ADDIS ABABA UNIVERSITY
SCHOOL OF GRADUATE STUDIES
FACULTY OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT

EXAMINING ATTERBERG LIMITS FOR EXPANSIVE SOILS

A thesis submitted to
The school of graduate studies of Addis Ababa University in partial fulfillment of the
requirement for the degree of Master of Science in
Geo-technical Engineering

Advisor Dr – Ing Samuel Tadesse

By Habtamu Kassahun
December 2006

Addis Ababa University
School of Graduate Studies
Department of Civil Engineering

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Declaration

I, undersigned declare that this thesis is my original work, has not been presented for a degree in any other universities and that all sources of materials used for this thesis have been duly acknowledge.

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Date of Submission: October, 2006

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Symbols & Abbreviations

LL (ω_L)	Liquid limit	percent
PI (I_P)	Plastic index	percent
PL (ω_P)	Plastic limit	percent
SL (ω_s)	Shrinkage limit	percent
ω	Moisture content	percent
LI	Liquidity index	
τ	Shear stress	$\frac{KN}{m^2}(Kpa)$
γ	Shear strain	
V	Volume	cm^3

V_{min}	Minimum volume	cm^3
W_d	Dry weight of soil pat	g
NGL	Natural Ground Level	
ΔV	Change in volume	%
V_f	Final volume after re wetting	cm^3
V_i	Initial volume, volume of dry pat	cm^3
ω_A	Additional moisture content	g
W_D	Mass of dry soil	g
W_W	Mass of wet soil	g
ω_{SA}	Additional moisture content with that of shrinkage limit	%
ω_f	Final moisture content	g
ω_{gs}	Moisture content for fully saturation	g
ω_{DM}	Additional moisture content with that of dry mass	%

ABSTRACT

Expansive soils have been responsible for many structural damages that result in great financial losses in many parts of the world including Ethiopia. Proper understanding of the properties of the soil helps to understand the actual causes of failure of structure.

The main objective of this research work is to examine Atterberg limits by giving emphasis on the range at which a reduction of water content will not cause a decrease in the volume of the soil mass, i.e. shrinkage limit. In addition to this an attempt has been made to define the maximum moisture content corresponding to the maximum swelling and the effect of initial moisture content on the swell pressure of expansive soils.

To achieve the research objective, disturbed and undisturbed samples of both Expansive and red clay soils were collected from Addis Ababa. The red clay soil was used as a methodology control of Atterberg limits .The following laboratory and field test were conducted to attain the purpose of this work

- Free swell and linear shrinkage
- Atterberg limits
- Swelling measurement with and without seating load
- Artificial flooding of test pit and moisture content
- Measuring of volume at various stages of desiccation (Drying process)
- Re wetting of the completely dry pat
- Swell pressure measurement by varying initial moisture content

Laboratory test results on both clay samples show that the volume change of expansive soil is more significant than that of red clay soil. The volume change on expansive soil is mainly due to the swell property of the soil and this volume change is sufficient enough to develop distress on structures.

The investigation of swell measurement and moisture content of undisturbed soil samples showed that the moisture content after a complete swelling is found with in the range of liquid

limit and plastic limit. In addition to this high moisture content soils experience less uplift, but the pressure required to maintain a constant volume is not altered.

Chapter one

Introduction, Scope and Organization of the thesis

1.1 General

The problem of expansive soils was not recognized by soil engineers until the latter part of 1930 [7]. Expansive soils are defined as plastic clay soils that exhibit high volume change when their environmental conditions are altered from dry to wet or vise versa. These soil types are very problematic materials to be used for construction material or foundation purposes. Constructions on these soils always impose problems to engineers for the fact that such soil is characterized with excessive swelling and shrinkage due to the variation of moisture content of the soil.

Today, there is a world wide interest in understanding/investigating expansive soils. These soils are major problematic soils in Africa and other parts of the world. Our country Ethiopia is one of the victims of black and gray expansive soils. The expansive soils of Ethiopia are residual, derived from weathering of basic volcanic rock. In the capital Addis Ababa it has been known that expansive soils cover large part of the city, in which recent construction areas are extensively covered with expansive soils. Damage due to expansive soils could occur on any type of buildings, highways, air field canal lining and other civil engineering constructions that are not properly designed and/or constructed for expansive soils. However, light structures like one or two story buildings, warehouses, retaining walls, roads, runways, and buried facilities are more vulnerable to damages because these structures cannot exert sufficient pressure to counteract the uplift pressure from the expansive soils. In this regard numerous building damages have been observed in areas covered by expansive soils in Addis Ababa [17].

In the field, expansive clay soils can be easily recognized in dry season by deep cracks in roughly polygonal patterns. Potentially expansive soils can typically be recognized in the laboratory by plastic properties, linear shrinkage, and free swell tests. These properties can be measured in the laboratory directly by testing remolded or undisturbed soil samples. Such a test enables an easy and accurate measurement of the swelling pressure of clay under various conditions.

One can also easily recognize expansive soil areas by simply looking the failures of lightly loaded buildings as shown in Figure 1.1. Swelling soils lift up lightly loaded foundations and frequently cause distress in floor slabs, and vertical and horizontal cracks develop in the walls. Diagonal cracks that develop below windows and above doors are strong indications of swelling movement. *(Figure 1.1 a & b)*


(a)


(b)

Figure 1.1 *(a) Sever cracking on light weight fence*

(b) Diagonal crack that develop below window

Misunderstanding soils and their properties can lead to construction errors that are costly in effort and material. The suitability of a soil for particular use should be determined based on its engineering characteristics. It has been noticed long ago that constructions on expansive soils face numerous problems and the cause of the problems are investigated in depth by different investigators. Most of the roads and buildings constructed on this type of soils fail before their expected design life or even after few months of completion.

Proper understanding of the properties of the soil help to understand the actual causes of failure of structure and will lead to proper designing and construction by avoiding or minimizing the failure and maintenance of the structure. It is the goal of this research to examine in detail Atterberg limits for expansive soils by giving emphasis on shrinkage limit for the expansive soils. In addition, the effects of initial moisture content on the swelling and swell pressure are studied.

The basic principle of Atterberg limit is developed for the Scandinavian soft non expansive clay of temperate zone. It is worthwhile to study in detail Atterberg limits for the soils found in Ethiopia which is located in tropical zone.

1.2 Scope of the study

This research addresses the above goal by undertaking a detailed investigation on study of Atterberg limits. For this intended purpose, disturbed and undisturbed samples were collected from representative locations of Addis Ababa which were conformed by different investigators to be covered by expansive soils. Atterberg limits tests were conducted on disturbed samples, and from undisturbed soil samples maximum swell and swell pressure by varying initial moisture content was determined from one dimensional consolidometer to know the amount of moisture needed for the development of complete swelling. The test was also conducted with and without seating load to know the time required for maximum swell. Atterberg limit tests are also done on Addis Ababa red clay soil for comparison purpose as Atterberg limit is originally developed and proved to be appropriate for non expansive soils.

1.3 Organization of the thesis

The thesis is organized into four Chapters. Chapter 1 presents the general description and major engineering problems associated with expansive soils and the aim and scope of this research work. The general review of expansive soils and the principles of Atterberg limits are discussed in Chapter two. Material tested, test procedures, results of the test and discussion of the results are presented in Chapter three. Finally, conclusion and recommendations are given in Chapter four.

Chapter Two

Literature review

2.1 General review on expansive soil

2.1.1 Origin , Distribution and characteristics of expansive soils

Tropical expansive soils are often called 'BLACK COTTON' soils. The latter name is believed to have originated in India where locations of these soils are favorable for growing cotton [3].These soils are residual deposits formed from basaltic and sedimentary rocks. Although many local terms are used , all expansive soils have very common characteristics which are of interest to the engineer i.e. they have high clay content with appreciable plasticity, dark or gray in color and the tendency to expand with moisture increase and undergoes excessive shrinkage when it gets dry.

The constituents of the parent material during the early and intermediate stages of the weathering process determine the types of the clay formed. The nature of the parent material is much more important during these stages than after intense weathering for long periods of time. The parent materials that can be associated with expansive soils are classified into two groups [10].The first group comprises the basic igneous rocks and the second group comprises sedimentary rocks that contain montmorillonite as a constituent, which breaks down physically to form expansive soil. The three most common clay minerals are montmorillonite, illite and kaolinite, which are crystalline hydrous alumino-silicate. Of these minerals it is montmorillonite that dominantly presents in most of the expansive soils problem.

Climate and topography play a very significant role in the formation of expansive soils. The situation in which montmorillonite can form require that leaching be restricted, so that magnesium, sodium and iron cations may be accumulated in the system. Thus the formation of montmorillonite is aided by alkaline environment, presence of magnesium ions and lack of leaching. Such conditions are favorable in semi-arid region in relatively low rainfall, particularly where evaporation exceeds precipitation. Under this condition enough water is available for the alteration process, but the accumulated cations will not be removed by flush rain.

Montmorillonite is a three layered mineral having a single octahedral alumna sheet sandwiched between two silica sheets to give a 2:1 lattice structure as shown in Figure 2.1[7]. The bonds are comparatively weak, and dipolar molecules of water can enter between the sheets causing them to expand readily. Thus, soils containing substantial amounts of montmorillonite will exhibit high shrinkage and swelling characteristics.

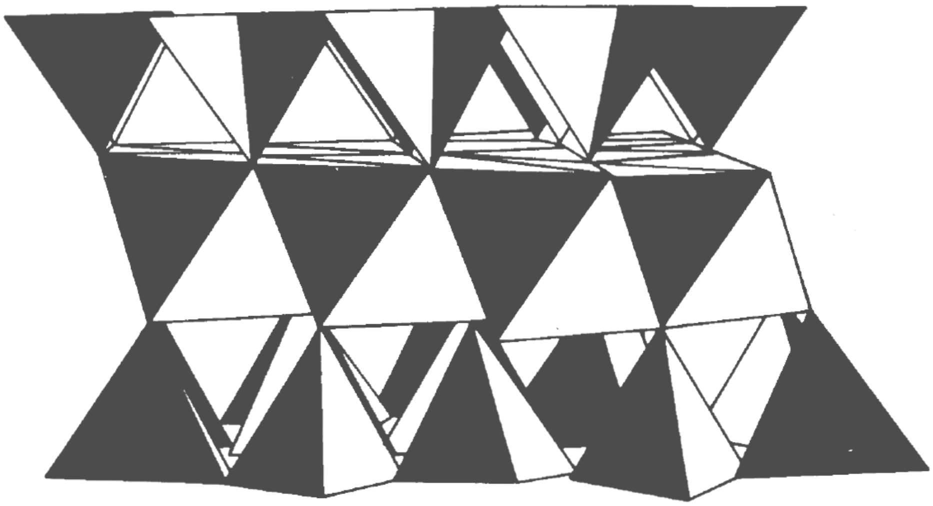


Figure 2.1 Model of a layer of montmorillonite.[7]

Potentially expansive soils can be found almost anywhere in the world; it is widely spread throughout the five continents. Expansive soils cover large parts of the United States, South America, Africa, Australia and Asia. More than 30 countries have been reported for occurrence of expansive soils [7].In Ethiopia expansive soils are observed in areas such as central Ethiopia, following the major trunk roads like Addis Ambo, Addis Welliso, Addis Debre Birhan, Addis Gohatsion, Addis Modjo[15]. Also they cover areas like Mekele, Gambella, and the most southern, south-west, and south-east part of the capital Addis Ababa areas in which the most major recent constructions are being carried out[8].The distributions are shown in Figure 2.2 and Figure 2.3..

