ADAMA SCIENCE AND TECHNOLOGY UNVERSITY

SCHOOL OF CIVIL AND ARCHITUCTURAL ENGINEERING

DEPARTMENT OF CIVIL ENGINEERING

SHEAR STRENGTH AND CONSOLIDATION CHARACTERISTICS OF ASELA LATERITIC SOILS

A thesis submitted to the school of graduate studies of Adama science and Technology University in partial fulfillment of the requirements for the degree of masters of Science in civil engineering (geotechnical engineering)

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Approved board of examiners

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DECLARATION

I, the undersigned, declare that this thesis is my original work performed under the supervision of my research advisor Dr. Yadeta Chimdessa and has not been presented as a thesis for a degree in any other university. All sources of material used for this work have also been duly acknowledged.

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LIST OF SYMBOLS AND ABBREVATIONS

Designation		units
C_u	Cohesion for Undrained Shear Strength	KN/m^2
ϕ	Angle of internal friction	-----
UU	Unconsolidated undrained	-----
E	Void ratio	-----
LL	Liquid limit	%
PL	Plastic limit	%
PI	Plasticity index	%
FS	Free swell	%
Gs	Specific gravity	-----
W	Water content	%
N	No. of blows	----
AD	Air dried	----
OD	Oven dried	----
AASHTO	American Association of State Highway and Transportation Officials	----
USCS	Unified soil classification system	----
ASTM	American Society for testing and Materials	----
H_0	initial height of spacemen	mm
D_0	initial diameter	mm
A_O	initial area of spacemen	cm^2
OCR	over consolidation ratio	%
OMC	Optimum moisture content	g/cm^3
MDD	Maximum dry density	g/cm^3
Cc	Coefficient of compression	
Cv	Coefficient of consolidation	cm^2/sec.
mv	Coefficient of volume compressibility	m^2/MN
H_{dr}	Length of drainage path	mm
K	coefficient of permeability	cm/sec

Pc	preconsolidation pressure	KN/m^2
P_0	Initial overburden pressure	KN/m^2
t_{50}	Time during which 50% of consolidation takes place	min.
t_{90}	Time during which 90% of consolidation takes place	min.
Ua	Pore air pressure	KN/m^2
Uw	Pore water pressure	KN/m^2
$б_1$	major principal stress	KN/m^2
$б_3$	minor principal stress	KN/m^2
Δбc	deviator stress	KN/m^2
$Є_f$	Strain	%
T	Shear stress	KN/m^2
γ_w	Unit weight of water	KN/m^3
CD	Consolidated drained	--------
CU	Consolidated undrained	--------
P	Pressure due to the applied load	KN/m^2
SP	Sample	-----
BS	British standard	-----
γ_d	dry unit weight	KN/m^3

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ABSTRACT

The most important engineering properties that determine the bearing capacity of soils are shear strength or ability to resist sliding along internal surfaces within a soil mass and the consolidation characteristics.

Thus, in this study the shear strength and consolidation characteristics of Asela lateritic soil is studied. The shear strength parameters (i.e. Cohesion (C) and internal friction angle (Φ)) and consolidation parameters are determined using UU triaxial and 1-D odometer tests, respectively, on disturbed soil samples remolded with different densities and water contents in order to simulate the field condition.

The shear strength test were done on the cylindrical samples of 38mm in diameter and a height of twice the diameter and the consolidation test were done on the samples of 75mm diameter and thickness of 20mm. In addition, the index test (specific gravity, grain size distribution, Atterberg limits and free swell tests) also conducted on air and oven-dried samples to understand the behavior and classification of the soils. The moisture content of the studied soil ranges between 17.35 – 32.51%, plasticity Index ranges between 11.8-26.4%, clay fraction ranges between 25.5-61.2, free swell ranges between 20-50% and specific gravity ranges between 2.59-2.95. The shear strength parameter, C and ϕ range from 89.63 to 161.48 Kpa and 17 º -24 º, respectively. The consolidation parameters: coefficient of compression ranges 0.193 to 0.581; coefficient of consolidation ranges 0.11 to1.06 cm^2/sec, coefficient of volume compressibility ranges 0.021 to 0.34m^2/MN.

1. INTRODUCTION

1.1 Background of the study

The safety of any geotechnical structure is dependent on the strength of the soil. If the soil fails, a structure founded on it can collapse, endangering lives and causing economic damages. Soils fail either in tension or in shear. However, in the majority of soil mechanics problems, such as bearing capacity, lateral pressure against retaining walls, slope stability, etc., only failure in shear requires consideration. The shear strength of soils is, therefore, of paramount importance to geotechnical engineers. The shear strength along any plane is mobilized by cohesion and frictional resistance to sliding between soil particles. The cohesion and angle of friction of a soil are collectively known as shear strength parameters.

In tropical and subtropical regions lateritic soils have been widely used as construction materials. Detail investigation of this type of soil is needed for the safety of civil engineering structures rest on it. Laterite was coined by Buchanan in India, and the name is derived from Latin word “Later” meaning “brick”. They are classified under residual soils of highly weathered material rich in secondary oxides of iron and aluminum (Fe_2O_3, Al_2O_3). The geotechnical properties of these soils are quite different from those soils developed in temperate or cold regions of the world (Morin, 1975)**.** Therefore, understanding the nature of shearing resistance and consolidation of lateritic soil is necessary in order to analyze soil stability problems such as bearing capacity, slope stability**,** lateral pressure on earth retaining structures and settlement of structures constructed on it such as pavement and foundation of buildings.

Generally the shearing resistance of soil is constituted of the following main components, the structural resistance to displacement of the soil because of the interlocking of the particles, the frictional resistance to translocation between the individual soil particles at their contact points, cohesion or adhesion between the surfaces of the soil particles (Arora, 2000)**.** When the stress increased due to the construction of the foundation or the other loads, the under laying layer of the soil will be compressed, these leads to the gradual consolidation of the soil caused by deformation of soil particles, relocations of soil particles and expulsion of water or air from the void spaces.

Asela is one of the areas where lateritic soils are abundantly deposited as specified by previous researcher. For example, (Belayun , 2012) studied some of engineering properties of soil found in Asela, a thesis presented to school of graduates study, Addis Ababa University. However, still the shear strength and consolidation characteristic; which are key parameters for computation of bearing capacity and slope stability analysis, of this soil are not studied so far. Therefore, this thesis work aimed to cover the lack of background research work on these topics, shear and consolidation characteristics of Asela lateritic soil.

1.2 Objective of the study

1.2.1 General objectives

The general objective of this work is to determine shear strength and consolidation characteristics of Asela lateritic soils

1.2.2 Specific objective

The specific objective of the thesis is to Determining the shear strength and consolidation parameters and index properties of the considered soils with different pretreatment conditions.

1.3 Methodology of the study

The specimen was taken at three different locations, where lateritic soil found, from seven test pits as shown in Figure 1.1. From each test pits two samples are collected from different depth. The laboratory tests on each sample were conducted in accordance with ASTM and British laboratory manual and different pretreatment methods have been carried out. The specimens were collected from the following sites.

- ✓ Around Kenenisa TVET college (SP1, SP2and SP3)
- ✓ Around Asela Catholic child home area (SP4 and SP5) and
- ✓ Around Asela hospital (SP6and SP7)

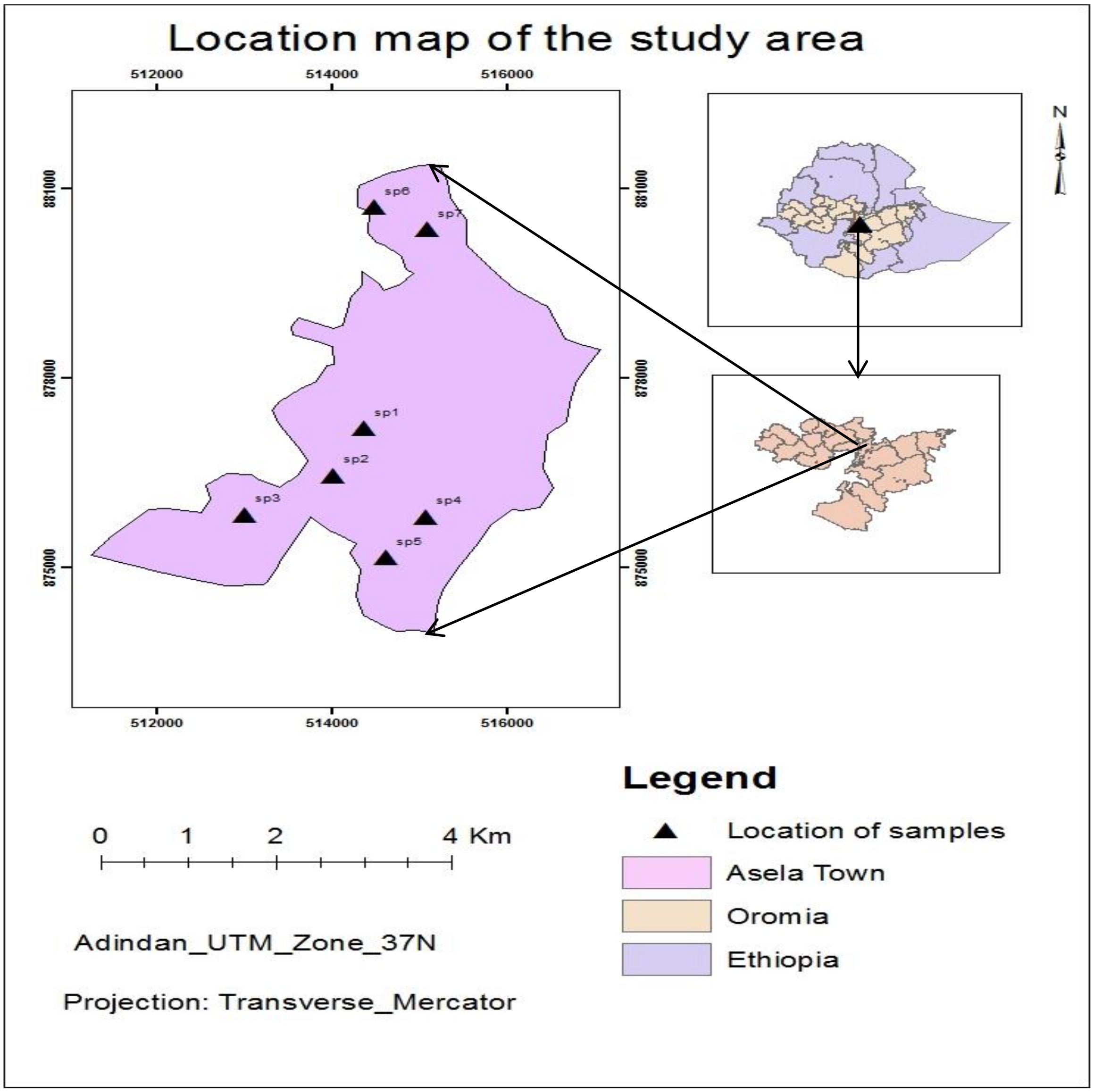


Fig1.1. Location map of the study area and test pit location (using GIS)

1.4 Scope of the study

The main purpose of the research work is to identify the shear strength and consolidation characteristics of Asela lateritic soil samples taken from different locations in Asela town at a depth of 1.5 and 3m. Different pretreatment methods have been applied to a number of samples tested in the laboratory. All tests are carried out on disturbed soil samples.

1.5 Structure of the thesis

The thesis consists of five chapters. The first chapter discusses the background, objective and a brief summary of the work. The second chapter consists of literature review that includes about formation of residual soils, engineering properties of laterites and lateritic soils, typical profiles of residual soils, Pedological and lithological classification of residual soils and the characteristics of shear and consolidation of lateritic soils and their measurements in laboratory. Chapter three describes about the study area of the thesis including topography and climates, soil and geology of the area. Chapter four covers all laboratory test results and discussions briefly. Finally chapter five discusses about the conclusion and recommendation made throughout the study.

2. LITERATURE REVIEW

2.1 General characteristics of residual soils

2.1.1 Formation

Residually soils are formed in different places however, a distinct type of soil called laterite and lateritic soils having its own engineering characteristics and importance developed in a certain favorable circumstances. Commonly they are formed in hot and humid tropically regions with an annual rainfall range between 750mm to 3000mm and abundantly in areas with significant dry seasons. These soils are formed due to, extreme weathering condition and when the chemical processes proceeded more extensively and rapidly, which are extremely leached soils, and are examples of such forms of residual soils. In humid tropical climates when intense weathering involving leaching occurs, it leaves behind a soil rich in Fe and Al oxides giving the soil a deep red color called Laterites. Soils under these classifications are characterized by forming hard, impermeable and often irreversible pans when dried (Blight, 1997). Some more descriptive terms, such as ferruginous soil, ferrallitic soils, and ferrisol are recommended in place of laterite soils (Lyon, 1971).

Laterites are formed by tropical and subtropical weathering, their chemical composition and morphological characteristics are greatly influenced by the degree of weathering to which the parent materials are subjected (Gidigasu, 1976). Laterites are highly weathered materials rich in secondary oxides of Iron, Aluminum or both. Primarily these soils are formed during weathering of Aluminous Silicates in the tropics, the silica alkaline earths are removed in solution by leaching during chemical processes while, the Alumina and Ferric Oxide became hydrated and remains behind. It may also contain large amounts of Quartz and kaolinite. The American Geological Institute (1972) defines laterite as "highly weathered, red subsoil or material rich in secondary oxides of iron, aluminum, or both, nearly void of bases and primary silicates, and may be containing large amounts of quartz and kaolinite". It develops in a tropical or forested warm to temperate climate, and is a residual or product of weathering. Laterite is capable of hardening after a treatment of wetting and drying. The Geological definition of lateritic soil is "A soil containing laterite; also any reddish tropical soil developed from much weathering". The last term is highly generalized but it is not always possible to be more precise in the field.

Laterization: - is a process of weathering of a parent material/mineral, which occurs under conditions favorable to tropical weathering, and when the weathering processes may be so intense and may continue so long that even the clay minerals, which are hydrous aluminum silicate, are destroyed. In the continued weathering, the silica is leached and the remainder consists merrily of aluminum oxide such as gibbsite, or hydrous iron oxide such as limonite or goethite derived from the iron. Laterization process takes place in three stages:

The first stage is the breakdown of primary rock-forming minerals occurs, and this results in the release or formation of clay minerals, mainly kaolinite, and constituent elements such as silica, alumina, iron oxides and oxides of other elements such as calcium and magnesium.

In the second stage, the silica and alkali (calcium and magnesium oxide, among others) are leached and accumulation of sesquioxides takes place. This occurs during Wet seasons of the year and its extent depends on the PH of the ground water and drainage conditions (Gidigasu, 1976)**.** Iron, being carried in ferrous form by Water, is mobile until it is oxidized to Ferric ions. Following the dry season, evaporation leads upward migration of ferrous Ions and opportunity for oxidation by atmospheric oxygen. Iron then precipitates as hydrated ferric oxide gel. Aluminum moves in solution until precipitated as an Alumina gel by dehydration or a change in PH. The sesquioxides (hydrated Ferric Oxide gel and Alumina gel) are adsorbed on the surfaces of the clay minerals. The adsorption occurs through the interaction of positively charged sesquioxides and negatively charged clay particles (Towensend, 1969).

At the third stage, partial or complete dehydration of hydrated colloidal sesquioxides occurs. Dehydration is accompanied by crystallization of amorphous iron colloids into dense crystalline forms in the sequence of Limonite, Goethite, and Hematite. This is accompanied by a change color from yellow or yellow-brown to red. Gelatinous, free iron oxide coats the soil particles, exerting a cementing effect upon the clay, silt and sand size fractions (Gidigasu, 1976).

Based on the above discussion, the three major processes responsible for the formation of Laterites can be summarized as follows:

a) Decomposition: physico-chemical breakdown of primary minerals and the release of constituent elements such as SiO_2, Al_2O_3, Fe_2O_3, CaO, MgO, K_2O, Na_2O, etc. these appear in simple ionic forms.

b) Laterization: is leaching under appropriate condition of combined silica, bases and the relative accumulation or enrichment of oxides, and hydroxides of sesquioxides such as Fe_2O_3, Al_2O_3 and TiO_2. The soil conditions under which the various elements are rendered soluble and removed through leaching or combination with other substances depend mainly on the PH of the ground water and the drainage conditions (Maiginien, 1956)**.** The level to which this processes is carried depends on the nature and the extent of the chemical weathering of primary minerals.

c) Desiccation/Dehydration: this involves partial or complete dehydration of the sesquioxide rich minerals and secondary minerals. The dehydration of colloidal hydrated iron oxide involves loss of water and the concentration and crystallization of amorphous iron colloids into dense crystals, in the sequence Limonite, Goethite with Hematite to Hematite. Dehydration may be caused by climate changes, upheavals of the land, or may also be induced by human activates, e.g. clearing the forests.

Laterization Indices

The degree of laterisation of the soil samples can be evaluated based on several Laterization indices ratios. To measure the extent of laterisation, different workers have defined a number of indices. One of the earlier attempts to define Laterites was based on Silica to Sesquioxide molar ratios. The sesquioxide is designated as R_2O_3 is the combination of aluminum oxide (Al_2O_3) and iron oxide (Fe_2O_3). The chemical formula SiO_2 designates the silica. Laterisation results in a progressive change in the proportions of silica and sesquioxides present in the weathering material, which is in the silica sesquioxide ratio. Iron enrichment, alumina enrichment and desilicification can therefore be expressed by these ratios, which lower as the silica content of clay is reduced and iron and alumina are accumulated. Can described as below

- ✓ $\mathbf{Kr = SiO_2/R_2O_3 = SiO_2/Al_2O_3+Fe_2O_3}$**,**thus depending on the above ratio,

If the ratio is less than 1.33 it is called true laterites and if it is between1.33 to 2.00 the soil is considered as lateritic soils, finally if the molar ratio is greater than 2.00 it is classified as non-lateritic tropically weathered soils.

2.1.2 The engineering properties of lateritic and laterites

Geotechnical characteristics and field performance of lateritic soils may be interpreted in the light of all or some of the following parameters (Gidigasu, 1976):

- Genesis and Pedological factors (parent material, climate, topography, and vegetation, period of time in which the weathering processes have operated),
- Degree of weathering (decomposition, Sesquioxide enrichment and clay-size content, degree of leaching),
- Position in the topographic site, and
- Depth of soil in the profile

2.1.3 Laterites and lateritic soil profile

The soil profile is a vertical cross-section of all the soil horizons from the surface to the fresh unaltered bedrock**.** The alteration of rock by the processes of chemical weathering takes place progressively through a series of events and stages which result in a profile of weathering. Some studies shows that tropically weathered residual soils have an upper clayey zone a few feet thick, underlain by a silty or sandy zone which in turn, passes through a very irregular transition in to weathered and finally into sound rock. The thickness of each member of the profile varies greatly from site to site because of the complexity of the interrelationships among the controlling soil-forming factors such as rain fall, temperature and time, character of the parent rock, topography, drainage conditions and vegetation. Moreover, at a given site there may be great difference in the depth and thickness of the weathered rock within lateral distances of only a few feet. The differences arise because of differences in lithology, such as the presence of a dike or a contact, or because of differences in the degree of jointing or extent of shearing. The latter features increase the permeability and the depth to which weathering can proceed.

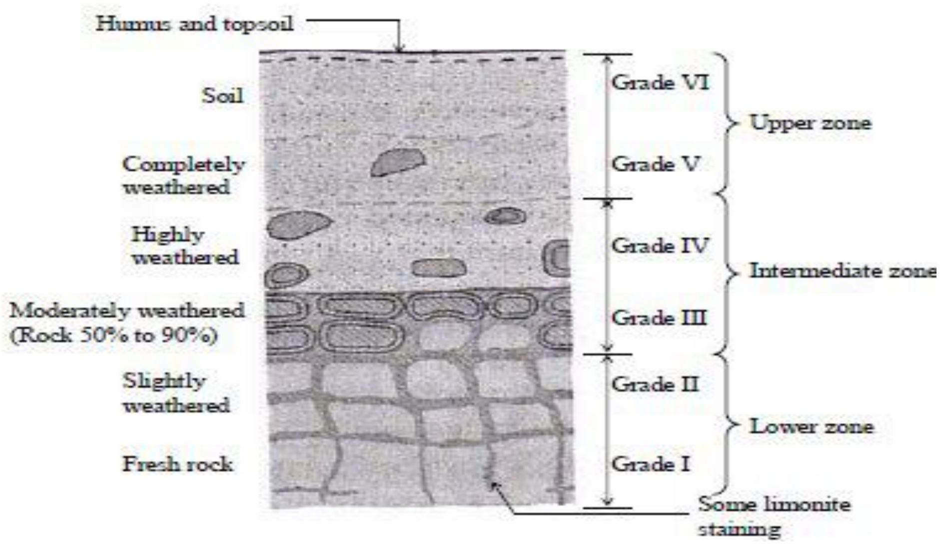


Fig.2.1.Typical weathering profiles of residual soil. (Source: Rahardjo et al. (2004))

2.1.4 Pedological and Lithological Classification of Residual Soils

A soil classification system is arrangement of different soils in to groups having similar properties. The purpose of soil classification is to make possible the estimation of soil properties by association with soils of the same class whose properties are known and to provide the engineer with accurate method of soils description. Conventional soil classification systems focus primarily on the properties of the soil in its remolded state. This is often misleading for residual soils. A practical system for classifying all residual soils based on mineralogical composition, micro and macro structures of the soil. The specific characteristic of residual soils which distinguish them from transported soils can generally be attributed either to the presence of specific clay or structural effects, such as the presence of unweathered or partially weathered rock, relict discontinuity and other planes of weathering and inter-particle bonds (Hanna , 2008)**.**

Special classification system for residual soils

Since classifying residual soils by conventional soil classification system is misleading, special classification system is required due to the following reason:-

I. Unusual clay mineralogy of some tropical and sub-tropical soils.
II. The soil mass in situ may display a sequence of material ranging from true soil to soft rock depending on degree of weathering.

The first step in the grouping of residual soils is to divide them into groups on the basis of mineralogical composition alone, without referring to their undisturbed state. The following three groups are often suggested (Blight, 1997):

Group A: Soils without a strong mineralogical influence

Group B: Soils with strong mineralogical influence derived from clay minerals also commonly found in transported soils.

Group C: Soils with a mineralogical influence deriving from clay minerals only found in residual soils. Properties of the groups are discussed in detail in table 2.1, below. Some additional description is given for sub-groups of group C.

Table 2.1 Characteristics of residual soil groups (Laurence, 2010)

Group		Examples	Means of identification	Comment on likely engineering properties and behavior.
Group A (soils without a strong mineralogical influence)	(a) Strong macro structure Influence	Highly weathered rock from acidic or intermediate igneous rocks and sedimentary rocks	Visual inspection	This is a very large group of soils (including the Saprolite) where behavior (especially in slope) is denoted by the influence of discontinuities ,fissures etc.
	(b) Strong micro structure influence	Completely weathered rocks formed from igneous and sedimentary	Visual inspection, and evaluation of sensitivity, liquidity index, etc.	Theses soils are essentially homogenous and form a tidy group much more amenable to systematic evaluation and analysis than group (a) above ,identification of nature and role of bonding (from relict primary bonds to weak secondary bonds) Important to understand behavior.
	(c)Little structural influence	Soils formed from very homogenous rocks	Little or no sensitivity, uniform appearance	This is relatively minor sub- group. Likely to behave Similarly to moderate over consolidated soils.
Group B (Soils strongly influenced by commonly occurring minerals)	(a) Semecticte (Montmorillonite group)	Black cotton soils, many soils formed in tropical areas in poorly drained conditions	Dark colour (grey to black and highly Plasticity.	These are normally problem soils found in flat or low lying areas of low strength, high swelling compressibility, and high swelling and shrinkage characteristics
	(b) Other minerals			This is likely to be a very minor sub group.

