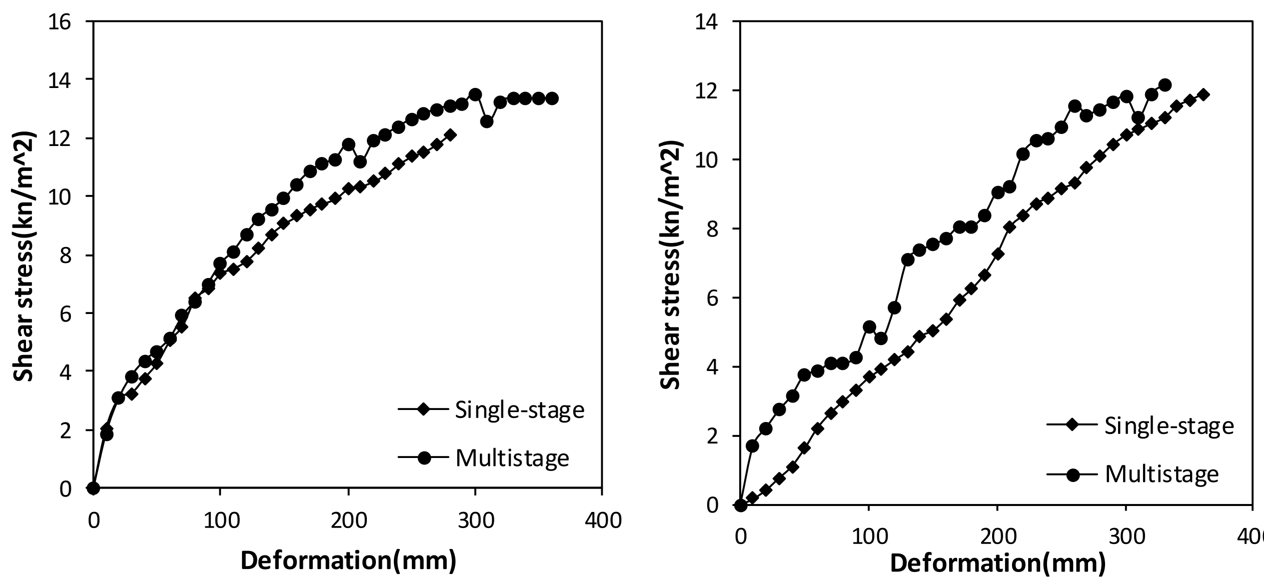
(a) (b)

Figure 31. Comparison between multistage and single stage shear test of pure BC at normal loads of (a) 150 kPa (b) 50 kPa



(a) (b)

Figure 32. Comparison between multistage and single stage shear test of BC+5% sand at normal loads of (a) 50 kPa (b) 150 kPa



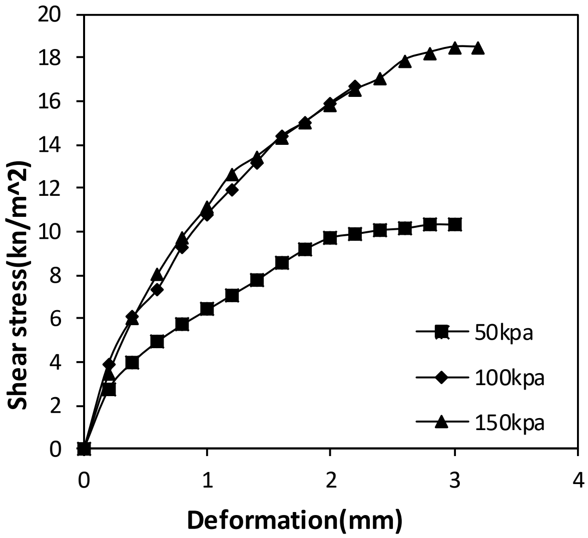
(a) (b)

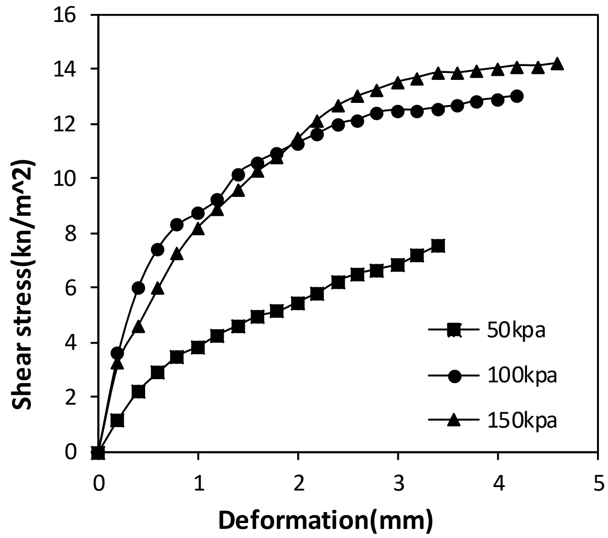
Figure 33. Comparison between multistage and single stage shear test of BC+25% sand at normal loads of (a) 50 kPa (b) 150 kPa

From above graphs, result is that in multistage shear test shear strength is more than in single-stage shear test due to the reason that in multistage test, there is some time gap (about 30 min), during this time slot there are inter molecule arrangement due to energy relaxation and sample molecules come in stable position. The maximum shear stress is increasing in the case of a multistage test as compared to a single-stage shear test.

5.3. Effect of normal loading in shear test

In every test of shear three normal loads (0.5, 1, 1.5 kg/cm^2) are applied. And comparison is shown below for pure soil sample.