Figure 2.2 Distribution of Expansive soils in Ethiopia [8]

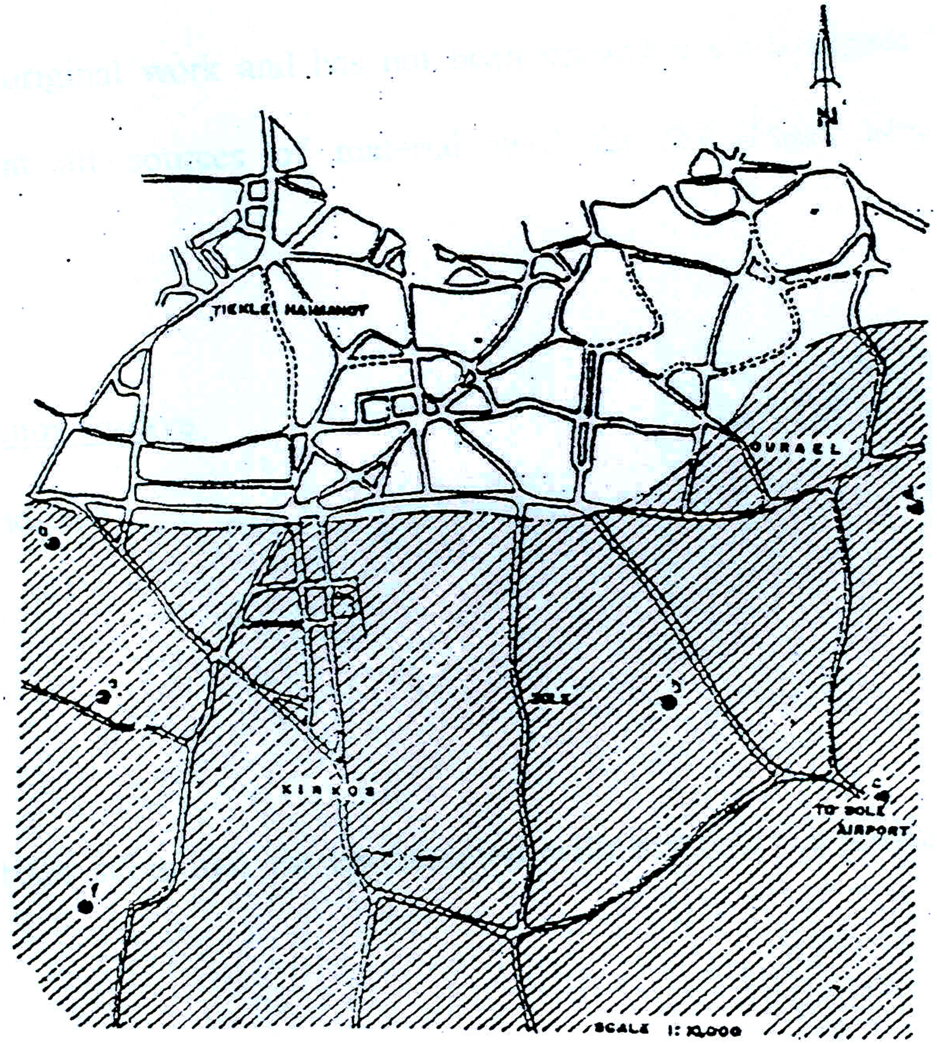


Figure 2.3 Distribution of Expansive soil in Addis Ababa. [17]

The properties of the very fine fractions of soils are of outstanding engineering importance. There will be no volume change if the moisture content of the clay element is unchanged. When the moisture content of the clay is increased, volume expansion in the vertical and the horizontal direction will take place. Complete saturation is not necessary to initiate swelling. Slight increase in moisture content in the magnitude of 1 to 2% is sufficient to cause detrimental swelling as shown in Table 2.1 and Figure 2.4. The most common moisture transfer is by gravity, in which the seepage of surface water, precipitation, and snow melting into the soil are common examples.

It is well recognized by soil engineers that the heaving of expansive soils may take place without the presence of free water. Vapor transfer plays an important role in providing the means for the volume increase of expansive soil. Water vapor at a temperature higher than its surroundings migrates toward the cooler area to equalize the thermal energy of the two areas. When water reaches the cooler area, generally the covered area beneath a structure, condensation can take place and provide sufficient moisture to initiate swelling.

Table 2.1 Effect of varying degree of saturation volume change and swelling pressure. [7]

Moisture content percent		Initial density	Volume Increase percent	Swelling pressure	Degree of saturation percent
Initial	Final	Kg/m^3		kpa	
9.66	13.07	1708	1.83	766.1	61.0
9.66	14.53	1698	3.35	742.1	67.0
9.66	17.58	1692	4.35	574.6	82.0
9.66	18.50	1709	5.53	814.0	86.3
9.66	19.93	1697	6.25	723.0	93.0

Complete saturation is not required to result in a large volume change. There is a misconception that by removing free water the swell can be controlled. It has been a common practice to install drain tiles around a building in an attempt to remove free water and to stop foundation movement. A sub drain does not arrest the migration of moisture. Consequently, swelling can be substantial. [7]

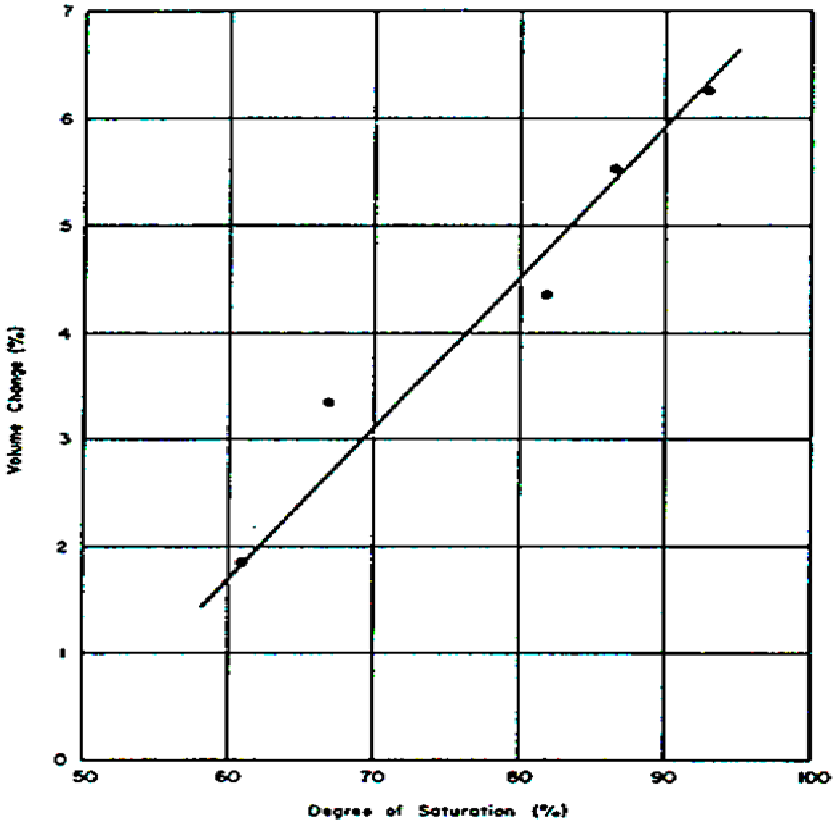


Figure 2.4 Degree of saturation vs. volume change [7]

Accrding to Mielenz and King (1955) two mechanisms are involved in the swelling of soils.

1. A relaxation of effective compressive strength related to enlargement of capillary films, and
2. Osmotic imbibition of water by clay minerals with an expanding lattice.

The high swelling of the montmorillonite type clays is the result of the property of this mineral to adsorb water between the individual silicate layers. [10]

2.1.2 Identification and classification of Expansive soils.

A. **Field identification: -** Currently it is evident that soil deposits can be recognized in the field through visual inspection. Some of the important field identification method that indicates the potential for expansiveness of a soil is the following [7]:

- Polygonal pattern of surface cracks in the dry season. Crack 2.5 cm wide and over 1 m depth is not uncommon. The cracks close down after rainy season (Figure 2.5 a and b).
- A shiny surface is easily obtained when a partially dry piece of the soil is polished with a smooth object such as the top of a fingernail.
- The wet samples of the soil are sticky and it will be relatively difficult to clean the soil from the hands. They are very hard when dry.
- They usually have a color of black and/or gray
- As a rough guide, the presence of distinct cracks on light weight buildings.

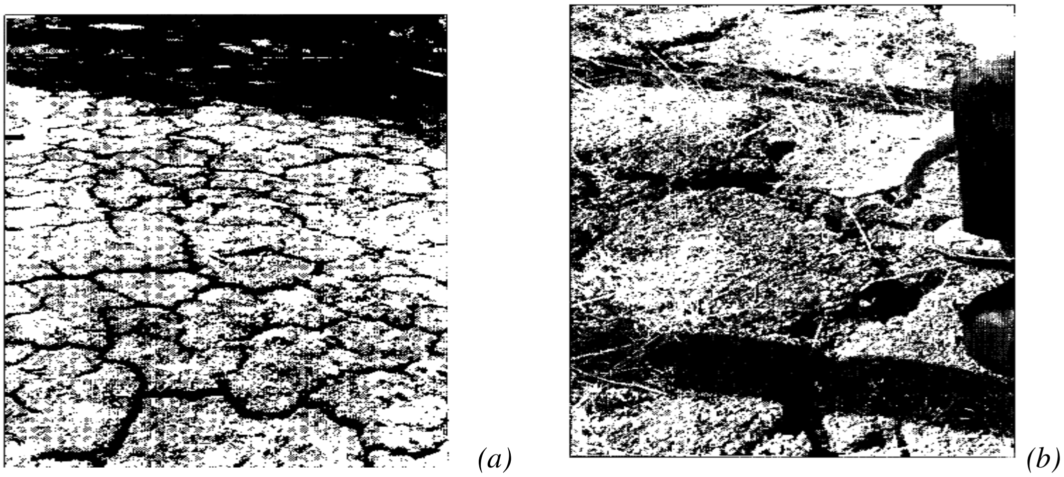

(a) *(b)*

Figure 2.5 surface cracks in dry season.(a) polygonal pattern of surface crack in dry season.(b)The crack is at least 32 inches(82cm) deep. Straight cracks may Extend much deeper. [9]

B. **Laboratory identifications**: - there are a number of laboratory tests that are useful in identifying expansive soils. Generally, these can be categorized as mineralogical identification, direct and indirect methods.

i. Mineralogical identification:- It is the nature of clay mineral in the soil, which is responsible for the swelling or shrinkage, and therefore, the methods that directly or indirectly enable to identify the types of minerals can be considered to be the most desirable ones. For this purpose, tests like X-Ray diffraction, electron microscope, differential thermal analysis, dye absorption, determination of silica-sesquioxide ratio and Base Exchange capacity, etc. are usually used to study the amount and type of clay minerals, by which the swelling characteristics of clay can be identified. These methods are mostly used for academic or research purposes. They are time consuming, require expensive test equipment, and the results are interpreted by specially trained technicians. Thus, for ordinary engineering works these methods are generally not often used.

ii. Direct methods: - They are the most accurate method and are preferably carried out on representative undisturbed samples .These methods are usually performed through actual measurement of swelling pressure and volume change of soil as well as free swell tests on disturbed samples. Swelling pressure is determined by measuring the pressure needed to prevent heaving of sample under the given condition of moisture, density and confinement. Swelling test provides complete swelling but due to varying initial conditions of moisture, density, etc, it is difficult to assess the swelling expected in the field. The methods provide quantitative information, which are very useful for design engineers.

iii. Indirect Methods:-These are usually the simplest methods, a practicing engineer resort to use for identifying expansive clay soils. They can be performed in average soil mechanics laboratory and yield excellent indices of expansive properties. Those tests comprised under these methods are Atterberg limits tests, and grain size distribution. Sometimes the use of Atterberg's limit in conjunction with grain size analysis enables the determination of clay activity which is defined as the ratio of plasticity index to the percentage of clay fraction(less than 2micron size) present in the sample. This ratio might also serve as useful characteristic for degree of expansiveness of different soils.

Expansive soils classification:-The key to all expansive soil classification systems is the method of measuring swell potential. Swell potential may be measured directly in swell test or indirectly determined by correlation with other test results of swell test data. In almost every case swell potential is evaluated in the laboratory in a consolidation test device. This may yield swell potential different from those for in-situ soils. Thus an accurate correlation between swell potential and other test results for a purpose of prediction of in-situ heave is difficult. These procedures, however, do provide good indicators of swell potential when the soil is subjected to the conditions used in the laboratory test.

i. ***Bureau of reclamation method:-***This method is based on direct correlation of observed volume change with colloid content, plastic index and shrinkage limit. The measured volume change is taken from odometer swell tests using 1 psi surcharge pressure from air dry to saturation. The degree of expansion and limits of correlated properties are shown in Table 2.2. Experience has shown that this method correlates reasonably well with expected behavior and provides a good indicator of potential volume change. **[7]**.

Table 2.2 Classification of expansive soils based on Bureau of Reclamation method [7]

Colloid content, %-1μm	***PI, %***	***SL, %***	***probable expansion %***	***Degree of expansion***
<15	*<18*	*>15*	*<10*	*low*
13-23	*15-28*	*10-16*	*10-20*	*Medium*
20-31	*25-41*	*7-12*	*20-30*	*High*
>28	*>35*	*<11*	*>30*	*Very high*

ii. Chen method:-*This method correlates between swell data (undisturbed sample with surcharge load) and percent passing the No 200 sieve, liquid limit, and standard penetration resistance. The resulting classification of the degree of expansion is as shown in Table 2.3. [7]*

Table 2.3 classification of expansive soils according to Chen

200 sieve, ***%***	***LL***	***SPT*** ***ows/ft***		
<30	<30	>10		
30-60	0-40	0-20		
60-90	0-60	0-30		
>95	>60	>30		

2.2 Atterberg limits

2.2.1 Historical background and basic principles

The Swedish soil scientist Albert Atterberg (1911) originally defined six ‘Limits of consistency’ to classify fine-grained soils, but in current engineering practice only three of the limits, i.e. liquid, plastic and shrinkage limits are used[12]. Atterberg was working in the ceramic industry, and at that time they had several simple tests to describe the plasticity of clay which were important both in molding clay into bricks, for example, and to avoid shrinkage and cracking when fired. After many experiments, Atterberg came to the realization that at least two parameters were required to define plasticity of clays, i.e. the upper limits and lower limits of plasticity. In fact, he was able to define several limits of consistency or behavior and he has developed simple laboratory tests to define these limits. They are:-

1. Upper limits of viscous flow.
2. Liquid limit – lower limit of viscous flow.
3. Sticky limit – clay loses its adhesion to a metal blade.
4. Cohesion limit – grains cease to cohere to each other.
5. Plastic limit – lower limit of the plastic state.
6. Shrinkage limit – lower limit of volume change.