Group C (Soils strongly influenced by clay minerals essentially found only in residual soils)	(a) Allophane Group	Soils weathered from volcanic ash in the wet tropics and in temperate climates	Very high natural water contents and Irreversible change on drying	These are characterized by very high natural water Contents and high liquid and plastic limits. Engineering properties are generally good through in some cases high sensitivity could make handling and compaction difficult
	(b)Halloysite group	Soils largely derived from Older volcanic rocks forming Especially tropical red clay	Reddish colour, well drained topography and volcanic parent rock are useful indicators	These are generally fine or course Soils, of low to Medium plasticity, but low activity. Engineering Properties generally good. (Note that there is often some overlap between Allophone and Halloysite Soils).
	(c)Sesquioxide Group	This soils group loosely referred to as Lateritic, or Laterite	Granular ,or nodular Appearance	This is a very wide group, ranging from silty clay to Coarse sand and gravel. Behavior may range from Low plasticity to non-plastic gravel.

2.2 Shear strength characteristics and its measurement in residual soils

2.2.1 General

The shear strength is measured in terms of two soil parameters, cohesion or inter-particle attraction, and angle of internal friction, the resistance to inter particle slip. Grain crushing, resistance to rolling, and other factors are implicitly included in these two parameters. This behavior is well represented by the Mohr-Coulomb failure criterion given as,

$$S = C + б \tan \Phi \quad \text{……………………………………………………………} [2.1]$$

Where: S= shear strength

б = normal stress on shear plane

C= Cohesion

The shear parameters are often taken as constant but they depend on drainage condition, previous stress history, and current state (particle packing or density or water content). Therefore, soils seldom exhibit unique strength parameters and obtaining accurate values is not a trivial task. Also while equation above (2.1) has a linear form, in real soils, for reasons just cited it is often nonlinear. The shear envelope defined by the above equation obtained from the locus of tangent points to a series of Mohr's circles constitutes the limiting state of stress, which can be imposed on the soil. In general, since two parameters are involved, two or more tests must be performed for a graphical (most common) or simultaneous equation solutions (Bowles, 1978)

In general many geotechnical problems such as bearing capacity, lateral earth pressures, and slope stability are related to the shear strength of a soil. The shear strength of a soil can be related to the stress state in the soil. The stress state variables generally used for an unsaturated soil are the net normal stress **(б − Ua)** and the matric suction **(Ua − Uw).** Shear strength is used in soil mechanics to describe the magnitude of the shear stress that a soil can sustain. Lateritic soils usually have relatively high to very high cohesion and internal friction angle. This behavior is generally attributed to the cementation which is taking place among the individual soil grains by the binding action of sesquioxides. The strength behavior of lateritic soils was observed to be very much dependent on the moisture content of the sample tested. Decreasing moisture content usually causes an increase both in cohesion and internal friction angle. This behavior is attributed to the variation in soil structure with varying moisture content, such that, as the soil dries out part of the hydrated colloidal iron and aluminum oxides dehydrates and forms strong bonds among certain soil grains which, in turn, causes an increase in shear strength. The shear resistance of soils is a result of friction and interlocking of particles, and possibly bonding or cementation at particle contacts. Due to interlocking, particulate material may expand or contract in volume as it is subject to shear strains (Rahardjo, 1993).

The high effective strength parameters found in most residual soils arise from several factors, including the following:

Residual soils contain clay minerals that tend to have good frictional properties. The exceptions are those derived from the weathering of soft sedimentary rocks, such as shale, and the black cotton soils in which montmorillonite is predominant. Most residual soils have significant micro structural effects, which contribute positively to the shear strength of the material. The microstructure also generally contributes a significant cohesive component to the shear strength of the material, that is, a significant C' value. Shear strength characteristics of Lateritic soils are generally influenced by genetic and compositional factors and the method of pretest sample preparations as well as testing procedures and condition of testing. In addition it depends significantly on the parent materials and degree of weathering (i.e. degree of composition, Laterization and desiccation) which depends on the position of the sample in the soil profile and compositional factors as well as the pretest preparation of the sample. Variation with depth in the shear strength parameters for soils over volcanic ash within the same pedologic horizon can affect the values of shear parameters. The higher angle of internal friction where noted at greater depths that means in the C-horizons. When comparing samples from B-horizons it was concluded that pedologic factor dominate the geologic parent materials as a control in determining the shear strength behavior of this group of residual soils.

In case of soils developed from the same parent material under varying amount of precipitation there is a decrease of cohesion with increasing rainfall. In addition the colloidal activity decreases and void ratio increases with increasing precipitation. These trends were found to be accompanied by a decreasing kaolinite and increasing sesquioxide content. Thus degree of weathering (Laterization) is an important control on the shear strength properties of some laterite soils. It was assumed that variations in shear strength behavior result from differences in the parent rocks therefore the higher the degree of Laterization the more favorable are the shear strength parameters.

In addition the void ratio, density and moisture content exert a considerable influence on shear strength parameters of undisturbed as well as compacted course and fine-grained laterite soils.

The shear strength parameters are very sensitive to moisture content and degree of saturation (Gidigasu, 1976).

2.2.2 Factors controlling shear strength of soils

The stress-strain relationship of soils, and the shearing strength, is affected by:-

a. **Soil composition (basic soil material):** mineralogy, grain size and its distribution, shape of particles, pore fluid type and content, ions on grain and in pore fluid.
b. **State (initial):** Defined by the initial void ratio, effective normal stress and shear stress (stress history). State can be described by terms such as: loose, dense, over consolidated, normally consolidated, stiff, soft, contractive, dilative, etc.
c. **Structure**: Refers to the arrangement of particles within the soil mass; the manner the particles are packed or distributed. Features such as layers, joints, fissures, slicken sides, voids, pockets, cementation, etc., are part of the structure. Structure of soils is described by terms such as: undisturbed, disturbed, remolded, compacted, cemented; flocculent, honey-combed, single-grained; flocculated, deflocculated; stratified, layered, laminated; isotropic and anisotropic.
d. **Loading conditions**: Effective stress path, i.e., drained, and undrained; and type of loading, i.e., magnitude, rate (static, dynamic), and time history (monotonic, cyclic) (Murthy, 2001)

2.3 Laboratory Shear strength parameters tests

2.3.1 General characteristics of laboratory shear strength tests

The shear strength of a soil is measured in terms of a limiting resistance to deformation offered by a soil mass or test samples when subjected to loading or unloading. The limiting shearing resistance corresponding to the condition generally referred to as '**failure**', can be defined in several different ways. It is the resistance developed from a combination of particle rolling, sliding, and crushing and reduced by any excess pore pressure that develops during particle movement. The shear strength of a test sample is measured in the laboratory by subjecting it to certain defined conditions and carrying out a particular kind of test. Failure can occur in the soil as a whole, or within limited narrow zones referred to as **failure planes.**

There are different criteria of 'failure', from which the shear strength of a soil is determined. They are (Murthy, 2001):

I. Maximum deviator stress

The maximum or peak deviator stress is the stress that is associated with **'failure'** in the testing of soil samples. It is the condition of maximum principal stress difference. If the vertical and horizontal total principal stresses are denoted by σ_1 **and** σ_3, respectively, the peak deviator stress is written $(\sigma_1 - \sigma_3)_f$, and the corresponding strain is denoted by ϵ_f.

II. Maximum principal stress ratio

If the principal effective stresses σ_1', σ_3' are calculated for each set of readings taken during an undrained test, values of the principal stress ratio σ_1'/σ_3' can be calculated and plotted against strain. The maximum value of the ratio occurs at about the same strain as the peak deviator stress in many undrained test on normally consolidated clays.

III. Limiting strain

This criterion is not often used in multistage drained triaxial tests. However, for soils in which a very large deformation is needed to mobilize the maximum shear resistance, a limiting strain condition might be more appropriate.

Three types of laboratory tests are commonly used to determine shear strength characteristics of soils. These tests are the direct shear test, the triaxial compression test and the unconfined compression test. The material characteristics that can be determined from these tests are the strength parameters (angle of internal friction, and cohesion). In some triaxial tests properties related to volume change such as modulus of elasticity and Poisson's ratio can be obtained. These parameters are used for analysis and design in conventional civil engineering problems relating to slope stability, bearing capacity and any other situations where shear strength controls. It should be noted, however, that laboratory strength test are meaningful only if the laboratory conditions of loading, drainage etc. adequately represent the actual field conditions and also the soil sample being tested is representative of the in situ soil. Out of the three types of tests mentioned above, the triaxial compression test is more versatile and simulates the in situ conditions better. Therefore, it is used for this study.

Triaxial compression test

Since stress conditions during tests are indeterminate and stress path cannot be established in direct shear test, triaxial test is better to measure the shear strength parameters of a soil in the laboratory. The triaxial test is usually performed on a cylindrical soil specimen enclosed in a rubber membrane, placed in the triaxial cell. The cell is filled with water or other suitable fluids and pressurized in order to apply a constant all-around pressure or confining pressure. The soil specimen can be subjected to an axial stress through a loading ram in contact with the top of the specimen.

- ✓ The application of the confining pressure is considered as the first stage in a triaxial test. The soil specimen scan either be allowed to drain (i.e. consolidate) during the application of the confining pressure or drainage can be prevented.
- ✓ The application of the axial stress is considered as the second stage or the shearing stage in the triaxial test.
- ✓ The shear strength test is performed by loading a soil specimen with increasing applied loads until a condition of failure is reached. There are several ways to perform the test, and there are several criteria for defining failure.
- ✓ The consolidated and unconsolidated conditions are used as the first criterion in categorizing triaxial tests.

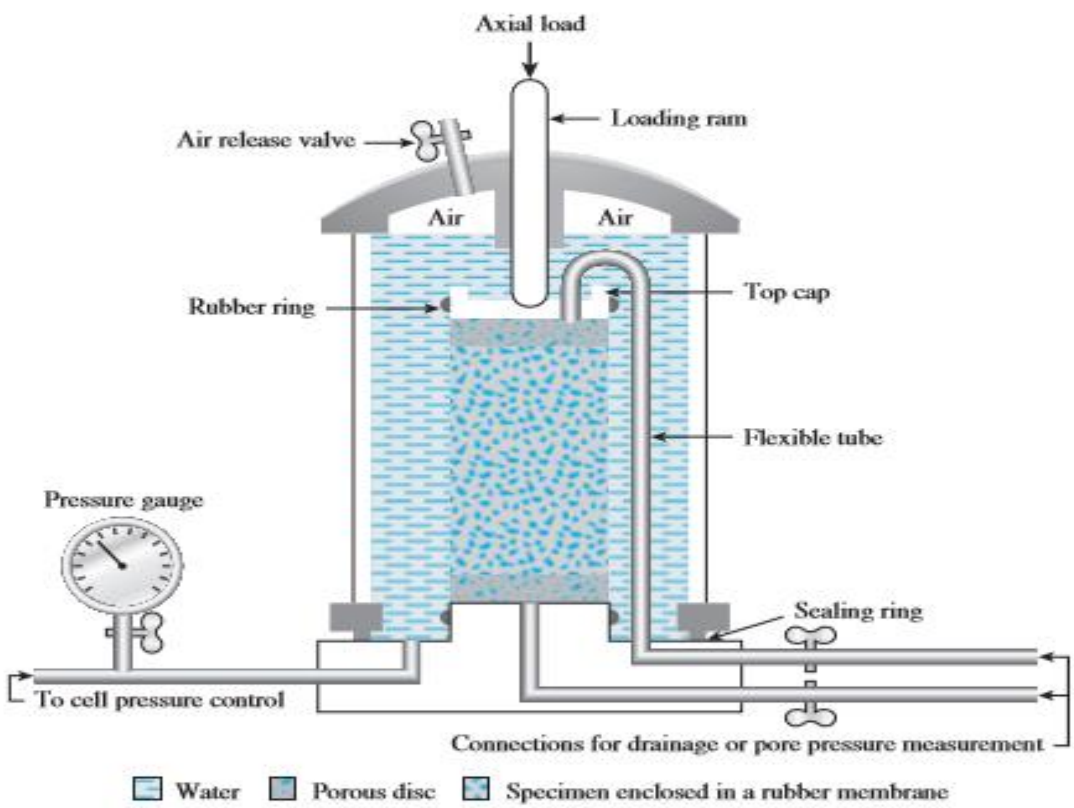


Fig 2.2 triaxial test machine (Source: Rahardjo et al. (2004))

Various triaxial test procedures are used for unsaturated soils based upon the drainage conditions adhered to during the first and second stages of the triaxial test. The triaxial test methods are usually given a two-word designation or abbreviated to a two-letter symbol. The designations are:-

1) Consolidated drained or CD test,
2) Consolidated undrained or CU test with pore pressure measurements,
3) Unconsolidated undrained or UU test

In the case of CD and CU tests, the first letter refers to the drainage condition prior to shear, while the second letter refers to the drainage condition during shear. The constant water content test is a special case where only undrained during shear (i.e. constant water content).

The pore air and pore water are not allowed to drain throughout the test for the undrained triaxial test. The unconfined compression test is a special loading condition of the undrained triaxial test. The air, water, or total volume changes may or may not be measured during shear.

2.4 Consolidation characteristics in residual soil

2.4.1 General

Consolidation is a time-dependent decrease in the volume of a soil mass under applied loading. In highway design, static loading is represented by the permanent load placed on the soil by embankments and structures. Depending on the configuration of the load and the subsurface conditions, the stress increase due to the externally applied loads may extend below the water table where all the voids are filled with water. An applied load will cause the soil grains to readjust to a more compact position to carry the load. This readjustment cannot take place until the water, which is incompressible, escapes from the voids. The displacements that developed at any given boundary of the soil mass can be determined on a rational basis by summing up the displacement of small elements of the mass resulting from the strains produced by a change in the stress system. The compression of the soil mass due to the imposed stresses may be almost immediate or time dependent according to the permeability characteristics of the soil. For example, coarse grained soils which are highly permeable are compressed in a relatively short period of time as compared to fine grained soils which are less permeable. Thus the consolidation of any soil can be defined by the following mechanisms:-

Clays are so impervious material that the water is almost trapped in the pores, When an increment of load is applied the pore water cannot escape immediately since the clay particles tend to squeeze together, the pressure develops in the pore water makes the fluid to flow out. The flow is rapid at the first but as it continuous the pressure drops and the rate of flow decreases. As the water is forced out from the clay samples, the particles can move closer together, finally the surface of specimen settles. The rate of settlement is rapid at the first, decrease to small value. The progress of consolidation can be observed by measuring the decrease of pore water pressure in the different part of the sample**.** This mechanism is called **consolidation**. Therefore, the rate of the readjustment of the soil particles is a function of the void size, which controls the rate at which the water can escape from the voids. The settlement associated with the readjustment of the soil particles due to migration of water out of the voids is Known as **primary consolidation**, and the **secondary consolidation stage** that follows the end of primary phase (Belayun , 2012).

Generally the consolidation of soils can be affected by the following factors: - External static loads from structures, Self-weight of the soils such as recently placed fills, lowering of the ground water table and Desiccation.
Mostly in residual laterite soils the consolidation characteristics depends on:-

- The nature of the soil
- The position of the sample in the profile
- The structural characteristics of the material deposit. In addition the degree of consolidation of residual soils also varies with the parent rock type from which the soils was residually formed, the depth of the sample (which is associated with the degree of weathering),the method of pretest sample preparation and the consolidation test procedure (Gidigasu, 1976).

2.4.2 Laboratory Consolidation tests

All residual soils behave as if over consolidated to some degree. Their compressibility is relatively low at low stress levels. The most method used for consolidation testing is one-dimensional test that are conducted by consoldometer (odometer) (Blight, 1997).

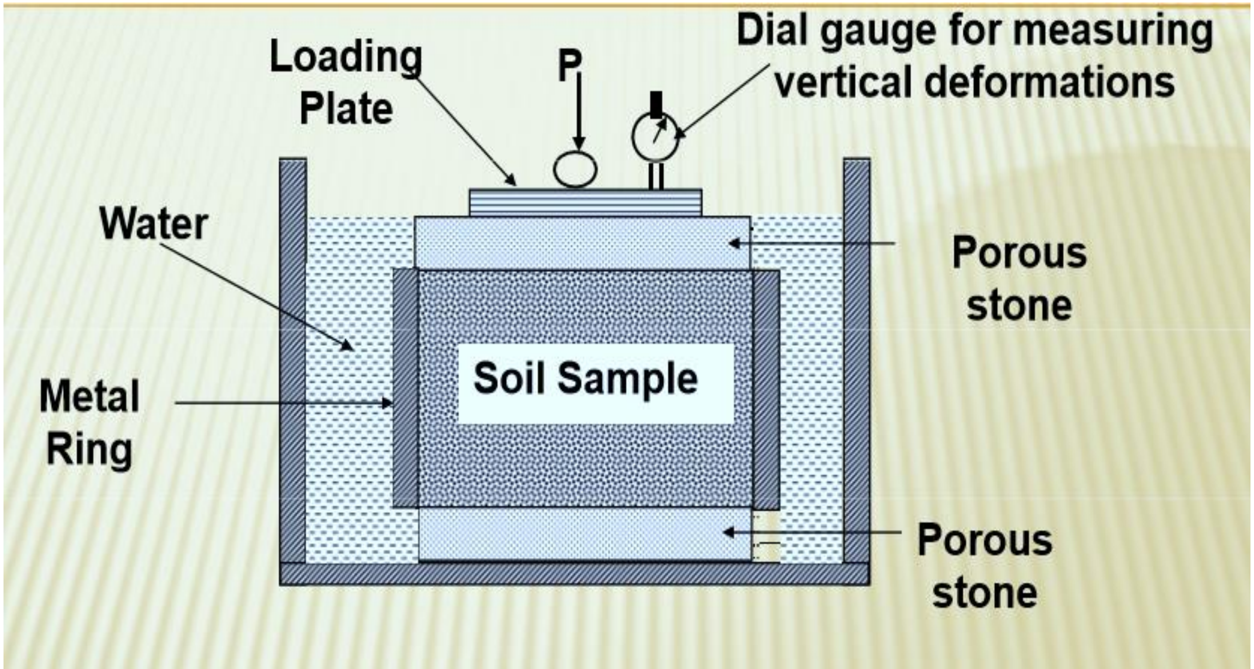


Fig 2.3 consoldometer (odometer) (Source: Rahardjo et al. (2004))

From consolidation test results the following consolidation parameters will determined:-

- Coefficient of consolidation (C_V)
- Coefficient of volume compressibility(m_v)
- Compression index(C_c)
- Coefficient permeability (k)

Generally the results of the odometer test are usually presented in the form of an e-P, e-log P, and dial reading- time plots.

Coefficient of consolidation (C_V)

The coefficient of consolidation is defined as decrease in void ratio per unit increase in effective stress, it becomes decrease with increase in effective stress i.e. the soil becomes stiffer or less compressible. Coefficient of consolidation is the indicators of the rate of drainage during consolidation.

The value of C_V may be determined by two methods from consoldometer test data, those are:-

- Square roots of time fitting method
- Logarithm of time fitting method

In theory, the square root time method should give good results except when nonlinearities arising from secondary compression cause substantial deviations from the expected straight lines

(Budhu, 2000). These deviations are most pronounced in fine grained soils with organic materials. The square root of time method has been used in this study.

Coefficient of volume compressibility

Coefficient of volume compressibility is defined as the volumetric strain per unit increase in effective stress. Lambe et al. (1969) mentioned some typical values of coefficient of volume compressibility (mv) to classify soils on the basis of compressibility. He mentioned a coefficient of volume compressibility value of **0.10-0.30 m^2/MN** for medium compressibility and **0.05-0.10 m^2/MN** for low compressibility clays. It is more commonly used parameters in practices than coefficient of compressibility.

Compression index (Cc)

In one dimensional consolidation, compression index is equal to the slope of linear portion of the void ratio verses logarithm of the effective stress plot. The values compression index indicates to how much consolidation or settlement will take place in a given types of soils. The compression index of clay is related to its index properties, especially the liquid limit. It is expressed as follows,

- For undisturbed soils $C_C = 0.009(LL-10)$
- For remolded soils $C_C = 0.007(LL-10)$

Some typical values of Cc for different types of clays are investigated by different authors. The compression index values of 0.2 to 0.8 for medium to low plasticity clay and 2.6 for montmorillonite clay (Head, 1982).

Coefficient of permeability (K)

The coefficient of permeability can be measured using field tests, or tests conducted in the laboratory. Coefficient of Permeability sometimes also estimated from one dimensional consolidation test (Teferra, 1999).

Thus, the coefficient of permeability can be obtained from the following relationships

$$K = C_V * M_v * \gamma_w \quad (2.2)$$

Where, C_V = coefficient of consolidation

m_V = coefficient of volume compressibility

γ_w = Unit weight of water

3. DESCRIPTION OF THE STUDY AREA

3.1 Location

Asela is located between **7^0 54’55’’N - $8^0$00’05’’N** latitude and **$39^0$06’10’’E- $39^0$10’00’E** longitude (Figure 1.1). It is a capital city of Arsi Zone and Tiyo district of Oromia regional National state. It is found at a distance of 75kms from Adama and 166kms from Finfine along the highway of Finfine-Adama and Bale Zone of Oromia regional National state.

In relative terms Asela is surrounded by peasant associations in all directions. It is bounded by the peasant associations of Gora Silingo in the North and Northeast, Lalo Cheka in the West and South west, Qonicha in the east and Burqa Chilalo in the South and Southeast.

3.2 Relief and topography

The relief of Asela is not uniform. The gradient of the town inclines from East to West direction. That means there is maximum altitude in the eastern peripheral areas of the town. This rises up to 2700 meters above sea level in the South east but declines up to 2210 meters above Mean Sea level in the west direction of the town. The overall altitudinal range of the town is 490 meters. Generally, rugged topographies are common in the eastern parts of the town but almost gentle slopes prevail in the western, southern and northern parts of the town. The presence of steeper gradient in eastern parts of the town is responsible for the presence of deep gully erosion along most stream and river valleys of the town. Topography is important in soil formation as it exercises a significant control on surface processes like erosion and drainage. Different topographic terrains with different landscapes uniquely affect soil development.

3.3 Soils of Asela town

The soil of Asela town ranges from red soil which is found in the southern parts of the town to black cotton soil which is generally observed in the Northern and other directions of the town. Both soils though have different colors are susceptible to serious soil erosion. As a result, the soil of the town and its surrounding areas are seriously degraded and eroded; these situation favor for the formation of **lateritic soil**. The vertical profile of the lateritic soil is shown in Figure 3.1.