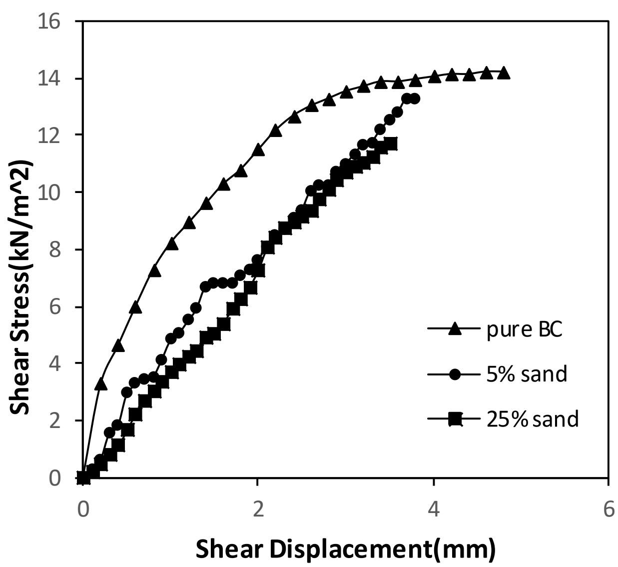
(a) (b)

Figure 34. stress vs strain graph at different normal loads of 50 kPa,100 kPa and 150 kPa for pure BC soil at shear rates of (a) 0.025 mm/min and (b) 0.25 mm/min

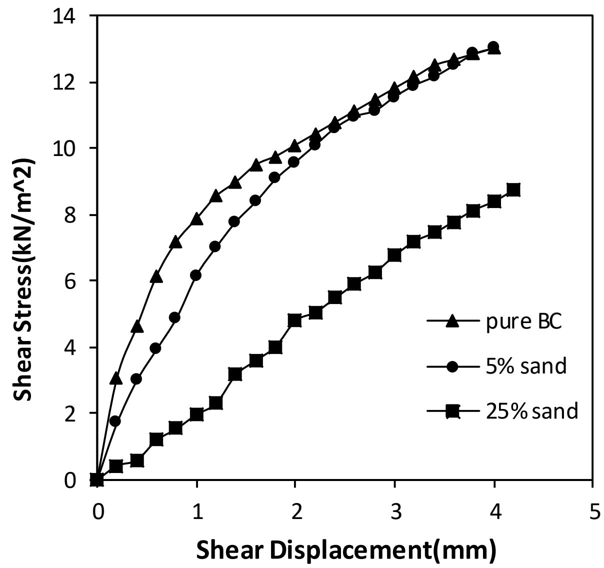
The result from above graphs is that increasing normal load increases the strength of soil sample. At the rate of 0.025 mm/min maximum shear stress is 18.5 kN/m^2 at 150 kPa load and it decreases to 10.3 kN/m^2. Same for rate 0.25 mm/min, here it decreases from 14.2 to 8 kN/m^2.

5.4. Effect of sand proportion on shear strength

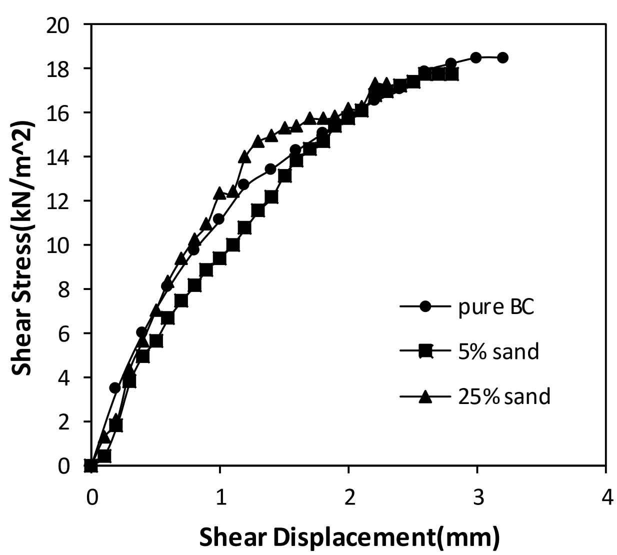
Direct shear test is done on different sand proportion soil samples with fixed rate. Comparison is done on three different rates; Graphs are plotted below in fig 26.



(a)



(b)



(C)

Figure 35. Stress vs strain plot for pure BC soil, BC+5% sand and BC+25% sand at normal load 150kPa at shear rate (a) 0.25 mm/min (b) 0.625 mm/min (c) 0.025 mm/min

From the graphs clearly, there is no significant difference in shear strength with increment of sand proportion. At 0.25 mm/min shear rate, maximum shear strengths are 14, 13.8, 12 kN/m^2 for pure black cotton, mixed with 5% sand, with 25% sand, respectively. For the shear rate of 0.025, maximum shear strength is almost same for all samples. So, in conclusion, it is clear that sand is not improving the shear strength of black cotton soil but, it is also not significantly decreasing shear strength of black cotton soil.

6. Conclusion

A series of multistage oedometer tests for black cotton soil mixed with different sand content were conducted to investigate the compressibility characteristics, creep behavior and swelling behavior during the loading stage and unloading stage, respectively. Direct shear test also conducted to check that how sand content effect the shear strength of black cotton soil. Engineering properties tests and free swell index test are also conducted to check that how increasing sand content effect these properties of black cotton soil. With increment of sand content long term swell strain and creep strain is decreasing.

1. As the number of loading-unloading cycles is increasing, the rate of strain is decreasing in both creep and swell.
2. The compression index (Cc) is obviously more significant than the rebounding index (Cr), average value of Cc is 0.35 and average value of Cr is 0.083 for all oedometer tests.
3. The sand content has a significant impact on soil behaviors. Compared with samples of low sand content, the high sand proportion results in (1) Low settlement and high resistance against loading (2) Low value of Cr and Cc, Cc is decreasing from 0.38 (pure BC) to 0.31 (BC+25%sand) (3) Less time at EOP.
4. The value of $C\alpha$ and Cs is less in a sample of 25% sand than in a sample of pure black cotton, $C\alpha$ is decreasing from 0.0044 (pure BC) to 0.003 (BC+25%sand) and Cs is decreasing from 0.0041 (pure BC) to 0.0027 (BC+25%sand). As the number of loading is increasing $C\alpha$ is decreasing because, during creep, the specimens re-arrange its particles to a denser structure than previous that is more difficult in compression.
5. The nonlinear function proposed by yin is more suitable than the linear function for predicting the long term creep and swelling behaviors as it has a good fitting with experimental results of oedometer data.
6. In direct shear test shear strength of sample is increasing as (1) the rate of shear is decreasing (2) normal loading is increasing (3) in the case of multistage shear test compare to single-stage shear test.
7. Sand has almost negligible impact on the shear strength of black cotton soil.
8. Mixing of Sand in black cotton soil reduces the swelling properties of this soil. The index properties of black cotton soil have been improved due to the replacement of expansive soil by non-expansive sand. Mixing of sand reduced the index parameter significantly.
9. Thus if the sand is an admixture in black cotton soil, maximum advantage in terms of volume stability can be achieved.

