Relation between Shrinkage Index and Swelling Pressure for Expansive Soil found in Addis Ababa

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By

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Abbreviations

AAiT	Addis Ababa institute of Technology
AAU	Addis Ababa University
Ac	Activity number
a.m.s.l	above mean sea level
AASHTO	American Association of Highway and Transportation Officials
ASTM	American Society of Testing and Materials
BS	British Standard
C	Clay content
CH	Inorganic clays of high plasticity
CL	Inorganic cays of low to medium plasticity
CSA	Central Statistic Agency
FS	Free Swell
Gs	Specific gravity
GSE	Geological Survey of Ethiopia
LL	Liquid Limit
M_D	Mass of dry soil
ML	Inorganic silts of low plasticity
MH	Inorganic silts of high plasticity
MC/NMC	Natural moisture content
n	Number of Sample
OH	Organic clays of medium of high plasticity
OL	Organic silts of low plasticity
PI	Plastic Index
PL	Plastic Limit
Ps	Swelling pressure
R^2	Correlation Coefficient
SI	Shrinkage Index
SL	Shrinkage Limit
USCS	Unified Soil Classification System

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Abstract

This research is inspired by the PhD thesis work of H. B. Nagaraj from Indian Institute of Science titled "*Prediction of Engineering Properties of Fine Grained Soils from their Index Properties*". The PhD thesis indicated that the swelling pressure has a poor correlation with the liquid limit, a good correlation with the plasticity index and a better correlation with the shrinkage index (liquid limit – shrinkage limit) of fine-grained soils. Nagaraj used a total set of ten remolded soil samples representing the three extreme clay mineral types namely Kaolinite, Illite and Montmorillonite. Also due attention was given to select pairs of soils with nearly the same liquid limit values, but having different plasticity properties. Consequently, the research title "*Relationship between Shrinkage Index and Swelling pressure for expansive soil found in Addis Ababa*" was selected for this Master's thesis work.

In contrast to the PhD work mentioned above, the results of this Masters study show that the relationship between the swelling pressure and the shrinkage index for Addis Ababa expansive clay (one type of soil) is very weak. However, a good approximation is obtained between the normalized shrinkage index (normalized by initial moisture content) and the swelling pressure with R^2 value of 0.89. In addition to this, a better estimation is observed when multiple soil parameters are involved in different combination in prediction of the swelling pressure rather than only the shrinkage index. The correlations developed in this work are compared with previously established equations such as David and Komornik (1969), Nayak and Christensen (1974), Daniel Teklu (2004), Vijayvergiya and Ghazzaly (1973), where the first two equations were found to be in a better agreement with the correlations developed in this work.

1. Introduction

1.1. General

Soil is formed by small pieces of mineral particles and it may contain water, air and organic materials. It supports every construction which is designed by human such as buildings, retaining walls and embankments. There are many types of soils. The main soil groups are granular soil, fine-grained soil and organic soil.

Some partially saturated clayey soils are very sensitive to variations in water content and show excessive volume changes. Such soils, when they increase in volume under applied loads of an increase in their water contents, are classified as expansive soils and exist in many parts of the world.

Expansive soil is fine-grained soil with small particles. This type of soil has high swelling and high compressibility characteristics. The swelling and compressibility behavior of a soil is a very important element to be observed in geotechnical field. It will bring effect to structures such as buildings, embankments and dams. The swelling phenomenon is considered as one of the most serious challenges, which the foundation engineer faces, because of the potential danger of unpredictable upward and downward movements of structures founded on such soils.

Among the different and most important geotechnical properties of clayey soils that have critical implications on the planning, designing, performance and maintenance of civil engineering infrastructure are their potential to expand and shrink with varying amount of moisture content. Expansive soils occur in many parts of the world. However, the problem of expansion and shrinkage is associated with high moisture changes. Hence it is restricted in areas where the seasonal variation in climatic conditions is high (Bell, 1983). The large volume change with the periodic cycle of wetting and drying can cause extensive damages to civil engineering infrastructure; mainly on small buildings, shallow foundations and other lightly loaded structures including roads and airport pavements, pipelines etc (e.g. Chen, 1988; Nelson and Miller, 1992).

Swelling Pressure test will be emphasized in this research. It is determined from the oedometer test. Atterberg Limits are used to describe the consistency of fine-grained soils with varying moisture content. Shrinkage Index is one of the Atterberg Limit parameters and is used to determine the moisture content of a cohesive soil which is between Shrinkage Limit and Liquid Limit. It is an important parameter used to classify fine-grained soil.

1.2. Background of the Research

The oedometer test is used to determine the swelling and compressibility characteristic of a soil. Oedometer test is a complex, time consuming and expensive test if compared to other soil index property tests. A large number of undisturbed samples are needed to acquire reliable data and it consumes more time to obtain the data. The Atterberg limits test is a simple test as it needs only a short time to obtain the results to extract the consistency of a cohesive soil.

In the past, effort had been put in to the research of establishing the relationship between the swelling characteristics and index properties of cohesive soil, aiming to simplify the procedures of swelling pressure determination.

Recent PhD study by Nagaraj (2000) indicates that there is a very good relation between swelling pressure and shrinkage index of fine-grained soils. Hence, the aim of this research is to establish the relation between shrinkage index and swelling pressure of expansive soils found in Addis Ababa. The swelling characteristics of soil will be identified based on the data to correlate these two types of parameters. By referring to the relation generated, the Swelling pressure value can be predicted in response to the execution of the Atterberg limits test. Such correlation will bring a lot of conveniences to the engineers of geotechnical field whenever they tend to do quick and simple identification of the Swelling pressure.

1.3. Expansive Soils in Ethiopia

Expansive soils cover a large portion of Ethiopia. According to information obtained from literature about 10% of the country's land is covered by Black Cotton soils (Berhanu, 1983). The dominant clay minerals of these soils are indicated to be the smectite groups and are mainly montmorillonite clays (Berhanu, 1983).

Expansive soils are known to create severe problems on construction activities; which can lead to expensive design and construction costs, mitigation measures as well as repeated and costly maintenance works. The following are the major engineering difficulties that are apparent due to the adverse properties (large changes in their volume with variation in moisture content which is related to periodic cycle of drying and wetting) of these expansive soils. Some of the main problems associated with expansive soils are discussed below.

1.3.1. Differential Settlement

Differential settlement can cause cracking, rutting and deformations in general distresses on road and runway pavements, failure of drainage structures (bridges, culverts) etc. Similar cracking and deformation problems can also be induced on foundation slabs and walls of small buildings, pipelines and sewerage systems and other similar lightweight structures.

As far as the road infrastructure are concerned, the differential settlement creates series of bumps or corrugations, potholes and patches on different road sections in various parts of the country which in turn reduces the riding quality of roads. Hence, costly and repeated maintenance requirements are also frequently demanded as a result of such problems.

1.3.2. Instability of Cut Slopes

Cut slopes on these soils are prone to instabilities and slope movements due to erosion, heaving and slumping. In places where they are overlain by stiff material the stiffness contrast can lead to even larger problems. Problems of clogging of road side ditches and culverts are common difficulties that demanded the allocation of high budgets for the clearing and maintenance of such drainage structures every year (ERA, 2001).

1.3.3. Gully Formations

Gully formation is associated with the poor permeability and erosion susceptibility nature of these soils. This poses negative and serious economical as well as environmental problems. Scouring of drainage structures seriously affects the overall performance of road infrastructure in many localities. Erosion of soils and hence degradation of environment is another issue of great national concern that is associated with the properties of these types of soils.

1.3.4. Difficult Ground Operations

Due to their sticky and slipery character when saturated with water, the workability of these soils is poor. This in turn is attributed to the difficulty in excavaton, accessibility restrictions to sites of construction and material production (borrow pits and quarry sites), hence time delays in construction activities. This has a negative implication on the cost of construction activities.

In general, engineering problems due to expansive soils have remained one of the biggest challenges in the construction sector in Ethiopia. Even if there is no statistics available on the cost consequences and the amount of damage caused by this problem, there have been serious economical losses and substantial increases in cost of constructions which in many times exceeds the initial cost estimates of construction projects and the allocated budgets and available fund.

1.4. Specific Problems

- Clay soils having a potential to undergo significant volume changes (expansion and shrinkage) with periodic cycle of drying and wetting, pose difficulties and damage on civil engineering infrastructures in the study area.
- Conventional standard methods of swelling pressure test is expensive and time consuming.
- Thus there is a need to identify these expansive soil types and to determine their swelling pressure in a method that is simple, cheaper and rapid as compared to the conventional methods of measuring the swelling pressure of these soils.

1.5. Statement of the Problem

Civil engineers always encounter difficulties in obtaining precise swelling pressure for design of light structures. Inadequate soil investigation data due to budget constraint and poor planning of soil investigation works regularly happen here in Ethiopia. In addition, conventional swelling pressure test requires undisturbed soil sample, is complicated and time consuming. Furthermore, the results sometimes are not accurate due to the poor quality of handling and laboratory testing on the soil samples. Thus, identification of factors that governs the swelling pressure such as shrinkage index of the soil can be used as a base of the finding on the validity of the swelling pressure.

1.6. Description of the study area

Addis Ababa the capital city of Ethiopia, is located between 08º 49´ to 09º 06´ North and 38º 38´ to 38º 54´ East. The study area is located on the Southern, Southeastern and Southwestern part of the city of Addis Ababa. Location map of the study area is shown in figure 1.1.

The climate of Addis Ababa is cool to temperate with a mean annual temperature of 16 degrees centigrade, and a mean annual rainfall of 1200 mm–1600 mm (EMA, 1988). The main rainy season in the area falls between June and mid September, which is winter. While the lesser rains of autumn fall between February and April, which sometimes extends up to mid May. Heavy and torrential rains that last from few minutes to several hours are common during both rainy seasons.

The topography of the study area is variable, that is rugged, hilly and mountainous at the north and northeastern parts whereas flat towards the south and southwest. Elevation ranges from 2700m (northern parts) to 2300m (southern parts) above mean sea level.

Addis Ababa is not only the capital city of Ethiopia, but also the biggest and the most highly populated city in the country. The city is situated in the main highland plateau with an average elevation of 2440m above sea level. According to recent census the inhabitants of the city are around 2,980,001 (CSA, 2011). Due to the rapid expansion of the city and the need to build more houses, currently there is an extensive construction activity going on in every direction of the city including and mostly on parts of the study area.

Extensive areas to the south, south eastern and southwestern part of the study area are covered by soils of expansive nature (GSE, 1990). Soil expansion and shrinkage is a big problem posing damage to buildings and other infrastructure like roads, pipelines and sewerage systems etc in areas of construction activities where such soils exist. Most of the damages are reported from the south and southwestern part (Bole and CMC) of the study area.

Figure 1.1 Location of soil samples tested

1.7. Research Objectives

The main objective of this thesis research will be to derive empirical models from correlation of soils for predicting swelling pressure of expansive soils.

Specific Objectives

- To study the relationship between the shrinkage index and swelling pressure of the expansive soil found in Addis Ababa.
- To compare the relation established with previous research.
- To establish empirical equations which assist in estimating the swelling pressure of expansive soils.

1.8. Research Questions

- It is possible to establish a relationship between the laboratory determined swelling pressure of expansive soils and their index properties; and develop prediction models to estimate the swelling pressure of these soils?
- Which index property test can best describe or are diagnostic of swelling pressure of expansive soils.

1.9. Research Hypotheses

Recent developments indicate shrinkage Index has a good relation with swelling pressure of expansive soil, since shrinkage limit takes care of the gradation of the soil fractions. Thus, by considering the shrinkage index, which is the difference of the liquid limit water content on one end and shrinkage limit water content on the other end, the primary physical properties of the soils namely the plasticity and the grain size distribution are considered.

1.10. Importance of Research

The oedometer test is important to examine the swelling behavior of soil. However, the oedometer test is time consuming, expensive and complex test. Initially, site investigation is carried out and the undisturbed soil sample would be obtained from the borehole or test pit and tested in laboratory. The test takes more time to obtain a complete set of data. In this research, relation between shrinkage index and swelling pressure is investigated. The swelling pressure of the soil sample will be determined by executing the Atterberg limit test; the test is simple and rapid. There is a helpful objective in this research, which is to solve the site problem in a quick and efficient way.

1.11. Thesis Structure

The thesis is divided in to seven separate Chapters. Chapter one introduces the topic. It also discusses the engineering problems and difficulties that are associated with potentially expansive soils and the overall objectives of the thesis and gives description of the study area and its location. Chapter two deals with relevant literature review and summarizes various aspects related to the research topic. In this Chapter, some conventional geotechnical techniques that are useful for identification of potentially expansive soils are presented.

Chapter three reviews the materials and methods used for this thesis research. It also includes details of fieldwork sample collection. Chapter four presents the tests that made on expansive soils and the results obtained from the tests. Discussions on the results obtained from the tests are given in this chapter. The statistical analysis using different statistical tools to predict swelling pressure of expansive soil from shrinkage index are illustrated in Chapter five. Chapter six evaluates empirical equations developed by various researchers to predict swelling pressure. Development of new empirical formula by taking two or more soil parameters in different combination using SPSS 19 software to predict swelling pressure of expansive soil is presented in this chapter. The conclusion and recommendations for future prospect are summarized in the last Chapter, Chapter seven.

2. Literature Review

2.1. Nature of Expansive Soil

Expansive soils swell when given access to water and shrink when they dry out. The moisture may come from rain, flooding, leaking water from sewer lines or from reduction in surface evapotranspiration when an area is covered by a building or pavement. Gourley et al. (1993) defined expansive soils as soils of clay composition that have a characteristic property to undergo significant volume changes upon wetting and drying. Such soils expand or swell when moistened; and shrink and crack when dried.

Expansive soils are largely recognized among other things, for the problems and considerable damage that they can pose in construction sector. The damage is adverse and is mainly restricted to light weight structures (e.g. Chen, 1988; Gourley et al., 1993) where the downward pressure exerted from the structure is exceeded by the uplift pressure from the soil swell. Chen (1998) mentioned that the damage from expansive soils in the United States of America exceeds the damage caused by all other natural hazards combined together. It has been reported that the damage caused by these soils contributes significantly to the burden that natural hazards pose on the economy (Nelson and Miller, 1992) of countries where the occurrence of these soils is significant. Ethiopia is among the list of countries in which the occurrence and spatial distribution of expansive soils (Chen, 1988) is recognized as significant.

2.2. Origin and Occurrence of Expansive Soils

Expansive soils are known to occur in many parts of the world. Some of these countries are Argentina, Australia, Burma, Canada, Cuba, **Ethiopia**, Ghana, India, Iran, Israel, Mexico, Morocco, Poland, South Africa, Spain, Turkey, U.S.A., U.S.S.R., Venezuela and Zimbabwe (Chen,1975). The parent material of expansive soils may be classified in to two groups. The first group comprises of basic igneous rocks. Here feldspar and proxyne minerals of the parent rocks decompose to form Montmorillite (the predominant mineral of expansive soil) and other secondary minerals. The second group comprises of sedimentary rocks that contain Montmorillite. The expansive soils of Ethiopia are derived from both groups (Teferra and Solomon, 1986).

Figure 2.1 Distribution of expansive soil in Ethiopia

2.3. Clay Mineralogy

The term clay can refer both to a size and to a class of minerals. As a size term, it refers to all constituents of a soil smaller than a particular size, usually 0.002 mm in engineering classifications. As a mineral term, it refers to specific clay minerals that are distinguished by small particle size, a net electrical charge, plasticity when mixed with water and high weathering resistance (Mitchell, 1993). Clay minerals are produced mainly from the chemical weathering and decomposition of feldspars, such as orthoclase and plagioclase, and some micas. They are small in size and very flaky in shape.

The thickness of clays is very small relative to their length and breadth, in some cases as thin as 1/100th of the length. Therefore, they have high to very high specific surface values. These surfaces carry a small negative electrical charge that will attract the positive end of water molecules. This charge depends on the soil mineral and may be affected by an electrolyte in the pore water. This causes some additional forces between the soil grains which are proportional to the specific surface. Thus, a lot of water may be held as adsorbed water within clay mass. The following table shows examples of mineral grain specific surfaces. The more elongated or flaky a particle is the greater will be its specific surface.

Table 2.1 Examples of clay mineral grain specific surfaces (Lesilie, 2000)

Mineral/Soil	**Grain width d (μm)**	**Thickness**	**Specific Surface m^2/N**
Kaolinite	2.0 - 0.3	≈0.2d	2
Illite	2.0 - 0.2	≈0.1d	8
Montmorillonite	1.0 - 0.01	≈0.01d	80

The various clay minerals are formed by the stacking combinations of basic sheet structures with different forms of bonding between the combined sheets. The main clay minerals are:

1. Kaolinite
2. Illite
3. Montmorillonite.

2.3.1. Kaolinite Mineral

Kaolinite is a layered silicate mineral, with one tetrahedral sheet linked through oxygen atoms to one octahedral sheet of alumina octahedral. Rocks that are rich in kaolinite are known as china clay or kaolin. The stacked layers of kaolinite are having a thickness of $7A^0$. Thus kaolin group of minerals are most stable and water cannot enter between the sheets to expand the unit cells (Mitchell, 1993).

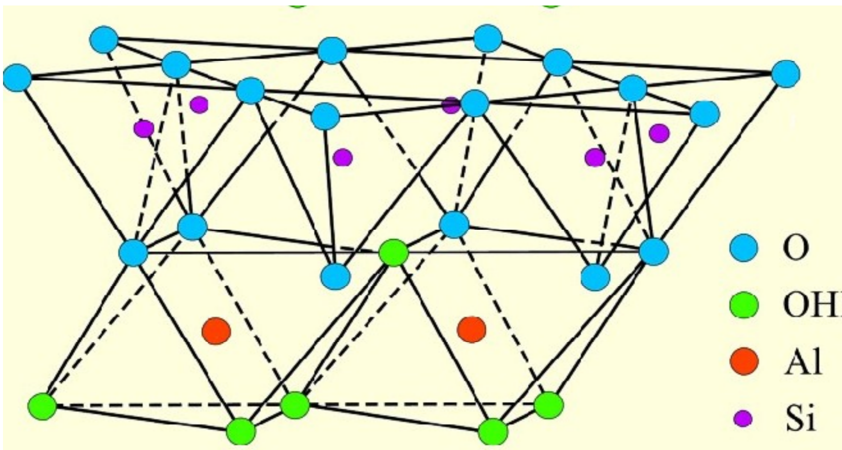

Figure 2.2 Diagrammatic sketch of the structure of Kaolinite layer

2.3.2. Illite Group

These minerals fall between the kaolinite and montmorillonite group so far as their structural arrangement is concerned. The spacing between the element silica gibbsite silica sheets depends upon the amount of available water to occupy the space. For this reason, Illite is said to have expanding lattice. Each thin platelet has a power to attract each flat surface, a layer of absorbed water approximately $200A^0$ thick thus separating palates a distance of $200A^0$ under zero pressure. In the presence of an abundance of water, the mineral can causes split up into about individual unit layers of $10A^0$ thick (Mitchell, 1993).