Fig 3.1.Typical soil profile of sample area

3.4 Climates of Asela town

The climate of a given place is the reflection of different factors such as latitude, altitude, pressure difference between high lands and low lands and distance from the Sea as well as the impact of topography and cloud cover. If one looks at the climatic condition of Asela town and its surrounding areas, it can be concluded that the climate of Asela is the reflection of altitude, cloud cover, impacts of winds from high land areas and rugged topography. Because of the impact of altitude, the agro climatic zone of the town is categorized as temperate climate because temperate climate prevails at the altitudinal limit the town is located. In order to analyze the temperature condition of Asela town the data of 32 years is used as summarized in Table 3.1.

Table3.1. Temperature condition of Asela town from 1971to 2006

(Source, Ethiopian Meteorological Agency)

Year	Mean annual temperatures (^{0}C)	Year	Mean annual temperatures (^{0}C)
1971	14.4	1989	14.2
1972	14.2	1990	14.7
1973	12.8	1991	16.3
1974	14.3	1992	16.0
1975	13.2	1993	15.5
1976	13.7	1994	16.2
1977	13.4	1996	14.7
1978	10.5	1997	16.5
1980	14.3	1998	15.3
1981	14.1	1999	16.2
1982	13.8	2000	16.7
1983	13.6	2001	17.1
1984	12.8	2002	17.3
1985	12.8	2003	16.4
1986	14.1	2004	16.1
1987	14.3	2005	16.5
1988	14.7	2006	17.2

The trend of temperature over the past 35 years indicates that the highest amount of average annual temperature over the specified period is 17.3^{0}C.This occurred in 1999 E.C. The lowest amount of average annual temperature over the specified period is 10.5°C. This occurred in 1978 E.C. The annual range of temperature from 1971 to 2006 is 6.8^{0}C. This is an indicator of that global warming is also a threat for Asela and its surrounding areas.

The other element of climate equally affected indirectly by human activity is rain fall. In order to analyze the rainfall of Asela the rain fall data of 32 years is used, the average annual rainfall for the study area is 700-850 mm. (Belayun , 2012)

The soil forming factors such as topography, amount of rain, climate and parent rock seem to favor formation of laterite and lateritic soil in the study area. Generally formation of laterite soils favored steep slope with good water runoff and distinct rainy season. Climate is perhaps the most important factor in soil formation. It governs the amount of precipitation and temperature available that in turn determines the various soil forming processes like the rate of rock weathering to the influence of vegetation on soil. As time passes, climate tends to play a more predominant role in soil formation and alteration.

4. LABORATORY TEST RESULTS AND DISCUSSION

4.1 Index tests of lateritic soils

4.1.1 General

Soil is a heterogeneous material in which its properties and characteristics vary from point to point. The tests required for determination of engineering properties are generally elaborated and time consuming. Sometimes the geotechnical engineer is interested to have some rough assessment of the engineering properties without conducting elaborate tests. This is possible if index properties are determined.

The index properties of soils are not of primary interest to the geotechnical engineers but which are indicative of the engineering properties (Arora, 2000). The index property tests are grain size analysis, Atterberg limits, free swell and specific gravity. Mostly they can serve for identification and classification purposes. Thus, the behavior of soils should be understood by conducting those tests on physical attributes of the soil particle and soil aggregate constituents.

4.1.2 Moisture content determination

The conventional definition of moisture content determination is based on the amount of water within the pore space between the soil grain when a soil or rock material is dried to a constant mass at a temperature between 105 and 110^{o}C. However in many lateritic residual soils some moisture exists as water of crystallization within the structure of minerals in the soil particles. Some of this structural moisture may be removed by drying above this depending on the behavior of the soil. Two drying temperatures are therefore recommended for water content determinations. One specimen should be oven dried at **105^{0}C** until successive weighing show no further loss of mass, the water content should then be calculated in the normal way. The second sample should be air-dried (if feasible), or oven-dried at a temperature of no more than **50^{0}C** and a maximum relative humidity (RH) of 30% until successive weighing show no further loss of mass. The two water content results should then be compared: a significant difference (4–6% of the water content obtained by oven-drying at **105^{0}C**) indicates that “structural’’ water is present and is driven off at high temperatures. This water forms part of the soil solids, and should therefore be excluded from the calculation of water content. If a difference is detected using the two different drying procedures, all subsequent tests for water content determination (including those associated with Atterberg limit tests, etc.) should be carried out by drying at the lower temperature (i.e., either air-drying, or oven-drying at **50^{0}C** and 30% relative humidity). If possible, the lower drying temperature of 50^{0}C should be used (Blight, 1997).

Moisture contents of the soil samples were determined in the laboratory according to ASTM 2216-92 (ASTM, 2004) .seven samples were taken from the site for moisture content determination. The test results are summarized in Table 4.1.

$$w = \frac{(Wi - Wf)}{Wi} * 100 \quad \text{……} \quad (4.1)$$

Where, w = moisture content

Wi = moist weight of the sample

Wf = dry weight of the sample

Table4.1. Laboratory test results of moisture content

Sample designation	Depth (m)	moisture content		Difference
		Pretreatment		
		OD 105^{0}C oven tem.	AD (15 days)	
SP1	1.5	28.69	24.23	4.46
	3	29.36	26.25	3.11
SP2	1.5	26.97	25.02	1.95
	3	27.93	26.98	0.95
SP3	1.5	30.25	27.39	2.86
	3	31.52	29.12	2.4
SP4	1.5	29.23	28.42	0.81
	3	31.53	28.52	3.01
SP5	1.5	19.43	17.35	2.08
	3	22.39	18.52	3.87
SP6	1.5	25.01	20.25	4.76
	3	32.51	27.82	4.69
SP7	1.5	23.21	19.92	3.29
	3	28.39	23.74	4.6

4.1.3 Atterberg limit

Since the formation of lateritic soils involves differential weathering as well as movement and deposition of dissolved materials, the variation of plasticity characteristics with depth cannot be predicted even in two similar profiles on different topographical sites (Lyon, 1971).

The Effective of pretest drying on Atterberg limit test

The effect of air-drying specimens prior to carrying out the Atterberg limit tests, rather than testing starting at natural water content has been observed to result in an apparent decrease in the liquid limit and plasticity index.

The effect of drying prior to testing may be attributed to:

- Increased cementation due to oxidation of the iron and aluminum sesquioxides, or
- Dehydration of Allophane and Halloysite

Usually, Atterberg limit tests are carried out on that part of the soil finer than 425μm (0.425). The above recommendation usually precludes separating this fraction. As far as possible, all large soil particles should be removed by working the undried soil with the fingers through a 2.38mm opening sieve, so as to remove the large particles without breaking them up. The results of Atterberg limit depending on different testing conditions are tabulated in Table 4.2

Table4.2. Laboratory test results of Atterberg limit test

Sample designation	Depth (m)	Atterberg limit					
		Pretreatment					
		OD(105^{0}C)			AD(50^{0}C)		
		LL	PL	PI	LL	PL	PI
SP1	1.5	47.5	30.2	17.3	50.0	35.4	14.6
	3	51.2	32.8	18.4	58.1	40.6	17.5
SP2	1.5	48.3	29.1	19.2	55.6	38.8	16.8
	3	51.5	25.1	26.4	52.3	26.7	25.6
SP3	1.5	47.6	35.3	12.3	52.8	35.09	17.71
	3	40.2	22.9	17.3	50.4	25.3	25.1
SP4	1.5	49.4	36.1	13.3	51.5	31.0	20.5
	3	38.6	21.5	17.1	45.8	25.9	19.9
SP5	1.5	54.1	32	22.1	52.9	34.6	18.3
	3	45.5	26	19.5	48.1	29.7	18.4
SP6	1.5	46.2	26.0	20.2	56.3	37	19.3
	3	58.6	35.1	23.5	55.1	43.3	11.8
SP7	1.5	45.3	24.5	20.8	60.4	21.8	38.6
	3	50.1	25.7	24.4	60.8	45.4	15.4

4.1.4 Specific gravity test

Specific gravity (Gs) is the ratio of the mass of a given volume of material to the mass of an equal volume of water and used to calculate volumetric parameters such as void ratio and porosity. Mostly the specific gravity has been used as a measure of degree of maturity (Laterization). The values of specific gravity in the lateritic residual soils are different from conventional soils. In lateritic residual soils, Gs may be unusually high or unusually low depending on whether the "particle'' consists of a void less solid particle of a heavy mineral, e.g., iron sesquioxide or a porous aggregation of elementary particles. The value of specific gravity of lateritic soils ranges between 2.55 to 3.4 (Lyon, 1971). Thus, the specific gravity of samples was determined in accordance with ASTM test designation D854 – 92 procedures method 'A' for oven dry; methods 'B' for air dry samples.

The values of specific gravity for the soils under investigation ranges between 2.5 to 3.3 and summarized in the Table 4.3.

Table 4.3 laboratory results of specific gravity

Sample designation	**Depth (m)**	**Specific gravity**	
		Pretreatment	
		OD	AD
SP1	1.5	2.71	2.74
	3	2.75	2.95
SP2	1.5	2.65	2.77
	3	2.85	3.00
SP3	1.5	2.78	2.89
	3	2.70	2.79
SP4	1.5	2.69	2.72
	3	2.70	2.82
SP5	1.5	2.80	2.95
	3	2.74	2.80
SP6	1.5	2.63	2.70
	3	2.59	2.67
SP7	1.5	2.62	2.69
	3	2.64	2.77

4.1.5 Free swelling test

The amount of swelling and the magnitude of swelling pressure are known to be dependent on the clay minerals, the soil mineralogy and structure, fabric and several physico-chemical aspects of the soil. Among clay minerals, montmorillonite influences the magnitude of swelling as compared to Illites and kaolinite (Haile Mariam, 1992). The simplest test conducted is free swell test. The test is performed by slowly pouring 10cm^3 of dry soil which has passed the No. 40 (0.425mm) sieve in to 100cm^3 graduated cylinder filled with distilled water. After 24 hours, final volume of the suspension is read. Hence, free swell is defined as

$$\textbf{Free swell} = \frac{\textbf{Final volume - Initial volume of the soil}}{\textbf{Initial volume}} * 100\% \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots.. [4.2]$$

Soils with free swell less than 50% are not likely to show expansive property, while soils with free swells in excess of 50 percent could present swell problems. Values of 100% or more are associated with clay which could swell considerably, especially under light loadings (Tefera, 1999). Free swelling test results ranges from 22% to 50% suggesting the soil under investigation shows low swelling property and summarized in Table 4.4.

Table4.4. Laboratory results of free swelling

Sample designation	Depth (m)	**Free swelling test (%)**	
		Pretreatment	
		OD	**AD**
SP1	1.5	28	35
	3	30	48
SP2	1.5	35	39
	3	20	35
SP3	1.5	45	50
	3	33	40
SP4	1.5	42	50
	3	25	37
SP5	1.5	20	22
	3	37	40
SP6	1.5	44	46
	3	48	50
SP7	1.5	42	47
	3	49	50

4.1.6 Particle size distribution of residual soils

Particle size distribution may provide the following information:

- A basis for identification and classification of soils.
- The compactibility characteristics
- Permeability
- Swellability and
- A rough idea of deformation characteristics of the soil mass.

Texturally lateritic soils are very variable and may contain all fractions sizes; boulders, cobbles, gravel, sand, silt, and clay as well as concretionary rocks.

Pre-testing preparation of lateritic soils for sieve analysis may have the following effects on the size distributions (Gidigasu, 1976):

I. Re-molding and removal of free iron oxides increases the content of fines between 35% to 65% (this will be a function of the dispersing agent)
II. Degree of drying and time of mixing of the sample prior to testing influence the degree of dispersion of some lateritic soils
III. Cementing effects of sesquioxides, which bind the clay and silt fractions into coarser fraction

Thus in particle size analysis, sieving and sedimentation tests are used to determine particle size distribution within a soil sample. The sieving test is normally used for particle size larger than **0.075 mm** whereas for particles smaller than **No 200 (0.075 mm)**, sedimentation in water test is used. The distribution is normally shown as a grading curve of in a plot of particle size (log scale) on the abscissa and percentage passing (linear scale) on the Y-axis as shown in Figure 4.1 and 4.2. The grading curve is particularly important in identification and classification of coarse grained soils; however the plasticity chart is used for classifying fine grained soils. The percentages of different particle size for the studied soils are summarized in Table 4.5.

Test procedures

In preparing tropical residual soils samples for sieving, breakdown of individual particles by crushing must be avoided**.** For this reason wet sieving is preferred to dry sieving. Too long exposure of the samples to the atmosphere especially in the dry region before wet sieving should also be avoided because it may lead to aggregation of fine particles to form larger particles which may remain bonded together even on rewetting (Blight, 1997). Longer working time also may lead to breaking of particles in the soil sample. The detailed procedure of this analysis can be obtained from the ASTM Designation (ASTM D422-63) and (ASTM D 421-85) for hydrometer and dry preparation respectively.

Table 4.5 laboratory results of particle size

Designation	Depth	Pretreatment		Percentage amount of Particle size			
			Gravel %	Sand %	% fine	Silt%	Clay %
SP1	1.5	OD	0.0	14.1	85.9	50.37	35.53
		AD	0.0	10	90	48.3	41.7
	3	OD	0.0	19	81	23.5	57.5
		AD	0.0	11.3	88.7	46.8	41.9
SP2	1.5	OD	0.0	10.9	89.1	48.14	40.96
		AD	0.0	10.7	89.3	26.2	63.1
	3	OD	0.0	12.3	87.7	37.6	50.1
		AD	0.0	12.3	86	39.05	48.65
SP3	1.5	OD	0.0	27.8	72.2	28	44.2
		AD	0.0	14.3	85.7	45.1	40.6
	3	OD	0.0	19.8	80.2	26.3	53.9
		AD	0.0	12.5	87.5	30.14	57.36
SP4	1.5	OD	0.0	24.3	75.7	20.9	54.8
		AD	0.0	19.1	80.9	45.08	35.82
	3	OD	0.0	21.2	78.8	31.4	47.4
		AD	0.0	14.7	85.3	26.33	58.97
SP5	1.5	OD	0.0	25.7	74.3	36.1	38.2
		AD	0.0	17.2	82.8	57.3	25.5
		OD	0.0	12.6	87.4	28.1	59.3
	3	AD	0.0	11.1	88.9	28.04	60.86
SP6	1.5	OD	0.0	26.1	73.9	26.3	47.6
		AD	0.0	17.9	82.1	32.35	49.75
	3	OD	0.0	30.1	69.9	25.2	44.7
		AD	0.0	11.7	88.3	51	37.3
SP7	1.5	OD	0.0	13.5	86.5	38.7	47.8
		AD	0.0	12.1	87.9	26.7	61.2
	3	OD	0.0	21	79	27.8	51.2
		AD	0.0	13.8	86.1	27.5	58.7

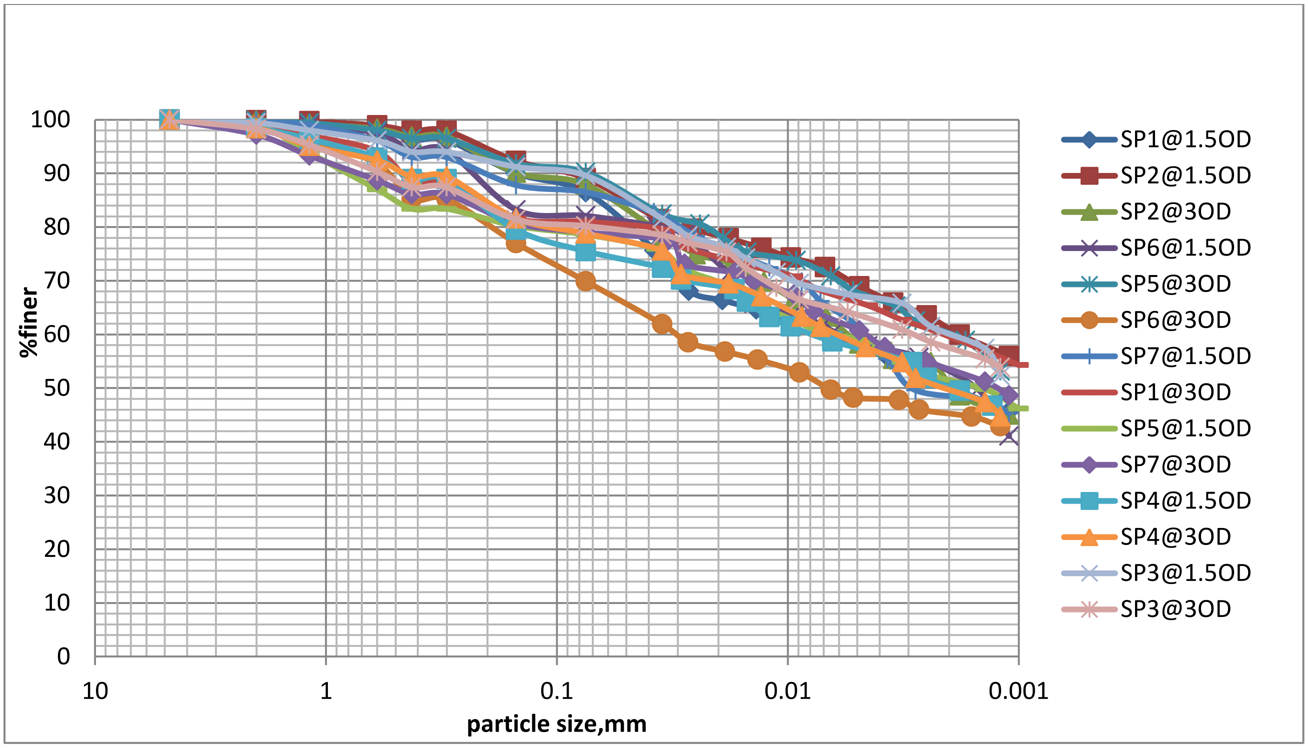


Fig4.1. particle size analysis for OD samples

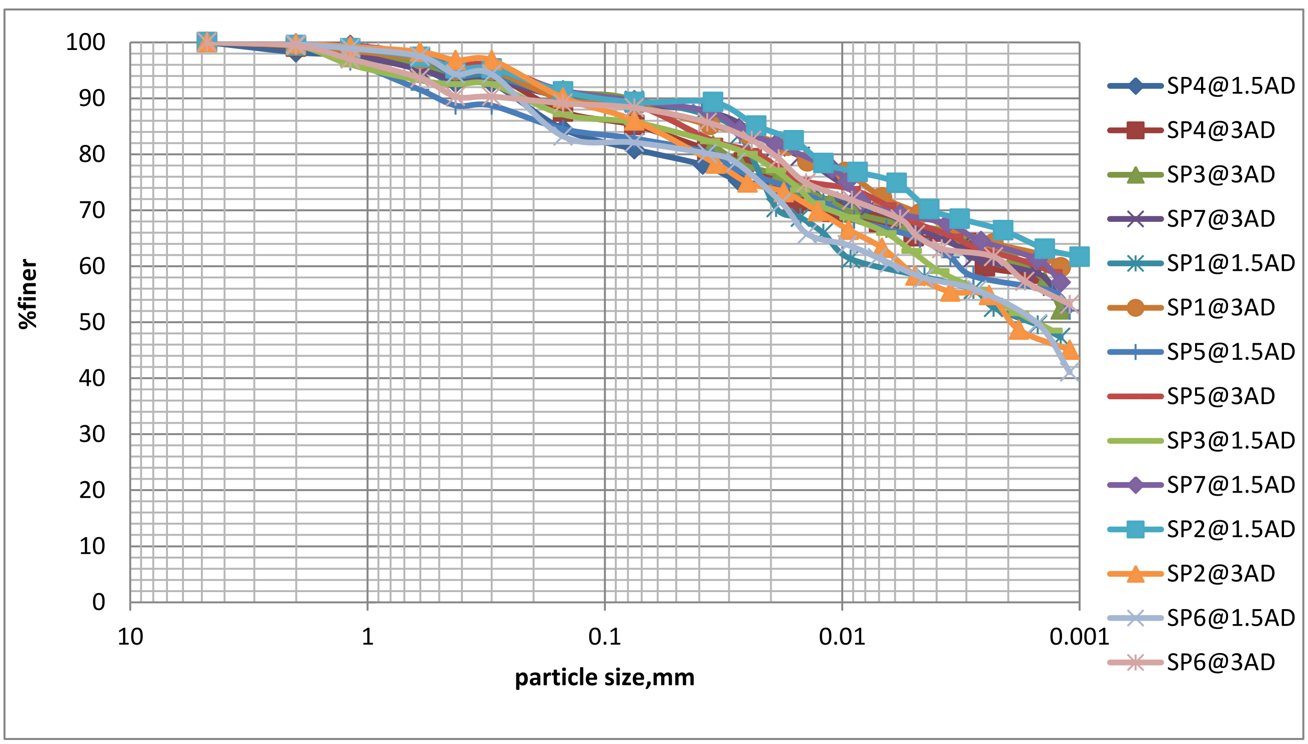


Fig4.2. particle size analysis for AD samples

4.2 Classification of soils

The importance of soil classification is to categorize their appearance and behavior by relating with previously established engineering properties and performance. The most widely used classification schemes are those that divide soils in to an orderly easily remembered system of groups or classes that have similar physical and engineering properties that can be identified by simple and in expensive tests. These groups ideally provide estimates of both the engineering characteristics and performance of soils for design and construction engineers.

The description of soils within the group of a given classification typically are represented by alphabetical or alphanumerical symbols for rapid identification in written material, graphical boring logs and on engineering drawings. The continued use of a few engineering soil classification systems is the result of the provision in each for the needs of civil engineers as well as the adaptability of the classification to the variety of soils encountered in engineering practices (Jiregna, 2008).For this study two most widely used classification systems are used. Those are American association of state highway and transportation officials AASHTO) and the unified soil classification systems (USCS).

4.2.1 Unified soil classification system (USCS)

Unified soil classification system (USCS) is system which is more preferred for geotechnical engineers for soil classification in accordance with American society for testing and material (ASTM) in D-2487 standard requirement. The system is discovered by Cassagrande in 1942 during engineering works in world war. It classifies coarse grained soils that are gravelly and sandy in nature with less than 50% passing through the No. 200 sieve. The group symbols are designated by GW, GP, GM, GC, SW, SP, SM and SC.

The symbol that starts with a prefix G stands for gravel or gravelly soil and S stands for sand or sandy soil. The other symbols such as W and P stands for well graded and poorly graded, respectively. However fine grained soils that are 50% or more passing through the No. 200 sieve represented by the symbols M which represents inorganic silt, C for inorganic clay or O for organic silts and clays.

Plasticity chart (for USCS)

High numerical value of plasticity index is an indication of the presence of high percentage of clay in the soil sample which indicates that, the plasticity values increase with the corresponding increase in clay contents. PI in relation to the LL can provide information regarding the types of

clay in the soil samples. This can be done by means of a plasticity chart developed from soils tested from different parts of the world (Budhu, 2000). Thus from experimental results of the soils tested from different samples were plotted on a graph of PI as ordinate Vs. LL as abscissa. From the graph it was found that clays, silts and organic soils lie in distinct regions of the graph.
A line that is defined by the following equation

$I_P = 0.73(W_{LL}-20)$ % is………………………………………………………..4.1

Called "A- line" which indicates that the boundaries between clays (above the line) and silts and organic soils (below the line) as shown on the fig below
A second line is called the U- line which is expressed as,

$I_P = 0.9(W_{LL} -8)$…………………………………………………………………..4.2

This defines the upper limit of the correlation between PI and LL. If the result of the soil tested fall above the U- line, one should be suspicious on the quality of the tests and should be repeated. All the soils of the study area fall below the U- line, hence the results are acceptable. They are plotted below and above the A- line in the Cassagrande plasticity chart in the Figure 4.3 & 4.4., showing that they are dominantly composed of clays and silts in accordance with grading curve.