7. Future scope

1. It is recommended to study the effect of sand content and identify the optimum value of reconstituting material to stable the black cotton soil.
2. It is prudent to utilize a variety of alternate waste material containing silica or silica compounds as a potential reconstituting material for stabilization of the black cotton soil.
3. Further studies are recommended to ascertain the finding by conducting large scale physical test.

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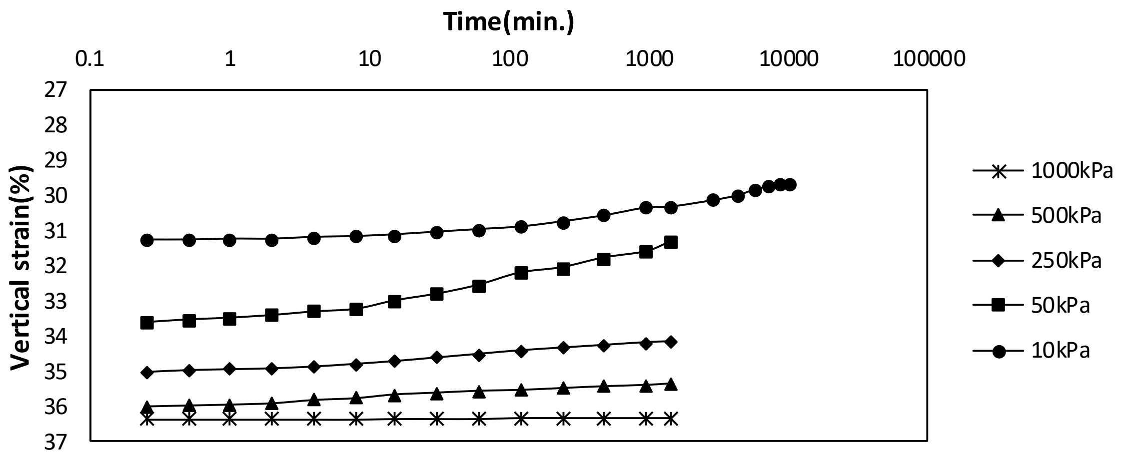
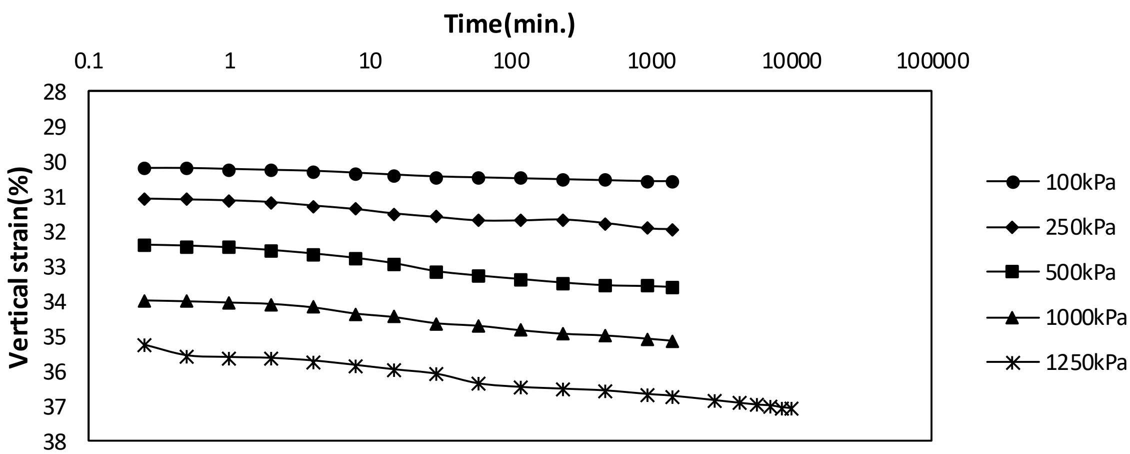
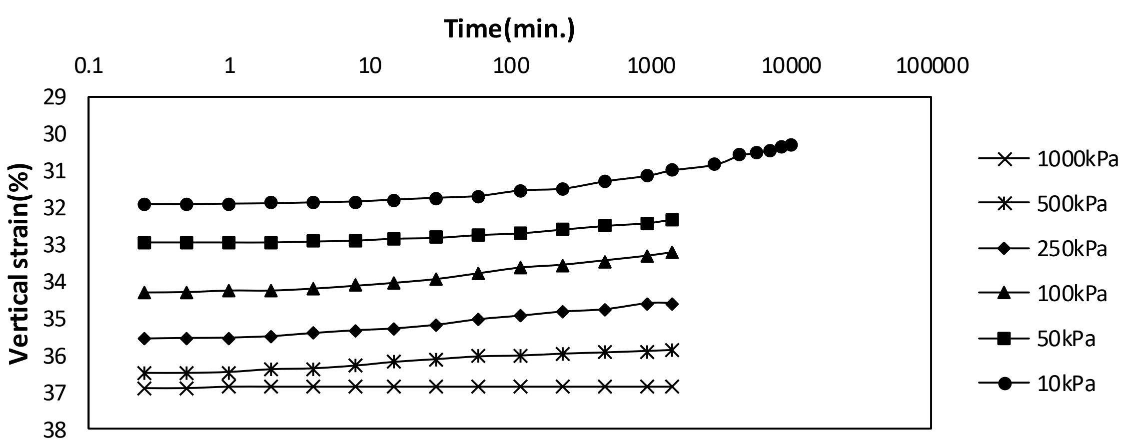
Appendix A



(a)

