ADDIS ABABA UNIVERSITY

(Addis Ababa Institute of Technology (AAIT))

SCHOOL OF GRADUATE STUDIES
FACULTY OF TECHNOLOGY
DEPARTEMENT OF CIVIL ENGINEERING

Major: Geotechnical Engineering

Correlation between Index Properties and Swelling Characteristic of Expansive Soil
(The case of Asella town)

A Thesis Submitted to the School of Graduate Studies of Addis Ababa University in Partial fulfillment of the Requirement of the Degree of Master of Science in Civil Engineering (Geotechnical Engineering)

Advisor: Samuel Tadesse (Dr.Ing)
By: Abraham Mengistie

July/2014

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By

Abraham Mengistie

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DECLARATION

I, the undersigned, declare that this thesis is my original work performed under the supervision of my research advisor 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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LIST OF SYMBOLS

SP-Swelling pressure

w, wi-Initial moisture content

γd-Initial dry density

LL,wl-Liquid limit

PL-Plastic limit

PI,IP-Plasticity index

SL-Shrinkage limit

CEC- Cation Exchange Capacity

FS-Free swell

Cc-Clay content

Gs-Specific gravity

A-Activity

LI-Liquidity index

CVC- constant volume change

PVC-Potential Volume Change

ABSTRACT

In some parts of Ethiopia large surface deposits are covered by expansive soils. Such soils have the characteristics of swelling and shrinkage up on change in water content with seasonal variation. These swelling and shrinkage are generally confined to the upper portions of the soil deposit and cause considerable damage to lighter structure such as small buildings and highway pavements if not adequately taken care of. This damage is due to development of swelling pressure when expansive soils come in contact with water. Therefore, main purpose of this research is to determine the swelling pressure of these soils and also to develop an equation for quick prediction of swelling pressure from easily determined soil properties for this research area.

In order to examine the relationship between index properties and swelling characteristic of the soil, both undisturbed and disturbed soil samples were obtained from seventeen different test pits in different locations at depth of 1.5 – 2.0m. Then laboratory tests including swelling pressure tests, mineralogical identification tests and index property tests were conducted. After having all the test results, new correlations of swelling pressure have been carried out by stastical analysis using linear regression analysis method (SPSS 20 Computer program) for prediction of swelling pressure using liquid limit, plasticity index, shrinkage limit, initial moisture content, free swell index, liquidity index and initial dry density of the soil.

Soils of the studied area are identified as moderately to high swelling expansive soils containing high plastic clay and high compressible silt particle. The analyses carried out have confirmed the existence of strong correlations between the swell pressure and its index properties. The correlations revealed a simple regression equation for a quick prediction of swell pressures from easily determined soil properties.

CHAPTER 1

INTRODUCTION

1.1 General

Expansive soils are clayey minerals which exhibit significant volume change when subjected to moisture variations. Expansive soils swells if its moisture content increases and shrinks when its moisture content decreases. Magnitude of expansion depends upon the kind and amount of clay minerals present, their exchangeable ions, electrolyte content of the aqueous phase, and the internal structure. The three most important groups of clay minerals are Montmorillonite, Illite and Kaolinite. Montmorillonite is the clay mineral that presents most of the expansive soil problems [1].

On drying expansive soils crack very badly and if prevented from swelling following exposure to moisture, the soils exert high swelling pressure. The pressure build up is usually responsible for cracking of buildings, distortion of pavement surfaces and damage to other structures [12].

The problem of expansive soil is widespread throughout the five contents. In the undeveloped nations, many of the expansive soil problems may not have been recognized. It is to be expected that more expansive soil regions will be discovered each year as the amount of construction increases.

Ethiopia is one of the countries in Africa in which expansive soils have been reported. The Ethiopian black soils are residual soils, derived from the weathering of basic volcanic rocks and are invariably clays or silty clays [1].

Many researchers have shown that substantial damage has been occurring in Ethiopia on buildings and roads that are constructed on expansive soils. Sisay A., 2004 and Sime A., 2006 are among many researchers that have found out damage of structures founded on expansive soils.