4.2.2 AASHTO classification system

According to AASHTO classification (Table 4.6), the soils under study are classified as A-7-5 and A-7-6 with group index range from 12 to 20 implying the soils are least suitable as highway material or subgrade.

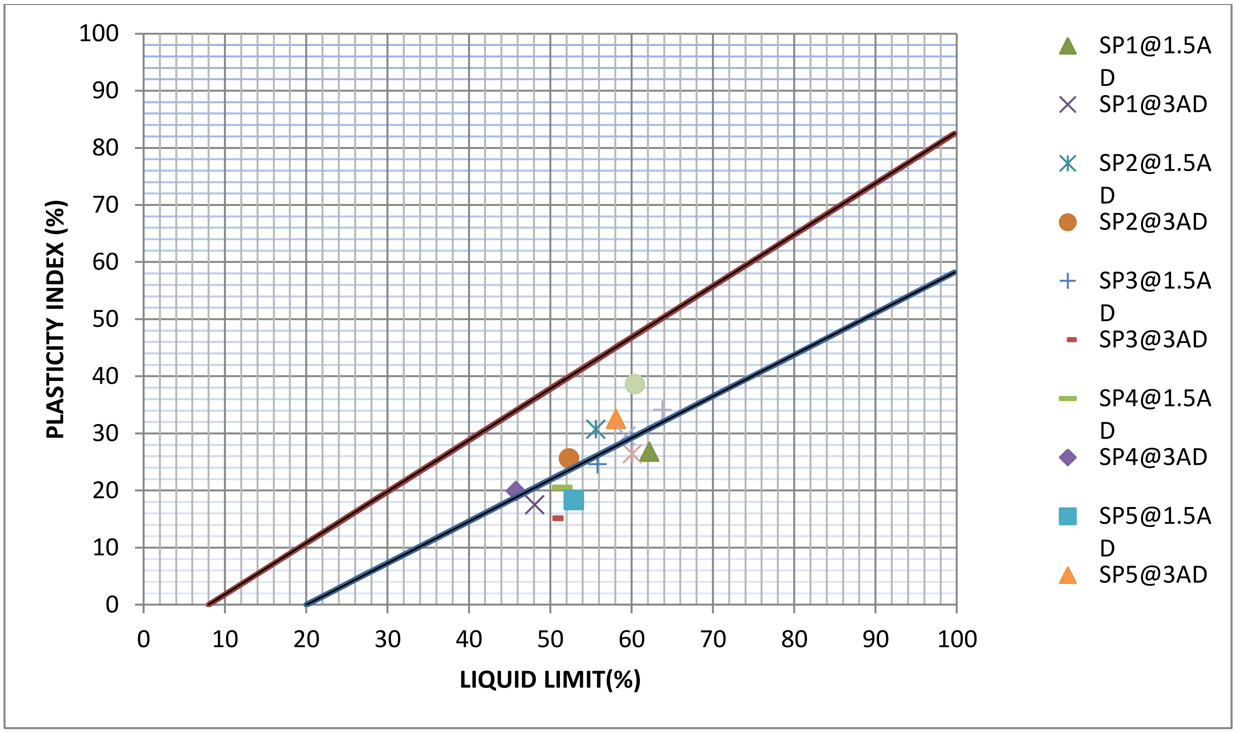


Fig4.3 The USCS Plasticity chart classification of air dried soil samples

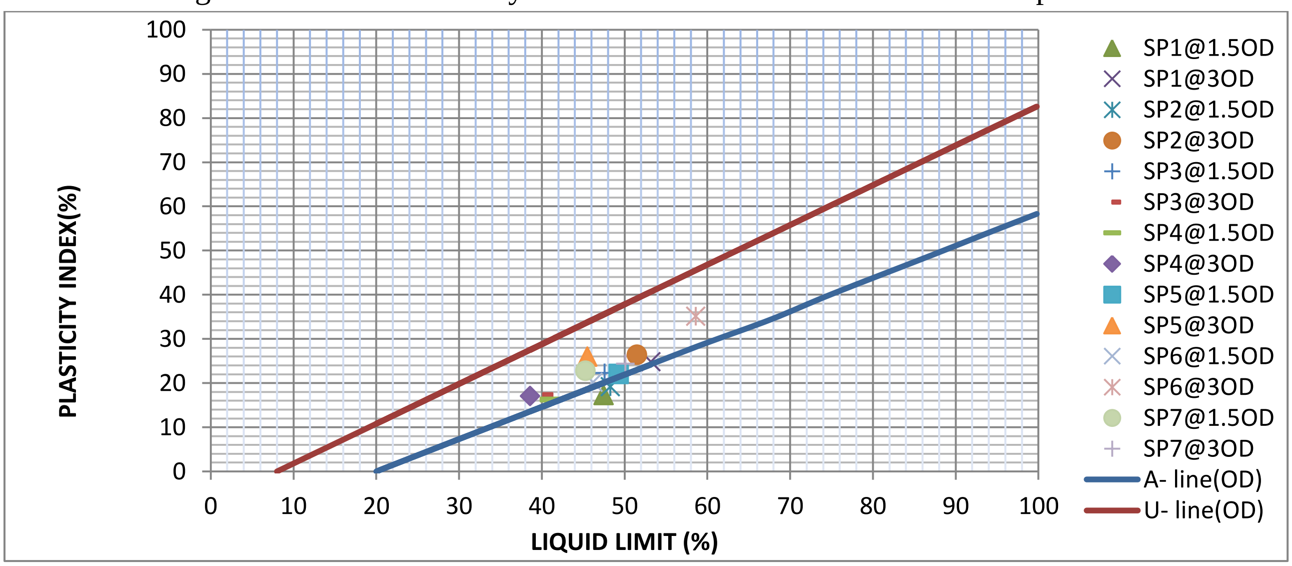


Fig4.4. the USCS Plasticity chart classification of oven dried soil samples

Table 4.6 Classification of the soil according to AASHTO and USCS

Test Pit	Depth	Pretreatment	LL (%)	PI (%)	% amount of passing			Classification according to AASHTO		% amount of particle size				Classification according to USCS
					2	0.425	0.075	Group	Group index	gravel	sandy	silty	Clay	
SP1	1.5	OD	47.5	17.3	99.7	96.2	85.9	A-7-5	17	0	14.1	50.37	35.53	ML
		AD	50.0	14.6	99.9	97.6	90.0	A-7-6	17	0	10	41.7	48.3	CH
	3	OD	53.2	24.8	98.6	88.3	81	A-7-5	17	0	19	23.5	57.5	CH
		AD	58.1	22.5	99.4	95	88.7	A-7-5	19	0	11.3	46.8	41.9	MH
SP2	1.5	OD	48.3	19.2	99.9	97.9	89.1	A-7-6	20	0	10.9	48.14	40.96	ML
		AD	55.6	16.8	99.5	95.3	89.3	A-7-6	20	0	10.7	26.2	63.1	CH
	3	AD	52.3	17.5	99.6	97.7	86	A-7-6	19	0	12.3	37.6	50.1	CH
		OD	51.5	26.4	99.8	96.8	87.7	A-7-6	20	0	14	37.35	48.65	CL
SP3	1.5	OD	47.6	22.3	98.8	84.3	72.2	A-7-6	18	0	27.8	28	44.2	CL
		AD	55.8	25.7	99.4	92.5	85.7	A-7-5	18	0	14.3	45.1	40.6	MH
	3	OD	40.2	17.3	98.2	87.4	80.2	A-7-6	14	0	19.8	26.3	53.9	CL
		AD	50.4	15.1	99.4	94	87.5	A-7-6	17	0	12.5	30.14	57.36	CH
SP4	1.5	OD	49.4	13.3	98.2	92.7	75.7	A-7-6	12	0	24.3	20.9	54.8	CL
		AD	51.5	20.5	98.4	88.9	80.9	A-7-5	19	0	19.1	45.08	35.82	MH
	3	OD	38.6	17.1	98.4	89.4	78.8	A-7-6	13	0	21.2	31.4	47.4	CL
		AD	45.8	19.9	99.2	94.3	85.3	A-7-6	19	0	14.7	26.33	58.97	CL
SP5	1.5	OD	54.1	22.1	98.9	83.4	74.3	A-7-6	17	0	25.7	36.1	38.2	CH
		AD	52.9	18.3	99.5	88.7	82.8	A-7-6	18	0	17.2	57.3	25.5	MH
	3	OD	45.5	26	99.5	94.5	87.4	A-7-6	18	0	12.6	28.1	59.3	CL
		AD	48.1	29.7	99.8	95.9	88.9	A-7-6	19	0	11.1	28.04	60.86	CL

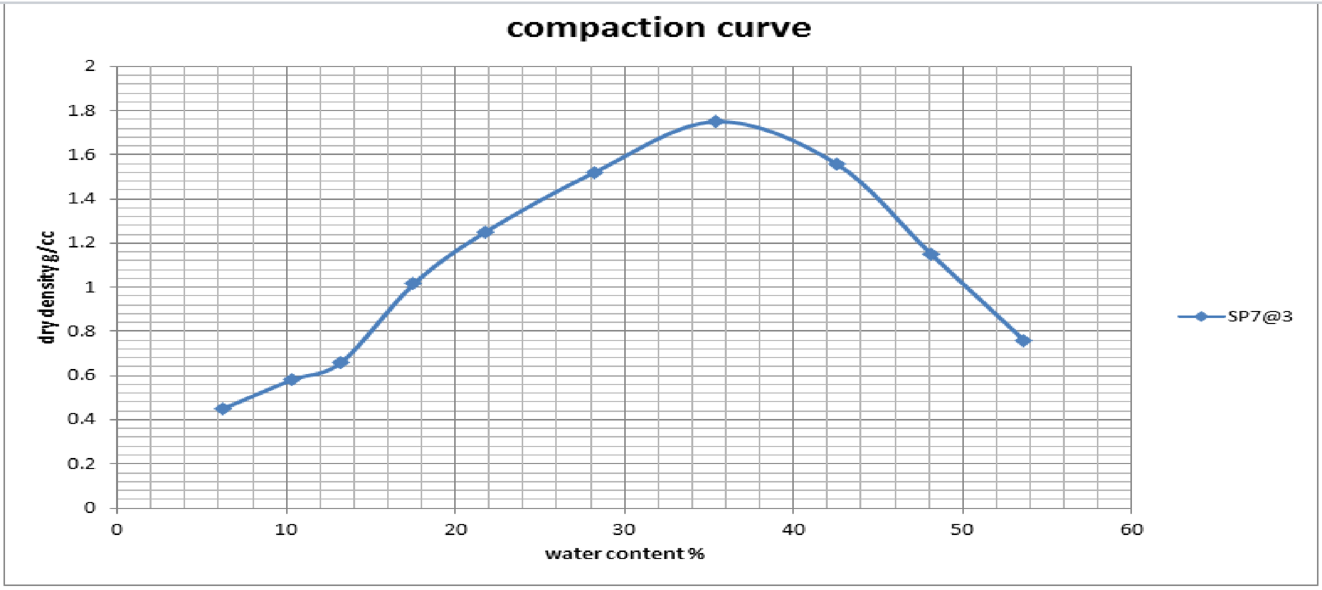
SP6	1.5	AD	59.3	20.3	99.6	94.3	82.1	A-7-6	20	0	17.9	32.35	49.75	CH
		OD	46.2	20.2	98.2	89.7	73.9	A-7-6	15	0	26.1	26.3	47.6	CL
	3	AD	60.1	16.8	99.4	90.3	88.3	A-7-6	16	0	11.7	51	37.3	MH
		OD	58.6	35.1	99	85.2	69.9	A-7-6	18	0	30.1	25.2	44.7	CL
SP7	1.5	OD	45.3	20.8	99.6	93	86.5	A-7-6	19	0	13.5	38.7	47.8	CL
		AD	60.4	13.6	99.8	94.2	87.9	A-7-6	19	0	12.1	26.7	61.2	CH
	3	AD	63.8	18.4	99.6	91.9	86.2	A-7-6	19	0	13.8	27.5	58.7	CH
		OD	50.1	24.4	97.2	86.1	79	A-7-6	20	0	21.0	27.8	51.2	CH

4.3 Compaction

Standard proctor compaction test is performed to determine the relationship between the moisture content and the dry density of a specific soil sample at a specified compactive effort. The compactive effort is the amount of mechanical energy that is applied to the soil mass. Soil specimens were oven dried, passing the No. 4.75mm sieve was soaked overnight. Then, they were compacted in a cylindrical mold of 944 cm^3 volume (10 cm diameter and 12 cm height) standard proctor mold by repeated blows from the mass of a hammer 2.5 kg falling freely from a height of 30.5 cm. The soils were compacted with different moisture content in three layers, each layer being subjected to 25 blows. The MDD and OMC that are used to remold the soil for shear strength and consolidation parameters test are summarized in Table 4.7. The compaction curve for selected sample is shown in Figure 4.5 and the rest are included in appendix B.

Table4.7. Values of OMC and MDD

Test pit designation	depth(m)	OMC, %	MDD, gm/cc	Wet density g/cc
SP1	3	25	1.8	1.85
SP2	1.5	36	1.4	1.94
SP3	1.5	26.5	1.38	1.88
SP4	1.5	28.56	1.49	1.97
SP5	3	30	1.48	1.59
SP6	1.5	27.85	1.65	1.72
SP7	3	35.36	1.75	1.84



Optimum moisture content	35.36%	Maximum dry density	1.75g/cc

Fig4.5. Typical standard compaction test curve for sp7@3

4.4 Shear strength test

4.4.1 Unconsolidated Undrained test (UU)

Unconsolidated Undrained (UU) test is called a quick test which is useful for testing the shear strength of clayey soils for foundation analysis. During construction on cemented clays only a small amount of consolidation would be taking place in actual field, causing a little change in moisture content. Similarly, for analyzing clay slopes of cuts, the shear strength can be determined by such undrained or quick tests.

4.4.2 Sample preparation for UU test

Seven samples collected from 1.5 m and 3 m depth below the natural ground level were prepared for this test. The samples were sealed by wax cover and rapped by plastic bag and moist towel to maintain natural moisture content. (Blight, 1997)Recommended that the specimen diameter should not be less than 76 mm for residual soil but due to the triaxial machine limitation 38 mm diameter samples were used for this study. The test specimen were prepared from the 100 mm diameter tube by hydraulically pushing the soil in to three 38 mm diameter tubes simultaneously from which a 76 mm long specimen was extruded and three specimen from the same level were used for each of triaxial test. Unsaturated soils specimen have high initial matric suction, therefore there is no need of saturating the sample. The usual strain rate for shear testing of unsaturated soil is 0.017 - 0.03%/s, thus 1.02/60/s is used as test strain rate. Therefore to conducted this test BS1377:1975 test 12 is used for Unconsolidated Undrained test. The three identical specimens are confined at 100KN/m^2, 200KN/m^2 and 300KN/m^2 by considering over burden pressure and load ranges from possible structures.

The stress-strain relationship at 100KN/m^2, 200KN/m^2 and 300KN/m^2 confining stress for selected samples are shown in Figure 4.6a and 4.7b and the rest included in appendix C. the peak value of the axial stress increases as increasing the confining stress. The shear strength parameters, undrained cohesion (Cu) and internal friction (Ø), of the partially saturated soils computed from the intercept and slope, respectively, of tangent line to the three Mohr circle as indicated in Figure 4.6b and 4.7 and the results are tabulated in Table 4.8. As indicated in the table the undrained cohesion (Cu) value ranges between 89.63 - 161.48 Kpa and the internal friction angle (Ø) value ranges from 17 º - 24 º.

Table4.8. C and Φ values of soils remolded with OMC and MDD under UU test

Test pit Designation	Depth (m)	Treatment condition	Soil description	MDD(g/cc)	OMC(g/cc)	Cu	Ø^0
SP1	3	OD	CH	1.8	25	161.48	18
SP2	1.5	AD	CH	1.4	36	126.56	19
SP3	1.5	OD	CL	1.38	26.5	124.3	17
SP4	1.5	OD	CL	1.49	28.56	100.69	22
SP5	3	OD	CL	1.48	30	144.46	18
SP6	1.5	AD	CH	1.65	27.85	157.81	15
SP7	3	OD	CH	1.75	35.36	89.63	24

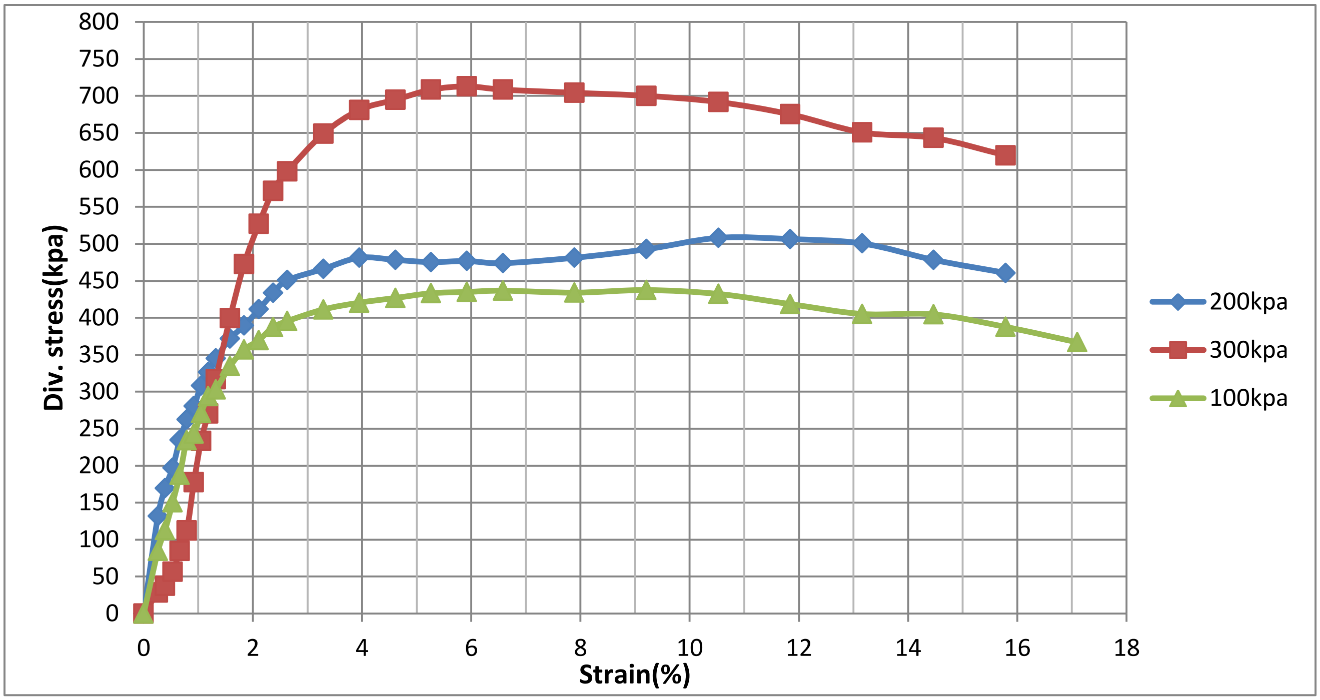


Fig4.6a. Axial stress vs. strain of SP1 at 3m depth

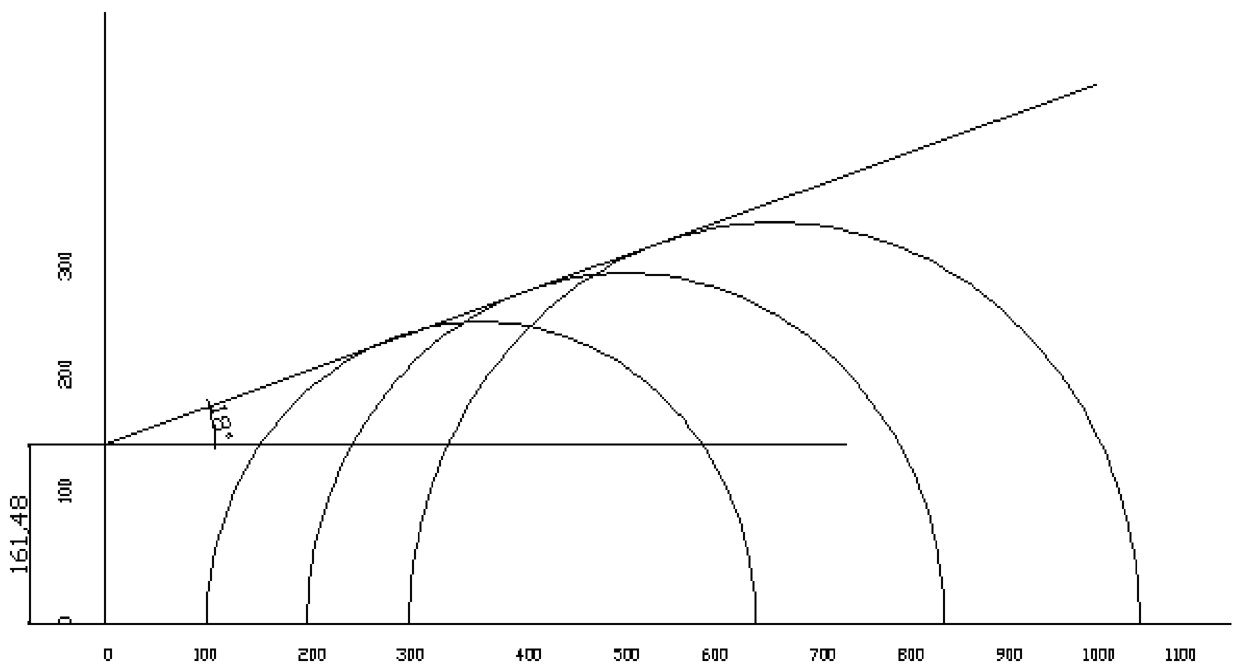


Fig4.6b. Mohr circle for cell pressures of 100,200,300 Kpa and failure envelope for SP1 at 3m

4.5 Consolidation test

Test procedures

One-dimensional consolidation tests were performed in accordance with ASTM D2435, on specimens trimmed from remolded samples for the samples collected from the area of lateritic soils of Asela. The test specimens were loaded at twenty-four hour intervals using a load-increment ratio of two, and they were unloaded at twenty-four hour.

The consolidation test were conducted in such way that sufficient time is allowed for the applied pressure (total stress) increment to be transmitted from the pore water, where it acts initially as an excess pore water pressures, to the soil structure where it ultimately becomes an applied effective stress increment. The time it takes for this transfer to occur is the basis for the process being called **consolidation.** Therefore, the effective stress corresponding to the applied pressure is generally plotted versus void ratio. The resulting consolidation curve permits an evaluation of the preconsolidation pressure and values for other parameters pertaining to the consolidation characteristics of the soil sample.

Plots of void ratio versus effective pressure on arithmetic and logarithmic scales are shown in Figure 4.14a to 15b. The apparent pre-consolidation pressures and consolidation parameters: coefficient of consolidation (Cv), coefficient of volume compressibility (mv) and the compression index (Cc) of the studied soils were determined from the consolidation curve. The values of coefficient of permeability (k) were determined from the values of mv and Cv.