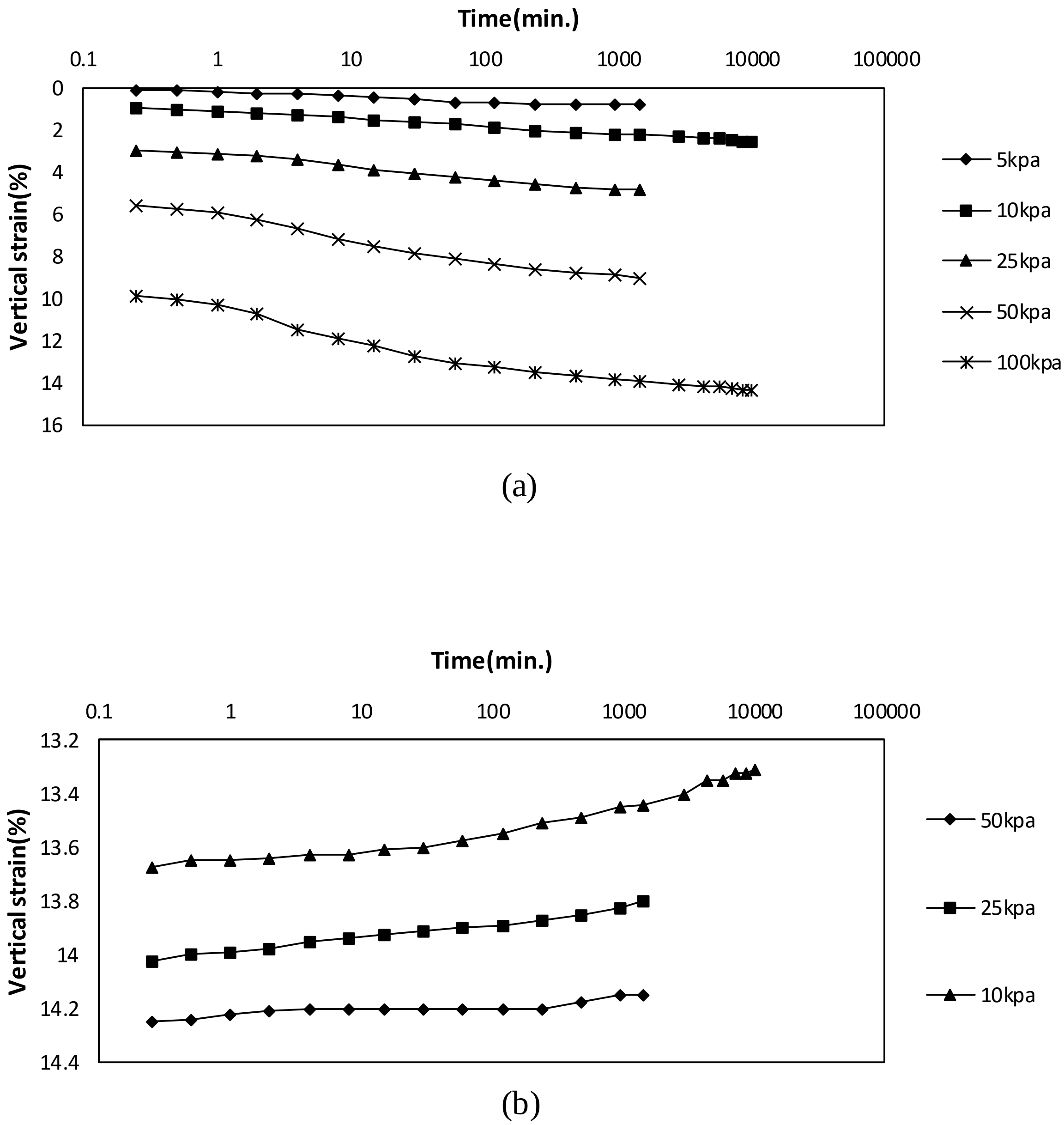


(b)



(c)

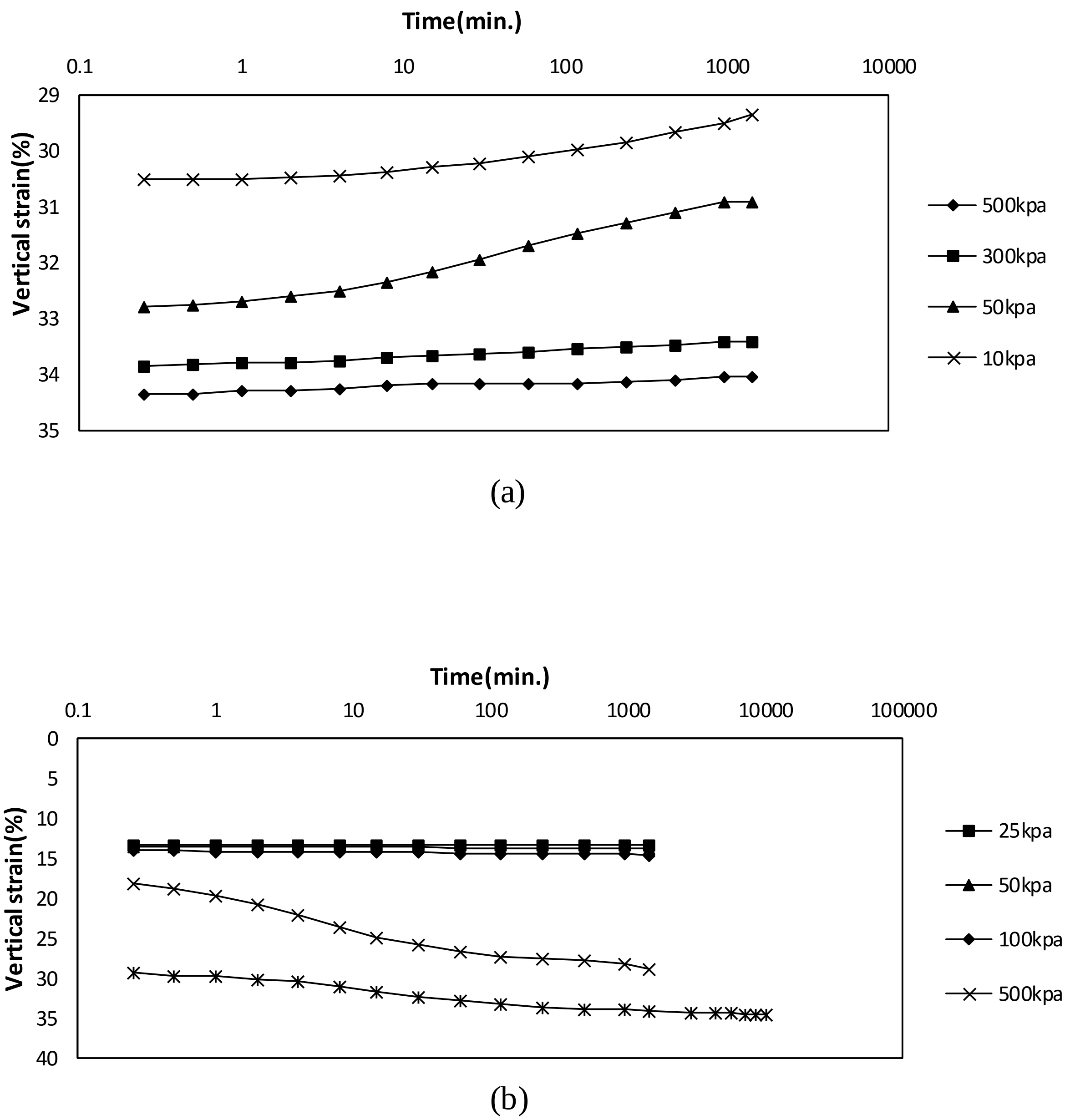
Figure 36. Vertical strain against time in log scale for pure black cotton soil for loading pattern T2 stage U3, R3, U4