He also defined the plasticity index, which is the range of water content where the soil is plastic, and he was the first to suggest it for soil classification. Later on in the late 1920's K.Terzaghi and A.Casagrande (1948), working for the U.S. Bureau of public Roads, standardized the Atterberg limits so that they could be readily used for soils classification purposes. In present geotechnical engineering practice we usually use the liquid limit (LL or ω_L), The plastic limit (PL or ω_p), and the shrinkage limit (SL or ω_s). The sticky and the cohesion limits are more useful in ceramics and agriculture. Since the Atterberg limits are water contents where the soil behavior changes, one can show these limits on a water content continuum as shown in Figure 2.6 .Also in the same figure shown are the types of soil behavior for the given ranges water contents and the generalized material response (stress – strain curves) corresponding to those states[11].

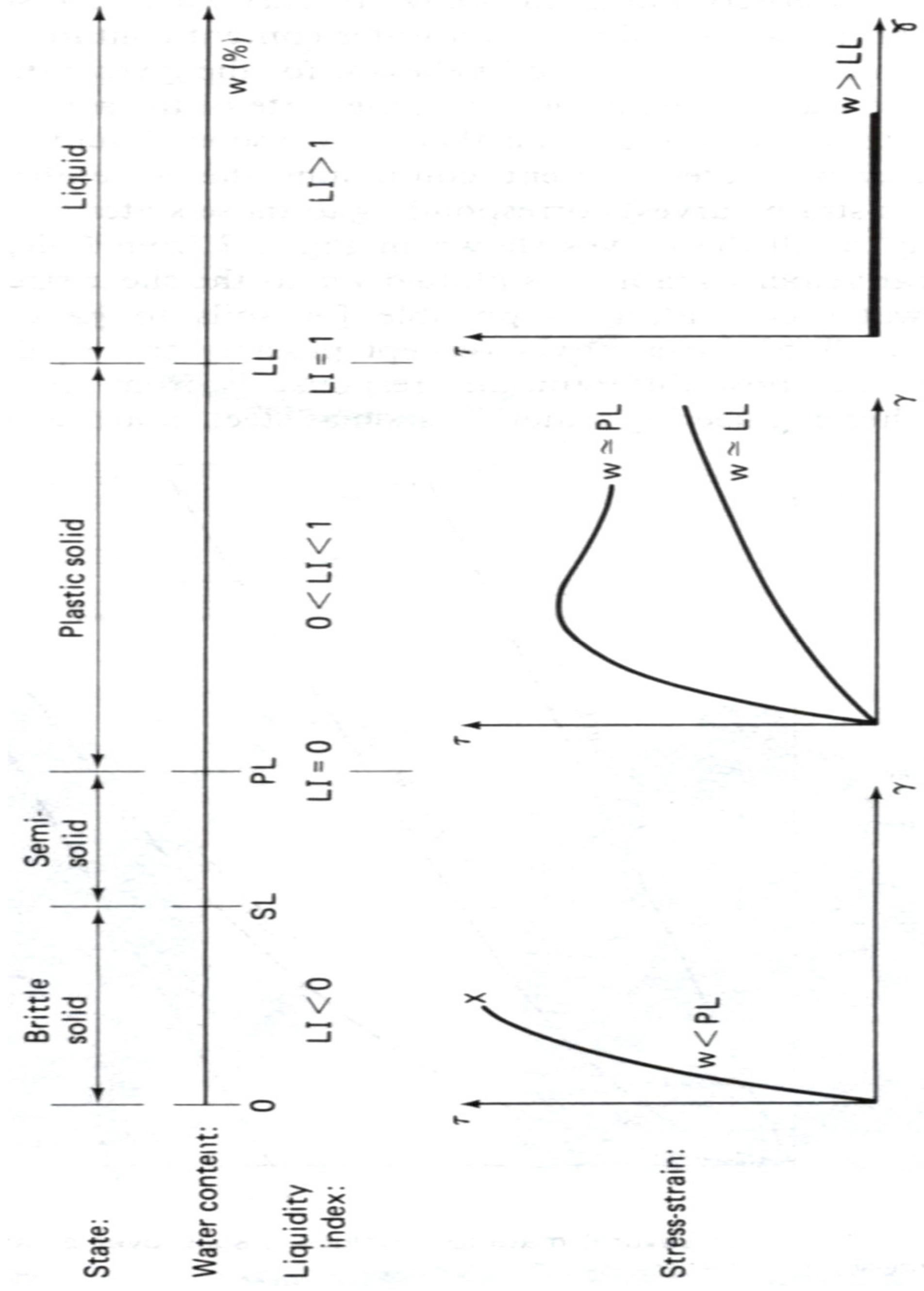


Figure 2.6 Water content continuum showing the various states of a soil as well as the generalized stress-strain response.[11]

2.2.2 Liquid limit

A fine grained soil can exist in any of several states; each state depends on the amount of water in the soil system. Liquid limit is the percentage of water content at which the soil has such small shear strength that it flows to close a grove of standard width when jarred in a specified manner. At the moisture content of LL the distance between particles is such that the force of interaction between particles is sufficiently weak to allow easy movement of particles relative to each other. It is dependent on the amount and type of clay, but the latter being very important [4].According to Skempton (1953) a clay – water system at the liquid limit has a shear resistance of the order of 0.1 psi $\cong 0.7 kpa$.

Figure 2.7 Equipment used for Liquid limit test

2.2.3 Plastic limit

Plastic limit is the water content at which the soil begins to crumble when rolled in to thread of specified size .At this limit sufficient water is required to wet all the surfaces and reduce cohesion so that the particles can move past one another under stress but maintain a new molded position.

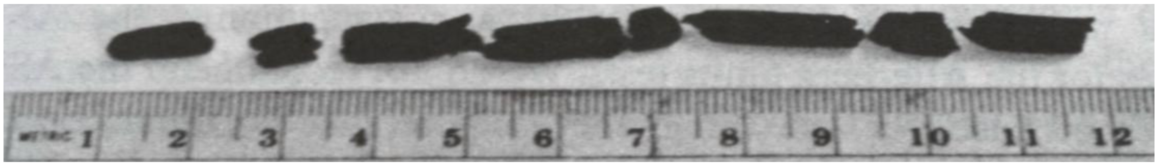


Figure 2.8 Rolled threads for plastic limit determination.

2.2.4 Shrinkage limit

Shrinkage limit is the maximum water content at which a reduction in water content will not cause decrease in the volume of the soil mass. If a mass of soft, saturated soil gradually dries, its volume is reduced by an amount equivalent to the volume of the water lost by evaporation. This volume change is caused by capillary forces acting on the surface of the soil mass. At a certain water content the sum of the capillary forces reaches its greatest value and the volume change ceases. On further drying the water begins to withdraw into the interior of the soil, whose color then changes from dark to light. The surface of the desiccating soil shows a characteristics pattern of shrinkage crack. The finer the particle of the soil, the greater is the amount of shrinkage. The water content below which further loss of water by drying does not result in reduction of the volume of the soil is called the shrinkage limit. Working on the above definition, the shrinkage limit is determined by completely drying out a lump of soil and measuring its final volume and weight. The volume of the oven-dried sample may be assumed to be equal to its volume at the shrinkage limit no appreciable volume change has taken place.

Since the degree of saturation at the shrinkage limit is unity (as it is throughout the process of description at water contents greater than ω_s, the shrinkage limit can be calculated from the minimum volume V_{min} and weight W_d of the oven dried sample

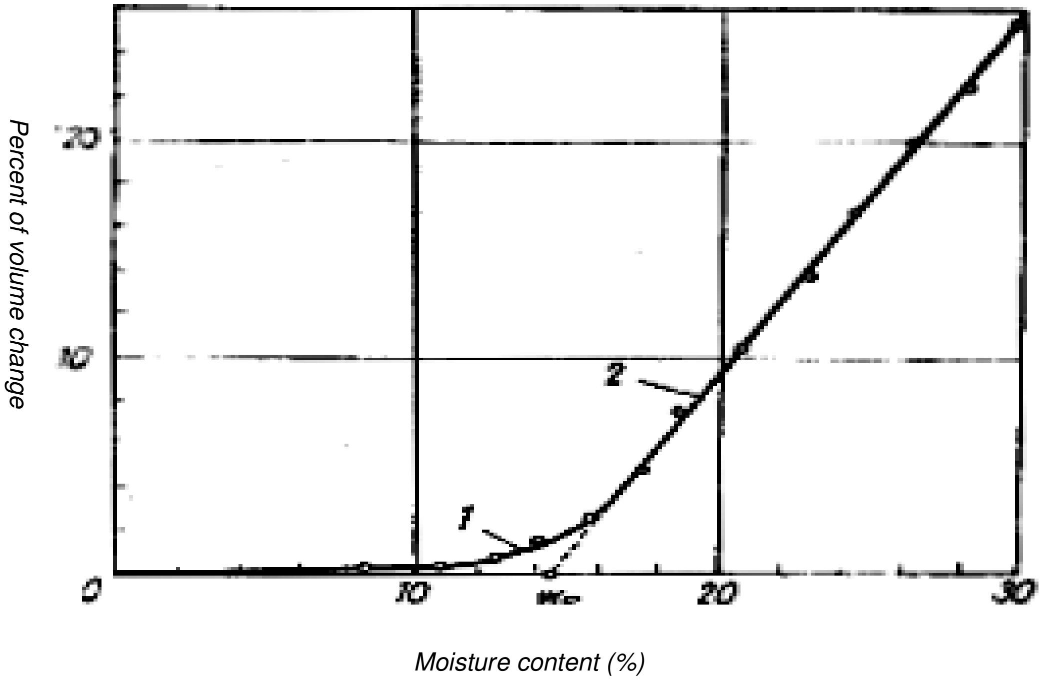


Figure 2.9 percent of volume change verses moisture content [2]*, 1.Shrinkage due to hydration forces. 2. Shrinkage due to capillary action*

A more accurate method of the determination of the shrinkage limit involves measuring the volume of the sample at various stages of the desiccation process and plotting the volume change $[V - V_{\min}]/[V_{\min}]$ in Percent against the water content a graph in Figure 2.9 shows the above result. At the shrinkage limit the curve shows a conspicuous/very noticeable break.But this not abruptly sharp, as some small volume change occurs even below the shrinkage limit due to hydration forces which act in the water film between the grains. The straight section of the curve is therefore extended to the $V= 0$ axis; the water content by the intercept on this axis is the shrinkage limit [2].

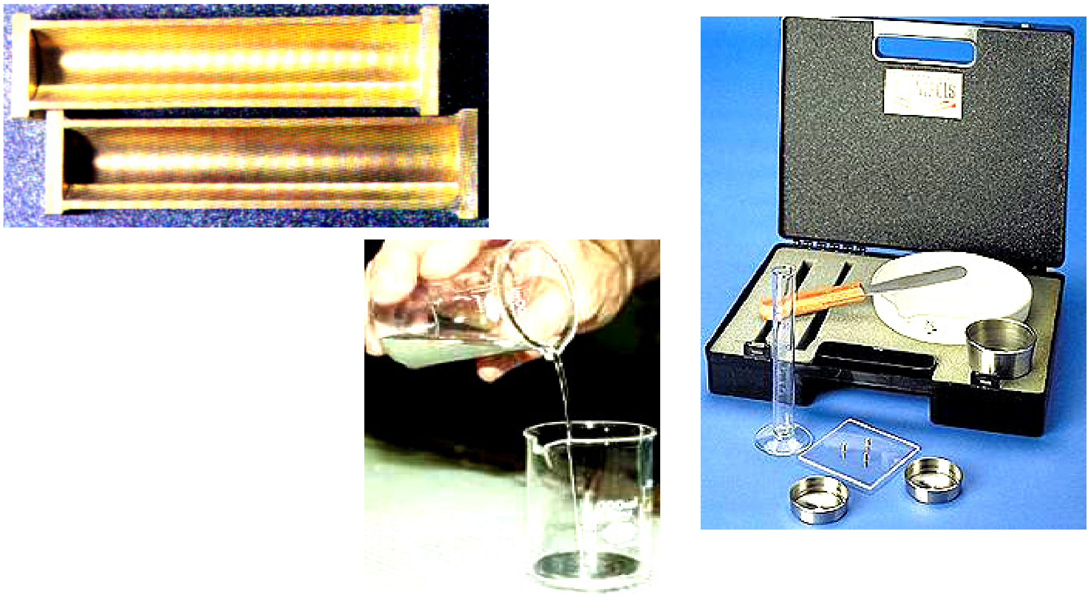


Figure 2.10 Linear and volumetric shrinkage limit test equipments.

2.2.5 The plasticity Index

The plasticity index is simply the numerical difference between the liquid limit and the plastic limit and indicates the magnitude of the range of moisture content over which the soil remains plastic.

$$PI = LL - PL$$

It is the measure of the cohesion qualities of the binder resulting from the clay content. Also it gives some indication of the amount of swelling and shrinkage that will result in the wetting and drying of that fraction tested. The plasticity index is nothing but a measure that gives the amount of water which must be added to change a soil from its plastic limit to its liquid limit.