In ordinary soils, settlement or shear strength is important in the design of foundation, in black cotton soils, heave or swelling pressure becomes important. Determining the swelling pressure requires undisturbed or remolded samples, which involves time and money, and also complicated laboratory procedures. Therefore, it is desirable to find simpler and quicker methods of testing, using the data of which the swelling pressure of expansive soils can be predicted satisfactorily especially for preliminary design purposes.

This can be achieved by investigating the correlation between index properties and swelling characteristics of expansive soil and by developing equations to predict swelling pressure from index tests of soils.

1.2 Objectives of the Study

1.2.1 General Objective

The general objective of this research is to determine the swelling characteristics of the expansive soils in the selected study area without the requirement of undisturbed or remolded sample and elaborate laboratory procedures. This can be achieved simply by developing correlation between index properties of the soils with its swelling characteristics particularly with its swelling pressure of soil samples taken from different places of the study area. Determining swelling pressure using this correlation will provide information for design consideration and construction precautions for any civil engineering structures to be constructed in this area and it can also be used to indicate problematic soils that should be tested further.

1.2.2 Specific Objective

- To evaluate the geotechnical properties and swelling characteristics of the soil in the study area.
- To study the mineralogical composition of the clay soils in the area.
- To study and classify the degree of expansiveness of the soil using different classification systems
- To develop correlation between index properties of soil and swelling characteristics of expansive soil that is best suited for the type of soils in the specified study area

1.3 Research Methodology

1.3.1 Sampling and Laboratory Tests

After identification of location of expansive soil; both undisturbed and disturbed soil samples are obtained from seventeen different test pits in different locations a depth of 1.5 – 2.0m. Undisturbed samples are taken using sampling tube. Undisturbed samples are kept to water content change and vibration. Swelling pressure and index tests are carried out in in the laboratory of Addis Ababa Institute of technology while CEC and exchangeable ions tests are performed in Debireziet Agricultural research center soil and water testing laboratory.

1.3.2 Data Analysis

Multiple regression analyses using a computer program (SPSS20) are carried out to correlate the logarithm of the measured swell pressures to the soil index properties. The analyses with high coefficients of multiple determinations are selected. Equations are derived for swelling pressure which is applicable to the study area and then comparison is done between the experimental results and calculated results. Finally, based on the developed correlation overall conclusions are made on the result obtained.

1.4 Scope of the Study

This research addresses the objectives stated by conducting detail laboratory tests on disturbed and undisturbed soil samples of the selected area. These laboratory tests includes: Index properties tests, swelling pressure test and mineralogical identification tests.

After conducting all necessary laboratory test; new correlations between index properties and swelling pressure is developed by statistical analysis using linear regression analysis method. Further, it tries to evaluate whether previously developed models for soils in other area by other researchers can predict the swelling pressure of the soil in the selected study area or not.

1.5 Organization of the Thesis

The thesis is organized in seven Chapters. Chapter one introduces expansive soils, objective of the study, research methodology and scope of the study. .

Chapter two reviews origin of expansive soils in relation with conditions or processes which determine clay mineralogy. Field identification methods, laboratory identification tests and classification schemes are presented in this chapter. Clay minerals with respect to their structural units, formation, classification, swelling characteristics and factors responsible for shrink-swell phenomenon are also discussed in this chapter.

In Chapter three, general overview related to location and foundation of the Asella town is given. Topography, soil, climatic characteristics and rainfall of the study area are also described in this chapter

Sampling, laboratory tests and their corresponding standard method of testing, discussions on different soil tests and values of test results are presented in tables in Chapter four.

In Chapter five, discussion of test results, classification of soil of the study area using different classification methods and relationship between different soil properties of the soil is graphically presented in this chapter.

Chapter six presents development of regression models, an overview of previous investigation, comparison between previously developed equations with measured swelling pressure values, selection of better equations and comparison of calculated and measured values. Finally in Chapter seven, conclusions and recommendations are given.