(a)

(b)

Figure 37. Vertical strain against time in log scale for black cotton soil mixed with 5% sand for loading pattern T2 stage L1 and U1



(a)

(b)

Figure 38. Vertical strain against time in log scale for black cotton soil mixed with 5% sand for loading pattern T2, stage U2 and L2

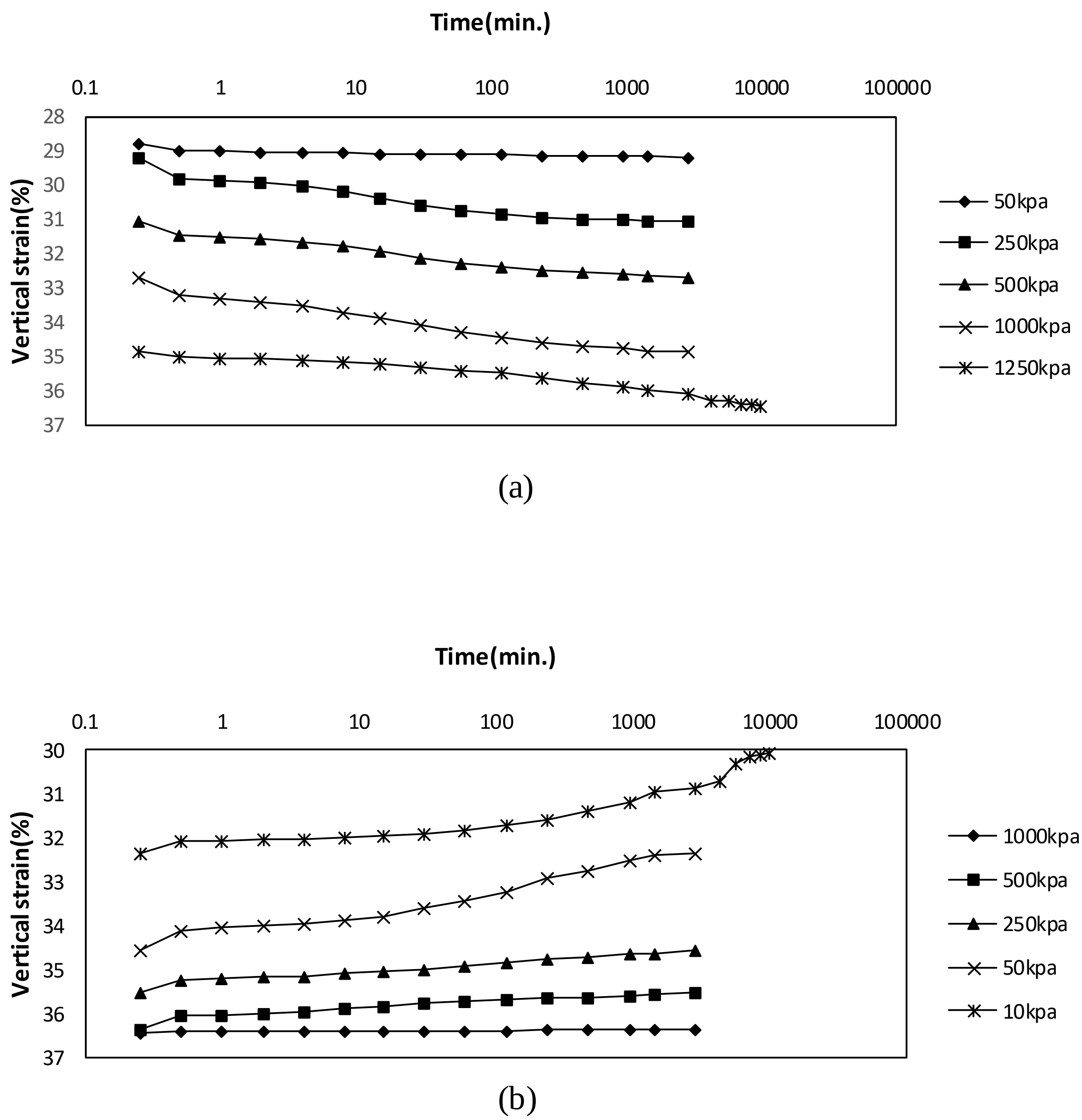
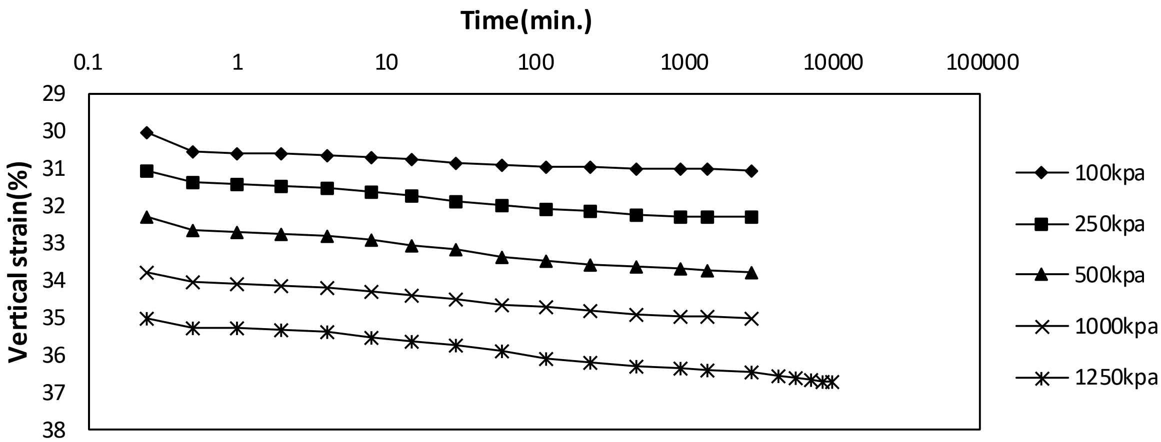
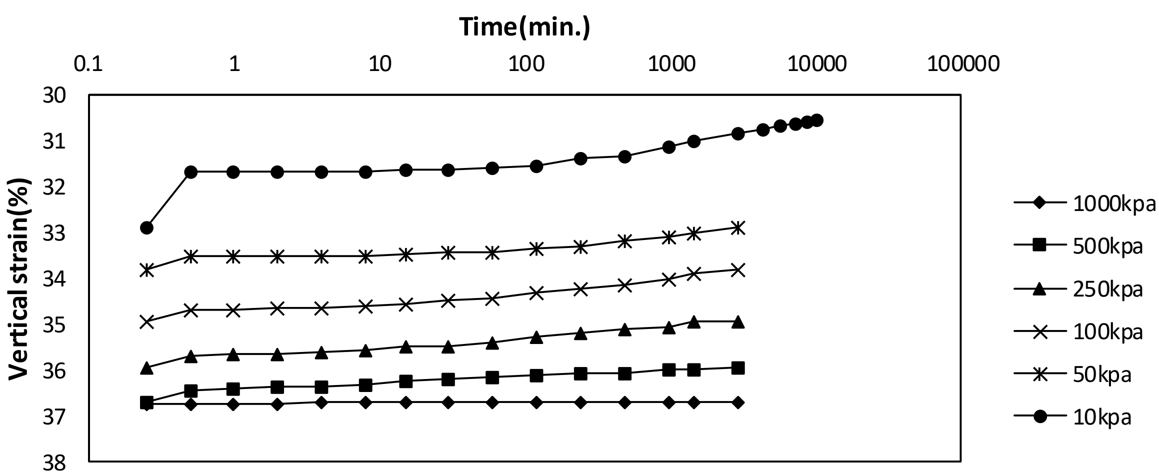