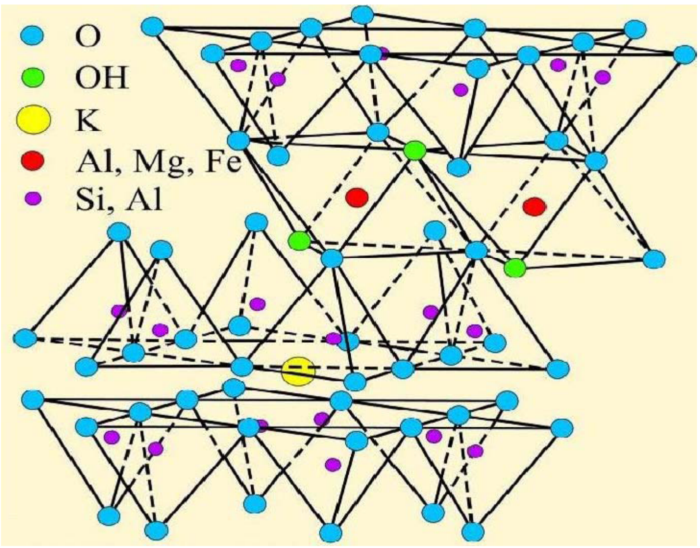

Figure 2.3 Diagrammatic sketch of the structure of Illite layer

2.3.3. Montmorillonite Minerals

This Crystals form weaker bondage between them. Soils containing higher percentage of montmorillonite minerals exhibit high swelling and shrinkage characteristics. Structural arrangement of montmorillonite mineral is composed of units made of two silica tetrahedral sheets with a central aluminum octahedral sheet. The silica and gibbsite sheets are combined in such way that the tips of the tetrahedrons of each silica sheet and one of hydroxyl layers of octahedral sheet form a common layer. The atoms common to both gibbsite and silica layers never participate in the swelling. Water can enter between the sheets causing them to expand significantly and these structures can break to $10A^0$ thick structural units. Thus, soils with montmorillonite minerals exhibit higher shrinkage and swelling characteristics depending upon the nature of exchangeable cation presence (Mitchell, 1993).

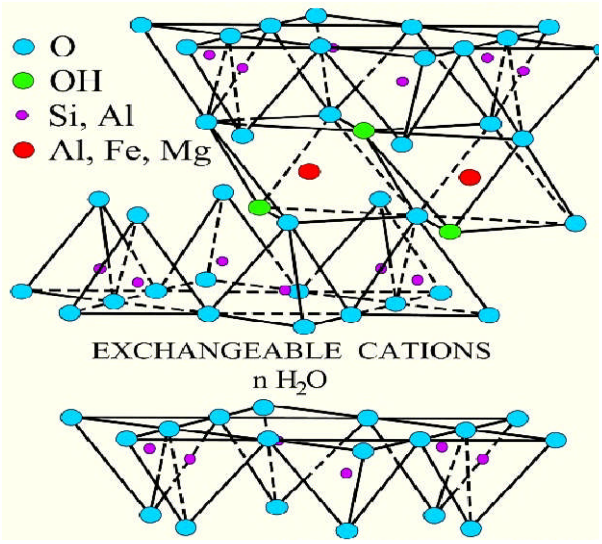

Figure 2.4 Diagrammatic sketch of the structure of Montmorillonite layer

Schemes for the basic clay minerals are shown on figure 2.5.

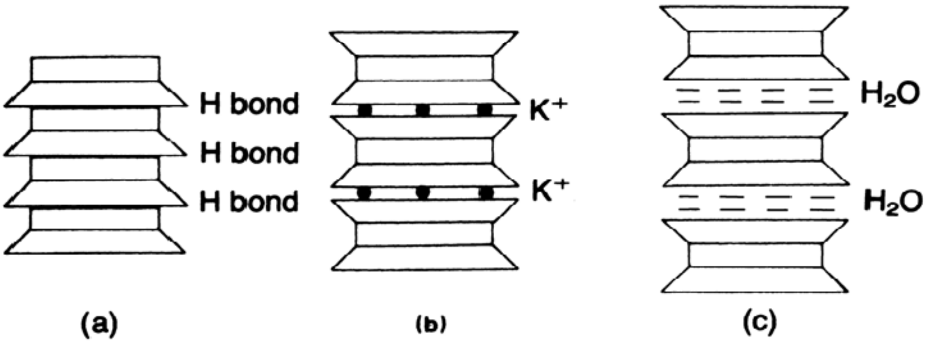

Figure 2.5 Schemes for a) Kaolinite b) Illite C) Montmorillonite (Craig,2004)

2.4. Factors Affecting Swelling of Soils

Expansion and shrinkage is a result of change in particle spacing and changes in the soil water system that disturb the internal stress equilibrium. Factors affecting the swelling of soils can be considered in three main groups (Nelson and Miller, 1992)

• Soil characteristics

• Environmental factors

• The state of stress

2.4.1. Influence of Soil Characteristics

Clay Content and Mineralogy

Clay minerals which typically cause soil volume changes are Montmorillonite, Vermiculites, and some mixed layer clay minerals. Swelling potential increases with the increasing amount of such clay minerals.

Soil Water Chemistry

Soil water has different type of dissolved minerals, which can react with the clay. Clay particles are platelets like in shape and they have negative charges on their surface and positive charges on their edge. The negative charges on the surface of these particles are balanced by the cations from the soil water. These cations are sodium, calcium, magnesium and potassium, which dissolve in the soil water and are adsorbed on the clay surface as exchangeable cations to balance the negative electrical surface charge. Thus, swelling is repressed by increased cation concentration and increased cation valance. For example, Mg^{+2} cations in the soil water would result in less swelling than Na^{+} cations.

Soil Structure and Fabric

Flocculated clays tend to be more expansive than dispersed clays. Cemented particles reduce swell.

Initial Dry Density

The dry density is an important factor in determining the magnitude of volume change. The swell or the swelling pressure of an expansive soil increases with increasing dry density for constant moisture content. The reason is higher densities indicate closer particle spacing, which mean greater repulsive forces between particles and larger swelling potential.

Initial Water Content

As the initial water content increases, the initial degree of saturation will also increase and the affinity of soil to absorb water will decrease, so the amount of swelling will decrease.

Fine Grained Fraction

As the amount of fine particles increase, the amount of swelling will increase due to the larger surface area.

Plasticity Index

In general, soils that exhibit plastic behavior over wide ranges of moisture content and that have high liquid limits have greater potential for swelling. Plasticity index is an indicator of swelling potential.

2.4.2. Influence of Environmental Conditions

Climate

Amount and variation of precipitation and evapotranspiration greatly influence the moisture availability and depth of seasonal moisture fluctuation. Greatest seasonal heave occurs in semiarid climates that have short wet periods.

Groundwater

Shallow water tables provide a source of moisture and fluctuating water tables contribute to moisture content of soils.

Vegetation

Vegetation (trees, shrubs, grasses, etc.) deplete moisture from the soil through transpiration, and cause the soil to be differentially wetted in areas of varying vegetation.

Permeability

Soils with higher permeabilities, particularly due to fissures and cracks in the field soil mass, allow faster migration of water and promote faster rates of swell.

Temperature

Increasing temperatures cause moisture to diffuse to cooler areas beneath pavements and buildings.

2.4.3. Influence of Stress Conditions Stress History

Loading

Magnitude of surcharge load determines the amount of volume change that will occur for a given moisture content and density. An externally applied load acts to balance interparticle repulsive forces and reduces swell.

Soil Profile

The thickness and location of potentially expansive layers in the profile considerably influence potential movement.

2.5. Identification and Classification of Expansive Soil

2.5.1. Field Identification of Expansive Soils

Expansive soil deposits can be recognized in the field through visual inspections. The method is simple and easy to use. Some of the important field identification method that indicates the potential for expansiveness of a soil are the following:

- ✓ 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 sample of the soil is sticky and it will be relatively difficult to clean the soil from the hands.
- ✓ The appearance of cracking in nearby structures.
- ✓ They usually have a color of black and gray.
- ✓ In the regions where there is seasonal moisture variation.
- ✓ Open or closed fissures, (a joint or similar discontinuity).
- ✓ Highly polished or glossy fissure surface.

2.5.2. Laboratory Identification of Expansive Soils

To be able to counteract the problems posed by expansive soils, it is necessary to identify them first. A successful design and construction of foundations on expansive soils requires a good understanding of swelling characteristics, namely, free swell index, swell potential, and swelling pressure. Soil scientists and geotechnical engineers to obtain information on the physical, chemical and mineralogical properties of expansive soils develop various methods and techniques. This is to help in the recognition and characterization of these soils, and to estimate the magnitude of damage that might be faced. Chen (1988), and Nelson and Miller (1992) grouped the methods in to the following categories; namely mineralogical identification, indirect correlations of swelling characteristics with simple index properties and direct measurement of swelling characteristics (swell potential and swelling pressure).

2.5.2.1. Mineralogical Identification

Among the mineralogical identification methods are x-ray diffraction (XRD), differential thermal analysis (DTA), dye absorption, chemical analysis and electron microscope resolution methods. These methods are important in research laboratories in exploring the basic properties of clays. But they are costly and hence not commonly used in soil mechanics laboratories (Chen, 1988).

Researchers have established thresholds of some chemical properties of soils for instance cation exchange capacity (CEC) of different clay types, and established relation with soil expansion potential (Chen, 1988). The following table shows the range of CEC in different clay minerals with their respective expansion potential.

Table 2.2 Ranges of cation exchange capacity of the three main clay minerals and the respective expansion potential

Clay minerals	CEC (meq/100g)	Expansion potential
Kaolinite	3-15	Low
Illite	10-40	Moderate
Montmorillonite	60-100	High

Cation exchange capacity is used as a measure of soil expansion and shrinkage potential in engineering applications. Clay minerals have high capacity to hold exchangeable cations due to their chemical structure (e.g. Chen, 1988; Perlof and Baron, 1976). However, various types of clay minerals are observed to exhibit different magnitudes of cation exchange capacities. While the Montmorillonite group clays show the highest CEC, the Kaolinite group clays exhibit the lowest CEC.

2.5.2.2. Indirect Methods

Under the indirect methods are Atterberg limits (liquid limit (LL), plastic limit (PL), plasticity index (PI), shrinkage limit (SL) and shrinkage index (SI)), free swell, colloid contents and linear shrinkage tests etc. These methods are the most widely used in engineering work for purpose of classification of soils and assessment of their expansion potential (e.g. Chen 1988; Day, 2001; Nelson and Miller, 1992).

Especially the Atterberg limits are considered very popular indicators of soil swelling potential. Atterberg limits are related to the amount of water attracted to the surfaces of the soil particles. Hence, they show the various states of consistency of cohesive soils in relation to their moisture content. Water has a significant influence on the physical properties of cohesive soils. Therefore, knowing the different states at which soils exist with varying amount of moisture content is of geotechnical interest. For example plasticity indices show the range of moisture content over which the soil remains plastic (Lambe and Whitman, 1979). The higher the plasticity index the higher the plasticity and compressibility of the soils also the greater the change in volume that they can exhibit. Atterberg limits are of high importance not only because they are indicators of soil expansion potential but they are also used in identifications and classifications of cohesive soils for engineering purposes. Besides, there is a good correlation between Atterberg limits and strength of cohesive soils (Perloff and Baron, 1976). Perloff and Baron (1976) have also mentioned the work of Skempton and Northey (1952) which showed that Atterberg limits can give an indication of sensitivity of soils when they are coupled with measurements of natural moisture contents of soils. Owing to this relevance, Atterberg limits are directly used in construction specifications for controlling the quality of construction materials that will be used in fill, embankment and sub-base constructions etc.

Thresholds are developed and relations are established between Atterberg limits and expansion potential of soils as well as the types of clay minerals in the soils (e.g. Nelson and Miller, 1992). Table 2.3 shows some of the established relationships. Researchers have observed an increase in the liquid limit and plasticity indices of soils with an increase in soil expansion potential (e.g. Chen, 1988; Lambe and Whitman, 1979). Whereas no specific conclusions were given about the plastic limits of soils on this respect; wide range of overlapping numbers are common for different types of clays with variable expansion potential.

Table 2.3 Relationships between expansion potential, Atterberg limits and types of clay minerals

Clay mineral type	Atterberg limits			Expansion potential
	LL %	PL %	SL %	
Kaolinite	10-20	30-100	25-29	Low
Illite	60-120	35-60	15-17	Moderate
Montmorillonite	100-900	50-100	8.5-15	High

2.5.2.3. Direct Measurement

Direct measurement involves the use of consolidation apparatus or Oedometer to measure the swelling pressure that can be exerted by the soil expansion. This method should be done in a sophisticated and controlled conditions with all the anticipated environmental conditions fulfilled. It gives an opportunity of directly observing the effects of soil expansion on different scale or magnitude of loadings that resembles the actual conditions. It is the most common and useful swell and heave prediction test method (Nelson and Miller, 1992). However, it might take several days and loading steps before the swell pressure is determined even for a single sample that in turn makes it an expensive and labor-intensive testing method.

2.6. Classification of Expansive Soils

The soil classification systems are used extensively by engineers to distinguish between the different types of soils within broad categories. Many classification schemes provide an expansion rating to provide a qualitative assessment of the degree of probable expansion. Expansion ratings may be such as high, medium and low, or critical and noncritical. A reasonable classification involves the use of soil properties and provides one or more of the following ratings:

i. Ranges of the values for either probable percentage of volume change, or probable swelling pressure
ii. A qualitative expansion rating, which is low, medium, high and very high expansion potential
iii. Some other classification based on grain size distribution such as the Unified Soil Classification System (USCS) and the American Association of State Highway and Transportation Officials (AASHTO) system. These classification based on particle size characteristics, liquid limit (LL) and plasticity index (PI) of soils. The systems that is quite popular amongst engineers.

2.7. Evaluation of the Swelling Potential of Expansive Soils by Single Index Method

Simple soil property tests can be used for the evaluation of the swelling potential of expansive soils (Chen, 1988). Such tests are easy to perform and should be used as routine tests in the investigation of building sites in those areas having expansive soil. These tests are:

1. Atterberg limits tests
2. Shrinkage limit and linear shrinkage tests
3. Free swell tests
4. Colloid content tests

2.7.1. Atterberg Limits

Holtz and Gibbs (1956) demonstrated that the plasticity index, PI, and the liquid limit, LL, are useful indices for determining the swelling characteristics of most clays. Since the liquid limit and the swelling of clays both depend on the amount of water a clay tries to absorb, it is natural that they are related. The relation between the swelling potential of clays and the plasticity index has been established as given in table 2.4.

Table 2.4 Relation between swelling potential and plasticity index

Plasticity Index, PI (%)	Swelling potential
0 - 15	Low
10 - 35	Medium
20 - 55	High
35 and above	Very high

2.7.2. Free Swell Index

Mohan and Goel (1959) suggested the degree of expansiveness based on FSI as shown in the following table.

Table 2.5 Degree of expansion based on free swell index

Liquid limit (%)	Plasticity index (%)	Free swell index (%)	Degree of expansion	Degree of severity
70 - 90	> 32	> 200	Very high	Severe
50 - 70	23 - 32	100 - 200	High	Critical
35 - 50	12 - 23	50 - 100	Medium	Marginal
20 - 35	< 12	< 50	Low	Non critical

2.7.3. Colloid Content

There is a direct relationship between colloid content and swelling potential. (Chen, 1988). For a given clay type, the amount of swell will increase with the amount of clay present in the soil. Holtz and Gibbs (1956) proposed some identification criteria based on the probable volume change in expansive soils.

Table 2.6 Data for estimating probable volume change

Colloidal content	Plasticity index (%)	Shrinkage limit (%)	Probable volume change (%)	Degree of expansion
> 28	> 35	< 11	> 30	Very high
20 - 31	25 - 41	7 - 12	20 - 30	High
13 - 23	15 - 28	10 - 16	10 - 20	Medium
< 15	< 18	> 15	< 10	Low

2.7.4. Shrinkage Limit and Linear Shrinkage

The swell potential is presumed to be related to the opposite property of linear shrinkage measured in a very simple test. Altmeyer (1955) suggested the values given in Table 2.7 as a guide to the determination of potential expansiveness based on shrinkage limits and linear shrinkage.

Table 2.7 Relation between swelling potential, shrinkage limits, and linear shrinkage

Shrinkage limit (%)	Linear shrinkage (%)	Degree of expansion
< 10	> 8	Critical
10 - 12	5 - 8	Marginal
> 12	0 - 5	Non-critical

2.8. Prediction of Swelling Pressure

Some researchers, (Komornik and David, 1969; Vijayvergiya and Ghazzaly, 1973; Nayak and Christensen, 1974 and Daniel T., 2004) based on experimental data relationships have established empirical relations from which swelling pressure can be estimated based on soil index tests. The empirical equations are as shown below.

1) Komornik and David (1969) carried out swelling pressure tests on a number of natural (undisturbed) soil samples and, on the basis of statistical (regression) analysis, developed the following relationship:

   $\text{Log Ps} = -2.132 + 0.0208(LL) + 0.00665(\rho_d) - 0.0269(MC)$ ……………..……….. (2.1)

   Where Ps is the swelling pressure in kg/cm^2

   LL is the liquid limit of the soil (%)

   ρ_d is the initial dry density in kg/m^3 and

   MC is the natural moisture content (%)

2) Vijayvergiya and Ghazzaly (1973) proposed the following correlations for swelling pressure of undisturbed samples tested under a surcharge of 10 KPa.