CHAPTER 2
LITERATURE REVIEW

2.1 General

Expansive soils are clay soils with high plasticity. These soils are mostly found in arid and semiarid regions. Expansive soils undergo volumetric changes upon wetting and drying, thereby causing ground heave and settlement problems. This characteristic causes considerable damages to structures, particularly light buildings and pavements if not adequately taken care of [2, 12].

Damages due to expansive soils are a major contributor to the burden that natural hazards place on the economy in many parts of the world especially at places where there are significant climatic difference between dry and wet periods [1].

Mitigation, through responsible engineering and construction, is essential in helping to alleviate the risk from this natural hazard. The first steps in mitigation are to recognize the problem and understand the preventive options that are available. The next step, which is essential, is to provide careful, responsive engineering and construction. Research is also important, as it provides knowledge and data for input into understanding of phenomena and design [2].

2.2 Origin of Expansive Soils

The origin of expansive soils is related to a complex combination of conditions and processes that result in the formation of clay minerals having a particular chemical makeup which, when in contact with water, will expand. The conditions or processes which determine the clay mineralogy include composition of the parent material and degree of physical and chemical weathering to which the materials are subjected [1].

2.2.1 Parent Material

Constitutions of the parent material during the early and intermediate stages of the weathering process determine the type of clay formed. The parent materials that give rise to expansive soil are classified into two. The first group comprises the basic igneous rocks, which are comparatively low in silica, generally about 45% to 52% and rich in metallic base such as pyroxenes, amphiboles, biotite and olivine. Such rocks include the gabbros, basalts and volcanic glass. The second group comprises the sedimentary rock that contains montmorillonite as a constituent. These include shales and claystones. Limestones and marls rich in magnesium can also weather to clay [1].

2.2.2 Weathering and Climate

The weathering process by which clay is formed includes physical, biological and chemical process. The most important weathering process responsible for the formation of montmorillonite is the chemical weathering, which include hydrolysis, hydration, oxidation, carbonation and solution.

The formation of expansive clays or the montmorillonite clays is favored by an alkaline environment and the absence of leaching, the presence of ferromegnesium minerals in parent materials and presence of bases. Such condition is favorable in semi-arid regions with relatively low rain fall or seasonal moderate rainfall particularly where evaporation exceeds precipitation or rainfall with restricted leaching [1].

2.3 Identification of Expansive Soils

2.3.1 Field Identification

It is evident that expansive soil deposits can be recognized in the field through visual inspections. This method is simple and easy to use.

Some of the important field identification methods that indicate the potential for expansiveness of a soil are the following [2, 7].

- 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 finger nail.

- A 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)
    - Slickenside, (highly polished or glossy fissure surface)
    - Shattering or micro-shattering, (presence of fissures forming granular fragments of clayey soils)

2.3.2 Laboratory Identification

There are three different methods of identifying potential expansive soil in laboratory. These are:

- Mineralogical methods of identification
- Direct measurement
- Indirect methods of identification

2.3.2.1 Mineralogical Methods of Identification

Type of clay mineral is a fundamental factor controlling expansive soil behavior. Clay minerals can be identified using a variety of techniques, some of the common types of mineralogical tests are:

X-ray Diffraction

Different minerals with different crystalline structures will have different x-ray diffraction patterns. The pattern of the common mineral are published and the diffraction pattern of the unknown will be compared with the pattern of known minerals. There is a problem, however, with soils which are mixture of clay minerals, contain organics and other non-clay mineral constitutes. In this method usually a detail quantitative analysis is impossible [13].

- **Differential Thermal Analysis (DTA)**

It consists of simultaneous heating a sample of clay and an inert control substance. A certain changes in temperature occur because of the particular structure of the clay minerals. The change occurs at specific temperatures for specific minerals, and the record of these changes may be compared with those of known minerals [13].