Figure 39. Vertical strain against time in log scale for black cotton soil mixed with 5% sand for loading pattern T2, stage L3 and U3

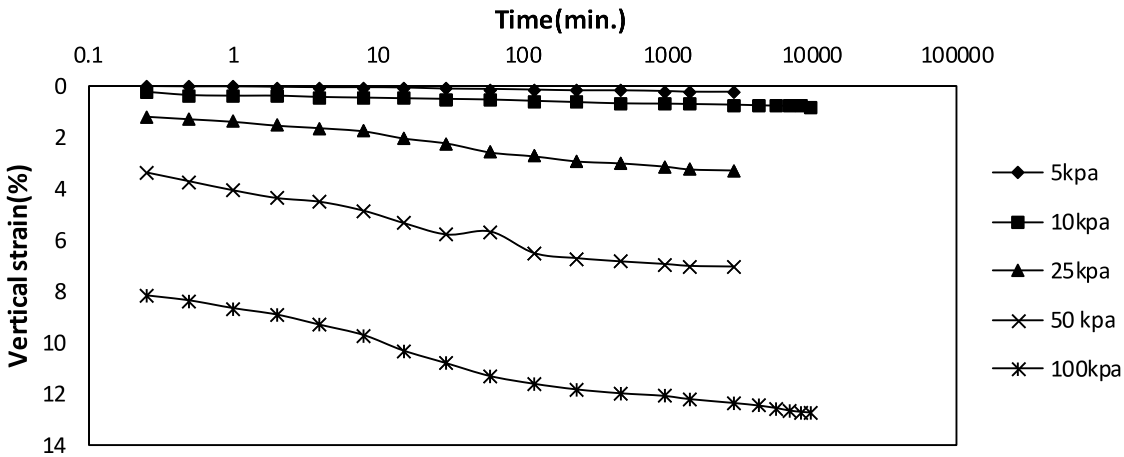


(a)



(b)

Figure 40. Vertical strain against time in log scale for black cotton soil mixed with 5% sand for loading pattern T2, stage L4 and U4