4.5.1 Coefficient of consolidation

Decrease in void ratio per unit increase in effective stress is known as coefficient of consolidation. It becomes decrease with increase in effective stress i.e. the soil becomes stiffer or less compressible. Coefficient of consolidation is the indicators of the rate of drainage during consolidation. The value of C_V determined by square roots of time fitting method from consoldometer test data, collected from laboratory. A graphical plot for the selected sample is shown is shown in Figure 4.13. The Cv values determined for the studied soils are included in Table 4.9.

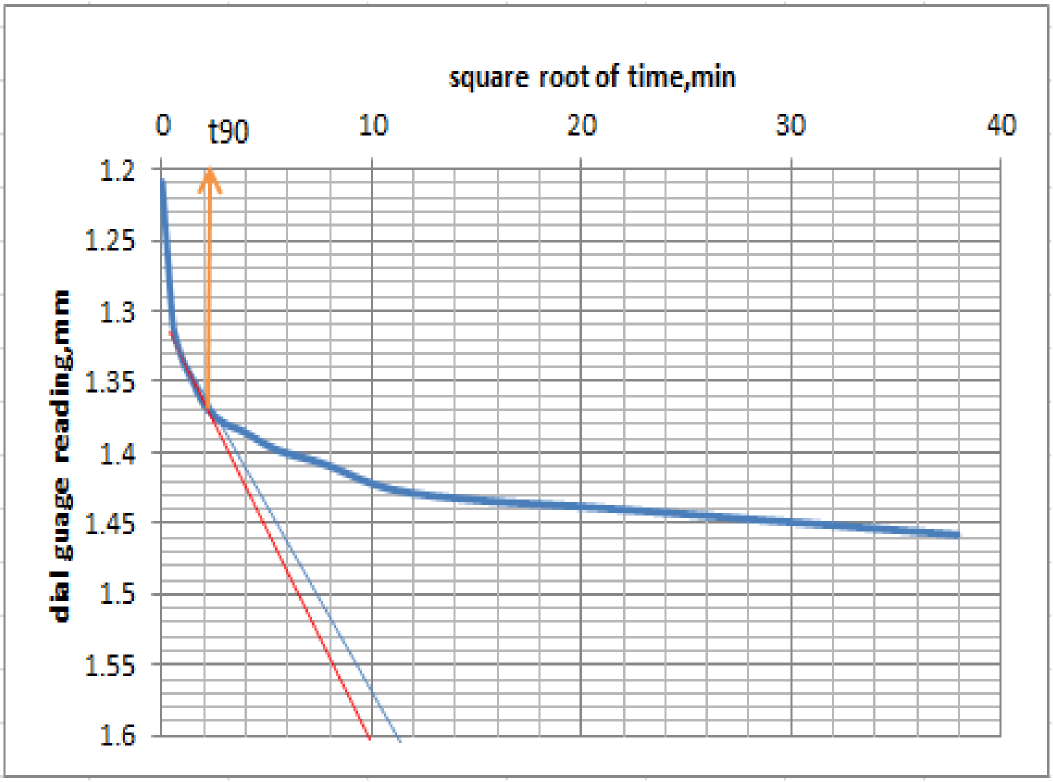


Fig. 4.7. Typical curve (square root of time method) for SP2 @1.5m depth, loading of 160kpa

4.5.2 Compression index (Cc)

Compression index is defined as the slope of linear portion of the void ratio verses logarithm of the effective stress plot (Figure 4.14a and b). The compression index of clay is related to its index properties, especially the liquid limit and related as follows for remolded soils,

C_C = 0.007(LL-10). Head (1982) mentioned some typical values of Cc for different types of clays. He quoted a compression index value of 0.2 to 0.8 for medium to low plasticity clay and a Cc value of up to 2.6 for montmorillonite clay.

The Cc values for the soil under study, as shown in Table 4.9, ranges from 0.193 to 0.581, suggesting the soils are medium to low plastic clay.

4.5.3 Void ratio versus P' curves

The void ratio versus log p' curves for seven selected samples obtained from laboratory Odometer tests were plotted fig below using Cassagrande construction method, the apparent pre-consolidation pressures were determined and summarized in table 4.7 below. The value ranges approximately from 150 Kpa to 200 Kpa.

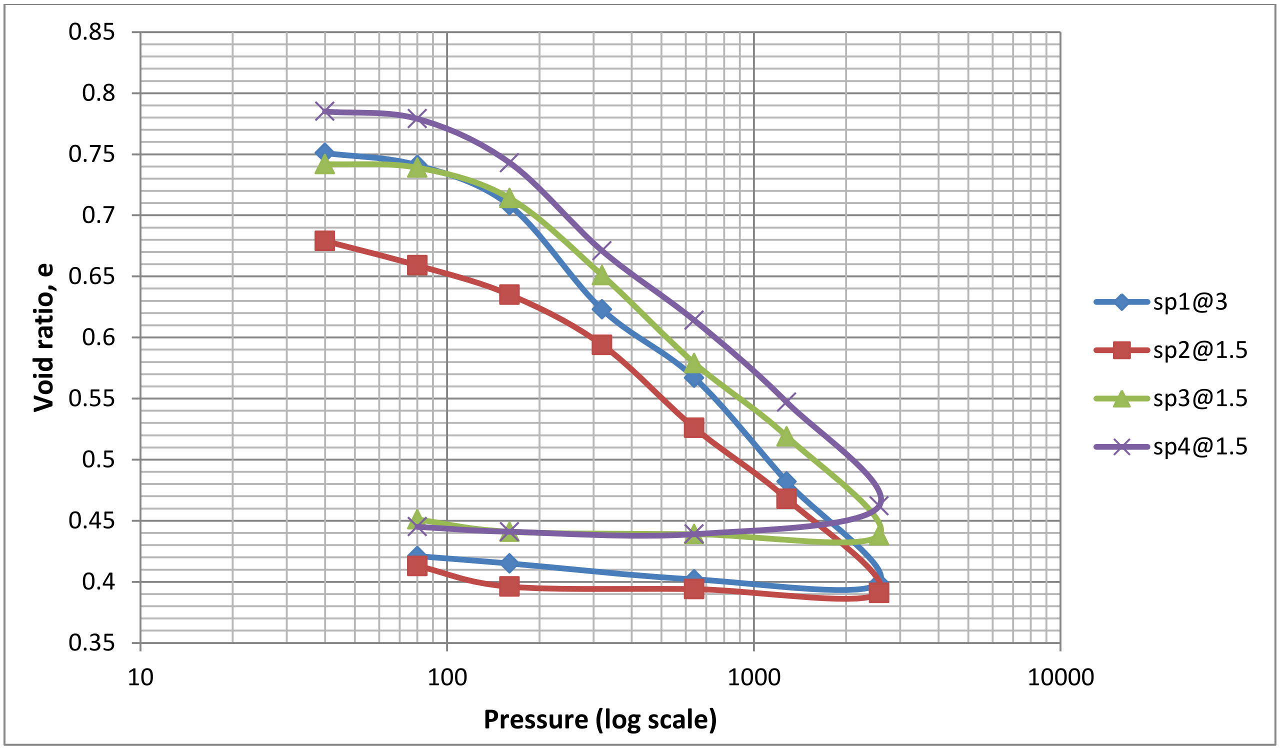


Fig4.8a. Void ratio Vs log pressure curve

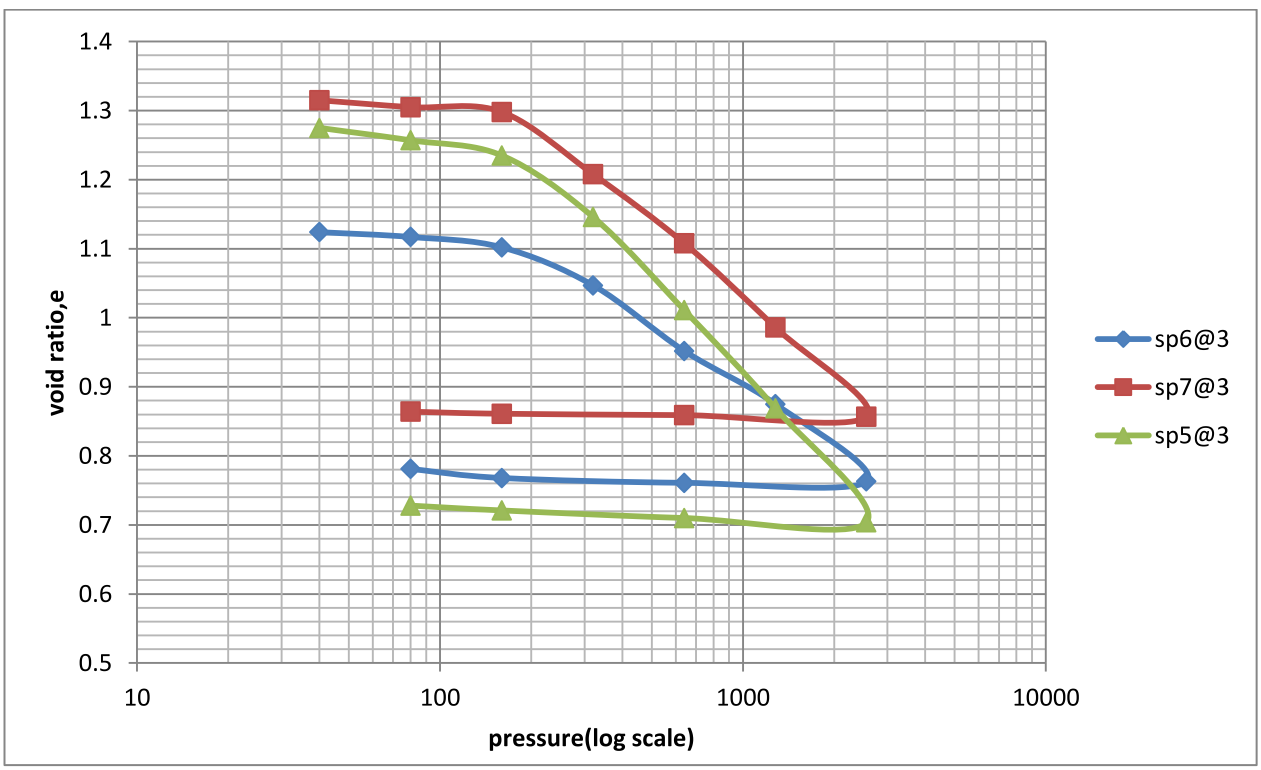


Fig4.8b. Void ratios log pressure curve

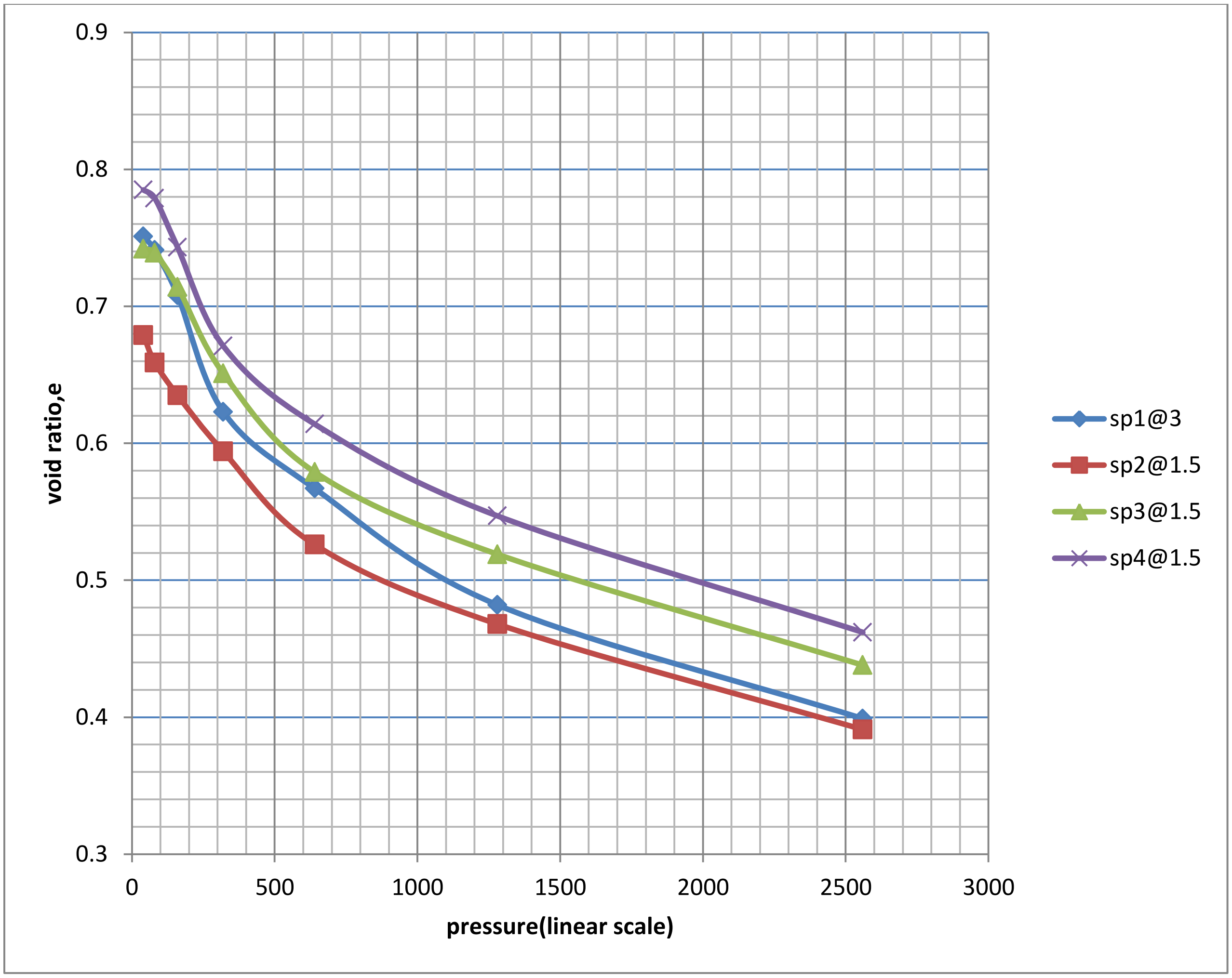


Fig 4.9a Consolidation test result (linear scale)

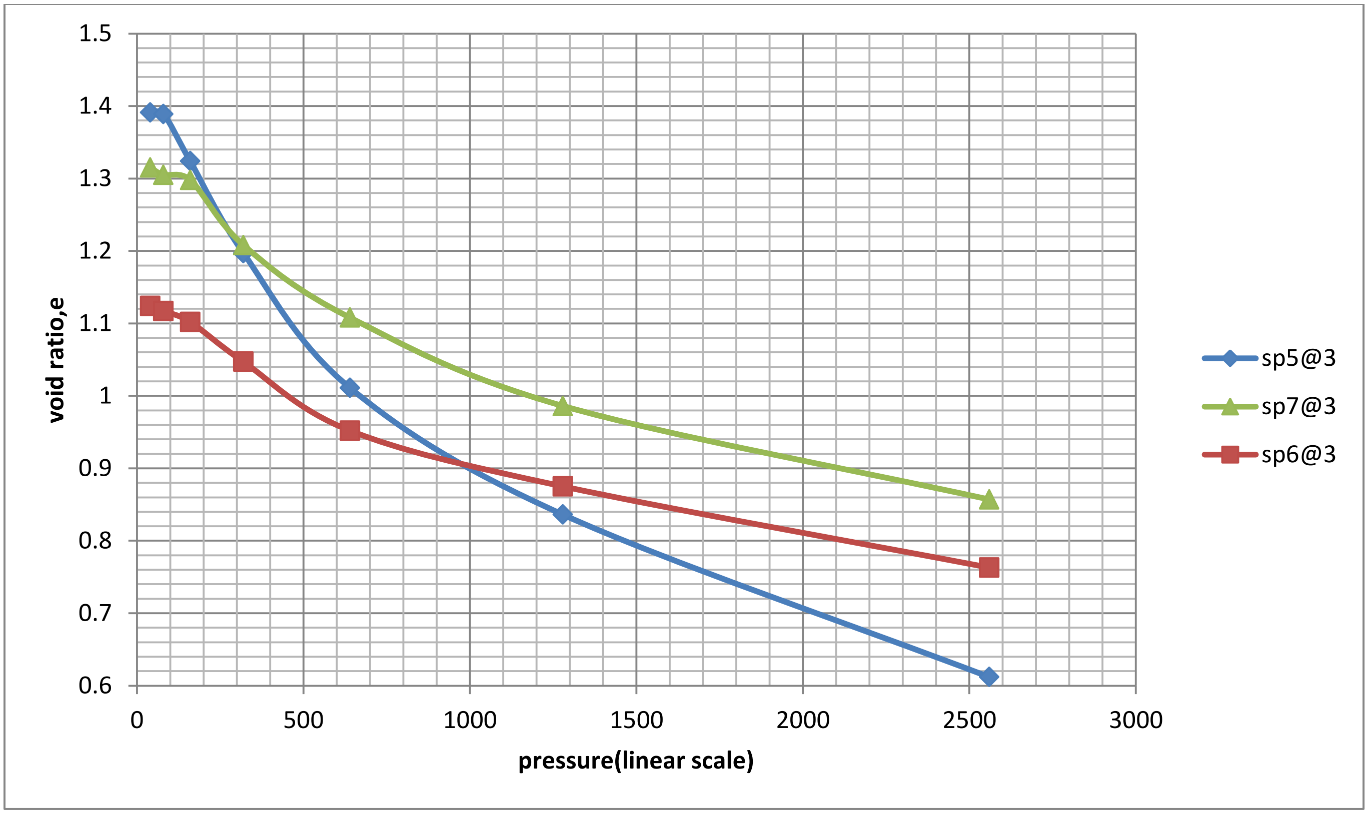


Fig 4.9b Consolidation test result (linear scale)

4.5.4 Coefficient of volume compressibility(mv)

Coefficient of volume compressibility is the volumetric strain per unit increase in effective stress. As it was mention by (Lambe et al. (1969)), some typical values of coefficient of volume compressibility (mv) are used to classify soils on the basis of their compressibility. He mentioned a coefficient of volume compressibility value of **0.10-0.30 m^2/MN** for medium compressibility and **0.05-0.10 m^2/MN** for low compressibility clays.

The values of coefficient of volume compressibility for the selected samples were calculated for each increment of pressures and the result obtained is tabulated in Table 4.9. The value for the studied soil ranges from 0.02 to 0.5 m^2/MN.

4.5.5 Pre-consolidation pressure

A soil may have been pre-consolidated during the geologic past by the weight of an ice which has melted away, or by other geologic overburden or/and structural loads which no longer exist. For example, thick layers of overburden soil may have been eroded or excavated away or heavy structures may have been torn down. Also capillary pressures which may have acted on the clay layers in the past may have been removed for one reason or another. The practical significance of the preconsolidation load appears in calculating settlements of structures (Jumikis, 1984). The tip to designer to limit the settlement of the structure would be to keep the applied foundation pressure lower than the preconsolidation pressure.

The apparent pre-consolidation pressures were determined using Cassagrande construction method as shown in Figure 4.15 and the results are summarized in Table 4.7. The value ranges approximately from 150 Kpa to 200

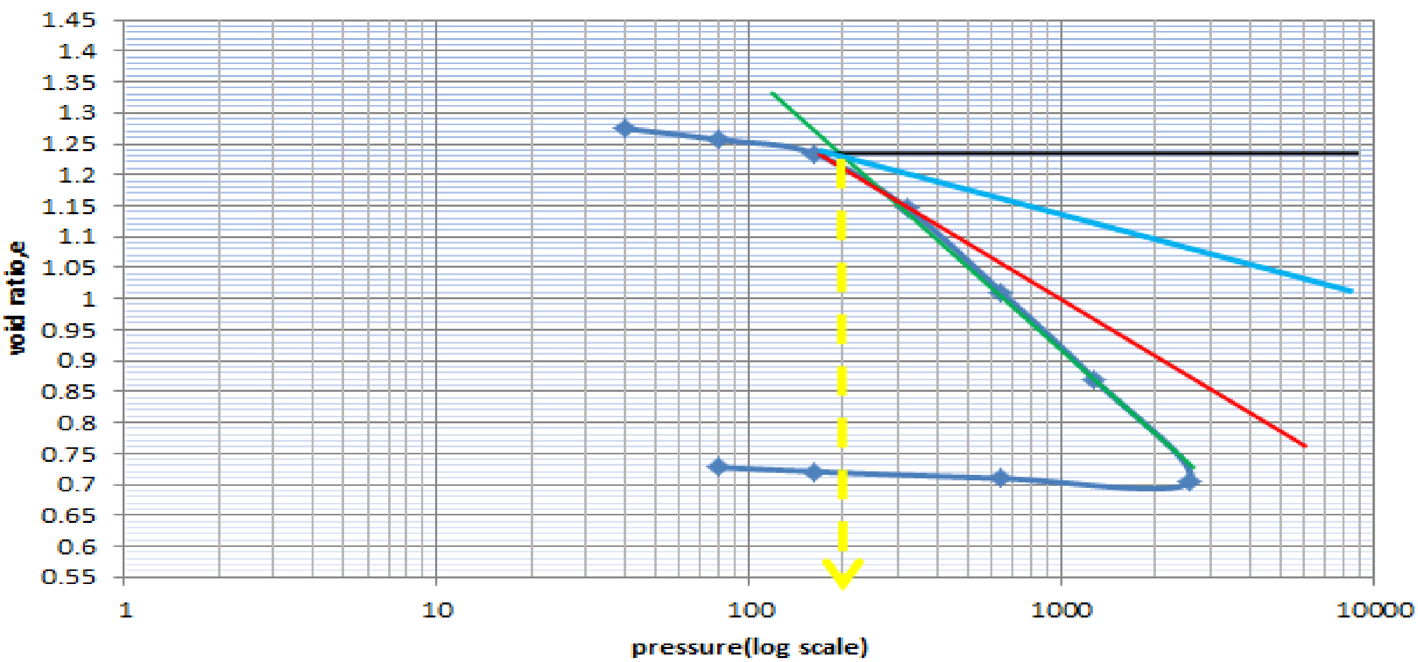


Fig 4.10. Determination of preconsolidation using Cassagrande method for sp5@3

The relative amount of pre-consolidation pressure is usually reported as the over consolidation ratio (OCR) which is defined as the ratio of pre-consolidation pressure to the present vertical effective stress (Equation 4.3). The results for the studied soils are summarized in Table 4.9. The OCR values for all soil samples are > 1, suggesting the studied soils are over consolidated.

$$OCR = P_C/P_0 \quad \text{..........} 4.3$$

Table 4.9 results of laboratory consolidation parameters tests

Designation	Depth (m)	Moisture content	Wet density (gm/cm3)	Pressure (Kpa)	Void ratio, e	Coefficient of consolidation(cv) cm2/sec.	Coefficient of volume compressibility(mv) m^2/MN	Coefficient of permeability 10^{-9} cm/sec	Compression index (Cc)	Pre-consolidation pressure, P'C (Kpa)	Overburden pressure, ,Kpa	OCR,%
SP1	3	40.2	1.61	40	0.751	1.06	0.280	2.91	0.282	160	47.38	3.38
				80	0.741	0.82	0.310	2.49				
				160	0.708	0.97	0.19	1.81				
				320	0.623	0.45	0.118	0.48				
				640	0.567	0.21	0.054	0.11				
				1280	0.482	0.19	0.042	0.08				
				2560	0.399	0.17	0.032	0.05				
SP2	1.5	39.5	1.88	40	0.679	0.73	0.29	2.08	0.193	150	27.66	5.42
				80	0.659	1.04	0.18	1.84				
				160	0.635	1.05	0.16	1.65				
					0.594	0.87	0.13					

				320				1.11				
				640	0.526	0.49	0.201	0.97				
				1280	0.468	0.15	0.08	0.12				
				2560	0.391	0.13	0.06	0.08				
				40	0.742	1.07	0.23	2.41				
				80	0.739	0.92	0.14	1.26				
SP3	1.5	41.2	1.86	160	0.714	0.83	0.059	0.48	0.199	150	27.37	5.32
				320	0.651	0.24	0.18	0.42				
				640	0.579	0.27	0.042	0.11				
				1280	0.519	0.31	0.024	0.07				
				2560	0.438	0.15	0.043	0.06				
				40	0.785	0.19	0.26	2.68				
SP4	1.5	35.8	1.97	80	0.779	1.02	0.25	2.50	0.223	160	29.55	5.41
				160	0.743	0.85	.065	0.54				
					0.671		0.11					

				320		0.39		0.42				
				640	0.614	0.21	0.043	0.19				
				1280	0.547	0.59	0.084	0.16				
				2560	0.462	0.17	0.024	0.04				
SP5	3	39.7	1.59	40	1.391	0.21	0.57	1.17	0.581	200	46.79	4.27
				80	1.389	0.14	0.52	0.71				
				160	1.324	0.17	0.26	0.43				
				320	1.197	0.24	0.12	0.28				
				640	1.011	0.22	0.095	0.21				
				1280	0.836	0.16	0.051	0.08				
				2560	0.612	0.18	0.021	0.04				
SP6	3	36.8	1.57	40	1.124	0.23	0.74	1.67	0.256	200	46.21	4.33
				80	1.117	0.27	0.53	1.40				
				160	1.102	0.31	0.24	0.73				
					1.047		0.15					

				320		0.11		0.16				
				640	0.952	0.29	0.05	0.14				
				1280	0.875	0.13	0.089	0.11				
				2560	0.763	0.14	0.02	0.03				
SP7	3	38.6	1.94	40	1.315	0.84	0.24	1.98	0.405	190	57.09	3.33
				80	1.305	0.52	0.11	0.56				
				160	1.298	0.31	0.14	0.43				
				320	1.208	1.07	0.04	0.42				
				640	1.108	0.25	0.09	0.22				
				1280	0.986	0.18	0.05	0.18				
				2560	0.857	0.17	0.03	0.05				

4.5.6 Coefficient of permeability

The rate of consolidation depends on the permeability of the soil, thus the coefficient of permeability can be measured either by using field tests, or tests conducted in the laboratory. Additionally, permeability is sometimes also estimated from one dimensional consolidation parameters (Cv and Mv) using Equation 4.4.

$$K = Cv * Mv * \gamma w \qquad \mathbf{4.4}$$

Where, Cv = coefficient of consolidation

Mv = coefficient of volume compressibility

Thus coefficients of permeability as a function of void ratio were calculated from the consolidation test results by using the above equation and summarized in table 4.9. The values of this coefficient normally range from 10^{-9} to 10^{-8} cm/sec, suggesting the soils under study are practically impermeable (Cassagrande et al. (1940))

The relationship between the coefficient of permeability and the consolidation pressure is shown in Figure 4.16. It can be seen from this figure that permeability is dependent on the applied consolidation pressure and the value decreases with increasing consolidation Pressure.