- **Electron Microscopy**

It provides a means of directly observing the clay patterns. Only quantitatively identification is possible based on size and shape of the particles using microscopy.

Other mineralogical methods include chemical analysis, infrared spectroscopy and dye adsorption [2].

2.3.2.2 Direct Measurement

This type of test directly measures the pressure that a swelling soil exerts on any structure resting on it. It is a convenient and more reliable test because it directly tells the likely in situ response of the soil for moisture variations. The test can be done by the use of a conventional one-dimensional consolidometer which is available in most soil mechanics laboratories.

The available techniques for quantitative measurement of expansive soils can be categorized in to three groups. These are oedometer test, soil suction test and empirical methodology. Among these techniques the oedometer tests are capable of simulating some of the factors which affect the swelling characteristics of expansive soils. Oedometer tests have limitations. The oedometer tests consider moisture as well as volume change in one dimension only. In the in-situ, the above changes take place in three directions. For simplicity, however, the oedometer testing techniques have become popular and are extensively used [12].

Some of these techniques are discussed as follows:

i. Constant Volume Test

This test is conducted by applying a small incremental load of 6.9kPa (1psi) on a clay specimen. Water is added to the sample. As expansion starts, pressure is added in small increment to prevent swelling. This is continued until the specimen cease to swell. The total load required to prevent swelling divided by the area of the sample defines the swelling pressure. (Mckeen, 1976)

ii. Swell-Consolidation Test

In this test the sample under a 6.9kPa applied load is wetted and allowed to fully swell. At this point a standard consolidation test is conducted by applying incremental loads starting with 25kPa and ending with 1600kPa. The pressure required to revert the specimen to its initial void ratio (height) is used to define the swelling pressure. (Mckeen, 1976)

iii. Double Oedometer Test

This method is based on subjecting the identical samples to consolidation test. The first sample is consolidated at its natural moisture content while the second sample is first allowed to absorb water and swell under light load followed by consolidation. For the two samples the applied pressures are plotted against vertical strain (or void ratio) on the same diagram. The pressure corresponding to the intersection point of the two curves is taken as the swelling pressure.

iv. Restrained Swell Test

This test consists of successively increasing the load on the specimen allowing it to attain equilibrium deformation at each pressure level. At a prescribed applied pressure, the sample is inundated and permitted to fully swell. The process is repeated with various inundation pressures on identical samples. Here the swell potential is calculated as the ratio of maximum expansion to the sample initial height. The pressure resulting in no expansion defines the swelling pressure.

2.3.2.3 Indirect Methods

This method is used to investigate the swelling potential of a soil by examining other parameters, which indirectly give information about the soil property. These include Index Property Tests, Cation Exchange Capacity (CEC), and Potential Volume Change (PVC) test.

i. Index Property Tests

These are the most commonly applied methods and simple soil property tests that can be used for the evaluation of the swelling potential of expansive soils in all soil testing laboratories. Such tests include:

a) Atterberg Limits tests
b) Linear shrinkage
c) Free Swell tests and
d) Colloidal content test
e) Activity

a) Atterberg Limits

Atterberg limit describes the consistency and plasticity of fine-grained soils with varying degrees of moisture content. For the portion of the soil passing No. 40 (0.425 mm) sieve, the moisture content is varied to identify three stages of soil behavior in terms of consistency. These stages are known as the liquid limit (LL), plastic limit (PL) and shrinkage limit (SL) of soils (Casagrande, 1948).
Holtz and Gibbs demonstrated in 1956 that the plasticity index and the liquid limit are useful indices for determining the swelling characteristics of most clay [1].

Liquid Limit: is the moisture content at which a soil begins to flow. Typically, clayey soils dominated by smectite will adsorb more water than clayey soils consisting of mostly non-expansive clays (kaolinite, mica, etc.) or limited expansion minerals (vermiculite).

Plastic Limit: is the moisture content at which a soil stiffens from a plastic to a semi-rigid and friable state. Soils with high specific surface areas, e.g., smectitic soils, can retain water at much lower moisture content than can kaolinitic or micaceous soils, and thus will have lower plastic limits.