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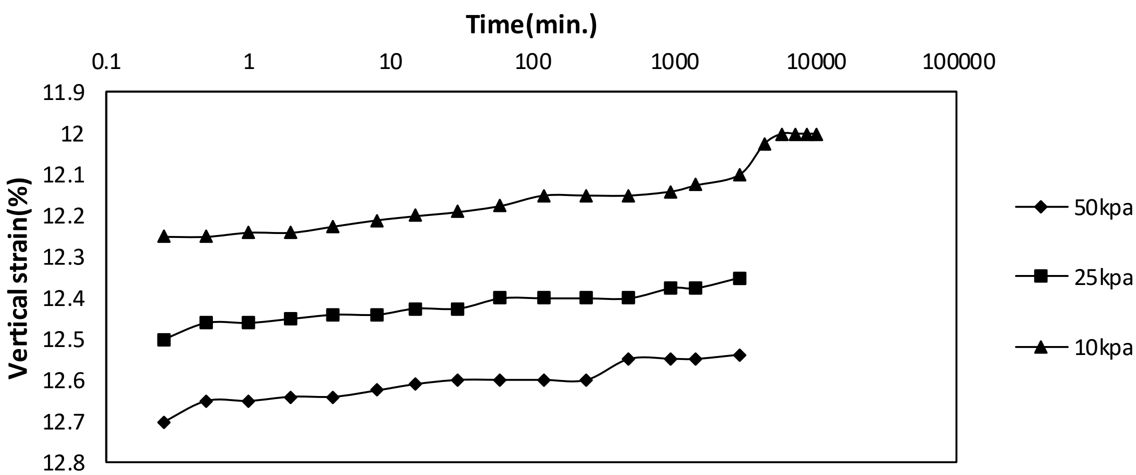
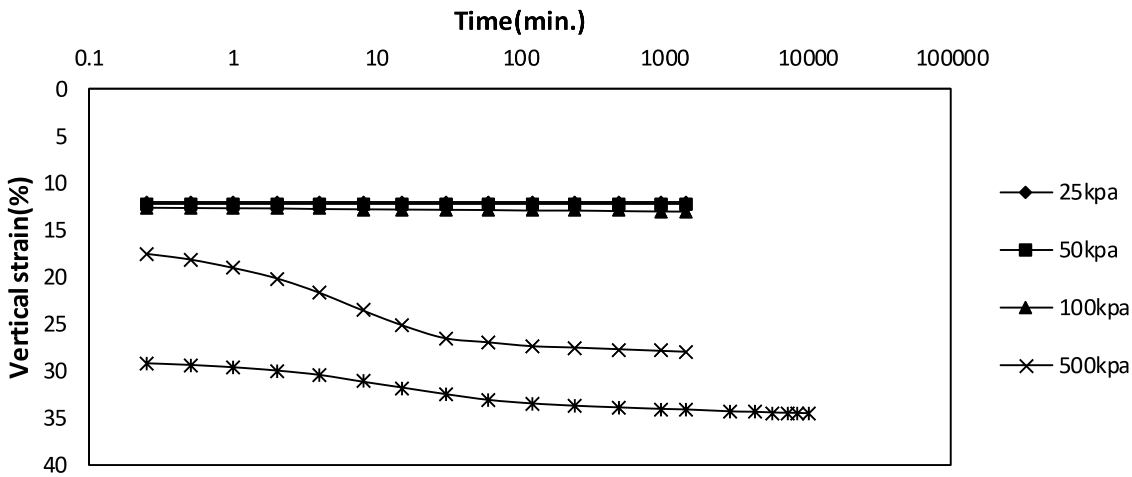


(b)

Figure 41.Vertical strain against time in log scale for black cotton soil mixed with 25% sand for loading pattern T2, stage L1 and U1



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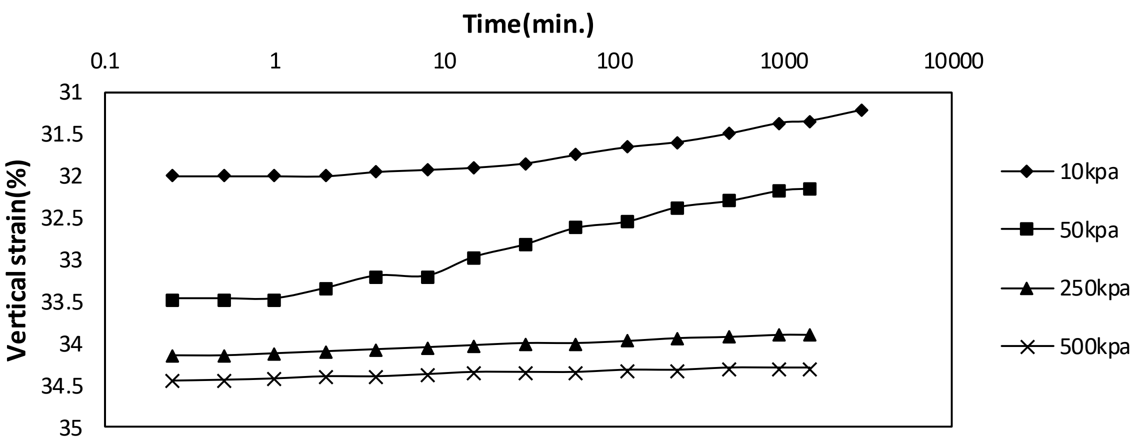
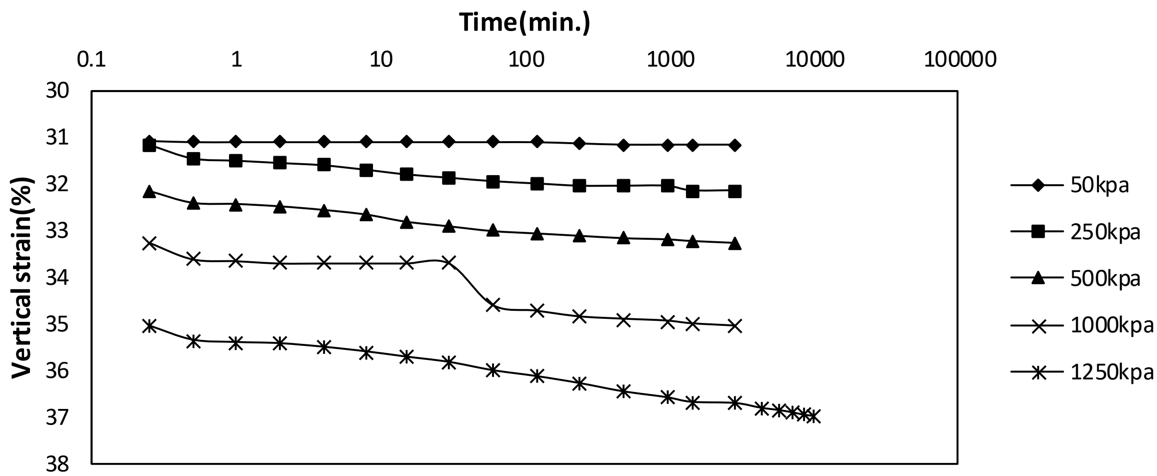
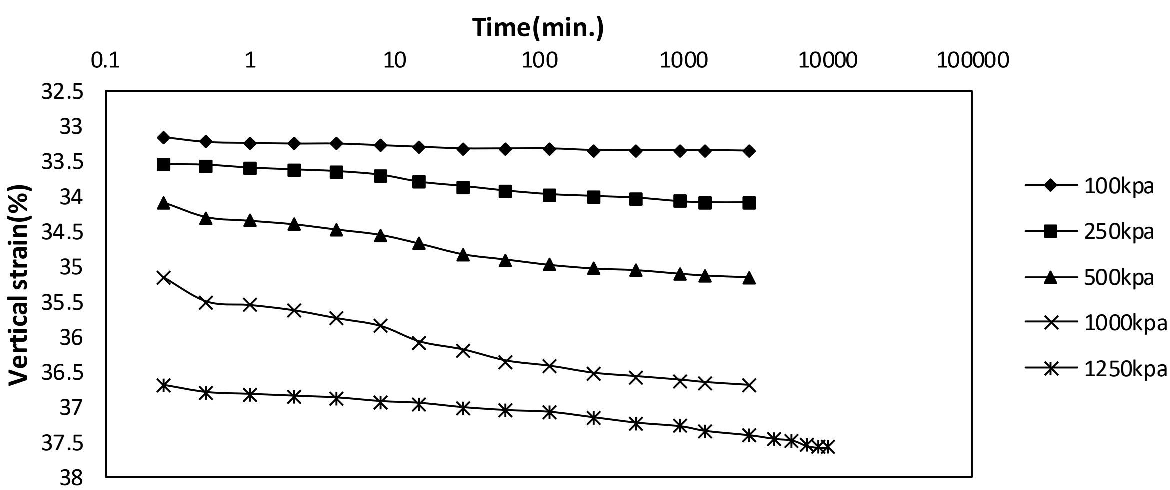