Generally the behavior of all soils and specifically clays considerably differs with the presence of water so one needs a reference index to clarify the effects. A clayey soil depending on the water content may be almost like a liquid as shown in Figure 2.11 or it may be quite hard too.

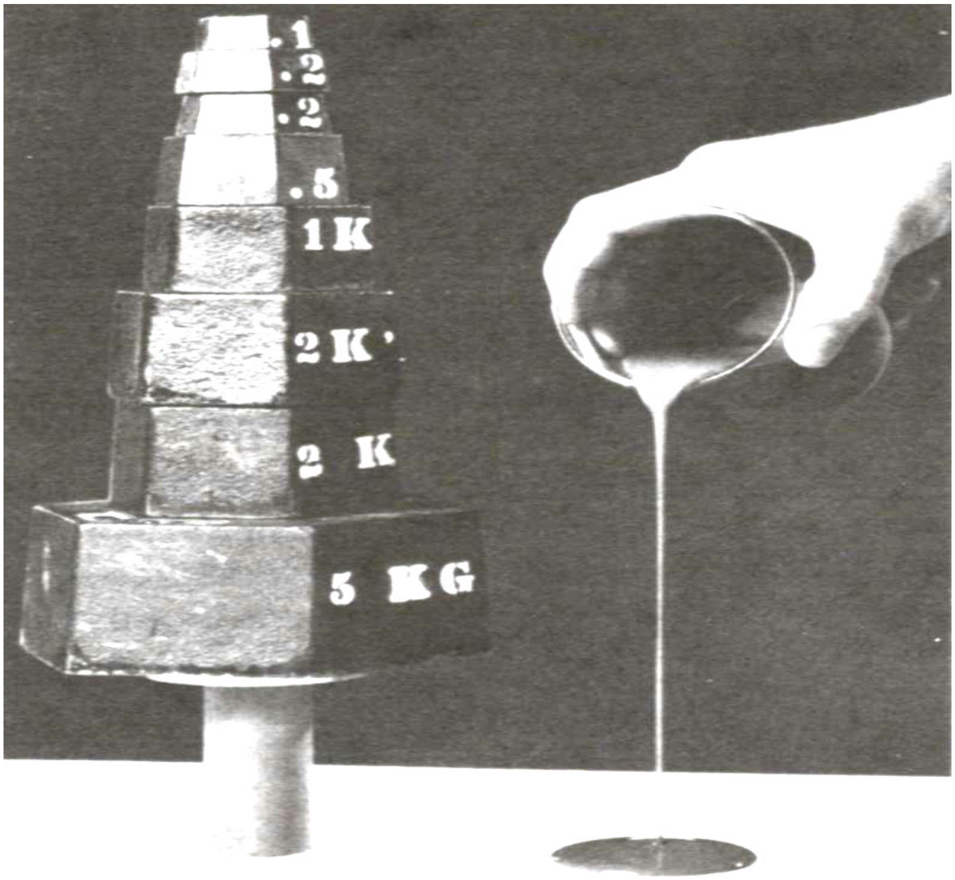
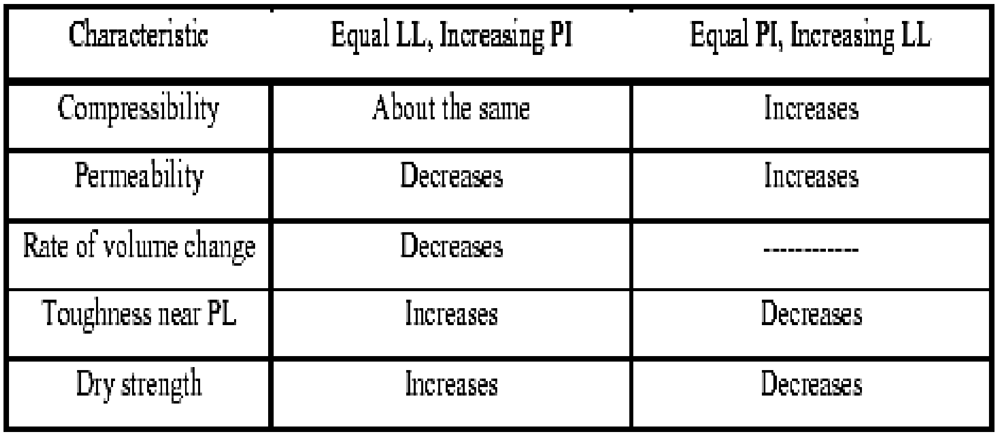


Figure 2.11 Undisturbed and thoroughly remolded samples of Leda clay from Ottawa, Ontario[11].

The chemical and mineral composition, size and shape of the soil, particles influence the adsorbed water films on the particles. Because such soil properties as compressibility, permeability, and strength, as well as the limits are dependent on the water films, approximate relationships exist between these properties and the limits. Some general relationships between the limits and engineering properties as given by Casagrande are listed in Table 2.4

Table 2.4 Approximate relation ships between soil properties and Atterberg limits [11]



Characteristic	Equal LL, Increasing PI	Equal PI, Increasing LL
Compressibility	About the same	Increases
Permeability	Decreases	Increases
Rate of volume change	Decreases	-----------
Toughness near PL	Increases	Decreases
Dry strength	Increases	Decreases

The Atterberg limits and related indices have proved to be very useful for soil identification and classification. The limits are often used directly in specifications for controlling soil quality for use in fill and in semi empirical methods of design [161Soils are classified by AASHTO and USCS based on their Atterberg limits and on particle size as determined by sieving.

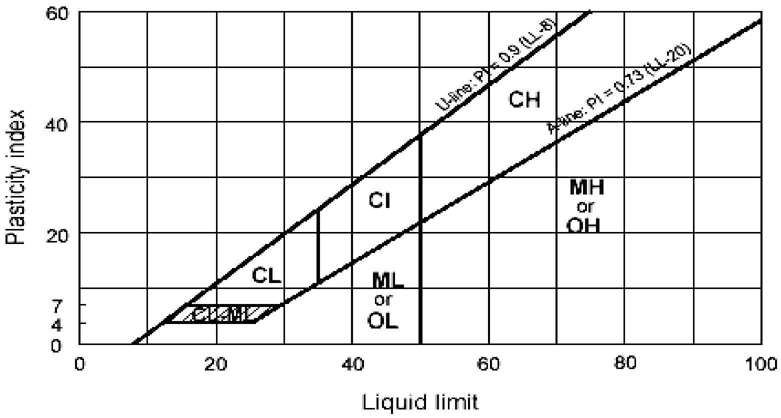


Figure 2.12 Casagrande's PI-LL chart [11]

Chapter Three

3.0 Laboratory tests results and discussions

3.1 Material tested

The soil profile of Addis Ababa varies from place to place .Generally, however, the soil profiles can be divided into two major groups, namely expansive and non expansive soils. For the laboratory works of this thesis, samples were taken from different sites in Addis Ababa where different investigators confirmed the presence of expansive soils. In addition to the confirmation of those investigations, the presence of distinct cracks on light weight buildings and surface cracks are used as an identification of the presence of expansive soils in the sampling field. For comparison purposes red clay soil of Addis Ababa were also sampled. Sampling area, depth, and types of sample are given in Table 3.1

3.2 Test procedures, results and discussions

In this research the standard laboratory tests were carried out in accordance with the ASTM and BS procedures for soil testing. For tests like measuring of the volume of the sample at various stages of the desiccation process, and rewetting of completely dry pat sample an attempt was made to develop test procedures that suit the goal of this research work. In order to carry out the above intended purpose the following laboratory tests were carried out.

- Free swell and linear shrinkage
- Atterberg limits
- Swelling measurement with and without seating load
- Artificial flooding of test pit and moisture content
- Measuring of volume at various stages of desiccation (Drying process)

Table 3.1 Samples description

Location	***Abbreviation***	***Test pit No***	***Depth of Sampling[m]***	***Color***	***Type of Sample***	***Test Conducted***
BOLE	B-1	1	1.0, 2.0, &2.5	Black	Disturbed	Atterberg limits, linear shrinkage, free swell, rewetting and drying
	B-2	2	1.0, 2.0, &2.5	Black & Gray	Disturbed	Atterberg limits, linear Shrinkage, free swell, rewetting and drying
	B-3	3	1.0, 2.0, &2.5	Black & Gray	Disturbed & Undisturbed	Atterberg limits, linear shrinkage, free swell, rewetting and drying ,swell with without seating load and the corresponding moisture content before and after the test, artificial flooding and moisture content and swell pressure with variable initial moisture content.
Lafto Lebu	L-1	1	1.5 & 2.8	Black	Disturbed & Undisturbed	Atterberg limit, linear shrinkage, free swell, shrinkage limit, rewetting and drying ,swell with without seating load and the corresponding moisture content before and after the test, moisture content of sample below water table, and swell pressure with variable initial moisture content.
	L-2	2	1.5 & 3.0	Black	Disturbed	Atterberg limit, linear shrinkage, free swell, shrinkage limit, rewetting and drying , moisture content of sample below water table
Asko	As	1	1.0 & 1.5	Red Clay	Disturbed	Atterberg limit, linear shrinkage, free swell, shrinkage limit, rewetting and drying
Addisu Gebeya	Ad	1	1.0 & 1.5	Red Clay	Disturbed	Atterberg limit, linear shrinkage, free swell, shrinkage limit, rewetting and drying

- Re wetting of the completely dry pat
- Swell pressure measurement by varying initial moisture content

3.2.1 Free swell and linear shrinkage

This test has not yet been standardized by ASTM and BS. The method was suggested by Holtz and Gibbs (1956) to measure the expansive potential of cohesive soils. The free swell test gives a fair approximation of the degree of expansiveness of the soil sample The procedure consists of pouring very slowly of 10 cubic centimeters of that part of the dry soil passing N<u>o</u> 40 sieve in to a 100 cubic centimeters graduated measuring cylinder and letting the content stand for approximately twenty four hours until all the soil completely settles on the bottom of the graduating cylinder. Then the final volume of the soil is noted.

$$Freeswell\ (\%) = \frac{Final\ Volume - Initial\ Volume}{Initial\ Volume} * 100\%$$

Linear shrinkage test follows a British standard BS 1377 1990 part 2, and covers the determination of total linear shrinkage from linear measurement on a standard bar of length 140 mm with a semi circular section of diameter 25 mm, the grove filled by a soil of the fraction passing 0.425 mm test sieve, originally having the moisture content of the liquid limit.

$$Linear\ Shrinkage\ (\%) = \frac{Initial\ Length - Oven\ dried\ Length}{Initial\ Length} * 100\%$$

The linear shrinkage value is the way of quantifying the amount of shrinkage likely to be experienced by clayey material. The results of free swell and linear shrinkage tests are tabulated in Table 3.2.

Table 3.2 Free swell and linear shrinkage results of samples

Location	Test pit no	Depth Sample [m]	Linear Shrinkage[%]	Free Swell [%]
BOLE	1	1.0	21	97
		2.5	26	92
	2	1.0	24	95
		2.5	25	90
	3	1.0	21	100
		2.5	21	105
LAFTO LEBU	1	1.5	20	97
		2.8	21	105
	2	1.5	18	98
		3.0	22	90
ASKO	1	1.0	5	35
		1.5	3	45
ADDISU GEBEYA	1	1.0	5	35
		1.5	4	40

From Table 3.2 one can see that the free swell of the soil from Bole and Lafto Lebu sites ranges from 90% to 110% and soil samples from Asko and Addisu Gebeya area (Red Clay soils) have a free swell value ranges 35-40 %. According to Holtz and Gibbs (1956) a free swell value above 50% are potentially expansive soils. Therefore samples from Bole and Lebu sites are potentially Expansive. On the other hand, samples from Asko and Adissu gebeya are (red clay soils) not potentially expansive. The linear shrinkage values are also in agreement with the above since the value of 8 % is the critical degree of expansion. The discrepancy showed in linear shrinkage and free swell is caused by the measurement according to BS standard.

3.2.2 Atterberg Limits

When clays are remolded over a wide range of water content the consistency can vary to a great extent. This behavior can be observed from the performance during the Atterberg limits determination.

Following the ASTM procedure, designation D 4318-98 and D 427-98, the soil samples obtained from different test pits were subjected to varying water contents and as a result the liquid limit, plastic and shrinkage limits of the sample as recorded in Table 3.3 were determined.