Plasticity Index: is the most commonly used indicator of soil expansive behavior. It is the difference between the liquid limit and plastic limit. It generally indicates a soil's potential plasticity and is widely used in engineering classification systems.

Shrinkage Limit: A low shrinkage limit indicates that a soil would begin to swell at low water content. The colloid content (<1μm fraction) constitutes the most active part of the soil contribution for swelling and high colloid content naturally means a greater possibility of expansion. Colloid content, plastic index and shrinkage limit parameters are used to indicate the criteria for identification of expansive soils by the U.S Bureau of Reclamation and are presented on table 2.1.

Table 2.1: Identification criteria for expansive Clays, After Holtz and Gibbs (1956) [2]

Colloid content, %	Plastic index, %	Shrinkage limit, %	Probable expansion, %	Degree of expansion
<15	<18	>15	<10	Low
13-23	15-28	10-16	10-20	Medium
20-31	25-41	7-12	20-30	High
>28	>35	<7	>30	Very High

b) Linear Shrinkage

The swell potential is presumed to be related to the opposite property of linear shrinkage measured in a very simple test.

Table 2.2: Relation of linear shrinkage, shrinkage limit with degree of expansion as per Altmeyer, 1955 [2]

Shrinkage limit, %	Liner Shrinkage, %	Degree of expansion
<10	>8	Critical
10-12	5-8	Marginal
>12	0-5	Non-critical

c) Free Swell Test

It is one of the simple swelling identification tests developed by the U.S Bureau of Reclamation (Holtz and Gibbs, 1956). In this test a small sample of 10 cm^3 of dry soil passing No. 40 sieve is added to a graduated cylinder and filled with water and observing the equilibrium swelled volume. The free swell is determined by comparing the initial volume with the final volume. For comparison highly swelling bentonite (mostly Na-montimorillinite) will have free-swell value of greater than 1200. Soils having free swell values greater than 100% are considered potential problems, whereas soils with free swell values below 50% probably do not exhibit appreciable volume changes.

d) Colloidal Content Tests

Grain size analysis helps to determine the amount of colloidal sized particles existing in a soil. The grain size characteristics of clay appear to have a bearing on its swelling potential, particularly the colloidal content. The term colloid describes a particle whose behavior is controlled by surface force (i.e electrostatic and adsorptive force) rather than by gravitational force. These colloidal particles greatly influence the plasticity characteristics and volume change behavior of the soil [1, 2].

e) Activity

Since the plastic property of soil is due to the adsorbed water that surrounds the clay particles, one can expect that the type of clay minerals and their proportional amounts in a soil will affect the liquid and plastic limits. Skempton (1953) observed that the plasticity index of a soil linearly increases with the percent of clay-size fraction (percent finer than 2μm by weight) present in it.

Activity A, of clay may be expressed as

$$\text{Activity (A)} = \frac{\text{Plasticity index}}{\text{\% clay fraction}} \qquad (2.1)$$

Activity is used as an index for identifying the swelling potential of clay soils. Skempton proposed three classes of clays according to the activity ratio as follows: inactive, A < 0.75, normal, A between 0.75 and 1.25, and active, A > 1.25. Active clays provide the most potential for expansion.

Table 2.3: Typical values of activities for various clay minerals [2].

Clay mineral	Activity)
kaolinite	0.33 – 0.46
Illite	0.9
Montmorillonite (Ca)	1.5
Montmorillonite (Na)	7.2

In mixed mineralogy soils as with most soil systems, the activity classification scheme does not accurately estimate shrink-swell potential.

ii. Cation Exchange Capacity (CEC)

Cations attracted to the negatively charged surface of soil particles are not strongly attached. These cations can be replaced by other ions and are, therefore, known as exchangeable ions.