   $\text{Log Ps} = (1/19.5)*(\rho_d + 0.65LL - 139.5)$ ……………..…………………..………… (2.2)

   Where Ps is the swelling pressure in $tons/ft^2$

   LL is the liquid limit of the soil (%)

   ρ_d is the dry density of the soil in lb/ft^3

3) Nayak and Christensen (1974) gave statistical relationships for swelling pressure as:

   $Ps = 0.25\,(PI)^{1.12} * (C^2/MC^2) + 25$ ……………………………………………...……… (2.3)

   Where Ps is the swelling pressure in kN/m^2

   PI is the plasticity index of the soil (%)

C is the clay content (%) and

MC is the natural moisture content (%)

4) Daniel Teklu (2004) from multiple regression analysis he recommended the following two equations:

$$\text{Log } P_s = -5.00 - 0.0002064(LL) + 0.003477(PI) + 0.005827(\rho_d) \quad (2.4)$$

$$\text{Log } P_s = -9.384 + 0.02748(MC) + 0.006307(PI) + 0.008359(\rho_d) \quad (2.5)$$

Where Ps is the swelling pressure in kPa

LL is the liquid limit in (%)

PI is the plasticity index of the soil (%)

ρ_d is the initial dry density in kg/m^3 and

MC is the natural moisture content (%)

2.9. Shrinkage Limit

The shrinkage limit of a soil is defined as the point at which no further volume decrease occurs, but where the degree of saturation is still essentially 100 % (Holtz and Kovacs 1981). The water content at which this occurs is defined as the shrinkage limit, SL, and it is one of the Atterberg Limits (Figure 2.7).

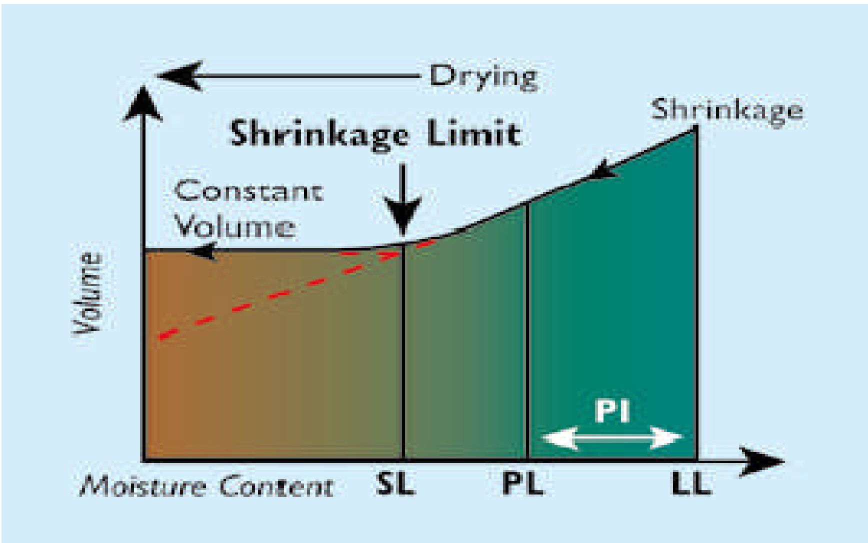

Figure 2.6 Consistency limits and plasticity

Although the shrinkage limit was a popular classification test during the 1920's, it is subject to uncertainty as it has some undesirable features including danger of mercury poisoning to the operator, errors resulting from entrapped air bubbles in the dry soil specimen, cracking during drying and weighing and other measurement errors (Holtz and Kovacs 1981). The standard, ASTM D 427, states to start with the water content near the liquid limit. However, this often results in a shrinkage limit greater than the plastic limit, which is meaningless. Casagrande suggested that the initial water content be slightly greater than the PL, if possible, but it is then difficult to avoid entrapping air bubbles (Holtz and Kovacs 1981).

It has already been stated in the literature (Perloff and Baron 1976; Sridharan and Rao 1971; Sridharan and Prakash 1998) that capillary forces initiate the shrinkage process. The capillary forces depend upon the pore size; the smaller the pore size, the higher the capillary forces. The shrinkage limit test is similar to a consolidation test with the difference being that the capillary stresses induce the shrinkage (Sridharan and Nagaraj 2000). Therefore, the void ratio, e, at the SL can be taken as a limiting void ratio beyond which compression will be significantly less. The range of void ratio change from the liquid limit state to the state of lowest void ratio is represented by the difference between the LL and the SL, termed the shrinkage index, SI (Sridharan and Nagaraj 2000).

The other hypothesis to explain the mechanism governing the shrinkage limit is the distribution of clay, silt and sand fraction in natural fine-grained clay. If the distribution is such that during shrinkage, larger void spaces between sand particles are filled with silt particles and smaller void spaces between silt particles are filled with clay particles and voids created by coarser clay particles get filled by finer clay particles so as to give the densest packing with the least average pore size, the soil reaches the shrinkage limit (Sridharan 1999). Such an optimal packing of soil particles gives the lowest shrinkage limit and the shrinkage limit is a result of the "packing phenomenon" and the function of relative grain-size distribution. It was then hypothesized that by mixing two soils with different shrinkage limits, the grain-size distribution will be in between their individual grain-size distributions. The resulting shrinkage limit will either be in between those of the soils used for the mixture or will be even less than that of the original soil of lower shrinkage limit if the soil mixture exhibits more improved grain-size distribution than the constituent soils so as to provide a denser packing (Sridharan 1999). In no case will the resulting soil mixture have a shrinkage limit more than that of the original soil of higher shrinkage limit. Sridharan (1999) presented eight soil mixtures in which the above hypothesis is proved.

The shrinkage of clay soils is often said to depend not only on the amount of clay, but also on its nature (Greene-Kelly 1974). High-swelling clays containing the mineral montmorillonite can absorb large amounts of water and hence are able to shrink considerably on drying. It is common experience that the notable shrinking soils are often rich in montmorillonite (Smith 1959).

Following along with the hypothesis that grain-size is the controlling factor in shrinkage limits, Sridharan (1999) showed results of six different soils with very similar grain-size curves, which also have almost the same shrinkage limits, irrespective of wide variations in the liquid limits or plasticity indices. It was also shown grain-size curves that are drastically different in shape, with completely different shrinkage limits.

All of the above observations rule out the possibility of considering the shrinkage limit as one of the plasticity characteristics of a soil or that it is primarily a function of soil plasticity, however, the tests performed by Sridharan (1999) showed that the grain-size distribution may be the main influence on shrinkage limit.

It has been found that a soil with a lower shrinkage limit compresses more than a soil with a higher shrinkage limit, even though their liquid limits are nearly the same. Soils with lower shrinkage indices compress faster (Sridharan and Nagaraj 2000).

2.10. Swell Collapse Behavior of Soils

Nagaraj (2000) gave an idea about swell/collapse behavior of five soil sets having nearly the same liquid limit. At different relative state of compaction that is minimum, intermediate and maximum dry densities the swell/collapse behavior of soils have shown to be different, though the liquid limit for each soil set is nearly the same. Also at any value of the liquid limit, the higher the shrinkage index or plasticity index, the soils have mostly shown higher swelling behavior, swelling increasing with increase in relative density and lesser the shrinkage index or plasticity index. This clearly brings out the fact that the liquid limit alone being used as a generalizing parameter to predict swell/collapse behavior has its limitation of not considering the plasticity characteristics of the soil. It is also clear that swelling mainly takes place when the soil contains predominantly Montmorillonite clay mineral and when the soil is Kaolinitic, collapse occurs. The amount of swell/collapse is primarily a function of amount of type of clay mineral present in the soil and the density of the soil.

Sridharan and Rao (1971), Sridharan et al. (1986a) and Sridhann et al. (1988b) have brought out in detail mechanisms controlling the liquid limit of soils. They have shown that primarily two mechanisms control the liquid limit water content of soils. Mechanism 1 in which the diffuse double layer essentially contributing to the liquid limit water content and mechanism 2, the shearing resistance at particle level and fabric controls the liquid limit. It has been brought out clearly that mechanism 1 is dominant in Montmorillonite soils (Sridharan et al. 1986b) and mechanism 2 being dominant in kaolinite soils (Sridharan et al. 1988b). Thus, the foregoing discussion shows clearly that swelling behavior, which is essentially controlled by diffuse double layer cannot be related to the liquid limit of Kaolinite soils. This further shows that for Montmorillonite soils the liquid limit may serve as a parameter representing the swelling behavior.

The correlation of swell/collapse with the liquid limit at three different relative states of compaction shows that the swelling type of soils (montmorillonite soils) and non-swelling type of soils (kaolinite and illite soils) show different correlation of swell/collapse behavior with the liquid limit. Similarly the relationship between the percent swell/collapse and the plasticity index and the relationship between the percent swell/collapse and the shrinkage index at the three relative state of compaction shows that the correlation of swell/collapse behavior is better with plasticity index than the liquid limit. Here, the difference between the swelling and non-swelling type of soils is reduced. Further, correlation of swell/collapse behavior with shrinkage index is better than plasticity index or the liquid limit. Here, the differences between the swelling and non-swelling soils are almost a non-existent. From the above discussion, it is clear that shrinkage index which is the range of water content between the liquid limit and the shrinkage limit is a better parameter for correlation with the swelling potential as well as collapse behavior of soils. Though, the free Swell index ratio has shown a good correlation with the swell/collapse behavior, shrinkage index has shown to have a better correlation with the swell/collapse behavior (Nagaraj 2000).

3. Site Description, Sampling and Laboratory Tests

3.1. Introduction

Literature review was carried out in the early stage of the study to enhance the understanding of expansive soils and swelling pressure. Books, Technical papers from the international proceedings, journals, unpublished thesis and published reports are reviewed to keep up to date on the correlations that relate the swelling pressure with the soil index properties. Summary of the literature review has been presented in the Chapter 2. Then soil samples were collected from field and laboratory tests are performed. The suitability of published correlations in predicting the swelling pressure on collected data was analyzed in this study. New correlations are developed and analyzed to fit the collected data.

Soil properties such as particle size distribution, Atterberg limits, dry density, specific gravity, moisture content, free swell and swelling pressure test were conducted for analysis, identification and substituted into the existing correlations to find the estimated swelling pressure. The estimated swelling pressure, which is derived from the existing correlations, was compared with the swelling pressure obtained from the laboratory. Comparison is made and to evaluate the appropriateness of the previous correlations for the expansive soils found in Addis Ababa.

This thesis research encompasses field work preparation, fieldwork data collection and post field work data analysis stages. Prior to the fieldwork, one of the major tasks was identification of expansion soils and studying which proper sampling methodology should be employed in the collection of soil samples. Gathering of as much information as possible about the study area were some of the other major activities in the pre-field work stage.

During the field work stage, soil samples were collected for subsequent laboratory analysis. In addition, any other data of relevance (reports and previous studies) were also collected. After obtaining the laboratory results of swelling pressure and soil index properties, it was proceeded in to the data preparation and compilation, data processing, analysis and interpretation stages. The following figure shows a briefly summarized schematic workflow of the various steps that were undertaken to achieve the objectives of the research.

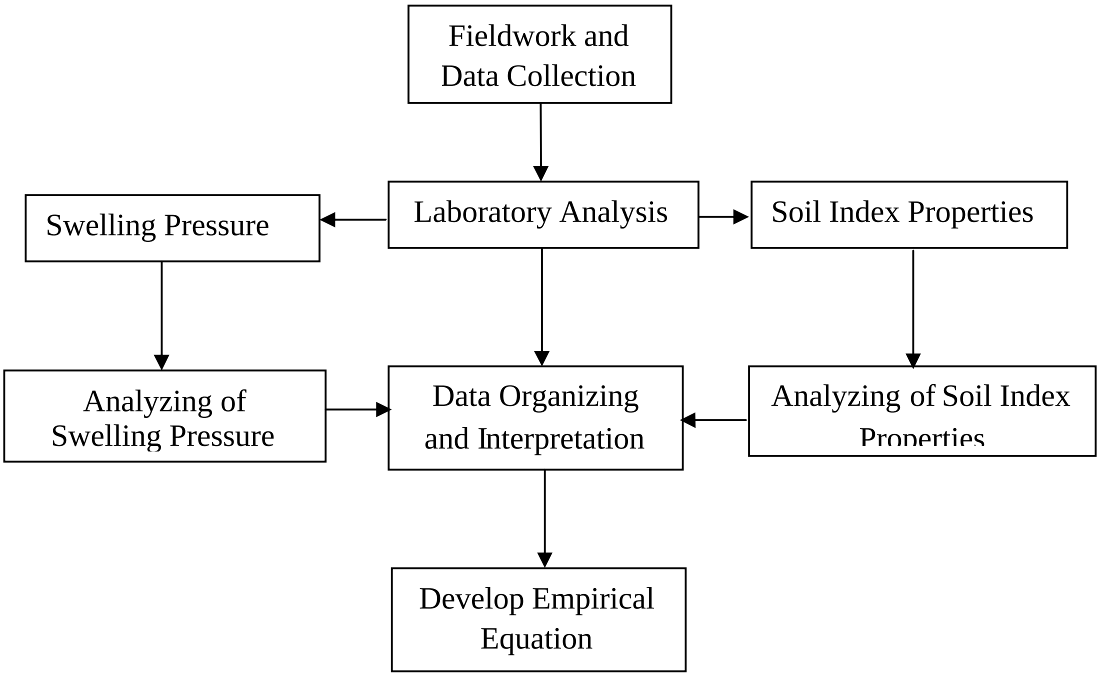

Figure 3.1 Schematic work flow diagram

3.2. Sample Collection

The data collection stage involved gathering of relevant information from literature followed by fieldwork soil sampling. During these fieldworks undisturbed and disturbed soil samples were collected and tested in the laboratory to determine the index properties and swelling pressure. Sampling sites were selected based on information from previous studies from the city of Addis Ababa.

The fieldwork data collection was started from December 25, 2010. During the time of sampling, the climatic condition was dry. The samples taken were all in dry condition. Most part of the study area was exposed bare soil. Both disturbed and undisturbed soil samples were taken from the depth 1.2m to 2.5m. Depth of excavation was dependent on the existing condition of the ground. In some places about 1.5m depth boulder was encountered and in one site (Kality) the ground water table was very near to the surface (1.3m depth).

For this study a total of nineteen samples were collected from fourteen test pits at different areas by excavating the ground using hand digging equipment like a spade and shovel. Each soil sample was taken from one soil profile and is representative of that same soil profile. Soil samples were not taken from areas where construction works and manmade deposits are suspected to impose effects on the nature of the soils.

A pit of 1.5m by 1.5m was dug at representative locations. Notes were taken and approximately 10-15 kg of disturbed soil samples were taken from each sampling location. Undisturbed samples of expansive soils are collected with sampling tubes of diameter 100mm. The samples were properly stored as per the requirements of handling soil samples. Prior to proceeding to the laboratory analysis, the disturbed samples were air-dried.

3.3. Laboratory Tests

Different laboratory tests were conducted on the disturbed and undisturbed soil samples that were collected from different localities of the study area. Sample preparation and testing followed the requirements of ASTM (American State Testing Materials) standard procedures and specifications. Most of the laboratory tests were performed in Water Works Design and Supervision Enterprise soil mechanics laboratory. Some laboratory tests were performed in AAIT geotechnical laboratory.

In order to achieve the objective of this thesis, the following laboratory tests were conducted:

- ✓ Atterberg Limit
- ✓ Grain Size Analysis
- ✓ Specific Gravity
- ✓ Initial Dry Density
- ✓ Natural Moisture Content
- ✓ Free Swell
- ✓ Swelling Pressure

4. Laboratory Test Results and Discussions

4.1. Atterberg Limit Tests

The Atterberg limits of selected soil samples were determined in the laboratory. From the results of Shrinkage limit (SL), Plastic limit (PL) and Liquid limit (LL), Shrinkage index (SI) and Plasticity index (PI) of the soils were calculated. Plasticity index is important in classifying fine-grained soils and is fundamental to use the Casagrande plasticity chart. The larger the plasticity index, the greater will be the engineering problems associated with using the soil as an engineering material, such as foundation support for residential building and road sub grades (Bowles, 1992).

The soil samples considered in this study have LL of 80% - 100%, PL of 29% - 40%, and SL of 10% - 12% (Table 4.1). Most of the soils of the study area fall in high plasticity type. The PI of the soil samples ranges between 45% to 63%; and the SI between 64% to 89%. The results of the test are given in table 4.1.

Table 4.1 Atterberg Limit test result of the study area

Sample No.	Location	Colour	Sample Depth (m)	Natural Moisture Content %	Liquid Limit %	Plastic Limit %	Plasticity Index (LL -PL) %	Shrinkage Limit %	Shrinkage Index (LL-SL) %
S1	Bole	Black	1.2	33.86	90.77	37.07	53.70	10.04	80.73
S2		Grey	2.5	35.67	94.25	35.21	59.04	11.32	82.93
S3	Gerji	Black	1.3	52.25	85.75	33.52	52.23	10.26	75.49
S4	Gurdshola (Egz Ab)	Black	1.5	32.39	91.20	28.75	62.45	10.46	80.74
S5		Grey	2.5	35.21	84.00	33.13	50.87	11.71	72.29
S6	Imperial	Black	1.5	37.43	90.50	39.74	50.76	11.71	78.79
S7		Grey	2.5	32.46	85.00	36.02	48.98	10.60	74.40
S8	CMC (Sumit)	Black	1.5	37.60	96.50	33.95	62.55	12.40	84.10
S9	CMC (Meri)	Black	1.5	34.20	87.20	37.13	50.07	10.53	76.67
S10	Ayat	Black	1.5	35.37	99.75	39.78	59.97	10.77	88.98
S11		Grey	2.5	37.56	88.75	36.15	52.60	12.22	76.53
S12	Hayahulet	Black	1.5	38.80	93.50	32.03	61.47	11.67	81.83
S13		Grey	2.5	31.75	89.20	34.85	54.35	10.13	79.07
S14	Old Airport	Black	1.5	45.23	92.75	37.05	55.70	11.53	81.22
S15	Lafto (Hana)	Black	1.5	31.78	86.50	32.08	54.42	11.85	74.65
S16	Jemo	Black	1.5	32.88	80.25	35.08	45.17	12.10	68.15
S17	Mekanisa	Black	1.5	49.51	96.85	40.21	56.64	11.73	85.12
S18	Kality	Black	1.3	56.27	86.25	35.21	51.04	11.01	75.24
S19	Akaki	Brown	1.5	37.39	83.60	34.06	49.54	11.53	72.07

Habtamu (2006) has obtained the liquid limit of expansive soils of Addis Ababa varying between 90% and 110%. According to Ayenew (2004), the plastic limit of expansive soils from Addis Ababa has a value ranging between 24% and 37%. As determined by Daniel (2004), the shrinkage limit of Addis Ababa expansive soils varying between 11% and 14%. This indicates that the measured Atterberg limits are not as such out of range.