Table 3.3 Atterberg limit test results of samples

Location	Test pit no	Depth Sample [m]	Liquid Limit [LL] (%)	Plastic Limit [PL] (%)	Plasticity Index[PI] (%)	Shrinkage Limit[SL] (%)
BOLE	1	1.0	96	36	60	12
		2.0	103	34	69	11
		2.5	97	36	61	12
	2	1.0	107	33	74	14
		2.0	97	37	60	12
		2.5	105	37	68	13
	3	1.0	94	29	65	13
		2.0	110	40	70	13
		2.5	100	35	65	13
LAFTO LEBU	1	1.5	98	36	62	13
		2.8	100	36	64	14
	2	1.5	102	39	63	12
		3.0	90	30	60	11

Location	Test pit no	Depth Sample [m]	Liquid Limit [LL] (%)	Plastic Limit [PL] (%)	Plasticity Index[PI] (%)	Shrinkage Limit[SL] (%)
ASKO	1	1.0	58	30	28	20
		1.5	60	28	32	22
ADDISU GEBEYA	1	1.0	65	25	40	19
		1.5	70	30	40	20

The data in Table 3.3 show the high liquid limits for expansive soils that range from 90 to 110%. On the other hand, the liquid limit values for red clay soils are 58 to 70 %. The plastic limits for expansive soils vary from29 to 39. The data also show 28 to 30 for Red clay soils. The shrinkage values indicate that 11 to 14 and 19 to 22 for expansive soils and red clay soils respectively.

Plasticity index represents the range in water content through which a soil is in plastic state. A high numerical value of plasticity index is an indication of the presence of high percentage of clay in the soil sample. Information regarding the nature of clay in the sample, however, may be obtained by considering the plasticity index in relation to the liquid limit. This is best done by means of a plasticity chart shown on Figure 3.1.

The empirical boundary designated by A-line on the chart separates inorganic clays from silts and organic clays. As seen from the chart both soil groups under investigation (Black and Red clay) fall in the region of inorganic clays with high plasticity.

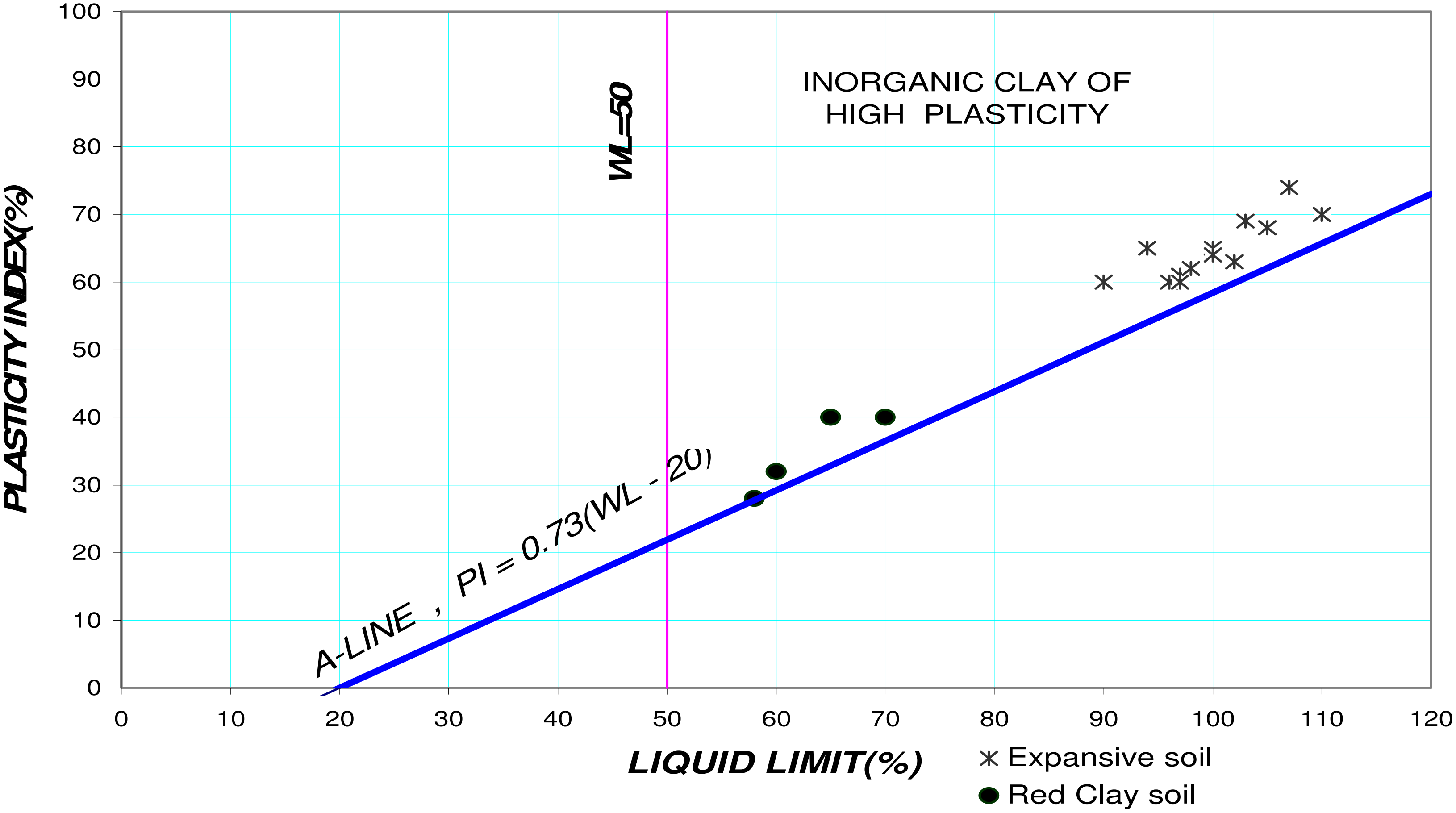


Figure 3.1 Plasticity Chart

3.2.3 Swelling measurement with and without seating load and moisture content.

Soils containing montmorillonite are likely to have significant potential for swell. The available techniques for quantitative measurement of swelling potential of expansive soils can be categorized into three groups. They are oedometer, soil suction test, and empirical methods. Among these techniques the odometer test is used in this research. The odometer test allows moisture and volume change in one dimension only, whereas in actual situation the above changes take place in all dimensions. For the intended purpose, swell potential is tested by following ASTM test method D 4546-96. This test method is used to measure a free swell and swelling pressure under a seating load of 7 kPa using consolidometer ring. In short it can be referred to as vertical strain under a surcharge that the sample undergoes following exposure to water.

Swelling without seating load test follows, the same procedure except the seating load is removed. For this a little modification is made on the conventional consolidometer apparatus as show in Figures 3.2 and 3.3.

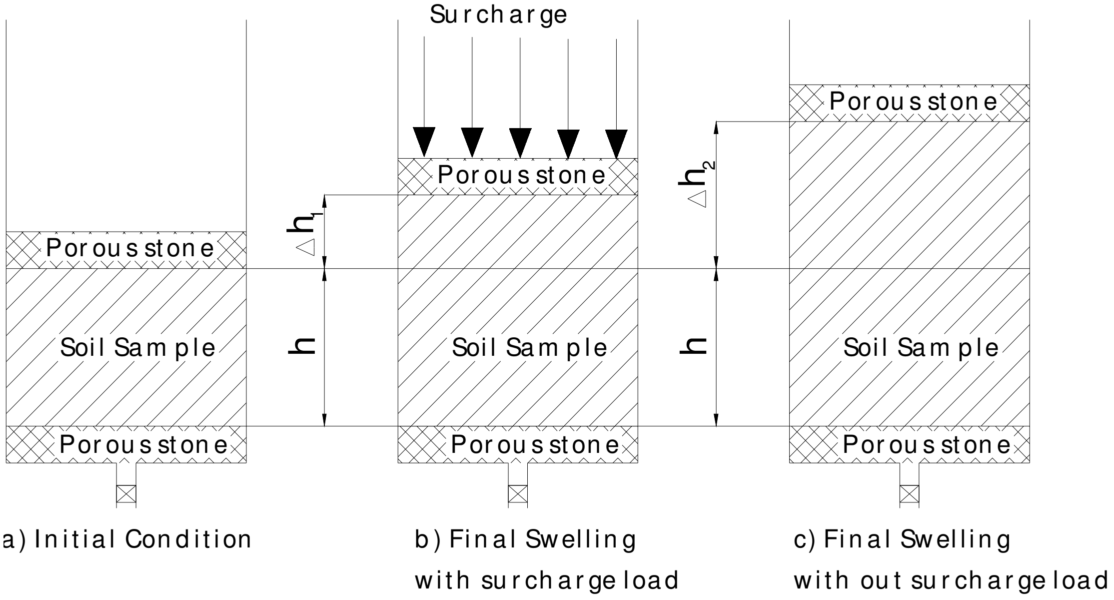
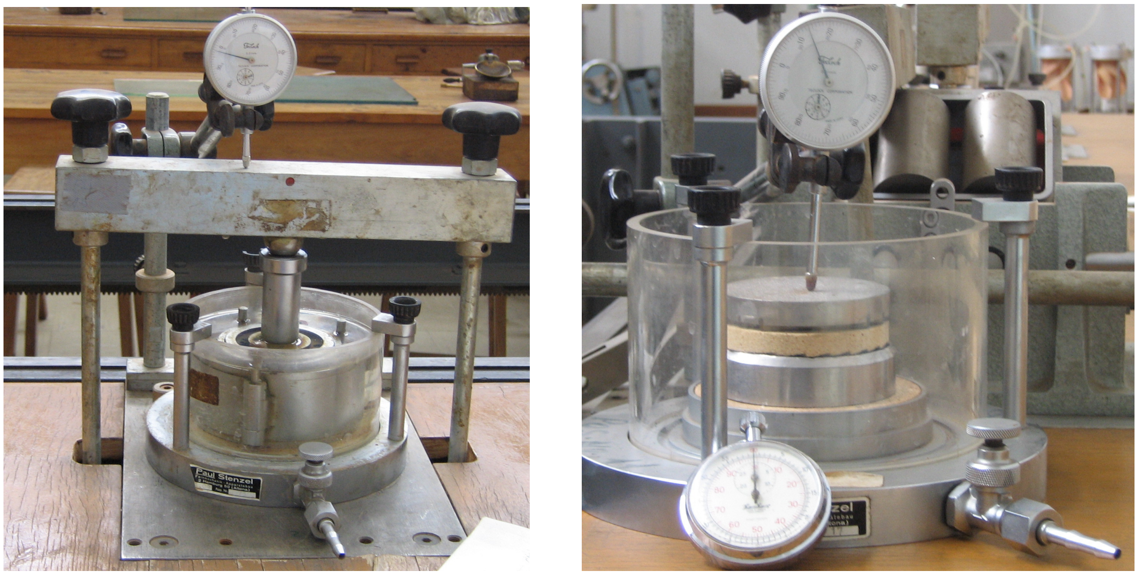


Figure 3.2 Diagrammatic description of swelling potential determination.

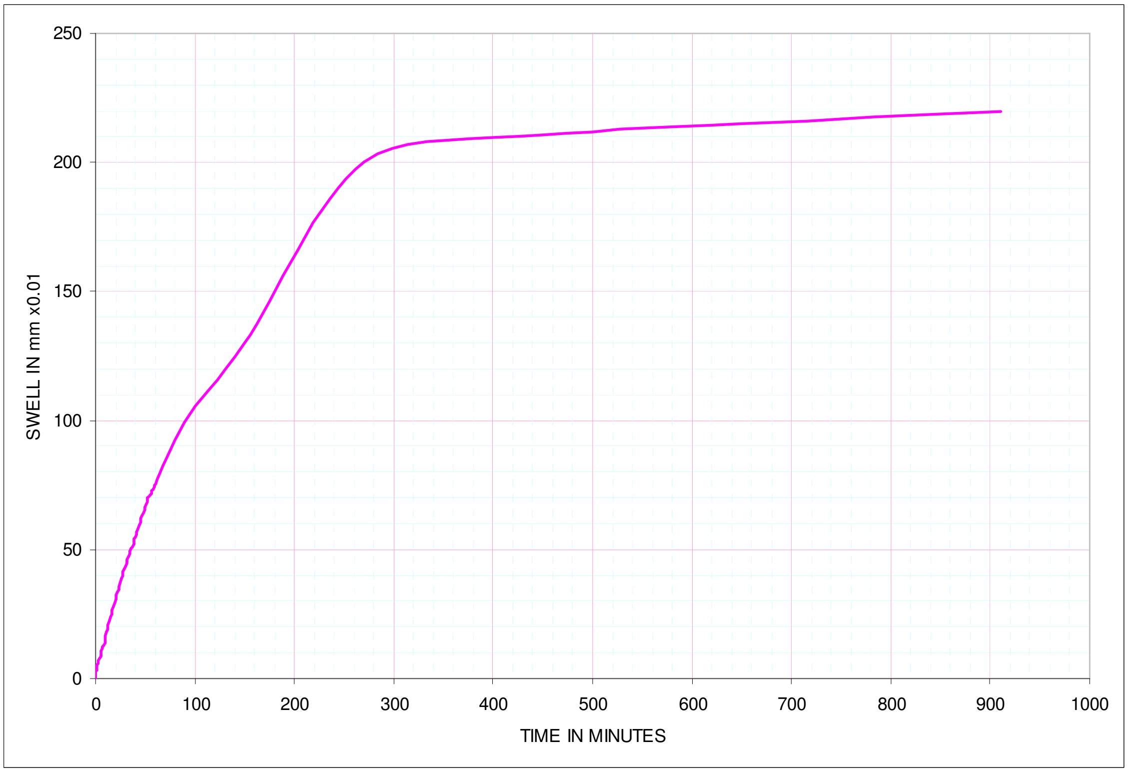
The soil sample extruded from sampling tube was placed inside a metal ring, with porous stones at the top and bottom. The specimen was kept under water throughout the test. The swelling was measured using a micrometer dial gage and reading was taken at a time interval of 24 hours. Figure 3.4 shows the plot of swell as a function of time.