The relationship between coefficient of permeability and void ratio is also shown in Figure 4.17. It is observed from Figure 4.17 that the permeability value of the samples increases with increasing void ratios.

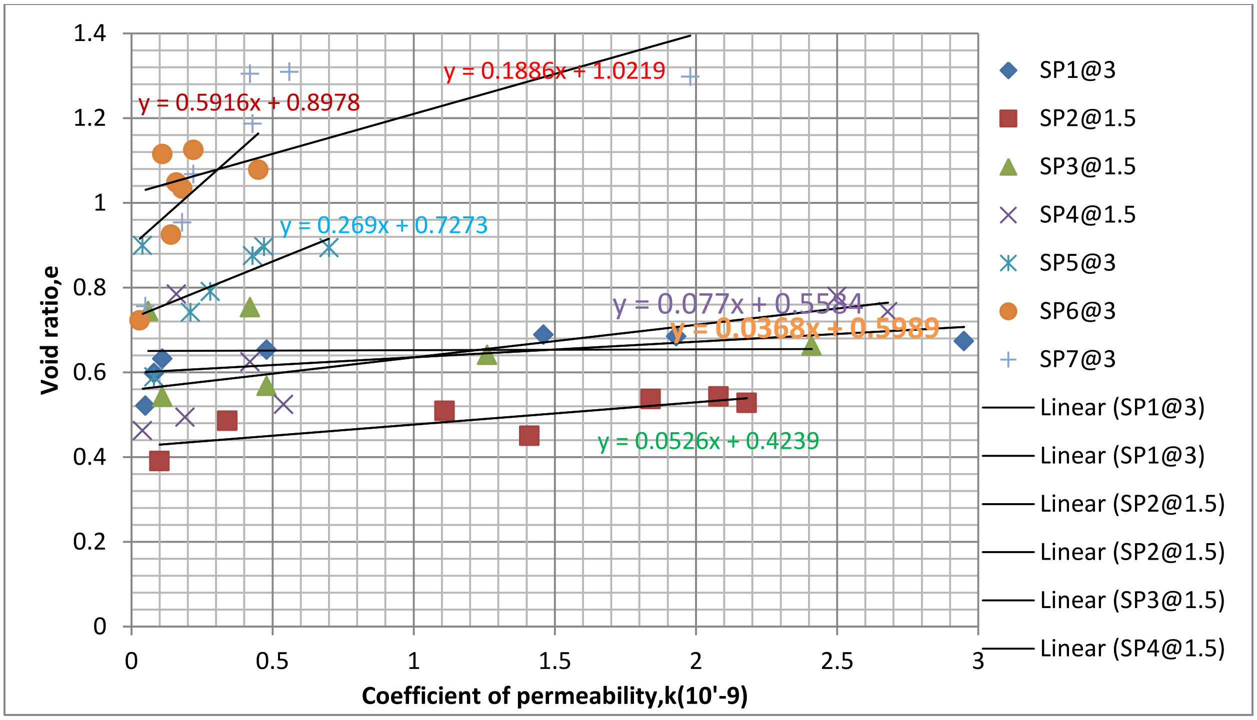


Fig.4.11. Relationship between void ratio and coefficient of permeability

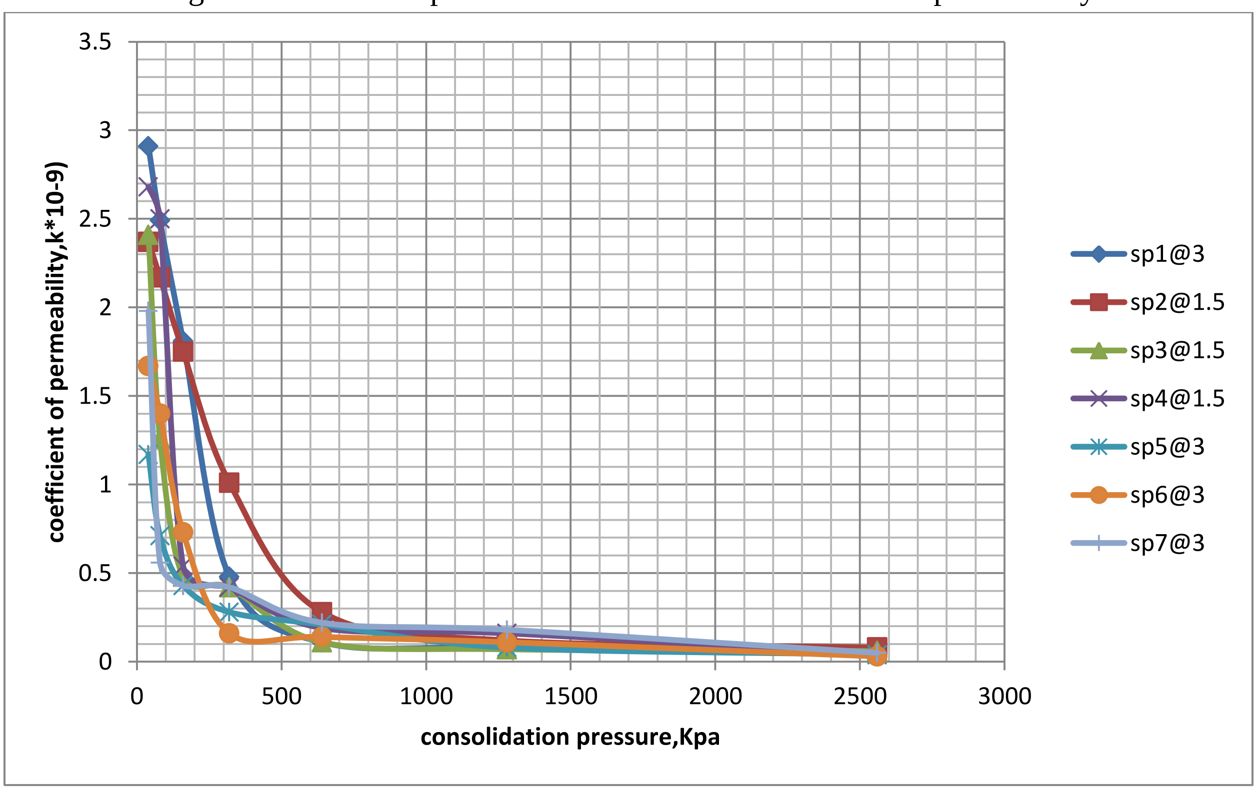


Fig 4.12 Relationship between consolidation pressure and coefficient of permeability

5. CONCLUSION AND RECCOMENDATION

5.1 CONCLUSION

The index property, shear strength and consolidation characteristics of Asela lateritic soils are investigated on disturbed soils after molding, from which different results are obtained. Based on the test results, the following conclusions are drawn:

1. The General soil classification systems, USCS and AASHTO, show that the lateritic soils of Asela are fine grained and unsuitable as construction material and sub grade material.
2. Almost all the samples have free swell value of less than 50%. This shows the soil in the study area is non-expansive with free swell value ranging from 22% to 50%.
3. From the mode of formation and comparisons of gradation charts, Asela soil could be grouped as Ferrisol soil.
4. The values of compression index, Cc, and coefficient of consolidation, C_V, of Asela lateritic soil as determined from the consolidation test on disturbed samples ranges from 0.193-0.581 and 0.11-1.06 cm^2/sec, respectively. The values of the consolidation parameters of the studied soils are relatively small, suggesting the soils are characterized by low compressibility and slow rate of consolidation.
5. The shear strength parameters Cu and Φ values obtained from UU test range from 89.63 to 161.48kPa and from 17^0 to 24^0respectively.
6. The soils under the study are over consolidated soils as the over consolidation ratio (OCR) is higher than one.

5.2 RECCOMENDATION

1. To increase the level of accuracy of the shear strength characteristics of Asela lateritic soils it is recommended to conduct further tests by using CU tests with more number of samples that are undisturbed.
2. In this thesis the consolidation parameters of the soil under studies are determined from disturbed samples. Thus it is recommended that to study the consolidation parameters of the considered soil on undisturbed samples for more accuracy.
3. The maximum cell pressure applied in this study is 300kpa; further detailed investigation has to be carried out by using higher cell pressure.
4. In this thesis the profile of the studied soil is within 1.5m to 3m, thus detailed profile investigation should have to be carried out on soil samples that are under studies.
5. Finally it is recommended that Cost benefit analysis should have to be made to use the soils as construction materials

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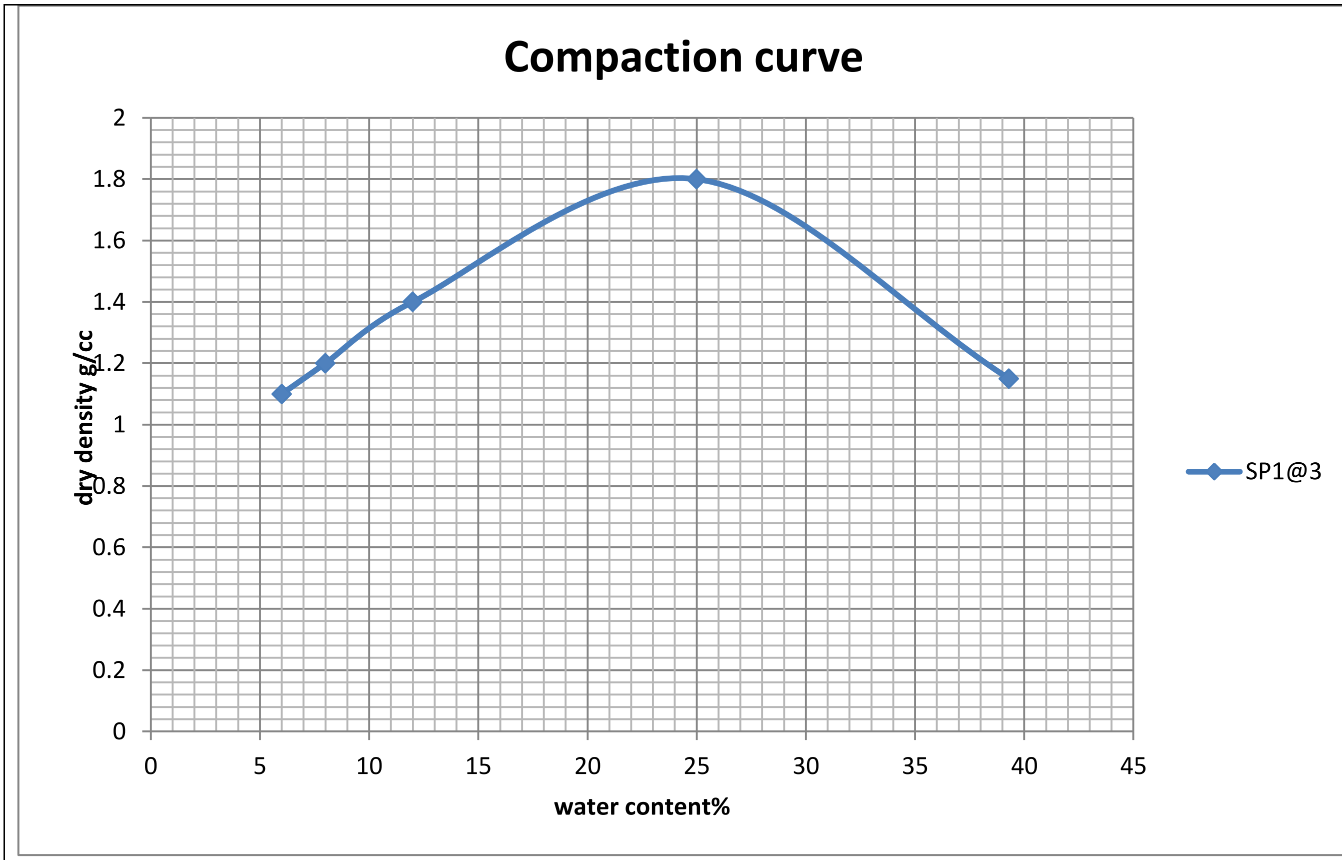
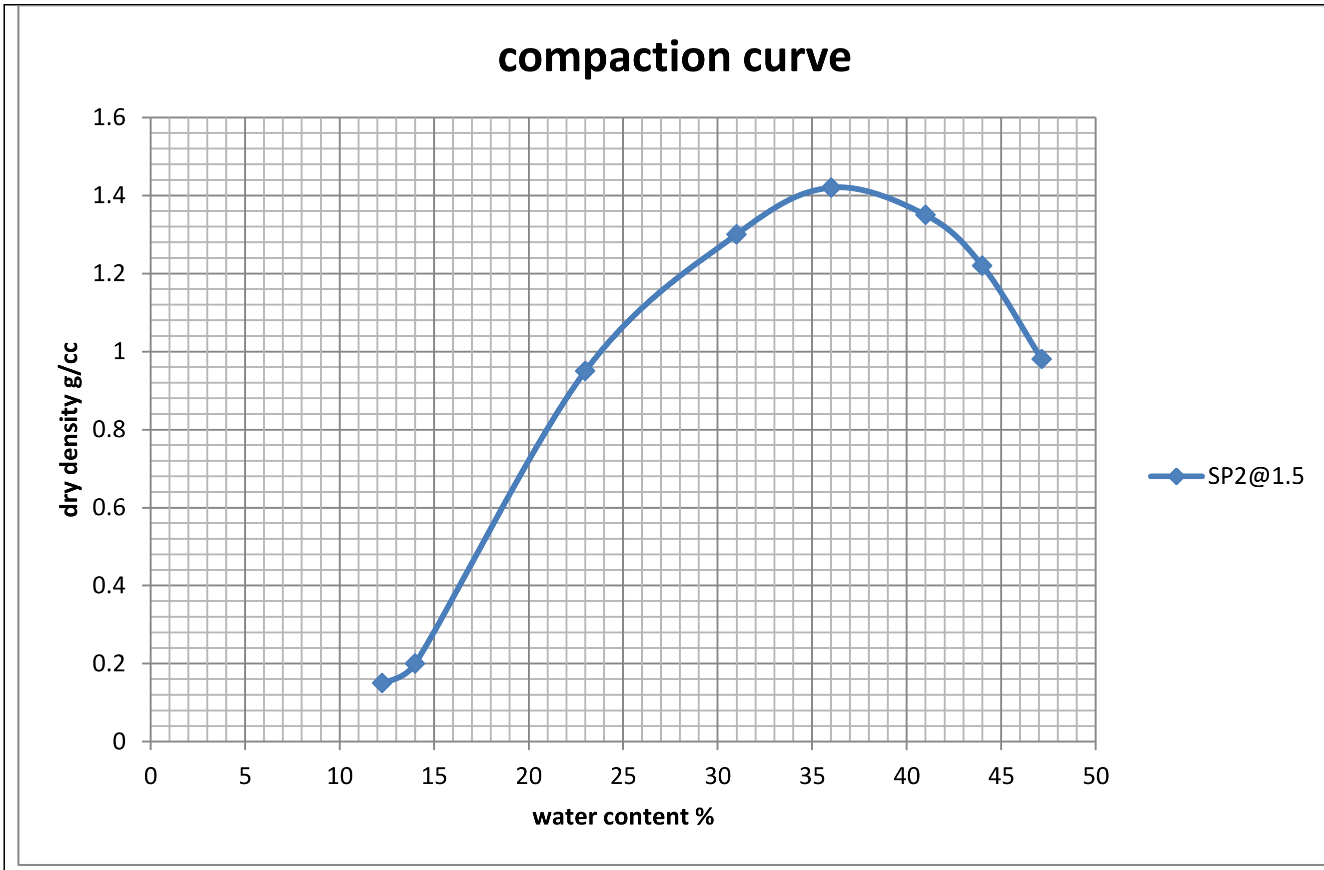
APPENDIX – A



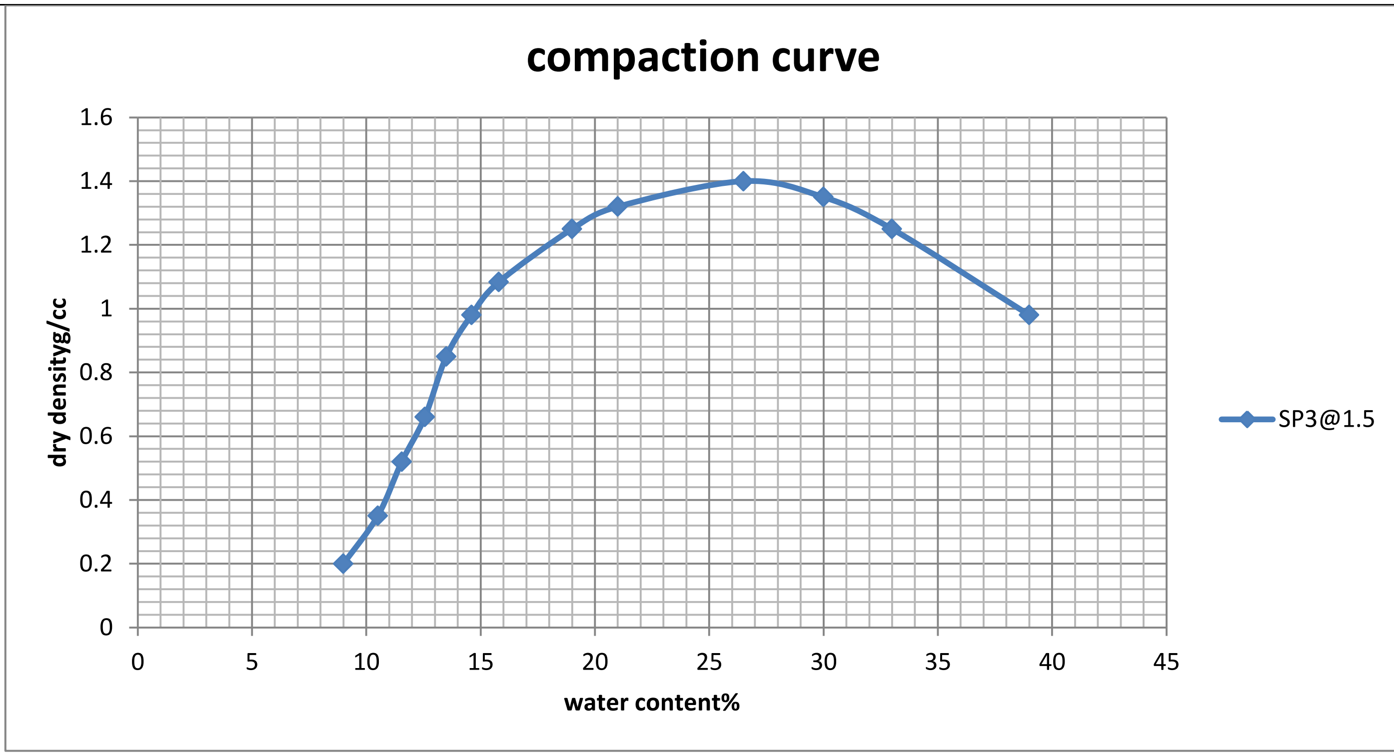
Optimum moisture content	25%	Maximum dry density	1.8g/cc