Table 4.2 Range of values of index properties of expansive soils of Addis Ababa

Soil Index Properties	Range of Values			
	Legesse (2004)	Ayenew (2004)	Daniel (2004)	Habtamu (2006)
Liquid Limit (LL), %	96-121	96-121	96-121	90-110
Plastic Limit (PL), %	24-29	24-37	26-47	29-40
Plastic Index (PI), %	70-87	70-84	54-84	60-74
Shrinkage Limit (SL), %				11_14
Free swell (FS), %	75-140			90-105
Clay Fraction, %			50-85	
Activity			0.76-1.26	

Plasticity Index is a parameter, which can be used as a preliminary indicator of the swelling characteristics of a soil (Seed, Woodward and Lundgreen, 1962). The following values are proposed to relate swelling potential with index property (Chen, 1998).

Table 4.3 Swelling potential and plasticity index range

Swelling Potential	Plasticity Index
Low	0-15
Medium	10-35
High	20-55
Very High	35 and above

Relating the Plasticity Index of the soil of the study area with the above given range discloses that the soil falls in the range of very high swelling potential.

Shrinkage Limit is used also as a guide to the determination of potential expansiveness. Altmayer (1956) suggested the following relation (Chen, 1998).

Table 4.4 Shrinkage limit and degree of expansion range

Shrinkage Limit	Degree of Expansion
Less than 10	Critical
10 - 12	Marginal
Greater than 12	Non Critical

Relating the shrinkage limit of the soil of the study area with the above given range reveals that the soil falls in the range of marginal degree of expansion. As previously, illustrated Daniel (2004) has obtained the shrinkage limit value of Addis Ababa expansive soils varying between 11% and 14%. This shows that the measured value has similar with the previous study. Therefore, it is difficult to show conclusive evidence between Shrinkage Limit and Potential Expansiveness (Chen, 1975).

4.2. Grain Size Analysis

Since grain size analysis is one of the index property tests, the soil of the study area is examined for its grain size distribution. Grain size divides soil into two distinctive groups, namely fine-grained and coarse-grained soil. Soil particles, which are coarser than 0.075 mm, are generally termed as coarse-grained and the finer ones like silt and clay are considered fine grained. The property of coarse-grained soil is greatly based on grain size distribution while the property of fine-grained soil is influenced by inter-particle force.

Two methods were used to find the particle-size distribution of the soil samples: sieve analysis, for particle sizes larger than 0.075 mm (No. 200) in diameter; and hydrometer analysis for particle-sizes smaller than 0.075 mm in diameter. During hydrometer analysis sodium hexametaphosphate ($NaPO_3$) was used as a dispersion agent, and all analysis was determined based on ASTM procedure. The results of grain-size analysis are presented in Table 4.5. The diameter or size range is adopted from BS standards as follows: > 2 mm gravel; 2 mm – 0.075 mm sand; 0.075 – 0.002 mm silt and < 0.002 mm clay.

Table 4.5 Grain size distribution of the study area

Sample No.	Location	Colour	Sample Depth (m)	Clay Fraction %	Silt Fraction %	Sand Fraction %	Gravel Fraction %	% passing 0.075mm
S1	Bole	Black	1.2	78.50	20.20	1.30	0.00	98.70
S2		Grey	2.5	70.00	21.66	4.84	3.50	91.66
S3	Gerji	Black	1.3	55.00	35.32	6.11	3.57	90.32
S4	Gurdshola (Egz Ab)	Black	1.5	59.70	31.36	4.42	4.52	91.06
S5		Grey	2.5	61.80	28.65	5.55	4.00	90.45
S6	Imperial	Black	1.5	69.75	19.85	6.90	3.50	89.60
S7		Grey	2.5	67.50	23.38	5.07	4.05	90.88
S8	CMC (Sumit)	Black	1.5	72.00	26.69	1.31	0.00	98.69
S9	CMC (Meri)	Black	1.5	64.80	33.67	1.53	0.00	98.47
S10	Ayat	Black	1.5	78.50	19.91	1.59	0.00	98.41
S11		Grey	2.5	68.00	26.59	5.41	0.00	94.59
S12	Hayahulet	Black	1.5	70.10	24.17	5.73	0.00	94.27
S13		Grey	2.5	57.20	33.13	5.82	3.85	90.33
S14	Old Airport	Black	1.5	70.00	27.46	2.54	0.00	97.46
S15	Lafto (Hana)	Black	1.5	71.20	25.98	2.82	0.00	97.18
S16	Jemo	Black	1.5	71.50	26.31	2.19	0.00	97.81
S17	Mekanisa	Black	1.5	75.00	22.84	2.16	0.00	97.84
S18	Kality	Black	1.3	44.20	46.41	3.41	5.98	90.61
S19	Akaki	Brown	1.5	64.50	28.01	2.75	4.74	92.51

According to Daniel (2004), the clay fractions of expansive soils from Addis Ababa have a value ranging between 50% and 85%. This indicates that the measured clay fractions are not as so much out of range.

4.2.1. Soil Classification

The basis for USCS (Unified Soil Classification System) is Liquid Limit and Plasticity Index of a soil. An “A-line” which is defined by an equation (i.e. 0.73*(LL-20)) separates the ‘MH or OH’ and the ‘CH or OH’ designation. In USCS MH means silt with high plasticity, CH means inorganic clays of high plasticity and OH means organic clay. According to this classification scheme most of the soil of the study area falls in CH region, which shows that the soil is inorganic clays of high plasticity.

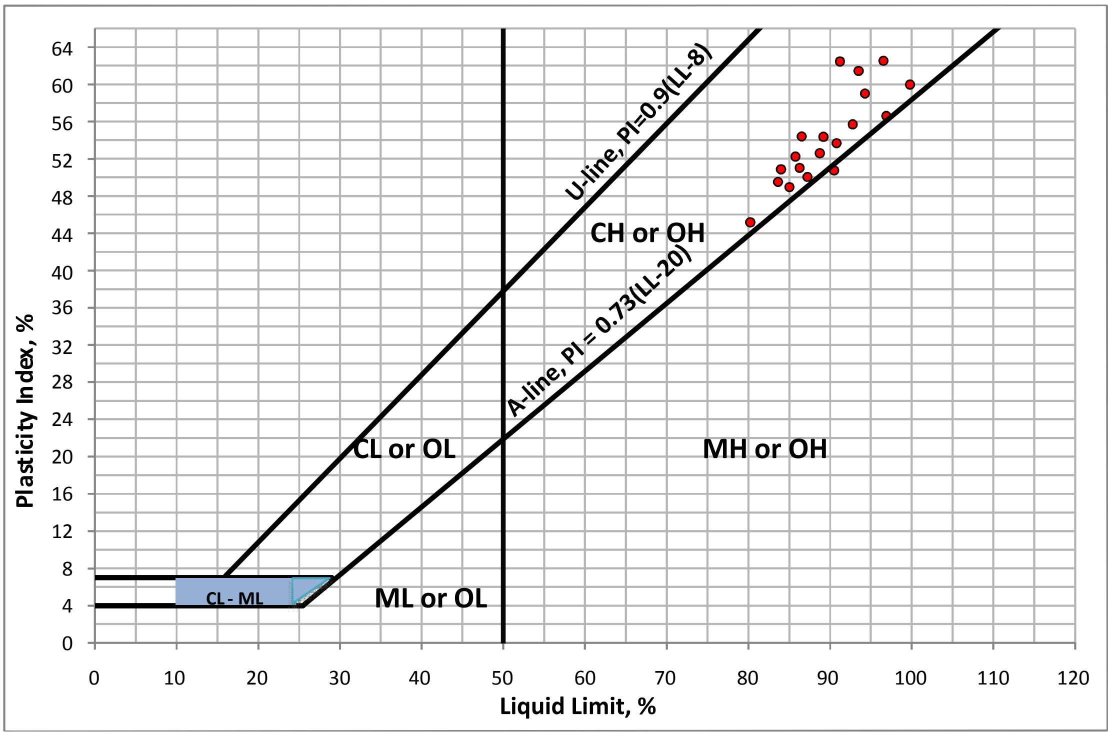

Figure 4.1 Plasticity chart of the study area

4.2.2. Other Classifications

Activity which is defined as the ratio of the plastic index to percent of clay fraction finer than 0.002mm is one means of classifying expansive soils based on their index property. According to Skempton (1953), clays are classified with respect to their activity and the following values show this classification scheme. In terms of potential expansiveness soils with activity less than 0.75 are low, 0.75-1.25 normal and those with greater than 1.25 are highly expansive (Teferra and Leikun 1999).

Table 4.6 Activity and degree of activity

Degree of Activity	**Activity**
Inactive Clay	Less than 0.75
Normal Clay	0.75 - 1.25
Active Clay	Greater than 1.25

These values are presented in the form of chart, which is called Activity Chart, and the soil of the study area is compared to the values and it falls in the range of normal clay (Figure 4.2.).

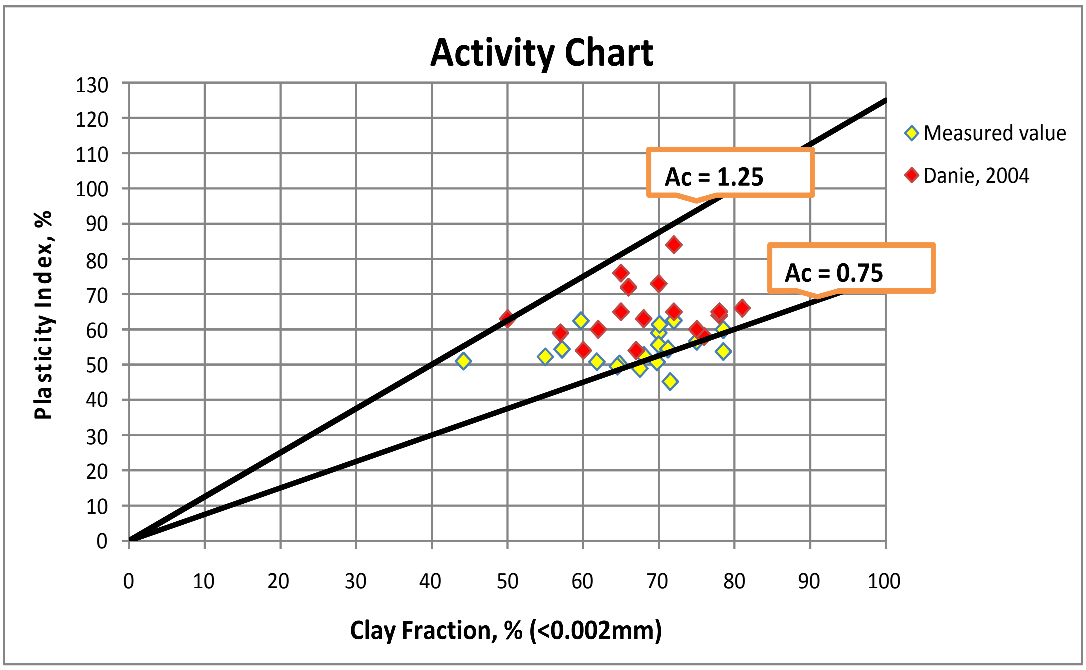

Figure 4.2 Activity chart of the study area

As illustrate in the above figure Daniel (2004) has obtained the activity value of expansive soils of the study area varying between 0.76 and 1.26. This indicates that the measured clay fractions are not as such out of range.

In general, the classification system developed based on single property only such as based on activity (Skempton, 1953), based on shrinkage limit (Altmeyer, 1956) are difficult to use alone as a classification system because they may lead to wrong conclusion.

4.3. Specific Gravity Test

Specific gravity of soil is the ratio of the unit weight of solids in the soil to the unit weight of water. Determination of specific gravity is important and it is useful to determine the diameter of the soil grains in hydrometer analysis. For the study area, the specific gravity of the samples is determined in laboratory and the result is given in the following table.

Table 4.7 Specific gravity of the soil of the study area

Sample No.	Location	Colour	Sample Depth (m)	Specific Grvity
S1	Bole	Black	1.2	2.72
S2		Grey	2.5	2.69
S3	Gerji	Black	1.3	2.78
S4	Gurdshola(Egz Ab)	Black	1.5	2.66
S5		Grey	2.5	2.61
S6	Imperial	Black	1.5	2.72
S7		Grey	2.5	2.61
S8	CMC (Sumit)	Black	1.5	2.81
S9	CMC (Meri)	Black	1.5	2.74
S10	Ayat	Black	1.5	2.71
S11		Grey	2.5	2.67
S12	Hayahulet	Black	1.5	2.69
S13		Grey	2.5	2.72
S14	Old Airport	Black	1.5	2.74
S15	Lafto (Hana)	Black	1.5	2.69
S16	Jemo (Haile Garment)	Black	1.5	2.7
S17	Mekanisa	Black	1.5	2.75
S18	Kality	Black	1.3	2.73
S19	Akaki	Brown	1.5	2.74

4.4. Natural Moisture Content (NMC)

For coarse and fine-grained soils, water content can have a significant effect on the soils behavioral properties when used for construction purposes and foundations. Moisture content affects the settlement (consolidation) condition; shear strength and suitability of soil for compaction. Moreover, the swelling-shrinkage condition of a particular soil is related to its moisture content and its change with time. Consistency of a fine grained soil also depends largely on its moisture content.

Generally, moisture content has an influence on the swelling potential of expansive soils. Natural moisture content of a soil is affected by climate, vegetation cover of the area and other artificial factors. Hence the same soil could have different moisture contents in different seasons of a year and in different times. Since such type of moisture content is likely to fluctuate any time it may not indicate the general property of the soil. Samples were collected and immediately tests were conducted to determine natural moisture content (NMC) of soils. Since in some places the ground water table was very near to the surface some of the NMC show large value while the soil samples were taken during dry season. The result (Table 4.8) shows that the soil samples have a moisture content of 31.75% – 56.27%.

Table 4.8 Natural moisture content values of the study area

Sample No.	Location	Colour	Sample Depth (m)	Natural Moisture Content %
S1	Bole	Black	1.2	33.86
S2		Grey	2.5	35.67
S3	Gerji	Black	1.3	52.25
S4	Gurdshola(	Black	1.5	32.39
S5		Grey	2.5	35.21
S6	Imperial	Black	1.5	37.43
S7		Grey	2.5	32.46
S8	CMC (Sumi	Black	1.5	37.60
S9	CMC (Meri	Black	1.5	34.20
S10	Ayat	Black	1.5	35.37
S11		Grey	2.5	37.56
S12	Hayahulet	Black	1.5	38.80
S13		Grey	2.5	31.75
S14	Old Airport	Black	1.5	45.23
S15	Lafto (Hana	Black	1.5	31.78
S16	Jemo (Haile G	Black	1.5	32.88
S17	Mekanisa	Black	1.5	49.51
S18	Kality	Black	1.3	56.27
S19	Akaki	Brown	1.5	37.39

4.5. Free Swell Test

The free swell test is one of the most commonly used simple tests for estimating soil swelling potential. Free swell tests consists of placing a known volume of dry soil (10cc of dry soil, passing through sieve No. 40 (0.425mm diameter), into a 100 cc graduated cylinder) in water and noting the swelled volume after the material settles, without any surcharge, to the bottom of a graduated cylinder. The difference between the final and initial volume, expressed as a percentage of initial volume, is the free swell value (Chen, 1975). Results of free swell tests of the soils of the study area are presented in Table 4.9.

According to Holtz (1956) cited in Bell (1983), soils having free swell value as high as 100% can cause considerable damage to lightly loaded structures, and soils having free swell value below 50% seldom exhibit appreciable volume change even under very light loadings. The free swell values of the soils of the study area vary from **110% to 185%**.

Table 4.9 Free swell test results of the study area

Sample No.	Location	Colour	Sample Depth (m)	Free Swell %
S1	Bole	Black	1.2	145.0
S2		Grey	2.5	135.0
S3	Gerji	Black	1.3	110.0
S4	Gurdshola(Egz Ab)	Black	1.5	142.5
S5		Grey	2.5	125.0
S6	Imperial	Black	1.5	130.0
S7		Grey	2.5	122.5
S8	CMC (Sumit)	Black	1.5	165.0
S9	CMC (Meri)	Black	1.5	165.0
S10	Ayat	Black	1.5	140.0
S11		Grey	2.5	132.5
S12	Hayahulet	Black	1.5	185.0
S13		Grey	2.5	165.0
S14	Old Airport	Black	1.5	160.0
S15	Lafto (Hana)	Black	1.5	115.0
S16	Jemo (Haile Garment)	Black	1.5	127.5
S17	Mekanisa	Black	1.5	160.0
S18	Kality	Black	1.3	130.0
S19	Akaki	Brown	1.5	125.0

4.6. Initial Dry Density Test

Density in general shows the spacing of particles in a system. Higher densities indicate closer particle spacing, which mean greater repulsive forces between particles and larger swelling potential. The core-cutter method is used for estimating the in-place density of undisturbed soil. The process involves measuring dimensions of the soil sample within the cylinder, then extrude and weigh the soil sample and determine moisture content. After that compute the volume of the soil sample and calculate the bulk density and dry density. The bulk density is the ratio of mass of moist soil to the volume of the soil sample, and the dry density is the ratio of the mass of the dry soil to the volume the soil sample.

The initial dry density values of the soils of the study area vary from **1.04 g/cm^3 to 1.30 g/cm^3.** Results of Initial dry density tests of the soils of the study area are presented in the following Table.

Table 4.10 Test result of initial dry density of the study area

Sample No.	Location	Colour	Sample Depth (m)	Natural Moisture Content %	Bulk Density (ρ_b) g/cm^3	Dry Density (ρ_d) g/cm3
S1	Bole	Black	1.2	33.86	1.71	1.28
S2		Grey	2.5	35.67	1.63	1.20
S3	Gerji	Black	1.3	52.25	1.77	1.16
S4	Gurdshola (Egz Ab)	Black	1.5	32.39	1.68	1.27
S5		Grey	2.5	35.21	1.66	1.23
S6	Imperial	Black	1.5	37.43	1.65	1.20
S7		Grey	2.5	32.46	1.67	1.26
S8	CMC (Sumit)	Black	1.5	37.60	1.71	1.24
S9	CMC (Meri)	Black	1.5	34.20	1.69	1.26
S10	Ayat	Black	1.5	35.37	1.67	1.23
S11		Grey	2.5	37.56	1.62	1.18
S12	Hayahulet	Black	1.5	38.80	1.68	1.21
S13		Grey	2.5	31.75	1.73	1.31
S14	Old Airport	Black	1.5	45.23	1.57	1.08
S15	Lafto (Hana)	Black	1.5	31.78	1.71	1.30
S16	JCMC (Haile Garment)	Black	1.5	32.88	1.63	1.23
S17	Mekanisa	Black	1.5	49.51	1.62	1.08
S18	Kality	Black	1.3	56.27	1.62	1.04
S19	Akaki	Brown	1.5	37.39	1.60	1.16

4.7. Swelling Pressure Test

Swelling Pressure is the amount of pressure a soil exerts upon swelling. The most reliable means of measuring swelling pressure is laboratory determination using one-dimensional consolidometer. This method is called direct measurement and the test is conducted on soils collected from the study area. The purposes of these tests are to make some correlation between shrinkage index and swelling pressure of expansive soil found in Addis Ababa. The results of the test are given in tabular form below (Table 4.11).