In order to know the amount of moisture needed for the complete development of swell, the initial natural moisture content and the final moisture content are determined. Like the preceding tests this test was also done in accordance with the ASTM procedure, Designation D 2216-92.The results are tabulated in Table 3.4.representative plots of swell against time are also presented in Figure 3.4.

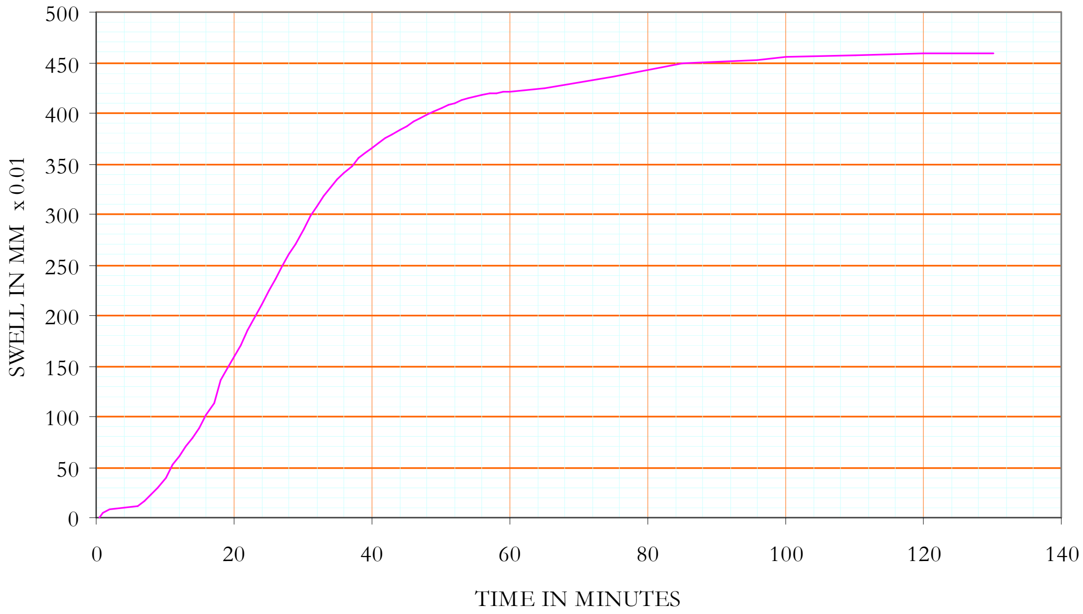


a) *Conventional consolidometer with loading b*ar b) c*onsolidometer without loading bar*

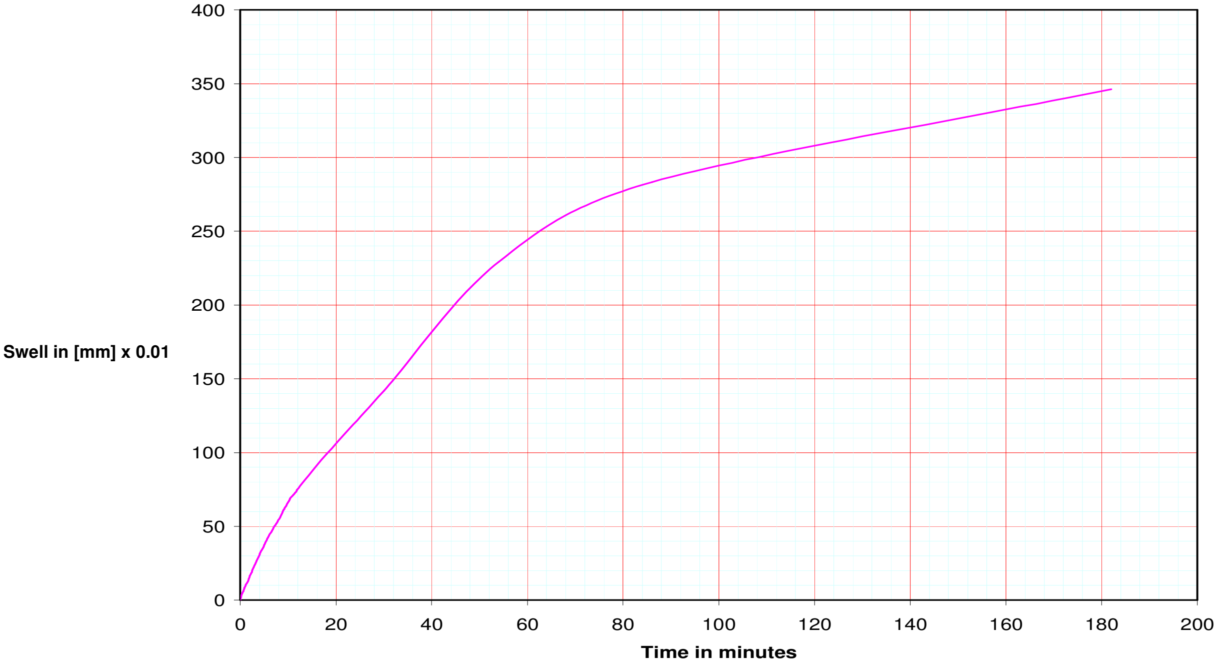
Figure 3.3 Conventional and modified consolidometer arrangement



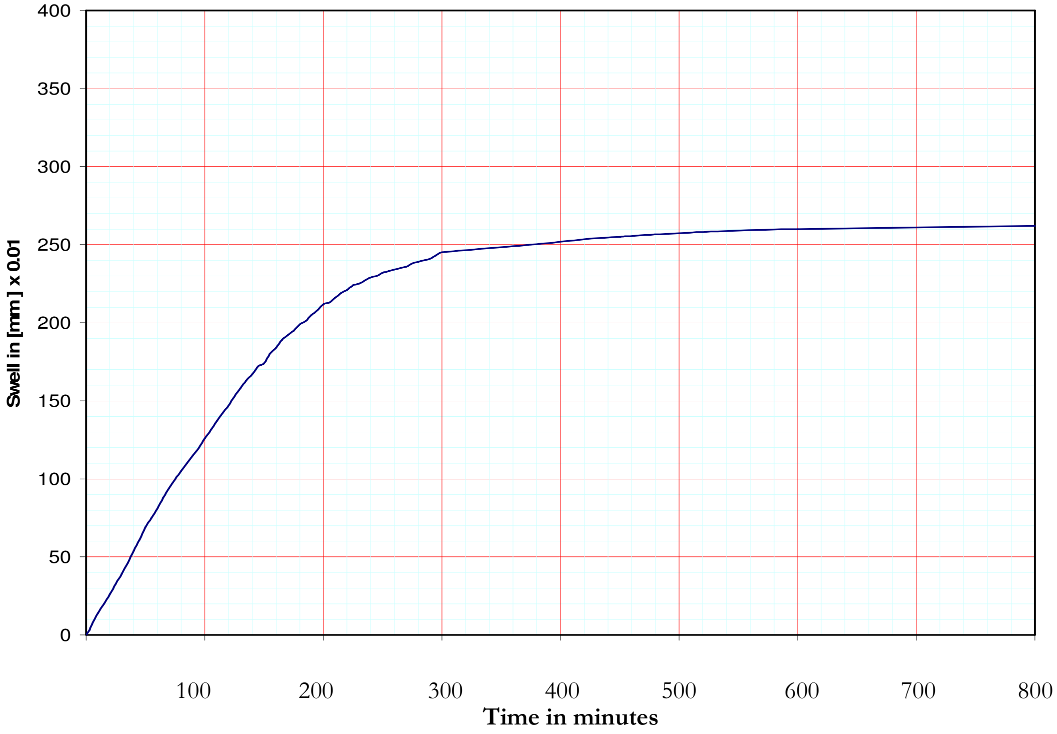
a) A plot of swell against time for Bole site with seating load



b) A plot of swell against time for Bole site without seating load



c) A plot of swell against time for Lebu site without seating load



d) A plot of swell against time for Lebu site with seating load

Figure 3.4 Representative plots of swell against time

Table 3.4 Moisture content before and after swelling

Location	Depth Sample [m]	Natural moisture content[%]	Moisture content after swell [%] Without seating load	Moisture content after swell [%] With seating load	Liquid limit [%]	Plasticity Index [%]	Plastic limit [%]
BOLE 3 [B-3]	1.0	30.2	61.08	51.66	94	65	29
	2.0	34.68	61.42	51.24	110	70	40
	2.5	36.1	66.57	57.05	100	65	35
LAFTO LEBU [L-1]	1.5	63.89	77.54	68.31	98	62	36
	2.8	55.83	70.61	62.38	100	64	36

From Figure 3.4, one can observe that samples with seating load more than 90 % of maximum swelling carried out from 5 to 8 hours. The sample without seating load the same percentage of swell recorded from 50 to 60 minutes. The test result on Table 3.4 also indicate that all samples exhibit moisture content in range of liquid limit and plastic limit after the swelling.

3.2.4 Artificial flooding of test pit and samples below water table

It is known that due to the presence of montmorillonite mineral in expansive soils, water holding capacity of such soils believed to be high. In order to compare this water absorbing capacity with that of the Atterberg limit two approaches are used. The first approach is by artificially flooding a test pit in the Bole area (Figure 3.5) for about a week and undisturbed soil samples are collected and moisture content test was conducted in accordance to ASTM procedure, Designation D 2216-92.The test pit was opened at dry season. The second approach is the sample collected from Lafto Lebu [L-1] site below water table. Purched water at Lafto Lebu was located at a depth of 1.5-2.0 meter from natural ground level [NGL].

Undisturbed block samples were taken for laboratory moisture content determination. The laboratory test data are summarized in Tables 3.5 and 3.6

Figure 3.5 Artificial flooding of BOLE -3

Table 3.5 Moisture content before and after artificial flooding

Location	Depth Sample[Side]	Moisture content before flooding [%]	Moisture content after Artificial flooding	Additional moisture content	Liquid limit [%]	Plasticity index [%]	Plastic limit [%]
BOLE 3 [B-3]	2.0 [L]	34.68	71.48	36.8	94	65	29
	2.0[R]	34.68	71.09	36.41	110	70	40
	1.5	-	72.53	-	100	65	35

Table 3.6 Moisture content below water table

Location	Depth Sample [m]	Natural moisture content[%]	Liquid limit [%]	Plasticity index [%]	Plastic limit [%]
LAFTO LEBU [L-1]	1.5	63.89	98	62	36
	2.8	55.83	100	64	36

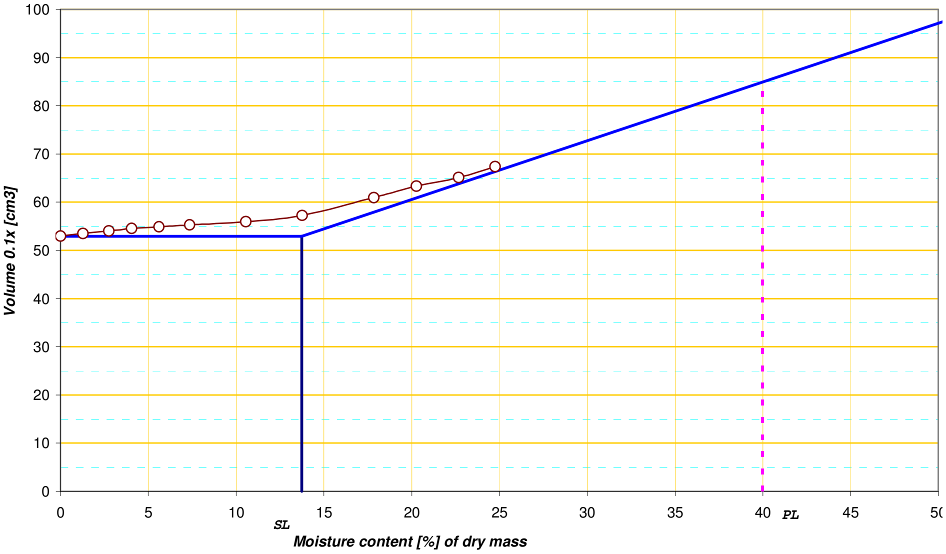
From Table 3.5 one can see that the additional moisture content is about 36 %. All the results show that the moisture content after artificial flooding and samples which are collected below water table are at the range of liquid limit and plastic limit.