Fig 13 standard compaction test curve for sp1@3



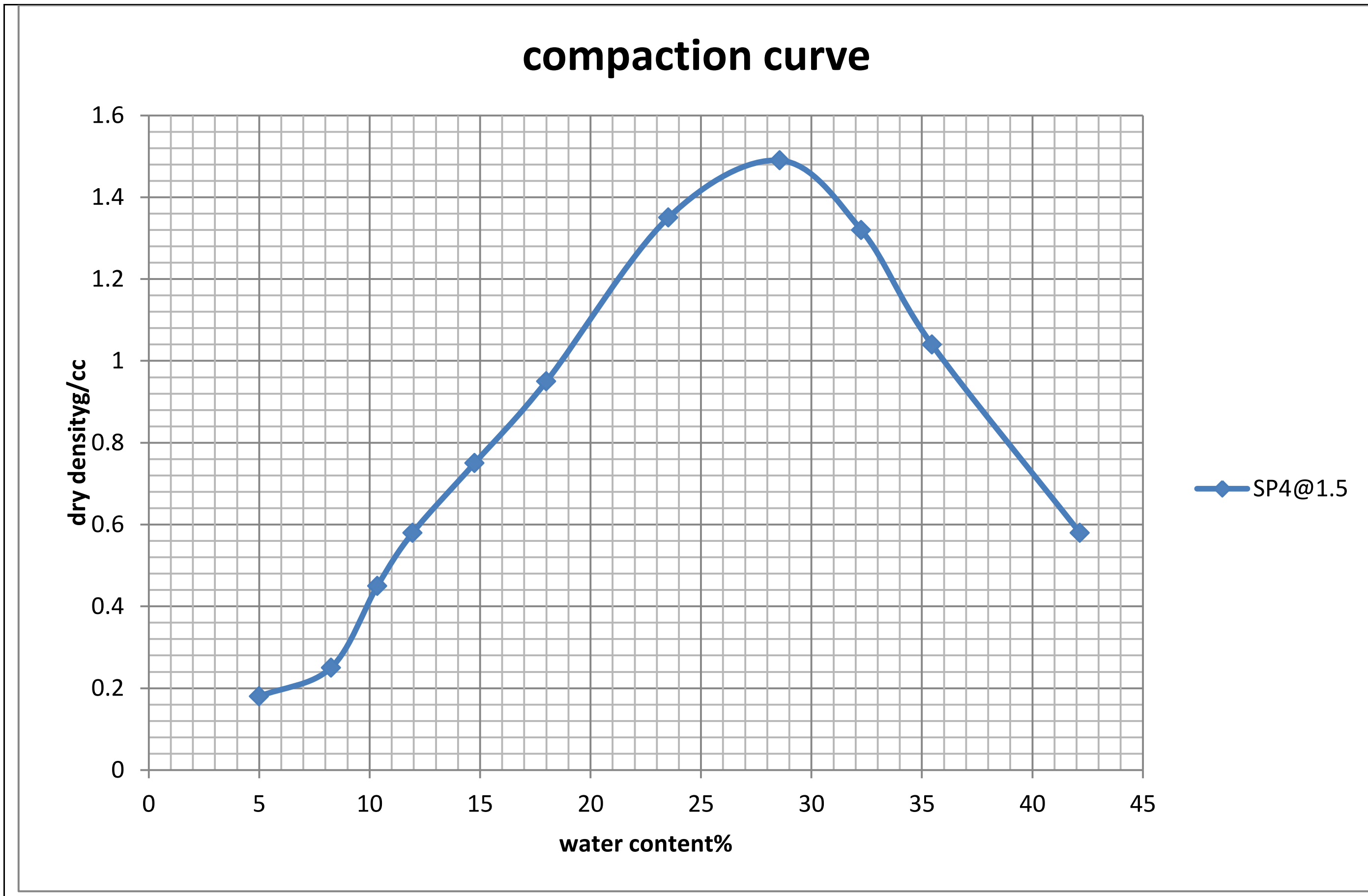
Optimum moisture content	36%	Maximum dry density	1.4g/cc

Fig 14 standard compaction test curve for sp2@1.5



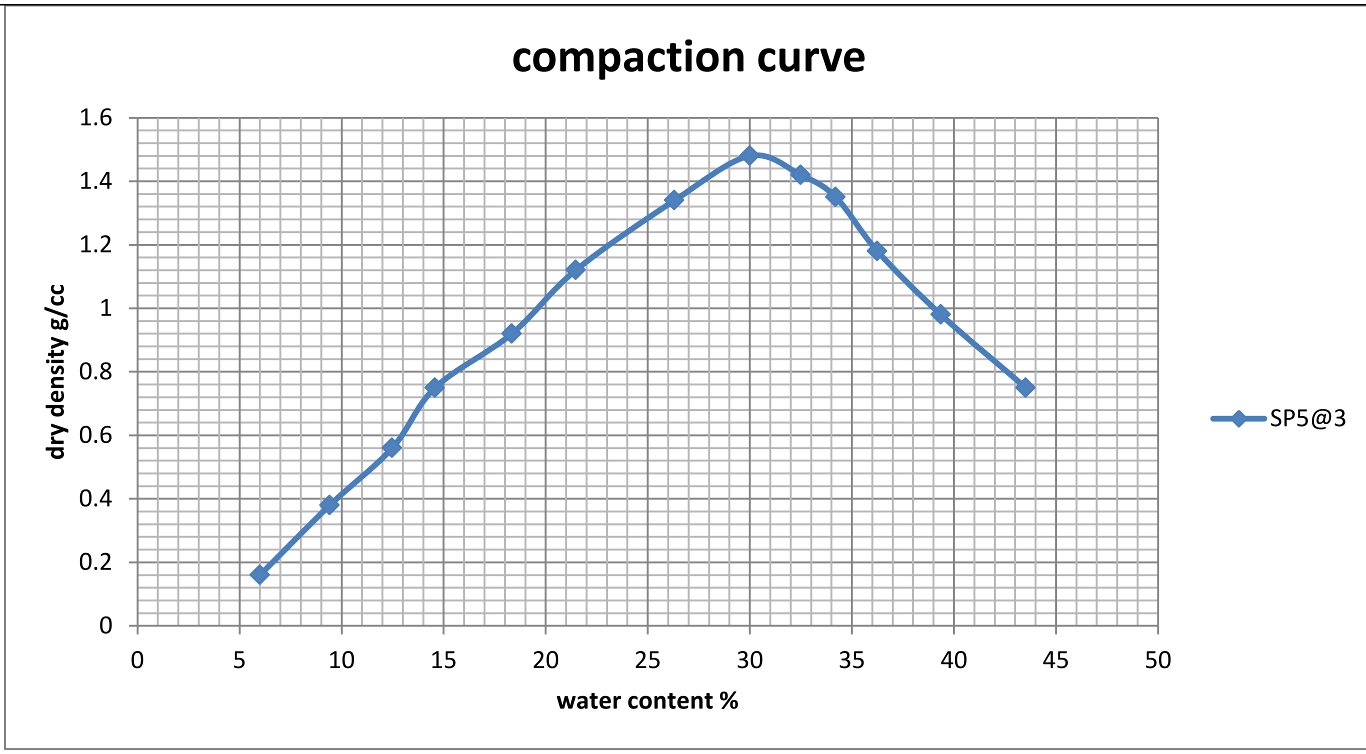
Optimum moisture content	26.5%	Maximum dry density	1.38g/cc

Fig 15 standard compaction test curve for sp3@1.5



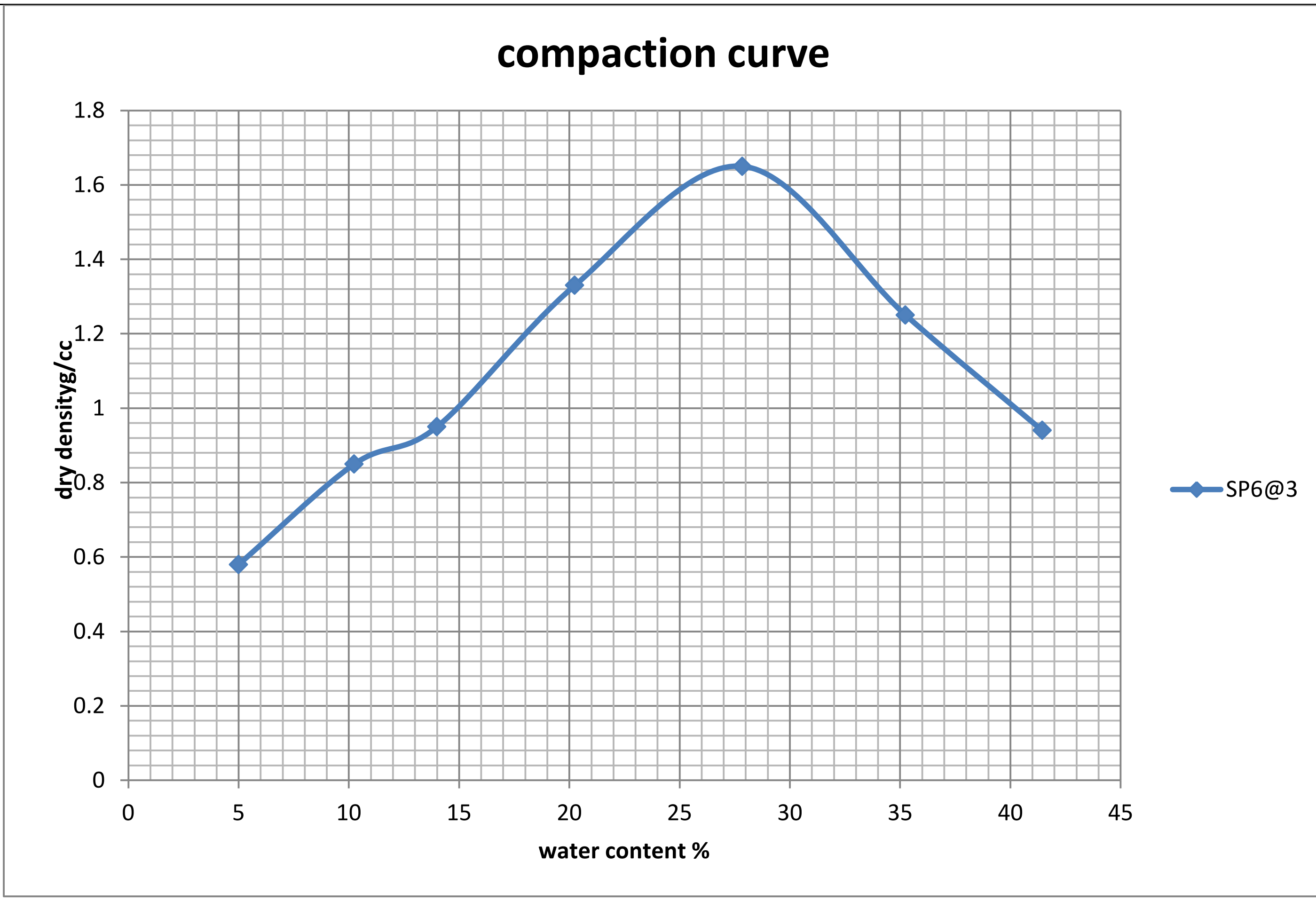
Optimum moisture content	28.56%	Maximum dry density	1.49

Fig 16 standard compaction test curve for sp4@1.5



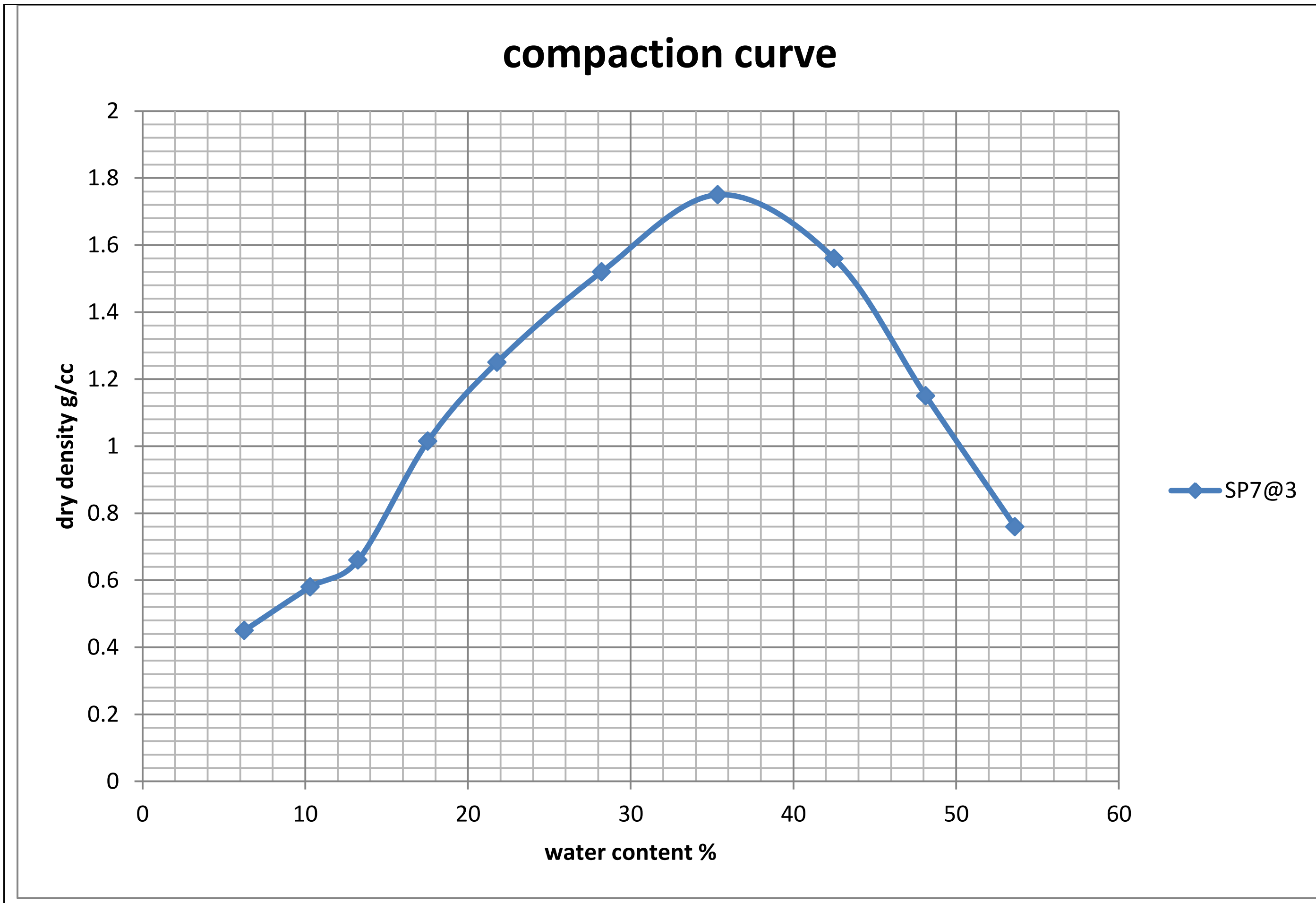
Optimum moisture content	30%	Maximum dry density	1.48g/cc

Fig 17 standard compaction test curve for sp5@3



Optimum moisture content	27.85%	Maximum dry density	1.65

Fig 18 standard compaction test curve for sp6@3



Optimum moisture content	35.36%	Maximum dry density	1.75g/cc

Fig 19 standard compaction test curve for sp7@3

APPENDEX- B

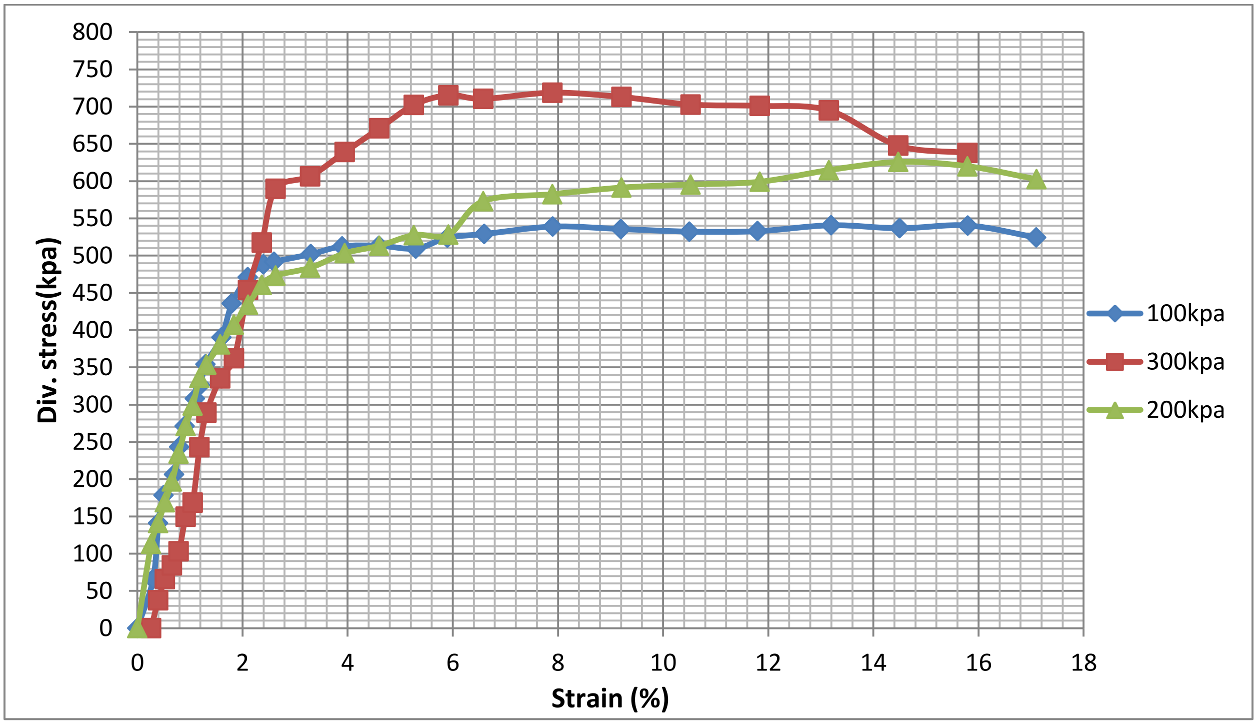


Fig.4.7a. Axial Stress vs. Strain for SP2 at 1.5

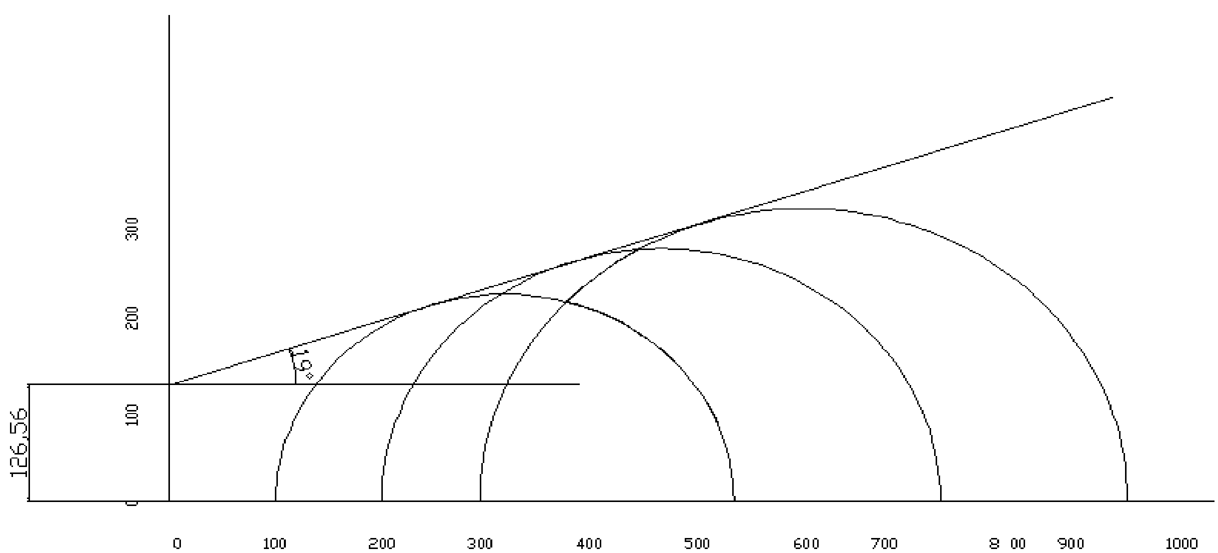


Fig 4.7b. Mohr circle for cell pressures of 100,200,300 Kpa and failure envelope for SP2 at 1.5m

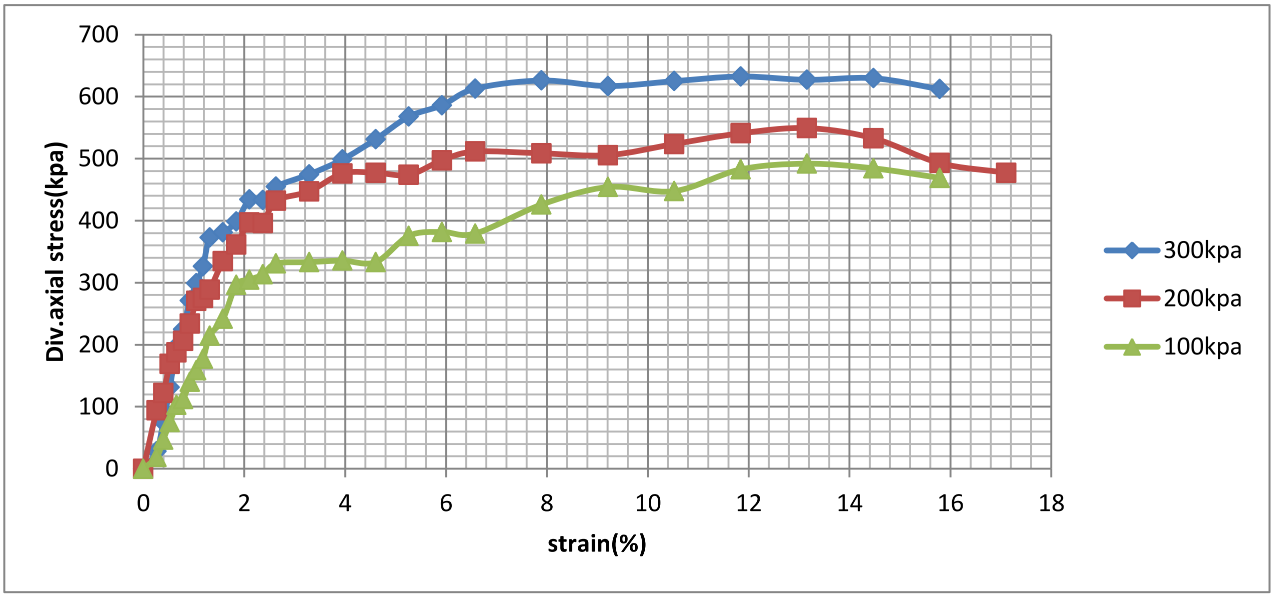


Fig 4.8a Axial Stress Vs Strain at 1.50 m depth for SP3

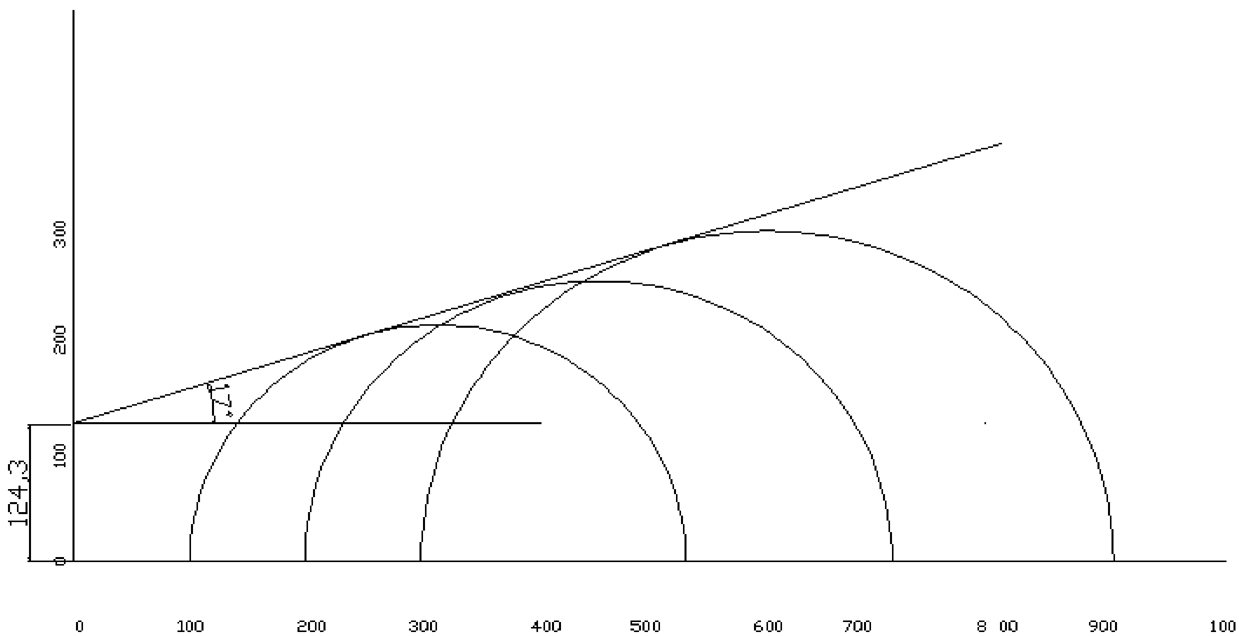


Fig 4.8b Mohr circle for cell pressures of 100,200,300 kPa and failure envelope for SP3 at 1.5m