The swelling pressure values of the soils of the study area vary from **9.45 kPa to 382.05 kPa**. The swelling pressure is dependent on the geology, environmental factors, soil characteristics and many other factors, which vary from place to place. Some of the samples showed small swelling pressure values due to samples have high natural moisture content during sampling and therefore they have already swelled in situ.

Table 4.11 Test result of swelling pressure of the study area

Sample No.	Location	Colour	Sample Depth (m)	Natural Moisture Content %	Dry Density (ρ_d) g/cm3	Swelling Presure kPa
S1	Bole	Black	1.2	33.86	1.28	382.05
S2		Grey	2.5	35.67	1.20	198.25
S3	Gerji	Black	1.3	52.25	1.16	9.45
S4	Gurdshola (Egz Ab)	Black	1.5	32.39	1.27	192.25
S5		Grey	2.5	35.21	1.23	116.34
S6	Imperial	Black	1.5	37.43	1.20	123.33
S7		Grey	2.5	32.46	1.26	141.31
S8	CMC (Sumit)	Black	1.5	37.60	1.24	138.31
S9	CMC (Meri)	Black	1.5	34.20	1.26	129.32
S10	Ayat	Black	1.5	35.37	1.23	170.28
S11		Grey	2.5	37.56	1.18	114.34
S12	Hayahulet	Black	1.5	38.80	1.21	156.29
S13		Grey	2.5	31.75	1.31	118.33
S14	Old Airport	Black	1.5	45.23	1.08	22.44
S15	Lafto (Hana)	Black	1.5	31.78	1.30	142.31
S16	Jemo	Black	1.5	32.88	1.23	136.31
S17	Mekanisa	Black	1.5	49.51	1.08	22.44
S18	Kality	Black	1.3	56.27	1.04	11.91
S19	Akaki	Brown	1.5	37.39	1.16	62.39

5. Statistical Analysis of Test Results

5.1. Correlation and Regression

Correlation and regression are useful statistical techniques to identify related variables. Correlation compares individual variables with one another and calculates or estimates of the strength, or magnitude of the statistical relationships.

The strength and trends of relationships between swelling pressure and shrinkage index were computed by conducting correlation analysis. The method used to analyze the results to determine if there is a correlation between the swelling pressure and the shrinkage index uses an 'xy' scatter plot and fits the best trend line to the plotted data. The computer program Microsoft Excel was used to carry out this analysis.

To be able to achieve a graph from this program the results were tabulated with shrinkage index against the corresponding swelling pressure. The results were then graphed in a scatter graph. A scatter chart has two axes with the x–axis showing one set of numerical data and the other value along the y-axis. This graph combines these two values into single data point and displays them on the graph where ever they occur.

Scatter charts are commonly used for displaying and comparing numeric values, such as engineering, statistical, and scientific data. The advantage of a scatter plot for this situation is that this chart allows different comparisons to be made. Scatter charts can be displayed with or without lines to connect the data points or fitted with a trend line that fits the data best. A trend line is a graphical representation of the trend or direction of data in a series. Trend lines are used generally to predict a value on the y-axis from data on the x-axis. The data was tested for different trend lines, which consisted of Linear, Logarithmic, Polynomial, Power and Exponential functions.

For each trend line both the equation and R^2 value of the trend line was determined using the Excel program. To determine the strength of each correlation the R^2 value for the trend line was calculated. R^2 is the relative predictive power of a model and is a descriptive measure between 0 and 1. The closer it is to one the greater the ability for the equation to predict an outcome.

5.2. Relation between Swelling Pressure and Shrinkage Index

The relationship between the swelling pressure and the shrinkage index for all of the tested samples is shown in figure 5.1. The best fitting trend line for this relationship is a polynomial regression function. The strength of this equation in predicting an outcome from the shrinkage index is around 26.6% or it has an $R^2 = 0.266$. As the strength of this correlation is only 26.6%, it is not reliable enough to be used as a predictor for the estimation of the swelling pressure.

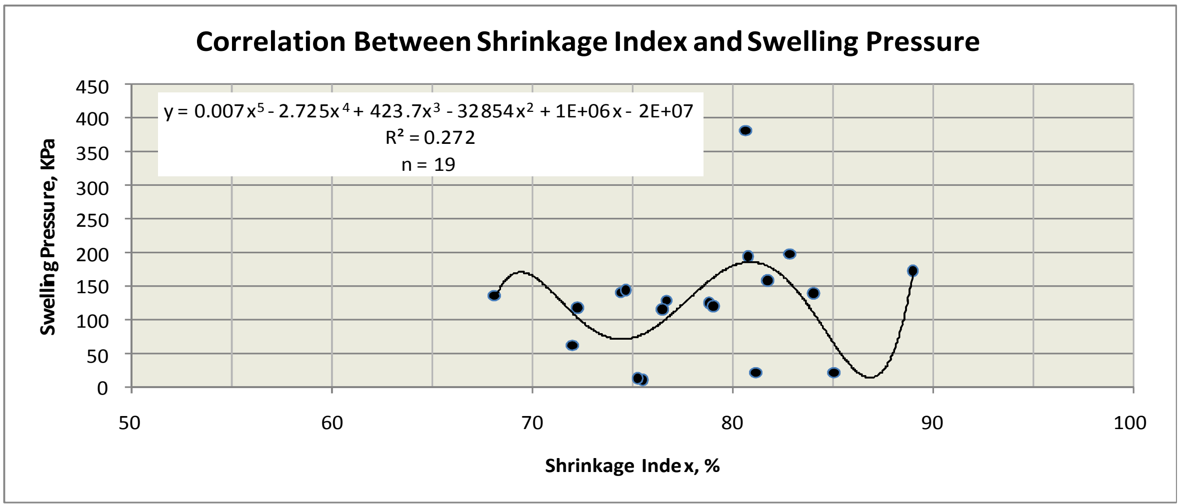

Figure 5.1 Correlation between shrinkage index and swelling pressure

As we see from the above figure (5.1), the relation between swelling pressure and shrinkage index of the study area shows the trend line widely scattered points. This shows that the relationship is weak. Since swelling pressure is a built-in property varying with the dry density of the soil, the amount and type of clay minerals and with the initial moisture content but the shrinkage index is not affected by the above properties. Therefore, the swelling pressure cannot satisfactorily predicted from one soil parameter alone. Hence, to attain a good result we decided to normalize the shrinkage index with natural moisture content.

5.3. Correlation between Swelling Pressure and Normalized Shrinkage Index

The relationship between the swelling pressure and shrinkage index of Addis Ababa expansive soils has been found not to be reliable enough to be used as a predictor. It was decided to investigate by normalizing the shrinkage index with the natural moisture content (shrinkage index divided by natural moisture content). The calculations of the normalized shrinkage index are shown in Table 5.2 with Figure 5.3 showing the resultant graph and trend line.

The trend line for this correlation is an improvement on the previous one. The most appropriate trend line to fit the plotted data is a power function which provides a predicted outcome from the normalized shrinkage index is 85.4% or it has an $R^2 = 0.854$. As the strength of this correlation is 85.4%, it has strong correlations and it can be used as a predictor for the estimation of the swelling pressure.

The equation of this trend line is $\mathbf{S_P = 1.894\ (SI/MC)^{5.294}}$**.** This Equation is valid for estimating swelling pressure. Therefore, the correlations above shall be good enough to estimate swelling pressure for expansive soil found in Addis Ababa based on normalized shrinkage index with natural moisture content for preliminary design.

Table 5.1 Calculation of normalized shrinkage index

Sample No.	Location	Colour	Sample Depth (m)	Natural Moisture Content %	Liquid Limit %	Shrinkage Limit %	Shrinkage Index (LL-SL) %	Swelling Presure kPa	Normalized Shrinkage Index (SI/MC)
S1	Bole	Black	1.2	33.86	90.77	10.04	80.73	382.05	2.38
S2		Grey	2.5	35.67	94.25	11.32	82.93	198.25	2.33
S3	Gerji	Black	1.3	52.25	85.75	10.26	75.49	9.45	1.44
S4	Gurdshola (Egz Ab)	Black	1.5	32.39	91.20	10.46	80.74	192.25	2.49
S5		Grey	2.5	35.21	84.00	11.71	72.29	116.34	2.05
S6	Imperial	Black	1.5	37.43	90.50	11.71	78.79	123.33	2.11
S7		Grey	2.5	32.46	85.00	10.60	74.40	141.31	2.29
S8	CMC (Sumit)	Black	1.5	37.60	96.50	12.40	84.10	138.31	2.24
S9	CMC (Meri)	Black	1.5	34.20	87.20	10.53	76.67	129.32	2.24
S10	Ayat	Black	1.5	35.37	99.75	10.77	88.98	170.28	2.52
S11		Grey	2.5	37.56	88.75	12.22	76.53	114.34	2.04
S12	Hayahulet	Black	1.5	38.80	93.50	11.67	81.83	156.29	2.11
S13		Grey	2.5	31.75	89.20	10.13	79.07	118.33	2.49
S14	Old Airport	Black	1.5	45.23	92.75	11.53	81.22	22.44	1.80
S15	Lafto (Hana)	Black	1.5	31.78	86.50	11.85	74.65	142.31	2.35
S16	Jemo (Haile Garment)	Black	1.5	32.88	80.25	12.10	68.15	136.31	2.07
S17	Mekanisa	Black	1.5	49.51	96.85	11.73	85.12	22.44	1.72
S18	Kality	Black	1.3	56.27	86.25	11.01	75.24	11.91	1.34
S19	Akaki	Brown	1.5	37.39	83.60	11.53	72.07	62.39	1.93

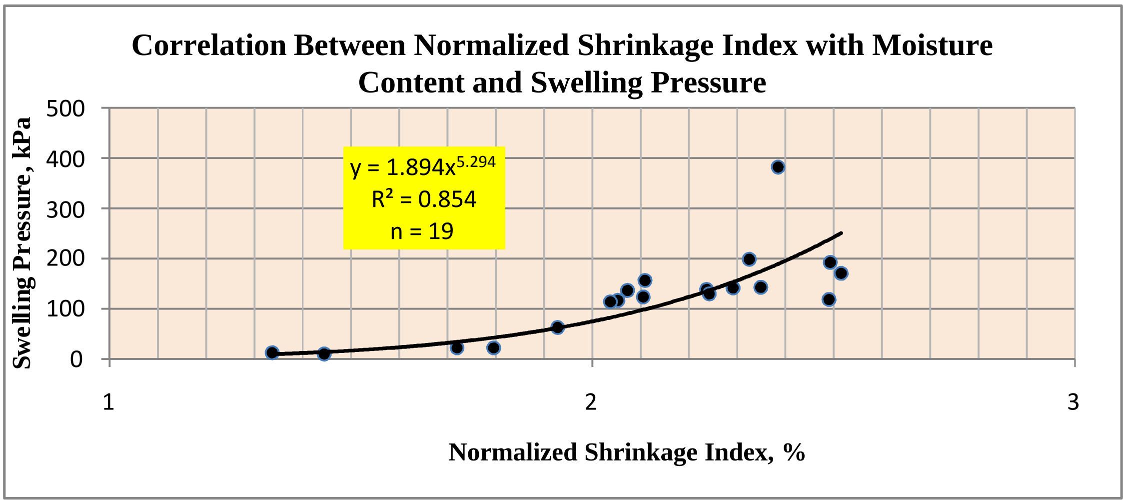

Figure 5.2 Correlation between normalized shrinkage index and swelling pressure

5.4. Improving Correlations

Looking at the spread of the data three points appeared to be an outlier. It was decided to re-calculate the trend line and not include these points, which are consider to be an outlier to determine if this had a major effect on the relationship. The graph of this correlation is shown in Figure 5.4 and with the outlier removed; the strength of the relationship has increased to an R^2 value of 0.902. The strength of the correlation with this outlier included was 0.854. By treating this point as an outlier has improved the correlation substantially. The equation of this trend line is $\mathbf{S_P = 1.623\ (SI/MC)^{5.549}}$ with an R^2 value of 0.902. Therefore, it is most appropriate enough to be used as a predictor for the estimation of the swelling pressure.

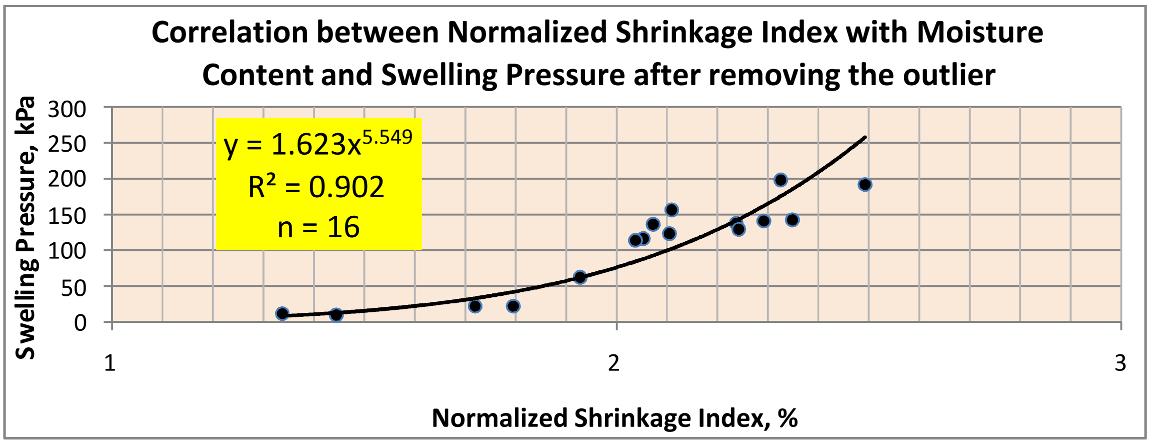

Figure 5.3 Correlation between normalized shrinkage index and swelling pressure after removing the outlier

5.5. Validation of Relationships Developed

The following graph is plotted to examine the results between the measured and predicted values by newly developed formulas. The measured and predicted values are plotted to observe the gap between the measured and the predicted values.

Figure 5.5 shows that for most of the soil samples, the predicted values are comparable to the measured swelling pressure. Therefore, the proposed equations could be used for the prediction of swelling pressure for expansive soil found in Addis Ababa.

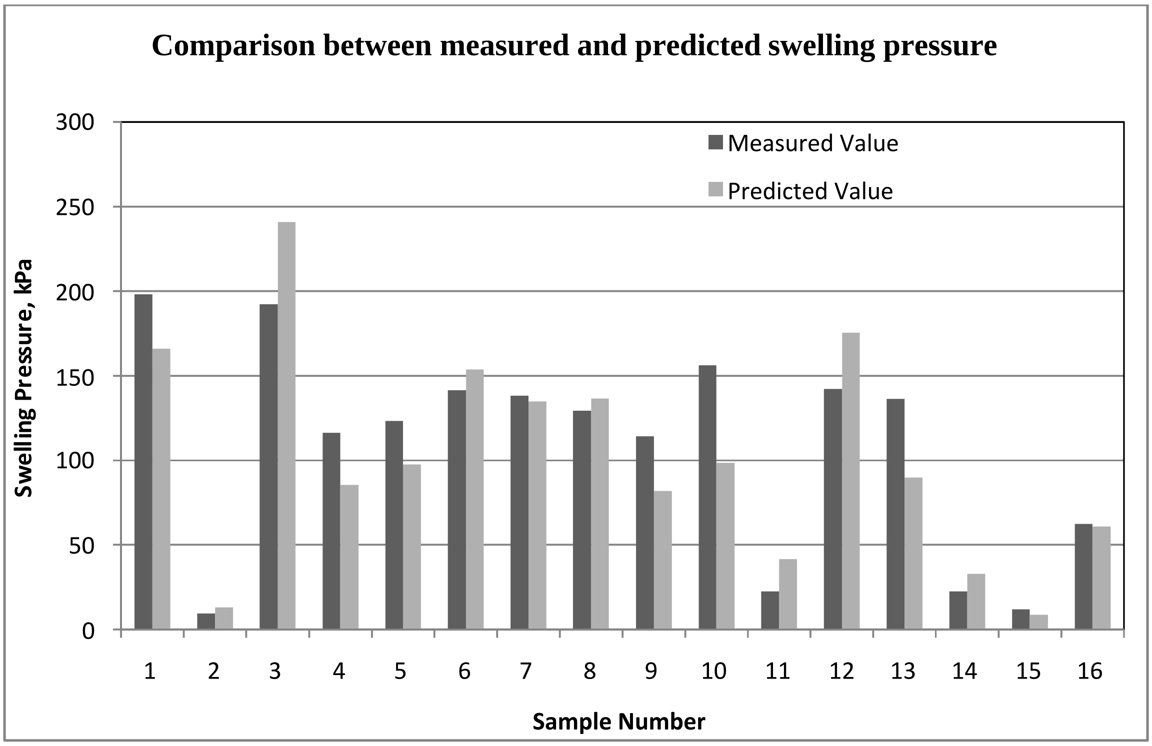

Figure 5.4 Comparison between measured and predicted swelling pressure

6. Evaluations of Existing Empirical Equations and Development of New Empirical Formula

6.1. Evaluations of Existing Empirical Equations

As discussed in the previous chapter 5 the prediction of swelling pressure from one soil parameter alone cannot satisfactorily predict the swelling pressure of the soil. The swelling pressure is dependent on the geology, environmental factors, soil characteristics and many other factors.

In the past, effort had been put in to the research of establishing the relationship between the swelling pressure and index properties of cohesive soil, aiming to simplify the procedures of swelling pressure identification. Oedometer test is a complex, time consuming and expensive test if compared to other soil index property test, it needs to be fully equipped in order to run the test. A large number of undisturbed samples are needed to acquire reliable data and it consumes more times to obtain the precise data. Therefore, there is a need for simple routine tests to be performed on disturbed samples to attain the same purpose.