The CEC is the quantity of exchangeable cations required to balance the negative charge on the surface of the clay particles. CEC is expressed in terms of the total number of positive charges adsorbed per 100gm of dry soil. It is measured in milliequivalent (meq.), which is equal to $6x10^{20}$ electronic charges. CEC is related to amount and type of clay present in a soil. High CEC values indicate a high surface activity. In general, swelling potential increases as CEC increases [8].

Typical values of CEC for three basic minerals are as follow (Mitchell, 1937) [2].

Table 2.4: Clay mineral vs CEC values

Clay mineral	CEC (meq/100gm)
kaolinite	3 - 15
Illite	10 - 40
Montmorillonite	80 - 150

The measurement of CEC requires detailed and precise testing procedures that are not commonly done in most soil mechanics laboratory. However, this test is routinely performed in many agricultural laboratories. The cations can be listed in approximate order of their replacement ability as:

$$Li^{+}<Na^{+}<H^{+}<K^{+}<NH^{4+}<Mg^{2+}<Ca^{2+}<Al^{3+}$$

iii. Potential Volume Change (PVC) Method

The soil PVC meter is a standardized apparatus for measuring swelling pressure. The swell index test is essentially a measurement of the pressure exerted by a sample of compacted soil when it tries to swell against a restraining force after being wetted. The PVC method can be used in the field or laboratory. Swelling categories based on the PVC method is given in Table 2.5. It should be pointed out that the PVC test should be used only as a comparison between various swelling soils [1].

Table 2.5: Values of PVC rating (Lambe, 1961) [2].

Category	**PVC rating**
Very critical	> 6
Critical	4 - 6
Marginal	2 - 4
Non- critical	< 2

2.4 Classification of Soils

Soil classification provides a systematic method of categorizing soils according to their probable engineering behavior and allows engineers to have a fairly good general idea of the way the soil will behave in the engineering situation.

It is possible to trace many soil classification systems such as the Casagrande unified soil classification system (USCS), the American Association of State Highway and Transportation Officials (AASHTO) system, Federal Aviation Agency (FAA) system, US public roads administration (PRA) system, and the textural classification system. Currently, the USCS and the AASHTO systems are in use in civil engineering practice.

Both USCS and AASHTO, base their classification of soils for engineering purposes on particle size characteristics, liquid limit (LL) and plasticity index (PI) of soils. According to USCS, subgrouping of coarse-grained soils is done with the help of parameters such as uniformity coefficient (Cu) and coefficient of curvature (Cc) to account for the gradation of soils. Subgrouping of fine-grained soils is entirely based on a plasticity chart [14].

Parameters determined from expansive soil identification tests have been combined in a number of different classification schemes. These parameters are used to indicate potential hazardous areas and a need for predication testing.

These classification schemes provide one or more of the following ratings:

- Ranges of values for either probable percentage of volume change, or probable swelling pressure.
- A qualitative expansion rating, i.e., low, medium, high and very high expansion potential [5].

2.4.1 Classification Specific to Expansive Soil

A parameter determined from the expansive soil identification tests have been combined in a number of different classification schemes to give qualitative rating on the expansiveness of the soil. But the direct use of such classification systems as a basis for design may lead to an overly conservative construction in some places and inadequate construction in some areas [2]. Hence, it is very important to emphasize that design decision has to be based on predicting testing and analysis, which provide reliable information.

Some of classification systems developed using expansive soil identification tests are:

a) Skempton (Mckeen, 1976)

This method is developed, by combining Atterberg limits and clay content into a single parameter called Activity.

Skempton suggested three classes of clays according to their activity.

Table 2.6: Soil expansivity predicted by Skempton[12]

Activity	Potential of expansion
Ac<0.75	Low (inactive)
0.75< Ac<1.25	Medium (normal)
Ac>1.25	High (active)

b) Altmeyer (Mckeen, 1976)

He suggests rating for degree of expansion based on shrinkage limit (SL) and linear shrinkage [2]

Table 2.7: Soil expansivity prediction by shrinkage limits and linear shrinkage (Altmeyer 1955)

Linear shrinkage (%)	Shrinkage limit,SL (%)	Degree of expansion
<5	>12	Noncritical
5 – 8	10 – 12	Marginal
>8	<10	critical

c) Bureau of Reclamation Method

This method is based on direct correlation of observed volume change with colloid content, plastic index, and shrinkage limit.