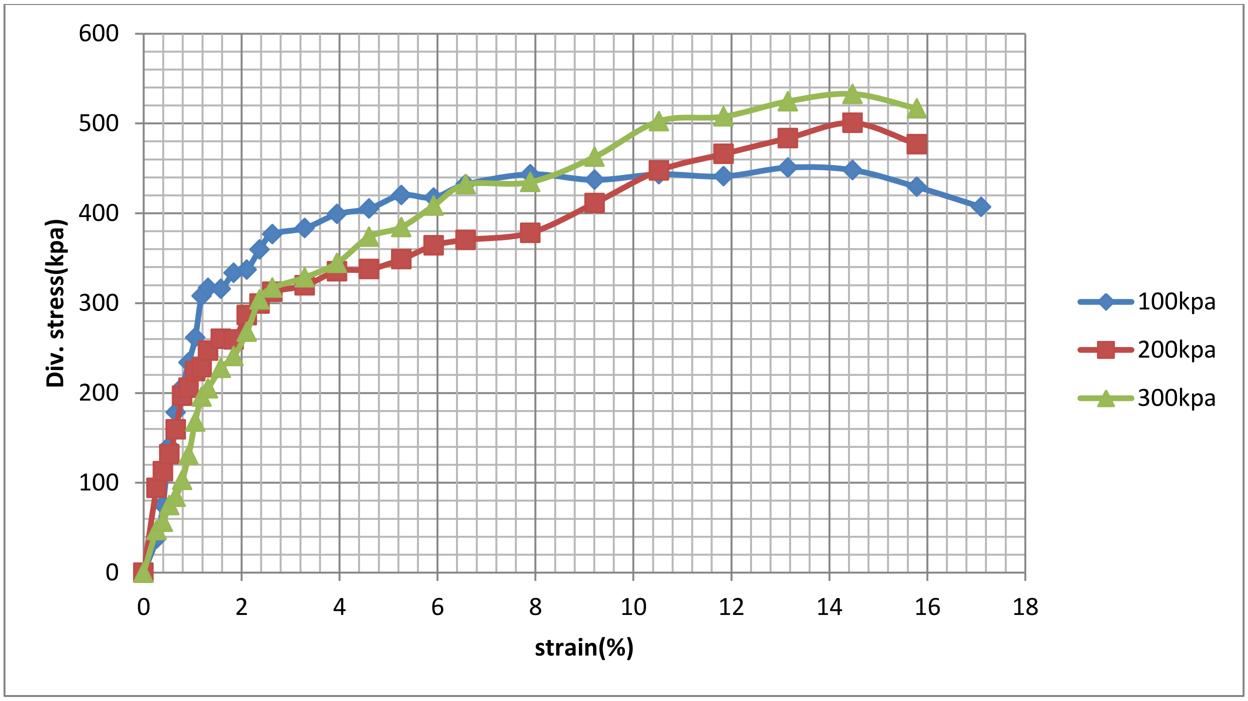


Fig 4.9a Axial Stress Vs Strain at 1.50 m depth for SP4

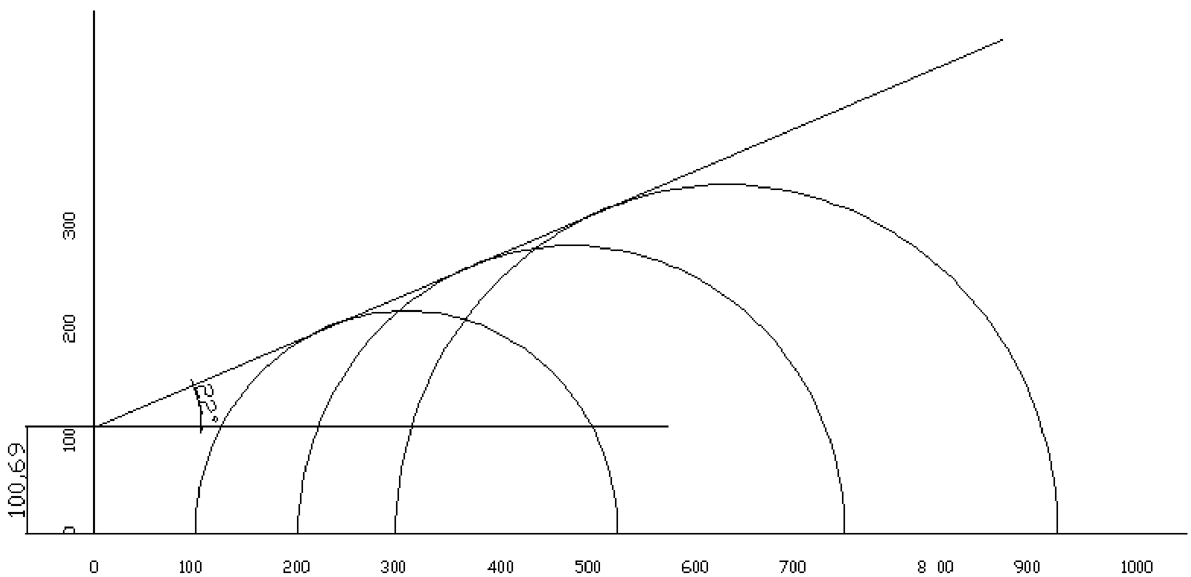


Fig 4.9b Mohr circle for cell pressures of 100,200,300 kPa and failure envelope at 3m for SP4

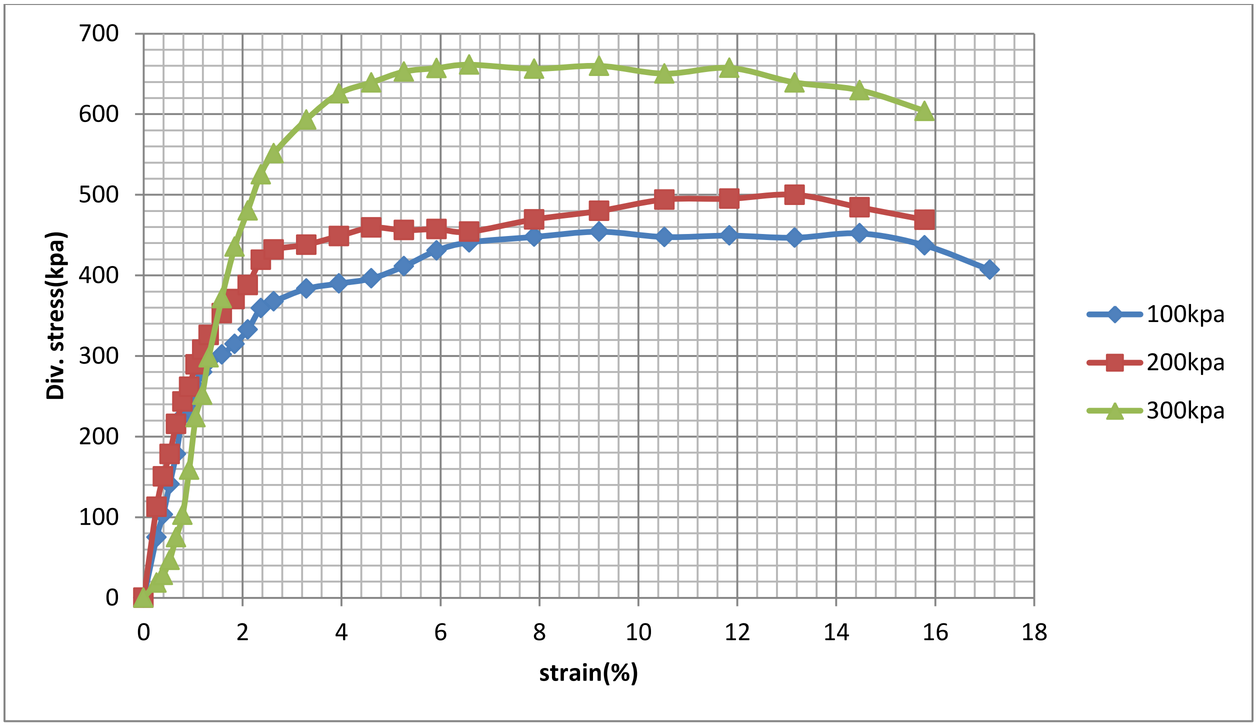


Fig 4.10a Axial Stress Vs Strain at 3 m depth at SP5

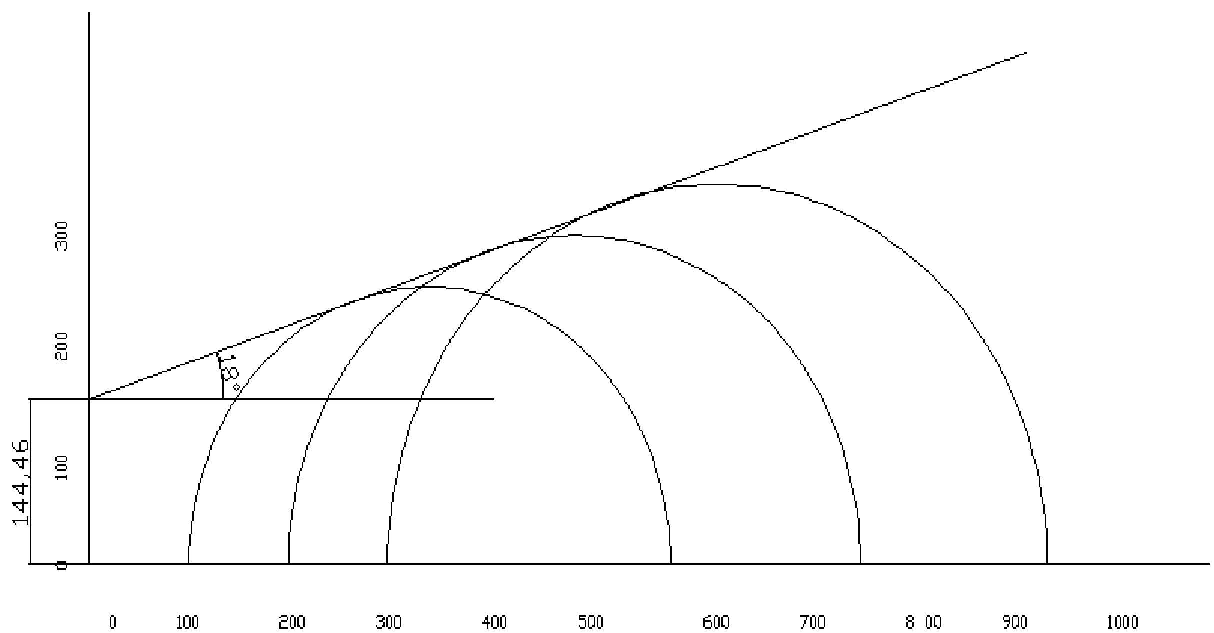


Fig 4.10b Mohr circle for cell pressures of 100,200,300 Kpa and failure envelope for SP5 at 3m

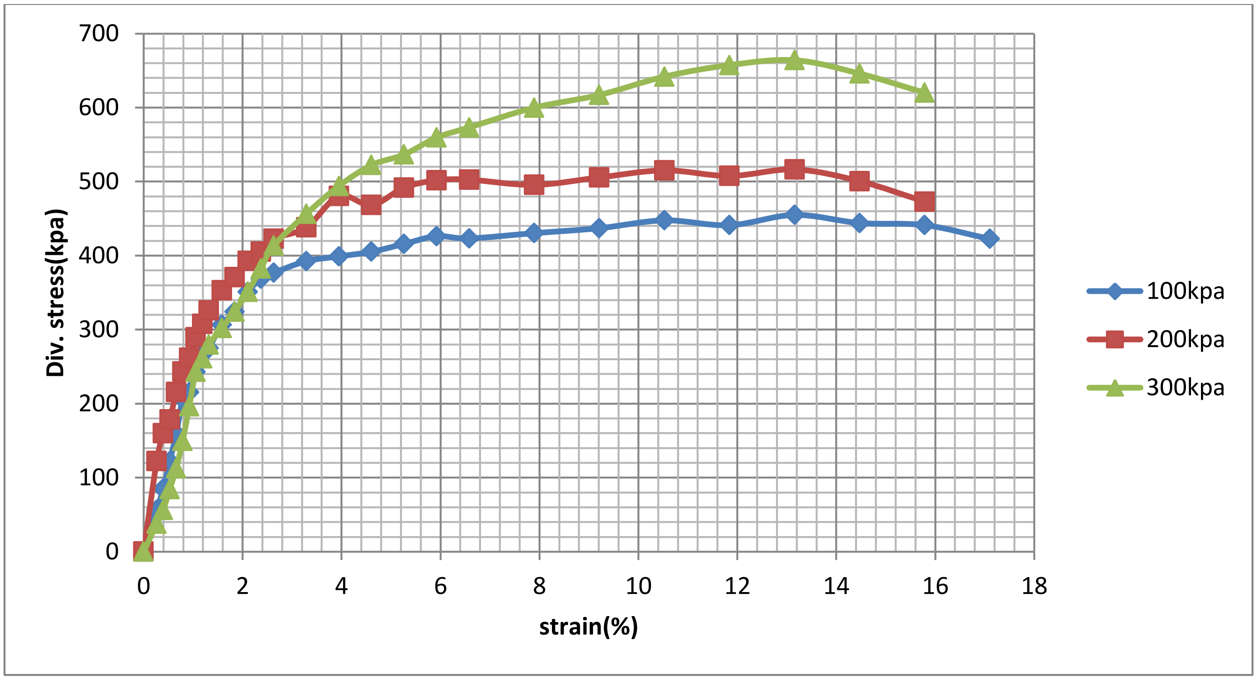


Fig 4.11a Axial Stress Vs Strain at 3 m depth at SP6

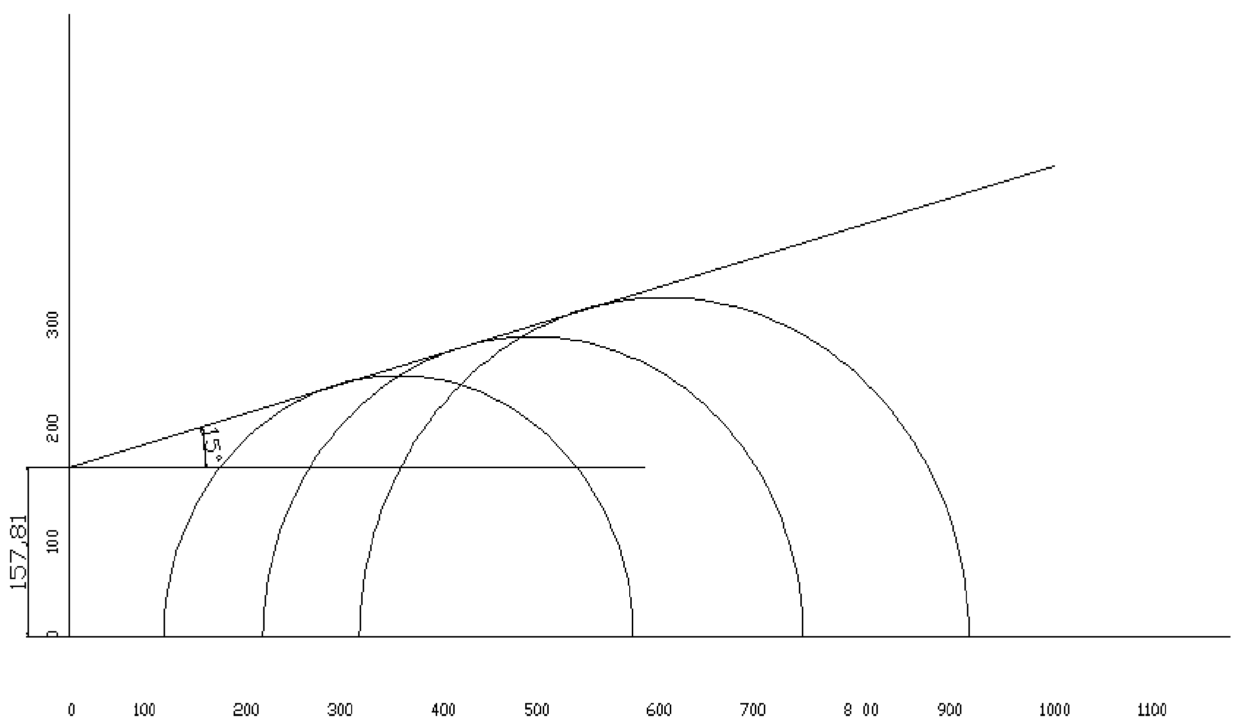


Fig 4.11b Mohr circle for cell pressures of 100,200,300 kPa and failure envelope at 3m for SP6

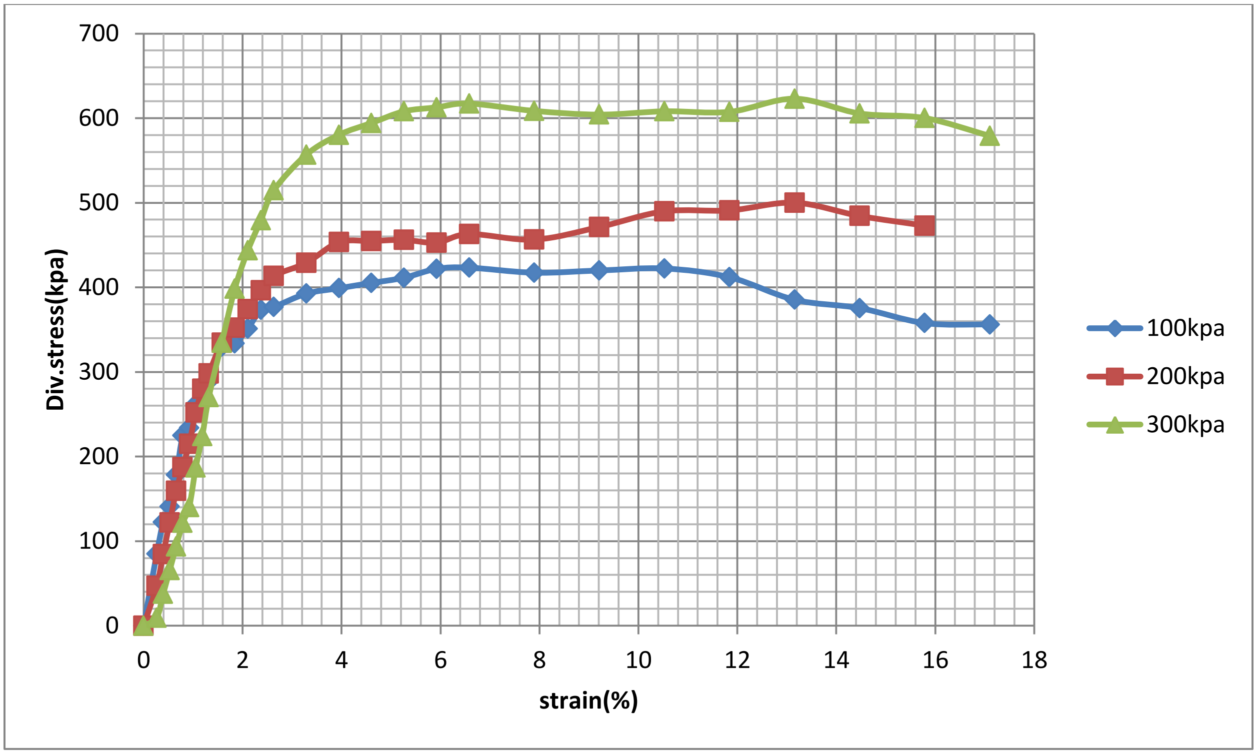


Fig 4.12a Axial Stress Vs Strain at 3 m depth at SP7

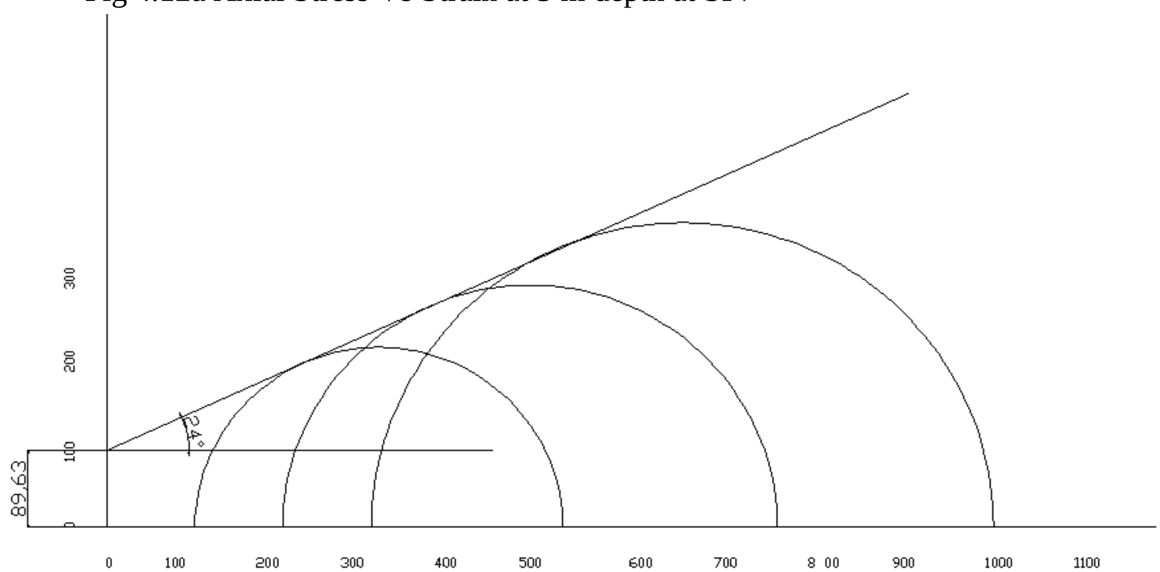


Fig 4.12b Mohr circle for cell pressures of 100,200,300 Kpa and failure envelope at 3m for SP7