3.2.5 Measuring of volume and moisture content at various stages of desiccation (Drying process) of shrinkage limit

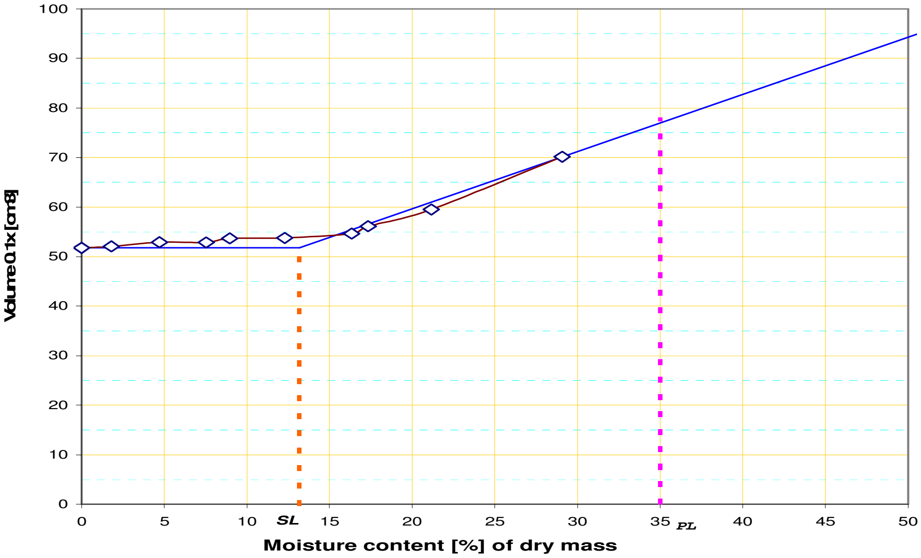
This test carried out to determine the actual pattern and relation moisture content and volume of both samples. There is no standard test procedure for the determination the shrinkage limit involving measuring the volume of the sample at various stages of the desiccation process. Here two attempts were done. For both attempts the standard test method of ASTM designation D 425-98 up to procedure number 9.5 are used with out modification.

The first modified method is used to measure volume and moisture content by fixing the oven temperature to 110±5^{0}C and data is taken with a constant time interval. The draw back of this method is that the cooling time of the sample takes relatively longer time in the dessicators.

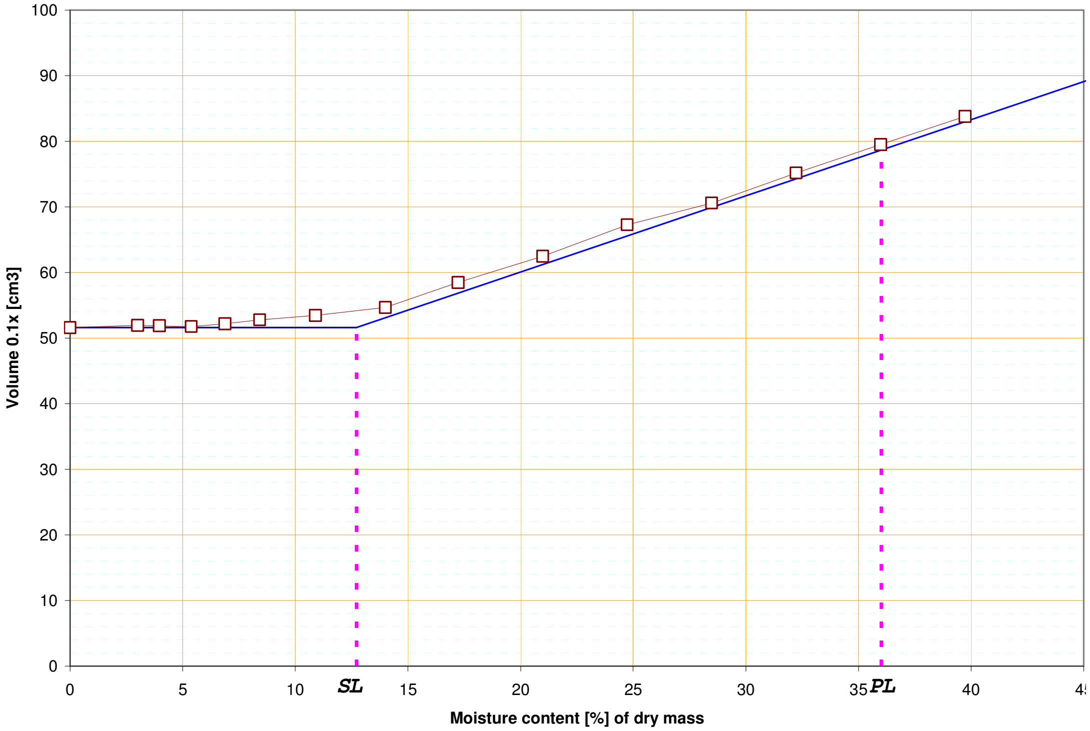
The second method involves varying the temperature at an interval of 20^{0}C and data is taken with a constant time interval. Both methods were carried out in parallel to the standard method of ASTM to check their applicability. The final results are the same as that of the standard methods. The results are plotted as shown in Figure 3.6



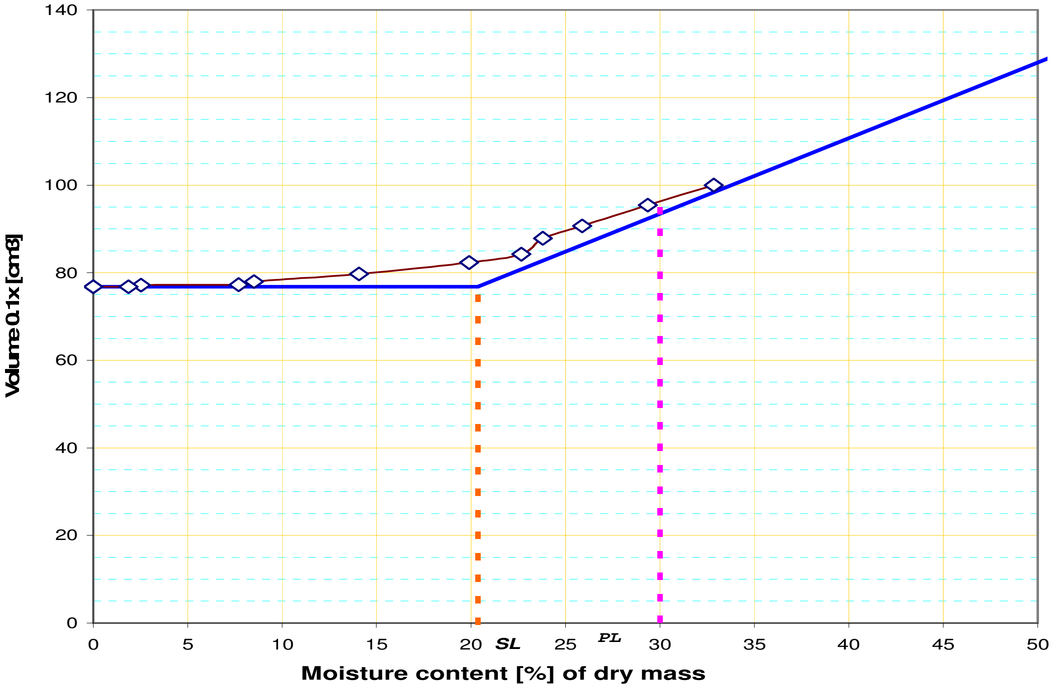
a) Bole 2 a plot of moisture versus volume



b) Bole 3 a plot of moisture versus volume



c) Lafto Lebu 1 a plot of moisture versus volume



d) Red clay soil a plot of moisture versus volume

Figure 3.6 Representative plots of moisture versus volume at different stages of shrinkage.

Figure 3.6 shows a similar pattern of volume change with moisture for both expansive and red clay soil samples.

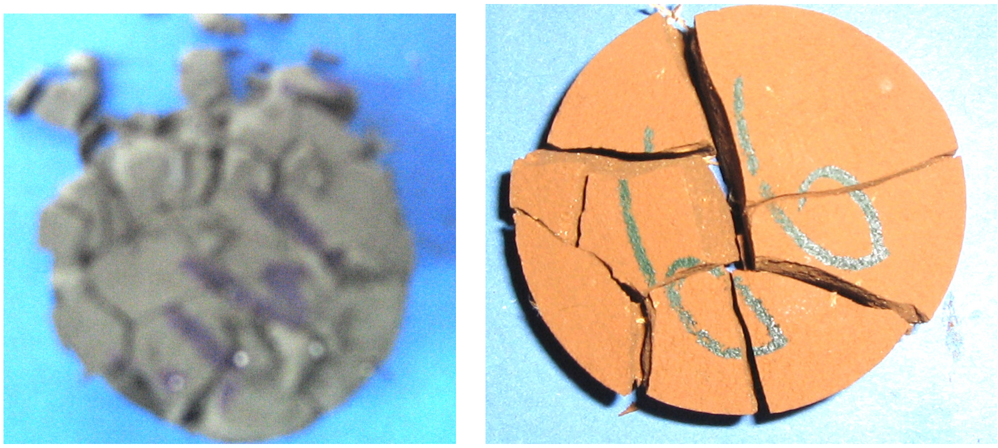
3.2.6 Rewetting of completely dry pat

The soil samples with moisture content below shrinkage limit expected to have a more or less constant volume. But previous research results disclosed that a complete saturation is not necessary to accomplish swelling. Slight change in moisture content in the magnitude of 1 to 2 % is sufficient to cause detrimental swelling. Whereas the shrinkage limit values of the soil under investigation ranges between 11 to 14 %. In this test an attempt was done to measure the volume of sample by rewetting with a moisture content below shrinkage limit.

There is no standard method of rewetting the dry pat used for the determination of shrinkage limit. Different attempts were done to have an efficient moisture transfer to the sample with out affecting the physical shape of the sample.
The procedure of the first trial is as follows

1 shrinkage limit of the soil sample is determined in accordance to ASTM test procedure
2 The amount of water required to fill the voids of a given sample is calculated
3 25%, 50%, 75% and 100% of the water calculated above injected on to the sample by using medical syringe and left for about 24 hours in dessicators for a uniform migration of moisture.

After 24 hours, almost all the samples were cracked and broken and the measurement of volume becomes difficult. The typical sample failure is as shown in the Figure 3.7.



a) Black cotton soil b) Red clay soil

Figure 3.7 Typical failures of samples in the first trial

Problems encountered during the first trial

- inability to measure water precisely
- failure of the sample after re wetting

Solution given

- ✓ Increase the size of the shrinkage dish from 15-16 cm^3 to 44-46 cm^3 and replace water measuring syringe by a precise one (precision of 0.01cm^3)
- ✓ To avoid the breakage of the sample reinforcement thread is used and the effect on the shrinkage limit also considered.

The second trial is carried out by incorporating the above solutions and all the procedures are the same except that in every layer of the wetted sample reinforcement thread introduced as shown in Figure 3.8.The weight of the reinforcing thread is so insignificant. The total weight for ten samples was 0.162g that is approximately 0.0162g for each sample. In order to see the effect of the thread introduction on the result of shrinkage limit, the test conducted for the same soil samples with and without thread. The recorded result show that no difference.

Problems encountered during the second trial

- The mode failure of the sample changed i.e. bulged and cracks at the top of the sample seen, but still there is a difficulty of measuring the volume of the sample.(Figure3.8)

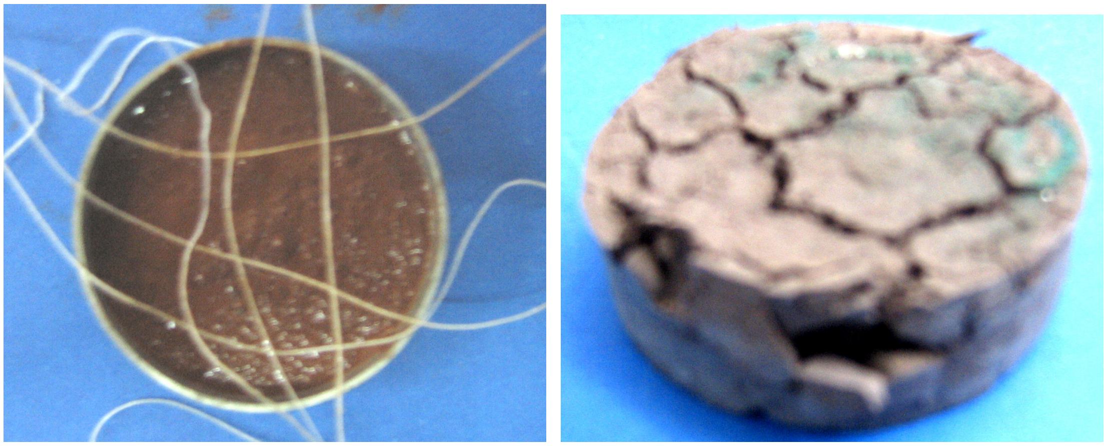


Figure 3.8 Procedure and failure of samples in the second trial

The 3rd and final trial provides a solution for the above problems by changing moisture transfer mechanism from injection to vaporizing the water. The mechanism of moisture transfer attained by diffusion, that the water vapor inters into the dry pat in all directions.

The shrinkage limit test procedure follows the standard ASTM D 427-93 except that the introduction of reinforcing threads. The main limitation of this test method is the amount of additional moisture content is not controlled.