To achieve this idea, empirical models had been set forwarded by different researchers to predict swelling pressure of a soil. These models comprise different soil parameters in different combinations. Index properties are the widely used parameters in these models because these properties have significance in indicating the swelling behavior of a soil.

As previously illustrated in Chapter two five published swelling pressure empirical equations (from equation 2.1 to equation 2.5) had been evaluated using the soil data obtained from the study area. The results are given in table 6.1.

A comparison is made between the measured and the predicted values developed by various researchers. The graph is plotted to examine the relation between the measured and published correlations.

Table 6.1 Comparison of previously developed equations with the measured value

Sample No.	Location	Colour	Sample Depth (m)	Measured Swelling Presure in Kpa	Proposed Swelling Pressure in kPa Equation 2.1 (Komornik and David)	Equation 2.2 (Vijayvergiya and Ghazzaly)	Equation 2.3 (Nayak and Christensen)	Equation 2.4 (Daniel Teklu)	Equation 2.5 (Daniel Teklu)
S1	Bole	Black	1.2	382.05	223.14	96.64	141.38	408.98	365.93
S2		Grey	2.5	198.25	73.64	72.10	117.75	153.76	102.71
S3	Gerji	Black	1.3	9.45	9.67	28.18	48.26	86.73	125.63
S4	Gurdshola(Egz Ab)	Black	1.5	192.25	219.13	93.83	112.11	391.43	321.62
S5		Grey	2.5	116.34	69.30	39.83	87.78	205.81	146.86
S6	Imperial	Black	1.5	123.33	54.44	53.72	95.59	142.49	100.14
S7		Grey	2.5	141.31	142.95	54.87	109.46	315.64	226.77
S8	CMC (Sumit)	Black	1.5	138.31	136.86	116.14	119.19	274.77	270.24
S9	CMC (Meri)	Black	1.5	129.32	139.43	64.27	96.86	311.89	250.10
S10	Ayat	Black	1.5	170.28	159.77	139.40	145.70	237.93	189.81
S11		Grey	2.5	114.34	34.95	39.66	94.34	106.38	66.68
S12	Hayahulet	Black	1.5	156.29	67.05	72.68	107.23	176.72	153.97
S13		Grey	2.5	118.33	407.08	111.41	96.24	663.70	641.87
S14	Old Airport	Black	1.5	22.44	5.99	26.45	79.03	29.77	17.65
S15	Lafto (Hana)	Black	1.5	142.31	281.71	80.79	135.32	540.23	477.89
S16	Jemo (Haile Garment)	Black	1.5	136.31	65.86	29.65	109.37	194.27	114.36
S17	Mekanisa	Black	1.5	22.44	5.81	36.90	77.74	30.96	24.61
S18	Kality	Black	1.3	11.91	1.12	11.58	37.62	15.86	14.12
S19	Akaki	Brown	1.5	62.39	22.58	24.25	83.87	87.28	49.04

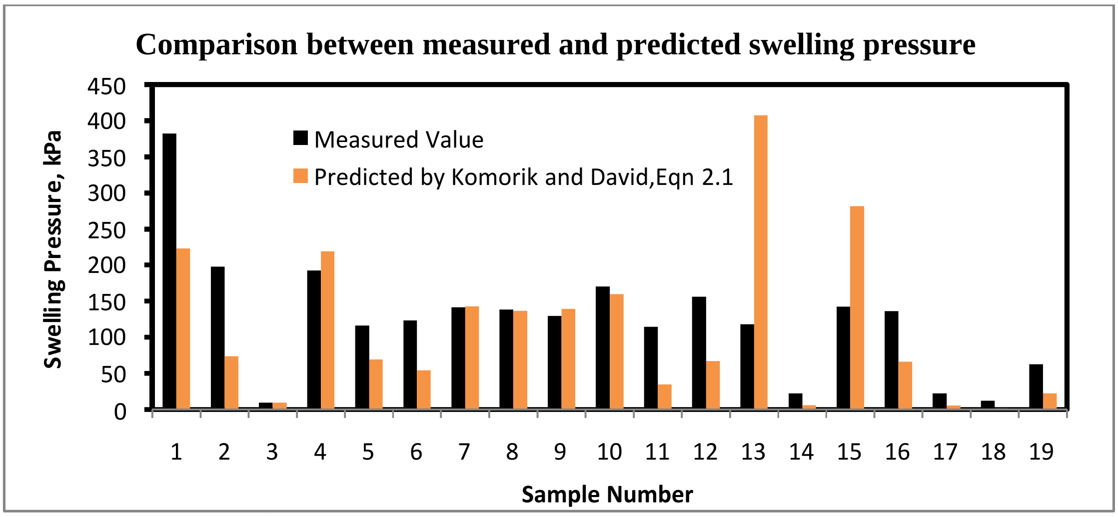

Figure 6.1 Comparison between measured and predicted swelling pressure (Komorik and David, Equation 2.1)

The above figure shows that for most of the soil samples the predicted values are comparable to the measured swelling pressure. Hence, it is suitable to apply as a predictor for the estimation of the swelling pressure of the study area.

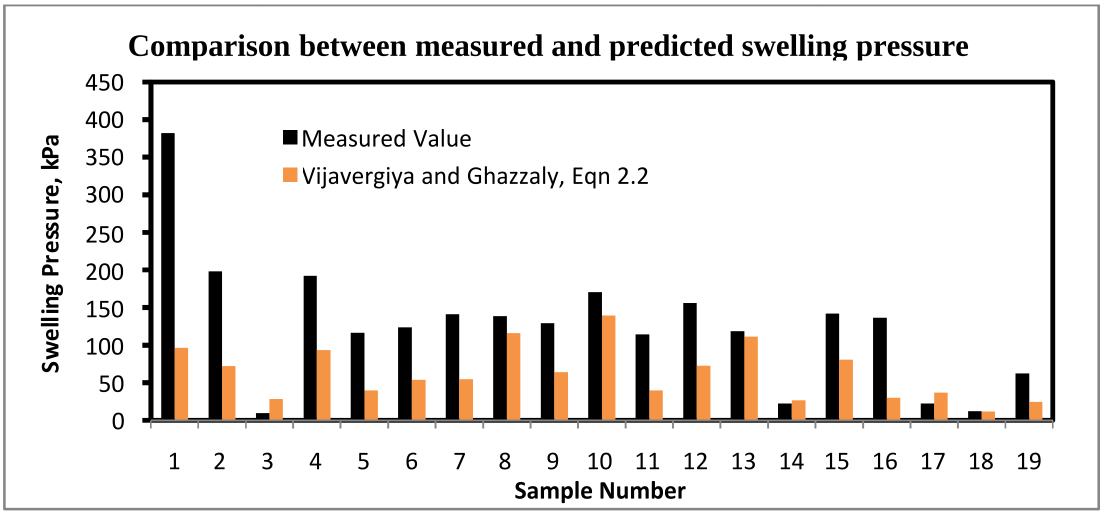

Figure 6.2 Comparison between measured and predicted swelling pressure (Vijayvergiya and Ghazzaly, Equation 2.2)

The above figure shows for most of the soil samples the predicted values are unrelated to the measured swelling pressure. Therefore, it is difficult to apply as a predictor for the estimation of the swelling pressure of the study area.

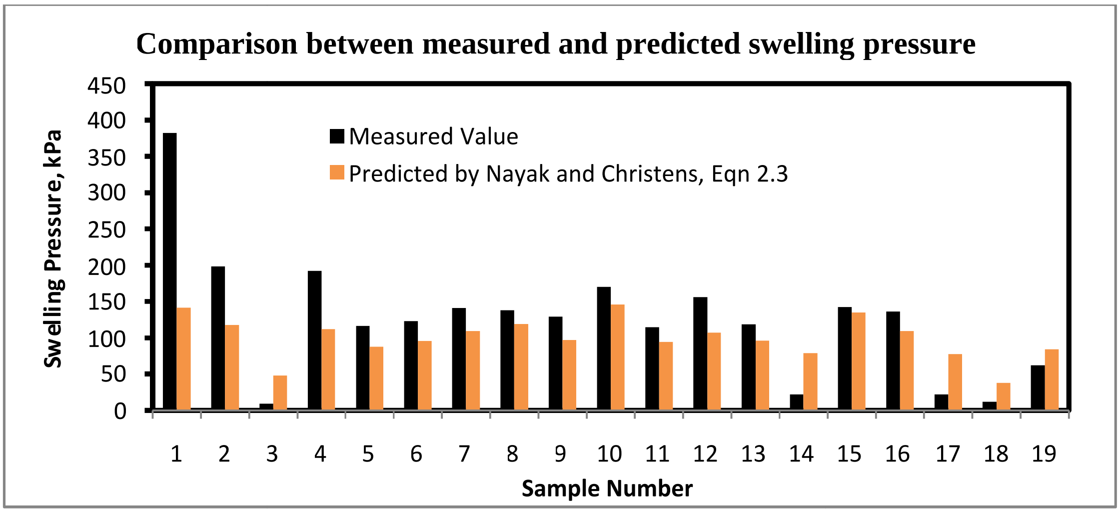

Figure 6.3 Comparison between measured and predicted swelling pressure (Nayak and Christensen, Equation 2.3)

The above figure shows for most of soil samples the predicted values are more comparable to the measured swelling pressure. Hence, it is suitable to apply as a predictor for the estimation of the swelling pressure of the study area.

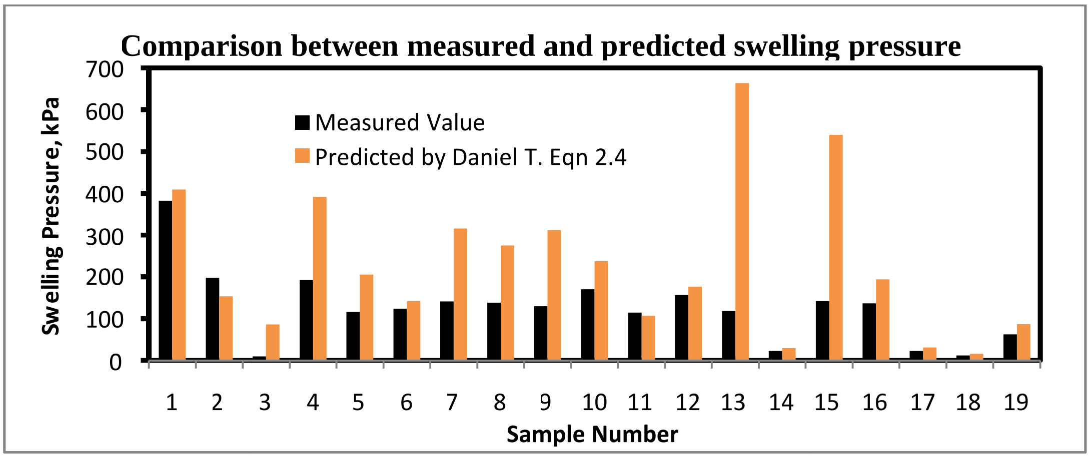

Figure 6.4 Comparison between measured and predicted swelling pressure (Daniel T. Eqn 2.4)

The above figure shows most of the soil samples the predicted values are different from the measured swelling pressure. Therefore, it is not suitable to apply as a predictor for the estimation of the swelling pressure of the study area.

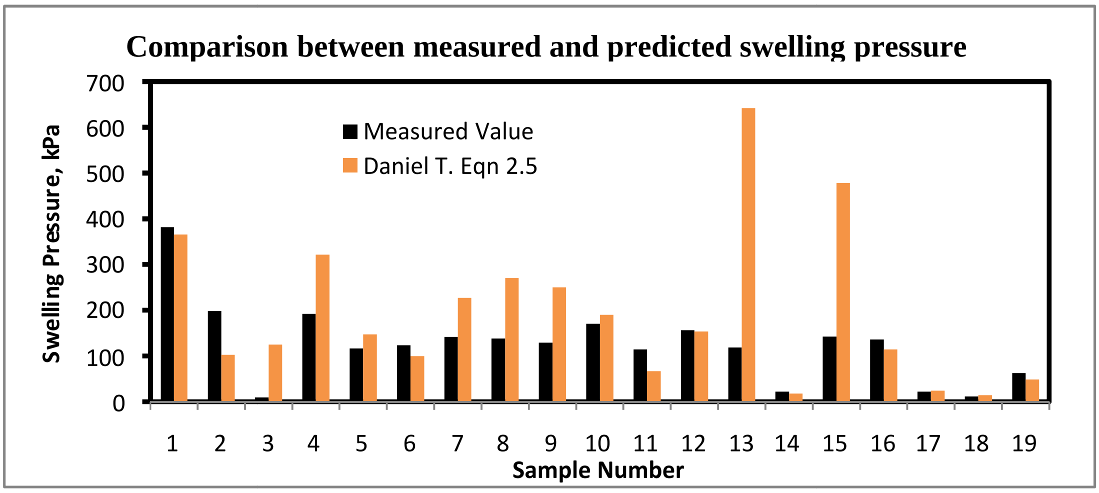

Figure 6.5 Comparison between measured and predicted swelling pressure (Daniel T. Eqn 2.5)

The above figure shows for most of the soil samples the predicted values are moderately compatible to the measured swelling pressure. Therefore, it is poor to apply as a predictor for the estimation of the swelling pressure.

To sum up, the results of the analysis shows that the two equations developed by Komornik and David (1969) and Nayak and Christensen (1974) correlations are more compatible for estimating the swelling pressure. But the correlation developed by Daniel T. (2004) and Vijayvergiya and Ghazzaly (1973) are moderately compatible to be used as for calculating swelling pressure. However, empirical equations are not expected to determine swelling pressure precisely and accurately for all soils. The formation and development of soil structure has very erratic nature and the swell potential is dependent on the geology, environmental factors, soil characteristics and many other factors, which vary from place to place.

6.2. Development of New Empirical Formula

6.2.1. Correlation of Swelling Pressure with Soil Index Properties

Soil index properties can be used to correlate with swelling pressure of expansive soils. This has been shown in many published swelling pressure correlations in the world. However, the published correlations are only reliable for predicting swelling pressure for the type of soil where the correlation is originated. Hence, it is important to find a correlation that can estimate swelling pressure based on soil index properties on expansive soil found in Addis Ababa.

Therefore, new relations are developed, by taking two or more of the five parameters (initial moisture content, dry density, liquid limit, plasticity index and shrinkage index) in different combination. The relations are developed, by taking 20 samples. Multiple linear regressions is made using IBM SPSS 19 for windows software. Equations with higher correlation coefficient were selected and using these equations the swelling pressure of the soil of the study area were calculated. Then a graph is plotted which shows the measured value against the predicted or calculated value. Finally two equations are selected which predicted the measured value better than the other equation. The input and outputs of the software are attached below.

The multiple regression equations take the form:

$$y = a_1 x_1 + a_2 x_2 + \ldots\ldots\ldots\ldots a_n x_n + C \qquad (6.1)$$

Where:

y = dependent variable

X = independent variable

a = regression coefficients

C = constant

The results of the regression analysis are as shown in the following tables. The dependent and independent variables along with the corresponding equations and the multiple correlation coefficients are listed. The multiple correlation coefficient, expressed as "R-squared," measures how well the regression model fits the data. R-squared values near zero are expected for completely random data, whereas, an R-squared value of 1.00 would imply all data to be falling on a straight line, which is the best possible fit. R-squared values for the regression equations are listed below.

The following relations are then developed. The relations are developed by taking two or more of the five soil parameters; natural moisture content, initial dry density, liquid limit, plasticity index and shrinkage index that affects the swelling pressure in different combinations.

Log Sp = 2.386 – 0.056(MC) + 0.068(ρ_d) + 0.018(LL) …………………..…. (6.2)

$R^2 = 0.878$

Log Sp = 3.986 – 0.055(MC) + 0.058(ρ_d) ……………...……………………..(6.3)

$R^2 = 0.834$

Log Sp = 3.668 – 0.058(MC) - 0.315(ρ_d) + 0.016(PI) …………...…….….…. (6.4)

$R^2 = 0.867$

Log Sp = 2.923 – 0.058(MC) - 0.126(ρ_d) + 0.018(SI) …………………….…… (6.5)

$R^2 = 0.877$

Log Sp = 3.007 + 0.015(SI) + 0.003(PI) - 0.168(ρ_d) – 0.058(MC) …………. (6.6)

$R^2 = 0.877$

Log Sp = -0.918 + 1.100(SI/MC) + 0.463(ρ_d) …..…………...……………......(6.7)

$R^2 = 0.833$

Where:

Sp = swelling pressure (kPa)

MC = natural moisture content (%)

ρ_d = initial dry density (gm/cm^3)

LL = liquid limit (%)

PI = plasticity index (%)

SI = shrinkage index (%)

Using the above new equations the swelling pressure of the study area is calculated. The results of the analysis are given in Table 6.2.

Table 6.2 Comparison of predicted value with measured value

Sample No.	Location	Measured Swelling Presure Kpa	Predicted Swelling Pressure					
			Equqtion 6.2	Equqtion 6.3	Equqtion 6.4	Equqtion 6.5	Equqtion 6.6	Equqtion 6.7
S1	Bole	382.05	162.39	157.69	144.85	178.36	158.69	197.75
S2		198.25	146.86	124.18	146.40	156.93	143.74	157.01
S3	Gerji	9.45	12.09	15.12	12.79	12.73	11.76	16.20
S4	Gurdshola(Egz Ab)	192.25	199.55	189.74	244.82	217.68	205.90	257.98
S5		116.34	102.26	132.03	112.98	106.52	98.99	81.09
S6	Imperial	123.33	100.13	99.31	85.32	104.48	93.01	89.83
S7		141.31	152.75	187.86	148.55	166.19	149.76	153.76
S8	CMC (Sumit)	138.31	126.45	97.74	124.89	125.72	116.57	131.15
S9	CMC (Meri)	129.32	133.63	150.63	122.67	144.77	129.41	135.18
S10	Ayat	170.28	192.61	129.48	153.98	207.84	183.19	263.30
S11		114.34	91.25	97.39	91.23	94.12	86.38	73.89
S12	Hayahulet	156.29	95.17	83.60	104.68	98.41	92.28	91.69
S13		118.33	200.86	206.97	191.66	218.43	196.79	268.78
S14	Old Airport	22.44	39.46	36.40	39.39	42.21	38.68	36.12
S15	Lafto (Hana)	142.31	178.47	205.76	193.52	181.95	169.34	184.78
S16	Jemo (Haile Garment)	136.31	118.24	177.36	125.14	122.54	112.67	85.15
S17	Mekanisa	22.44	26.94	21.17	22.98	27.99	25.12	29.84
S18	Kality	11.91	7.21	8.94	7.84	7.64	7.09	10.78
S19	Akaki	62.39	75.19	99.33	84.17	80.33	74.54	55.14

6.2.2. Validation of Relationships of the Measured and Calculated Values

The following graphs are plotted to investigate the approximation accuracy of the newly developed formulas. The measured and calculated values are plotted (Figure 6.2 to Figure 6.7).