(b)

Figure 42.Vertical strain against time in log scale for black cotton soil mixed with 25% sand for loading pattern T2, stage L2 and U2

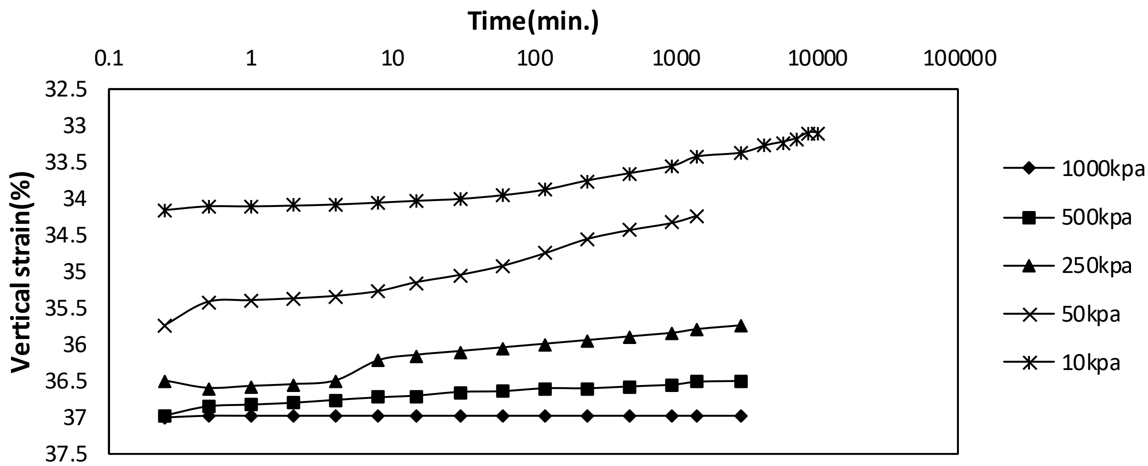


(a)



(b)

Figure 43. Vertical strain against time in log scale for black cotton soil mixed with 25% sand for loading pattern T2, stage L3 and L4



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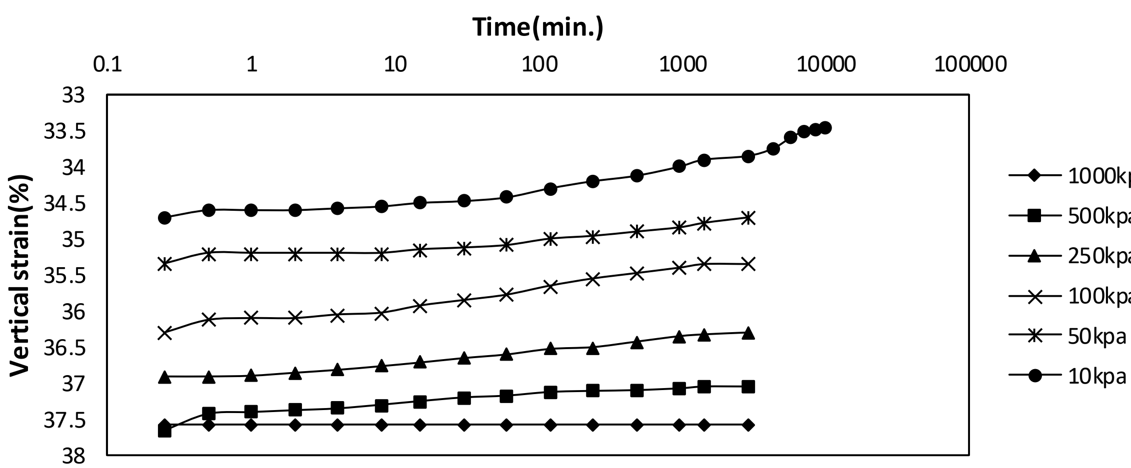

(b)

Figure 44.Vertical strain against time in log scale for black cotton soil mixed with 25% sand for loading pattern T2, stage U3 and U4