Apparatus needed

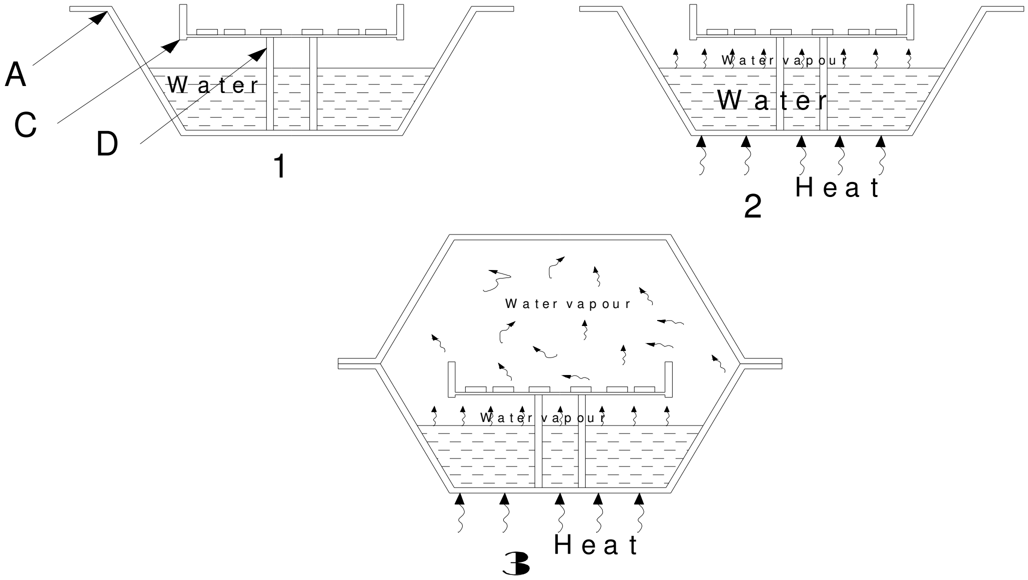
A. Shallow pan a diameter greater than that of standard sieve and the depth of the pan to hold water for boiling
B. Hot plate(stove) a capacity of 1000 W
C. Sieve opening 0.5 to 0.6 mm
D. Metallic circular stand Figure 3.9 gives diagrammatic representation.

Note: - all apparatus used to determine standard shrinkage limit test procedure of ASTM also used in this test method

Test procedure

1. Place the dry soil pat from shrinkage limit test in the sieve. The sieve also placed on metallic stand. The arrangement is shown in Figure 3.9
2. Turn on the hot plate after the placement of the sample and keep it from 20-30 minute or until the hissing sound starts which is the indication of boiling of the water
3. Cover the shallow pan with the same type of pan to confine the vapor and to accelerate the moisture transfer. This procedure may take from 1 hr to 1.5 hr to complete
4. Allow the soil pat to cool in dessicators for about half an hour.
5. Measure and record the weight and volume of partially wetted sample by using a balance and mercury displacement method.
6. Determine and record the change in moisture content and volume change if any.

Note: - All procedures used to determine standard shrinkage limit test procedure of ASTM also used in this test method including moisture content determination and volume measurement.



Letter designations refer to Apparatus used and numbers procedure.

Figure 3.9 Diagrammatic description Apparatus and test procedure of vaporized moisture transfer

Calculation

Volume change

$$\Delta V = \frac{V_f - V_i}{V_i} \times 100\%$$

Additional moisture content

$$\omega_A = W_D - W_W$$

Percentage of Additional moisture content with that of shrinkage limit

$$\omega_{SA} = \frac{\omega_A}{\omega_{gs}} \times 100$$

Percentage of Additional moisture content with that of dry mass

$$\omega_{DM} = \frac{W_W - W_D}{W_D} \times 100$$

Where:

ΔV = Change in volume [%]

V_f = Final volume after re wetting [cm^3]

V_i = Initial volume, volume of dry pat [cm^3]

ω_A = Additional moisture content [g]

W_D = Mass of dry soil [g]

W_W = Mass of wet soil [g]

ω_{SA} = Additional moisture content with that of shrinkage limit [%]

ω_f = Final moisture content [g]

ω_{gs} = Moisture content for fully saturation [g]

ω_{DM} = Additional moisture content with that of dry mass [%]

Using the above equations, the additional moisture absorbed and the corresponding volume change of the soil under investigation is calculated and the result are tabulated in Table 3.7 (Detail data are shown in Appendix)

Table 3.7 Test result of moisture variation and the corresponding volume change

Sampling Location	***Depth of Sampling[m]***	***Shrinkage Limit***	***Volume of wet pat[cm3]***	***Additional moisture[g]***	***Moisture content % of shrinkage limit***	***Moisture content % of dry mass***	***Change in volume [%]***
BOLE-1	1.0	12.00	17.12	2.91	77.90	9.35	8.81
		12.30	17.05	2.88	74.99	9.21	7.84
		12.73	16.68	3.01	77.06	9.82	7.00
	2.5	11.73	16.16	2.68	75.90	8.90	5.17
		12.44	16.61	2.89	75.72	9.42	7.54
		11.57	5.66	0.93	74.6	8.61	8.97
BOLE -2	1.0	14.20	17.64	2.96	65.03	9.23	7.57
		14.29	17.56	2.90	64.41	9.21	8.58
		14.68	17.19	3.13	68.53	10.06	7.28
		14.44	16.83	2.79	62.05	8.96	4.97
	2.0	13.13	17.27	3.57	88.66	11.64	9.24
		12.97	17.19	2.90	72.55	9.41	5.34
		13.32	17.05	3.00	71.15	9.47	4.44
Lafto Lebu-1	2.8	14.13	6.36	1.78	50.89	7.19	4.16
		15.06	6.34	0.94	58.11	8.75	6.42
		13.89	6.25	1.73	49.25	6.84	4.96
Lafto Lebu-1	3.0	10.60	15.42	2.20	72.36	7.66	9.42
		10.42	16.61	2.59	78.99	8.23	9.76
		11.79	16.46	2.10	56.14	6.62	8.78

Table 3.7 Cont'd

Sampling Location	*Depth of Sampling[m]*	*Shrinkage Limit*	*Volume of wet pat[cm3]*	*Additional moisture[g]*	*Moisture content % of shrinkage limit*	*Moisture content % of dry mass*	*Change in volume [%]*
ASKO	1.0	19.72	5.74	1.52	25.70	5.07	0.78
		20.67	5.57	1.41	19.65	4.06	0.55
		19.27	6.06	1.67	32.25	6.21	1.11
	1.5	22.13	24.28	2.99	32.16	7.12	0.92
		21.65	23.62	1.87	20.50	4.44	1.27
		19.86	22.88	1.99	24.56	4.88	0.98

From Table 3.7 one can see that for the moisture content of 6.62 to 11.64 % by dry mass of the expansive soil samples exhibits 4.16 to 9.76 % of volume change. On the other hand, the change in moisture content of 5.07 to 7.12 % of red clay soil shows, 0.55 to 1.27 % of volume changes. For comparison the variation plotted in Figure 3.10 as shown below. From Figure 3.10 the moisture absorption capacity below shrinkage limit of Red clay soil under the same condition with that of Black cotton soil is very small.

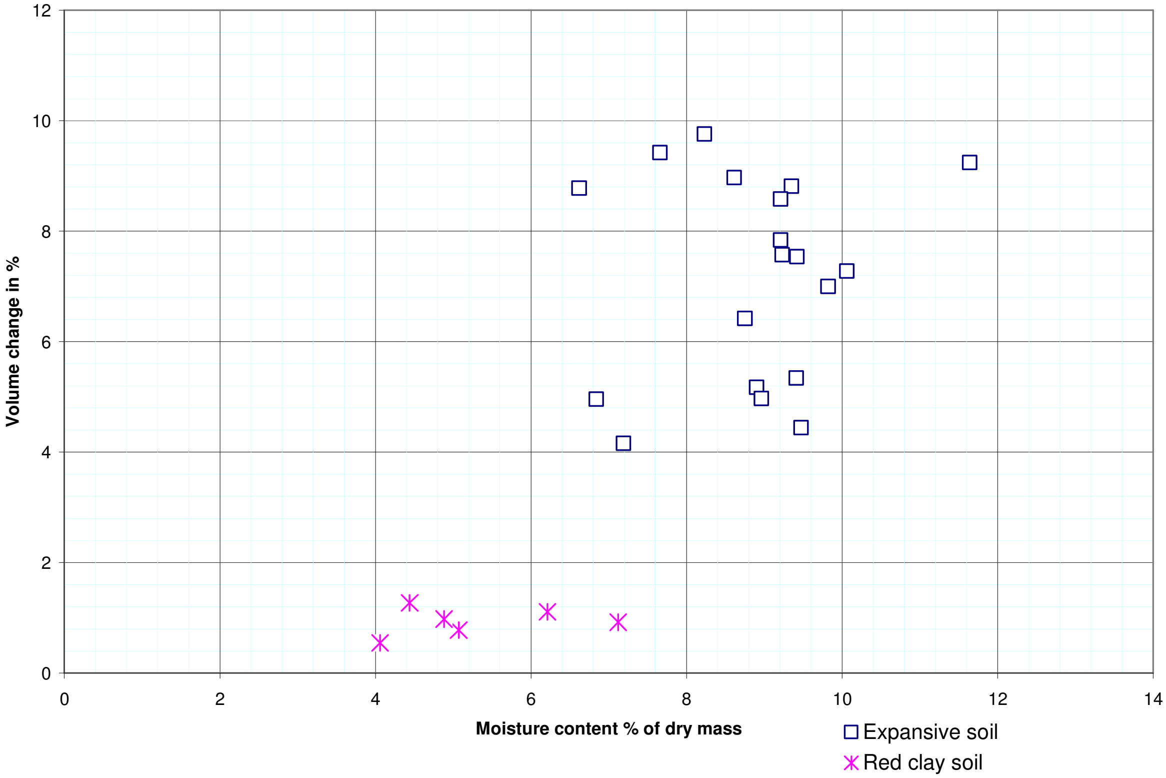


Figure 3.10 Moisture content Vs Volume change.

3.2.7 Measurement of swelling pressure by varying initial moisture content

Expansive soils are not subjected to volume change unless there is a change in moisture content. In this investigation a series of swelling tests were conducted to determine the effect of initial moisture content on the swelling property of an expansive soil found in Bole and Lebu sites. In order to simulate the actual construction site conditions, undisturbed soil samples were collected and the initial moisture content varied in the laboratory. Swell- Consolidation testing method following ASTM D4546 procedure was adopted for measuring the amount of swelling pressure using one dimensional consolidometer apparatus. In this series of tests, an attempt was made to determine the effect of varying the initial moisture content on the volume change as well as swelling pressure. Table 3.8 shows the laboratory test results indicating the amount of volume change and swell pressure by varying initial moisture content.

As it was expected the soil with low initial moisture content showed high volume change. However, the swelling pressure required for zero volume change remained practically constant. The test results on Table 3.8 indicate that the swelling pressure for the various moisture contents ranges from 455 to 548 kPa for Bole site and 190.60 to 234 kPa Lebu. Table 3.8 and Figure 3.11 indicate the variation of moisture content verses volume change. The result shows an increase of initial moisture content will decrease the amount of swell. From the test results one can conclude that soils with an initial high moisture content experience less uplift, but the pressure required to maintain a constant volume remains constant. (Detail data are shown in Appendix E)

Chapter Four

4.0 Conclusion and recommendations

4.1 Conclusion

In this thesis an attempt has been made to examine Atterberg limits for expansive soils. In order to meet the objective different laboratory and field tests were carried out. Based on the test results the following conclusions were made:-

1. From field tests i.e. artificial flooding and moisture content below water table revealed that the water holding capacity of expansive soil is found between liquid limit and plastic limit.
2. The desiccation process of expansive soils, and red clay soils, gives the same pattern. However; it is observed that re wetting of completely dry pat of expansive soils, at the moisture content below shrinkage limit shows from 4.16 to 9.76 % of volume change for the additional moisture content ranges from 6.62 to 11.64 % of dry mass. Whereas the Red clay soils shows a volume change of 0.55 to 1.27 % for an additional moisture content of 4.06 to 7.12 % of dry mass. The volume change of expansive soils is more significant than that of red clay soil. The observed volume change on expansive soil is large enough to develop distress on structures.

In addition to this it was observed that during the moisture transfer process of vaporized water, the moisture absorbed by red clay soils on the same process and condition shows a lesser percentage compared to that of expansive soil.

3. The test results show that the moisture content of undisturbed soil samples after a complete swelling was within the plastic range of Atterberg limit. In addition to this, swell pressure test on expansive soils with variable initial moisture content indicate that the samples with low initial moisture content swell most and high moisture content soils experience less uplift, but the pressure required to maintain a constant volume is not altered.

4.2 Recommendations

Though this research work is limited in its time and budget allocation, it is a good starting point for further research in the area. For a further improvement in this area the following recommendations are given.

- This study did not consider the effect of time and temperature during the vaporized rewetting of the sample. It is recommended extending the research by considering the effect of time and temperature by conducting a series of tests on the rewetting process and refinement may be needed for the procedure.

- The effect of initial moisture variation on the swelling pressure should be investigated in detail.

- The validity of hydrometer analysis for expansive soils should be checked. This may have an indirect effect on the swelling measurements(free swell and swell with seating load)