As seen from the graphs, Equations 6.2, 6.5, and 6.6 gave a better estimation of the measured swelling pressure with the coefficient of determination of 0.878, 0.877, and 0.877 respectively. Among the three equations, equation 6.2 describes the relation better than the others. Thus, one may use these suggested equations for the estimation of the swelling pressure of the study area alternatively.

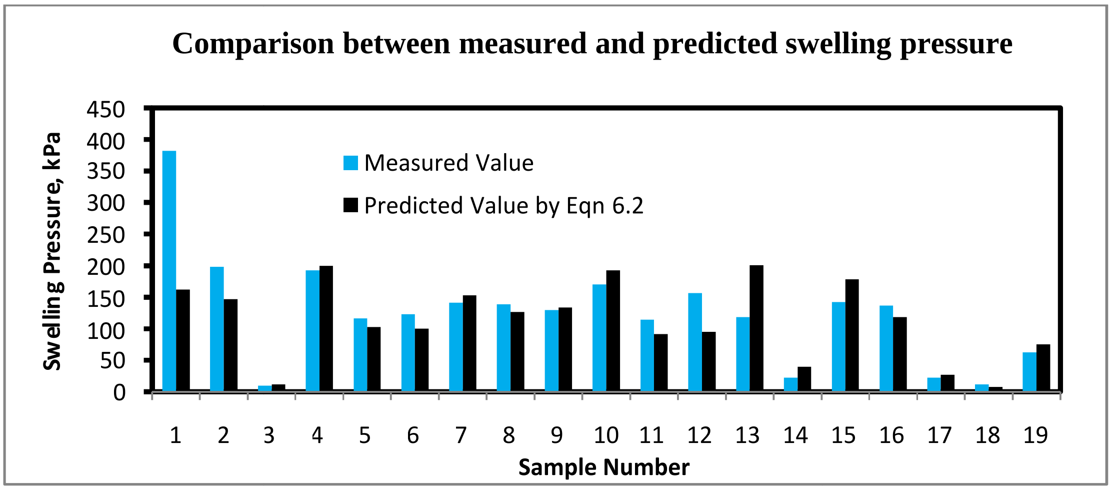

Figure 6.6 Comparison between measured and predicted swelling pressure (Eqn 6.2)

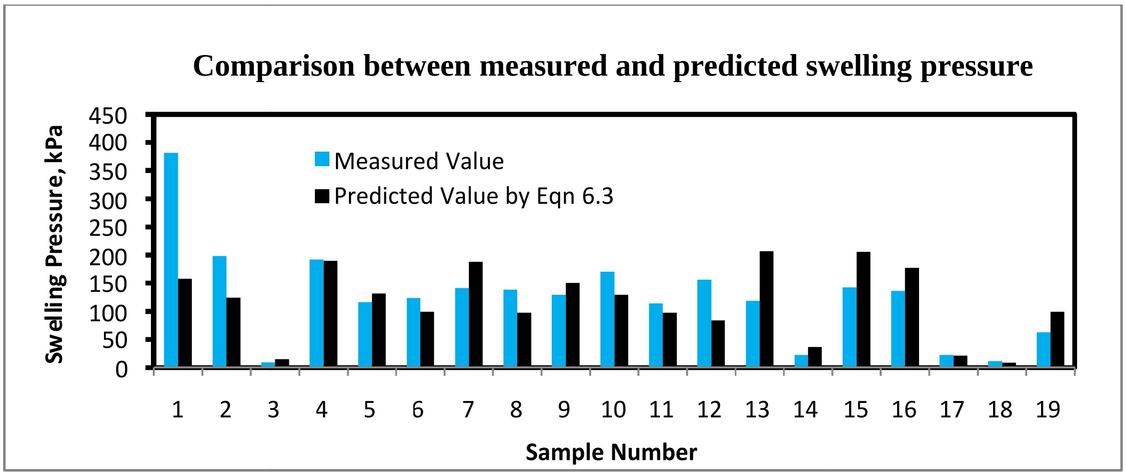

Figure 6.7 Comparison between measured and predicted swelling pressure (Eqn 6. 3)

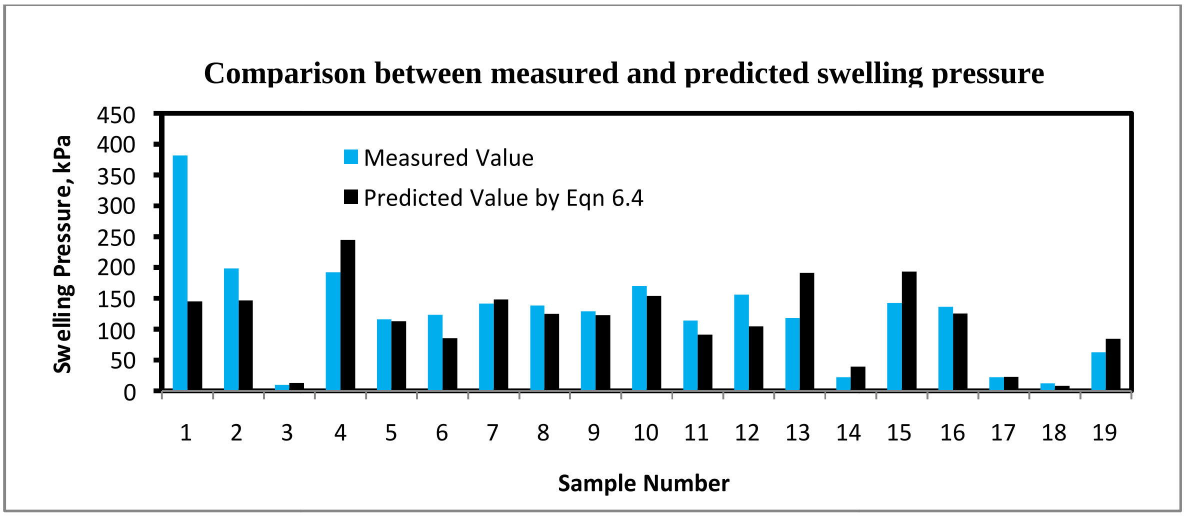

Figure 6.8 Comparison between measured and predicted swelling pressure (Eqn 6. 4)

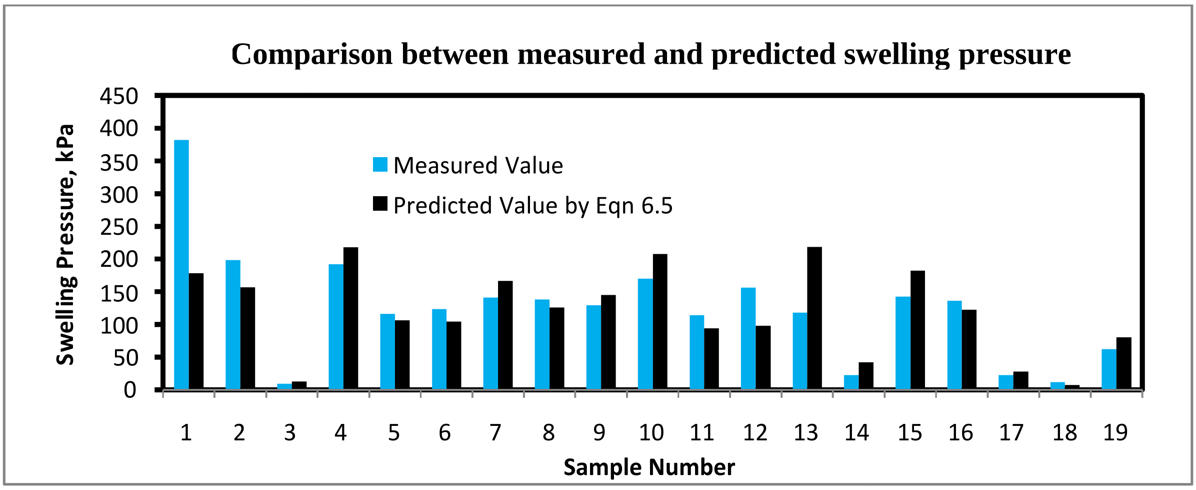

Figure 6.9 Comparison between measured and predicted swelling pressure (Eqn 6.5)

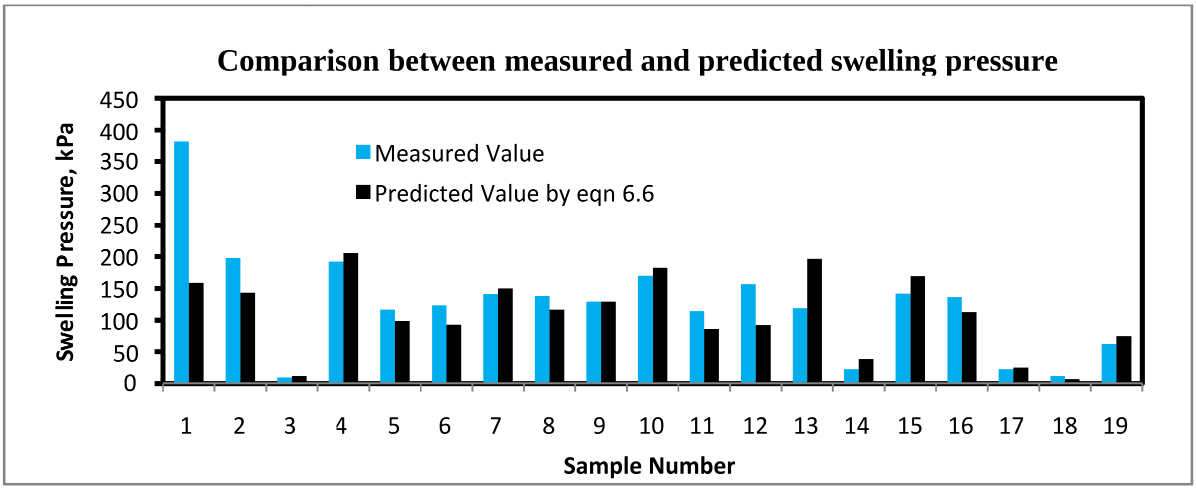

Figure 6.10 Comparison between measured and predicted swelling pressure (Eqn 6.6)

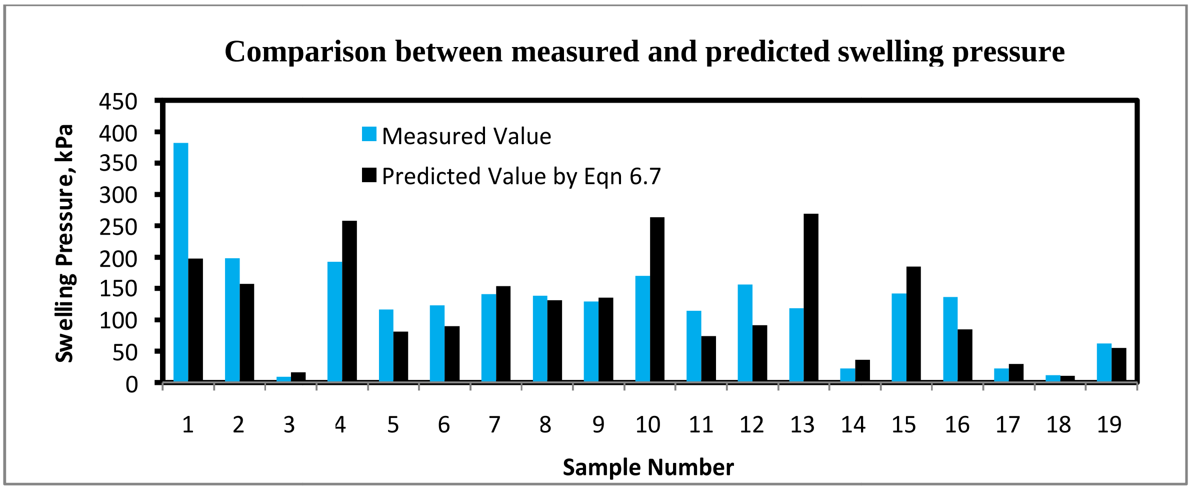

Figure 6.11 Comparison between measured and predicted swelling pressure (Eqn 6.7)

7. Conclusion and Recommendations

7.1. Conclusions

The following points can be concluded from the test result and regression analysis:

- The relationship between the swelling pressure and the shrinkage index is not reliable enough to be used as a predictor for the estimation of the swelling pressure. Since swelling pressure is a built-in property varying with the dry density of the soil, the amount and type of clay minerals, and with the initial moisture content whereas the shrinkage index is not affected by the above properties.
- The correlation between swelling pressure and normalized shrinkage index is appropriate to estimate swelling pressure. The equation of this trend line is $\mathbf{S_P = 1.623\,(SI/MC)^{5.549}}$.
- One soil parameter alone cannot satisfactorily indicate the swelling pressure of the soil.
- The equation developed by David and Komornik (1969) and Nayak and Christensen (1974) are more compatible for estimating the swelling pressure. But the correlation developed by Daniel Teklu (2004) and Vijayvergiya and Ghazzaly (1973) are moderately compatible to be used as for calculating swelling pressure.
- The results showed that some of the commonly measured soil properties such as initial moisture content, initial dry density and liquid limit used in combination to lead to reliable estimates of the swelling pressure.
- Equations 6.2 gave a better estimation of the measured swelling pressure with the coefficient of regression (R^2) value of 0.878. The equation of this developed formula is $\mathbf{Log\ Sp = 2.386 - 0.056(MC) + 0.068(\rho_d) + 0.018(LL)}$.
- A good approximation is also obtained by a model consisted of four input parameters moisture content, dry density, plasticity index and shrinkage index.

7.2. Recommendations

Further research is recommended to support the conclusions made on this research. Recommended for future research are as follows:

- All the developed formulas obtained in this study are based only on data obtained from 20 samples. Therefore, adding more samples to the dataset can result in improved prediction ability and a better representation. As more data are added, these correlations should be updated.

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Appendices

Appendix A: Some Examples of Swelling Pressure Test Results

Swelling Pressure Test Parameters and Formula used		
Ring height =		2.0 cm
Ring diameter =		7.5 cm
Cross sectional area (cm^2) =		44.17865
Volume (cm^3) =		88.35729
Sp = (M/A)*L*98.07		
Where :	Sp = Swelling Pressure in kPa	
	M = load on hanger in kg	
	A = Cross Sectional Area in cm^3	
	L = Lever Arm Multiplier = 9	
Project :- M.Sc. Thesis		
Location :- Bole (Infront of Millinium Hall)		
Sample No.:- S1		
Sample Type:- Undisturbed		
Test Type :- Swelling Pressure		
Test Date :- 19/02/2011		
Load on hanger = 19123 gm = 19.123 kg		
Sp =(M/A)*L*98.07= **382.0518** kPa		

Swelling Pressure Test Parameters and Formula used		
Ring height =		2.0 cm
Ring diameter =		7.5 cm
Cross sectional area (cm^2) =		44.17865
Volume (cm^3) =		88.35729
Sp = (M/A)*L*98.07		
Where :	Sp = Swelling Pressure in kPa	
	M = load on hanger in kg	
	A = Cross Sectional Area in cm^3	
	L = Lever Arm Multiplier = 9	
Project :- M.Sc. Thesis		
Location :- Lafto (Infront of Revera Hotel)		
Sample No.:- S16 (Black)		
Sample Type:- Undisturbed		
Test Type :- Swelling Pressure		
Test Date :- 12/02/2011		
Load on hanger = 7123 gm = 7.123 kg		
Sp =(M/A)*L*98.07=	**142.308**	kPa

Appendix B: Some Examples of Volumetric Shrinkage Test Results

Test Type : Volumetric Shrinkage Test

Sample Location :- Bole (In front of Millennium Hall) Black

Sample No. S1

Tested Date 02/03/2011

Can No.	C-5	27
Weight of can in g (W_1)	19.734	24.636
Weight of can + wet soil in g (W_2)	42.083	93.637
Weight of wet soil in g (W)	22.349	69.001
Weight of can + dry soil in g (W_3)	31.311	60.322
Weight of dry soil (W_3-W_1) = Ws	11.577	35.686
Weight of can + Mercury (g)	226.500	661.500
Weight of Mercury (g)	206.766	636.864
Volume of the wet soil in cm^3 (V_1)	15.282	47.071
Weight of displaced mercury in g	76.440	235.500
Volume of the dry soil in cm^3 (V_2)	5.650	17.406
Shrinkage Limit = $(((W-W_s)-\rho_w(V_1-V_2))/Ws)$ in %	9.849	10.228
Average Shrinkage Limit (%)	10.038	

Test Type : Volumetric Shrinkage Test

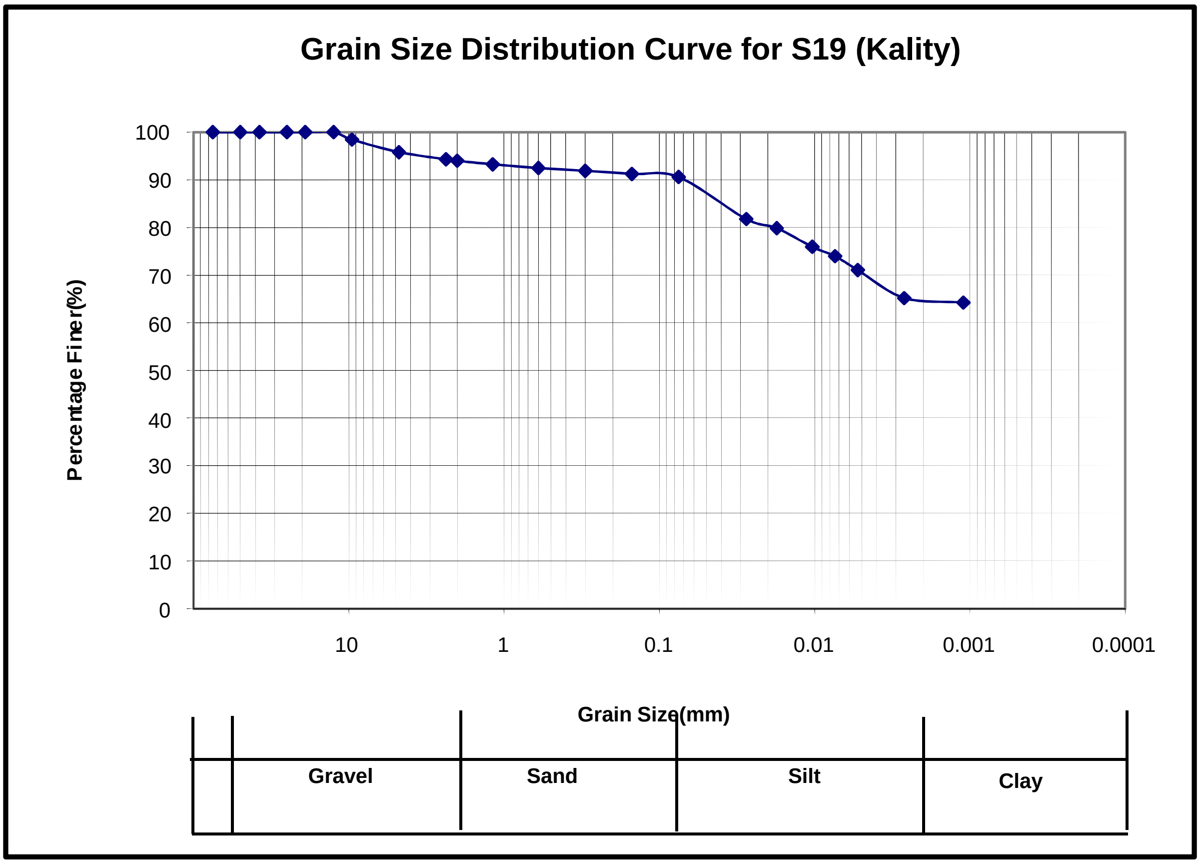
Sample Location :- Hana (Infront of Revera Hotel)

Sample No. S16

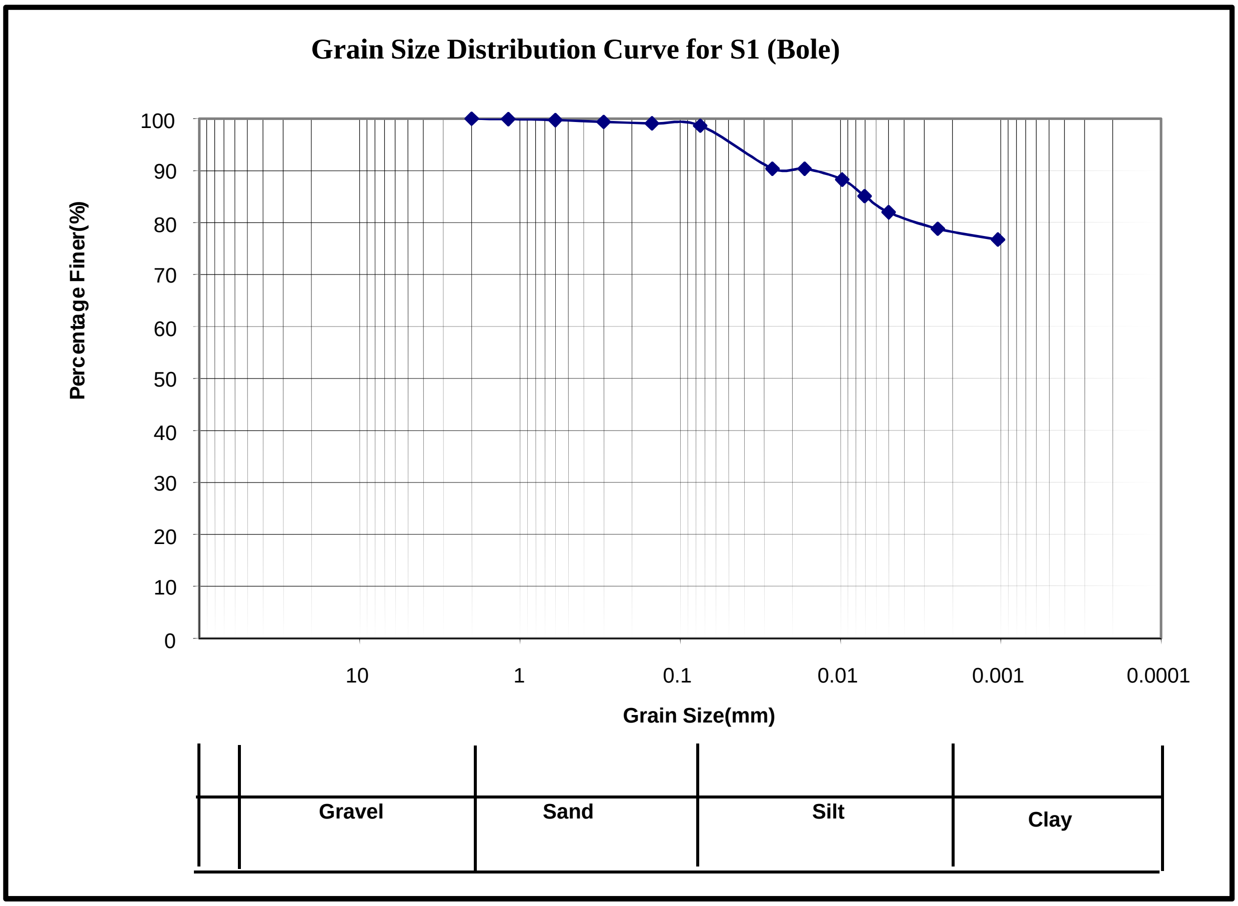
Tested Date 01/03/2011

Can No.	A	D
Weight of can in g (W_1)	24.395	24.201
Weight of can + wet soil in g (W_2)	92.869	93.343
Weight of wet soil in g (W)	68.474	69.142
Weight of can + dry soil in g (W_3)	60.957	61.220
Weight of dry soil (W_3-W_1) = Ws	36.562	37.019
Weight of can + Mercury (g)	642.500	641.500
Weight of Mercury (g)	618.105	617.299
Volume of the wet soil in cm^3 (V_1)	45.684	45.624
Weight of displaced mercury in g	243.000	244.000
Volume of the dry soil in cm^3 (V_2)	17.960	18.034
Shrinkage Limit = $(((W-W_s)-\rho_w(V_1-V_2))/Ws)$ in %	11.456	12.244
Average Shrinkage Limit (%)	11.850	

Appendix C: Some Examples of Grain Size Distribution Curves



Clay Fraction	65.10%
Silt Fraction	25.51%
Sand Fraction	3.41%
Gravel Fraction	5.98%



Clay Fraction	78.50%
Silt Fraction	20.20%
Sand Fraction	1.30%
Gravel Fraction	0.00%

Appendix D: Some Examples of Atterberg Limits Test Results

Sample No S11(Black)

Sample type : Disturbed

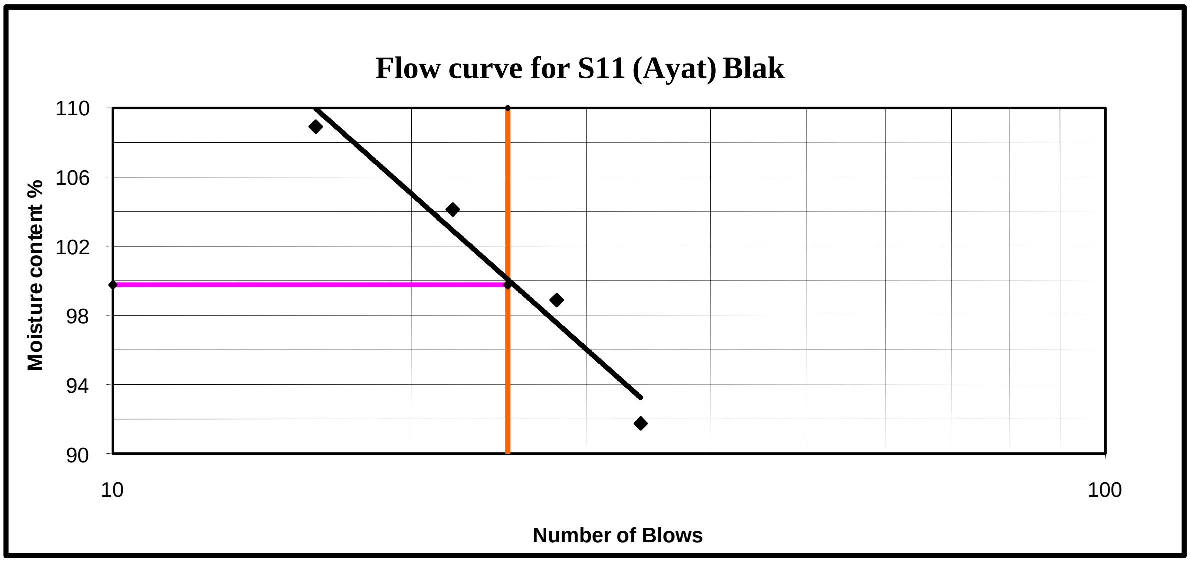
Test type : Atterberg Limit

Date 17/02/2011

Location : Ayat near to Ayat square

Depth(m): 1.5

Type of test	Liquid Limit				Plastic Limit	
Can No.	F11	11	F16	14	129	L
No. of Blows	34	28	22	16		
Wt. of wet sample + Can	35.64	41.52	37.42	39.15	30.940	29.360
Wt. of dry sample + Can	26.43	29.21	26.78	27.17	26.890	25.710
Wt. of water	9.21	12.31	10.64	11.98	4.050	3.650
Wt. of Can	16.39	16.76	16.56	16.17	16.800	16.450
Wt. of dry soil	10.04	12.45	10.22	11.00	10.090	9.260
Water content %	91.73	98.88	104.11	108.91	40.139	39.417
Avg. Water content %					39.778	



Results

LL =	99.75%
PL =	39.78%

Sample No S1(Black)

Sample type : Disturbed

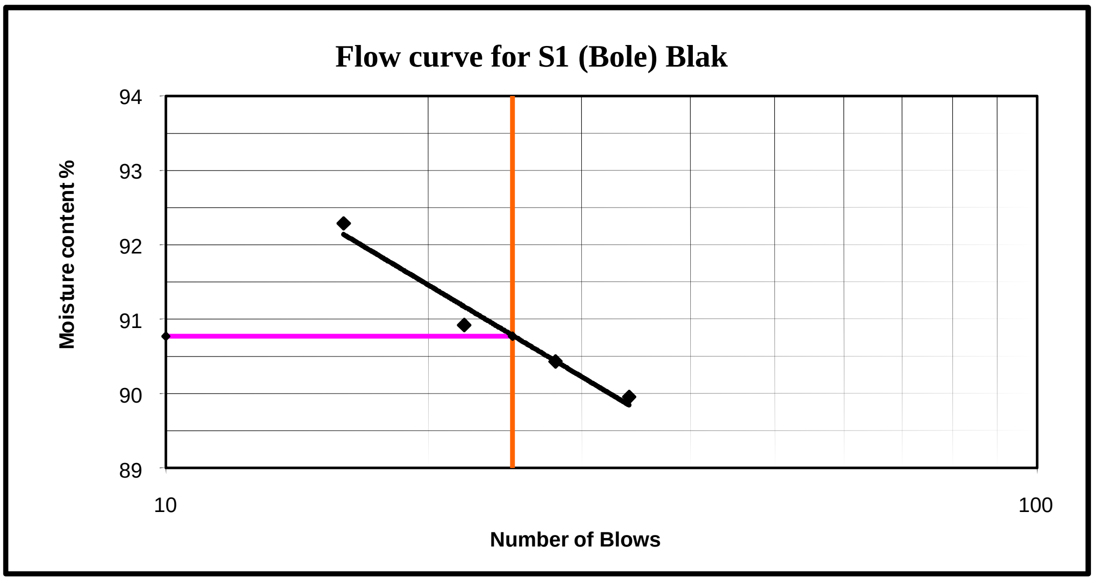
Test type : Atterberg Limit

Date 17/02/2011

Location : Bole in front of Millennium hall

Depth(m): 1.2

Type of test	Liquid Limit				Plastic Limit	
Can No.	66	92	16	C	M12	P5
No. of Blows	34	28	22	16		
Wt. of wet sample + Can	36.72	36.98	32.58	32.20	39.34	39.21
Wt. of dry sample + Can	27.05	27.15	24.87	21.31	33.11	33.03
Wt. of water	9.67	9.83	7.71	10.89	6.23	6.18
Wt. of Can	16.30	16.28	16.39	9.51	16.33	16.33
Wt. of dry soil	10.75	10.87	8.48	11.80	16.78	16.70
Water content %	89.95	90.43	90.92	92.29	37.13	37.01
Avg. Water content %					37.07	



Results

LL =	90.77%
PL =	37.07%

Appendix E: Inputs and Outputs of the SPSS 19 Software

Regression analysis for Equation 6.2

Variables Entered/Removed[b]

Model	Variables Entered	Variables Removed	Method
1	LL(%), Dry d (g/cm3), MC(%)	.	Enter

a. All requested variables entered.

b. Dependent Variable: Log SP(kPa)

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.937[a]	.878	.854	.1675574753

a. Predictors: (Constant), LL(%), Dry d (g/cm3), MC(%)

ANOVA[b]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	3.042	3	1.014	36.120	.000[a]
	Residual	.421	15	.028		
	Total	3.463	18			

a. Predictors: (Constant), LL(%), Dry d (g/cm3), MC(%)

b. Dependent Variable: Log SP(kPa)

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	2.386	1.884		1.266	.225
	MC(%)	-.056	.012	-.923	-4.809	.000
	Dry d (g/cm3)	.068	1.119	.012	.061	.952
	LL(%)	.018	.008	.213	2.351	.033

a. Dependent Variable: Log SP(kPa)

Regression analysis for Equation 6.3

Variables Entered/Removed[b]

Model	Variables Entered	Variables Removed	Method
1	MC(%), Dry d (g/cm3)	.	Enter

a. All requested variables entered.

b. Dependent Variable: Log SP(kPa)

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.913[a]	.834	.813	.1897929321

a. Predictors: (Constant), MC(%), Dry d (g/cm3)

ANOVA[b]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	2.887	2	1.444	40.075	.000[a]
	Residual	.576	16	.036		
	Total	3.463	18			

a. Predictors: (Constant), MC(%), Dry d (g/cm3)

b. Dependent Variable: Log SP(kPa)

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	3.986	1.990		2.003	.062
	Dry d (g/cm3)	.058	1.268	.010	.046	.964
	MC(%)	-.055	.013	-.904	-4.165	.001

a. Dependent Variable: Log SP(kPa)

Regression analysis for Equation 6.4

Variables Entered/Removed[b]

Model	Variables Entered	Variables Removed	Method
1	PI(%), MC(%), Dry d (g/cm3)	.	Enter

a. All requested variables entered.

b. Dependent Variable: Log SP(kPa)

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.931[a]	.867	.841	.1751404595

a. Predictors: (Constant), PI(%), MC(%), Dry d (g/cm3)

ANOVA[b]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	3.003	3	1.001	32.637	.000[a]
	Residual	.460	15	.031		
	Total	3.463	18			

a. Predictors: (Constant), PI(%), MC(%), Dry d (g/cm3)

b. Dependent Variable: Log SP(kPa)

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	3.668	1.844		1.989	.065
	Dry d (g/cm3)	-.315	1.186	-.054	-.266	.794
	MC(%)	-.058	.012	-.957	-4.734	.000
	PI(%)	.016	.008	.186	1.947	.071

a. Dependent Variable: Log SP(kPa)

Regression analysis for Equation 6.5

Variables Entered/Removed[b]

Model	Variables Entered	Variables Removed	Method
1	SI(%), Dry d (g/cm3), MC(%)	.	Enter

a. All requested variables entered.

b. Dependent Variable: Log SP(kPa)

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.936[a]	.877	.852	.1687715272

a. Predictors: (Constant), SI(%), Dry d (g/cm3), MC(%)

ANOVA[b]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	3.036	3	1.012	35.531	.000[a]
	Residual	.427	15	.028		
	Total	3.463	18			

a. Predictors: (Constant), SI(%), Dry d (g/cm3), MC(%)

b. Dependent Variable: Log SP(kPa)

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	2.923	1.830		1.598	.131
	Dry d (g/cm3)	-.126	1.130	-.022	-.111	.913
	MC(%)	-.058	.012	-.951	-4.897	.000
	SI(%)	.018	.008	.209	2.288	.037

a. Dependent Variable: Log SP(kPa)

Regression analysis for Equation 6.6

Variables Entered/Removed[b]

Model	Variables Entered	Variables Removed	Method
1	PI(%), MC(%), SI(%), Dry d (g/cm3)	.	Enter

a. All requested variables entered.

b. Dependent Variable: Log SP(kPa)

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.936[a]	.877	.842	.1744657771

a. Predictors: (Constant), PI(%), MC(%), SI(%), Dry d (g/cm3)

ANOVA[b]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	3.037	4	.759	24.946	.000[a]
	Residual	.426	14	.030		
	Total	3.463	18			

a. Predictors: (Constant), PI(%), MC(%), SI(%), Dry d (g/cm3)

b. Dependent Variable: Log SP(kPa)

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	3.007	1.941		1.549	.144
	MC(%)	-.058	.012	-.954	-4.736	.000
	SI(%)	.015	.015	.181	1.057	.309
	Dry d (g/cm3)	-.168	1.189	-.029	-.141	.890
	PI(%)	.003	.015	.033	.192	.851

a. Dependent Variable: Log SP(kPa)

Regression analysis for Equation 6.7

Variables Entered/Removed[b]

Model	Variables Entered	Variables Removed	Method
1	SI/MC, Dry d (g/cm3)	.	Enter

a. All requested variables entered.

b. Dependent Variable: Log SP(kPa)

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.913[a]	.833	.812	.1901840565

a. Predictors: (Constant), SI/MC, Dry d (g/cm3)

ANOVA[b]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	2.885	2	1.442	39.877	.000[a]
	Residual	.579	16	.036		
	Total	3.463	18			

a. Predictors: (Constant), SI/MC, Dry d (g/cm3)

b. Dependent Variable: Log SP(kPa)

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	-.918	.992		-.925	.369
	Dry d (g/cm3)	.463	1.187	.079	.390	.702
	SI/MC	1.100	.265	.843	4.148	.001

a. Dependent Variable: Log SP(kPa)

Declaration

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

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Signature:________________________

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Addis Ababa

Date of submission: December 2011

I certify that this thesis satisfies all the requirements as a thesis for the degree of Master of Science in Civil Engineering.

Signature: ________________________

Name of Advisor: Dr. Hadush Seged

Date: December 2011