The Atterberg limits and swell potentials of clays depend on the quantity of water that clay can imbibe.

- The higher the plasticity index, the greater the quantity of water that can be imbibed by the soil and hence the greater would be its swell potential.
- A low shrinkage limit indicates that a soil would begin to swell at low water content.

- The colloid content (<1 μm) constitutes the most active part of the soil contributing to swelling and a high colloid content naturally means a greater possibility of expansion.

The United States Bureau of Reclamation uses these three parameters to indicate the criterion for identification of expansive soils as indicated in table 2.8 [2]

Table 2.8: Identification criteria for expansive clays as per USBR (After Holtz and Gibbs, 1956)

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

d) Holtz and Gibbs (1956) and Seed *et al.* (1962)

Based on the oedometer swell potential values, Holtz and Gibbs (1956) and Seed *et al.* (1962) have classified the relative expansivity of the swelling soils. Holtz and Gibbs' (1956) classification is based on the swell potentials of undisturbed specimens that were inundated under 7 kPa pressure. Seed *et al.* (1962) criterion is based on the swell potential of remolded specimens that were compacted at their Standard Proctor MDD and OMC values and inundated under 7kPa pressure.

Table 2.9: Classification of expansive soils after Holtz and Gibbs (1956) and Seed et al. (1962)

Degree of expansion	*Holtz and Gibbs'(1956) classification of percent swell*	*Seed* et al'*s (1962) classification of percent swell*
Low	0 -10	0 – 1.5
Medium	10 -20	1.5 – 5
High	20 -35	5 – 25
Very high	>35	>25

Generally, Holtz and Gibbs (1956), Altmeyer (1955), Seed et al. (1962), and Daksanamurthy and Raman (1973) have evolved different methods to identify expansive soils based on the percentage of clay content, shrinkage limits, plasticity index, liquid limit, and shrinkage index. They classified soils into low, medium, high, and very high degrees of potential expansiveness. However this classification of potential expansiveness does not give the same assessment of the swelling potential.

Table 2.10: Classifications for degree of expansion (swelling potential) based on different methods [5].

Degree of expansion	Chen (1983)	Seed et al. (1962)	Daksanamurthy and Roman (1973)	USBR (Holtz and Gibbs, 1956)
Very high	$LL > 60$	$PI > 35$	$LL > 70$	$Cc > 28$
High	$40 < LL \leq 60$	$20 < PI \leq 35$	$50 < LL \leq 70$	$20 < CC \leq 31$
Medium	$30 \leq LL \leq 40$	$10 \leq PI \leq 20$	$35 \leq LL \leq 50$	$13 \leq CC \leq 23$
Low	$LL < 30$	<10	$20 \leq LL \leq 35$	$CC<13$

2.5 Clay Mineralogy

According to the clay mineral concept, clay materials are essentially composed of extremely small crystalline particles of one or more members of a small group of minerals that are commonly known as clay minerals. These minerals are essentially hydrous aluminum silicates, with magnesium or iron replacing wholly or in part for the aluminum, in some minerals. Many clay materials may contain organic material and water-soluble salts [3].

2.5.1 Structure of Clay Minerals

Clay mineral is composed of two structural units:

1) A silicon–oxygen tetrahedron unit
2) An aluminum or magnesium octahedron unit

An initial study of the crystal structure of clay minerals leads to a better understanding of the behavior of clays under different conditions of loading.

The Tetrahedral Unit: consists of four oxygen atoms (or hydroxyls, if needed to balance the structure) placed at the apices of a tetrahedron enclosing a silicon atom which combines together to form a shell-like structure with all the tips pointing in the same direction. The oxygen at the bases of all the units lies in a common plane.

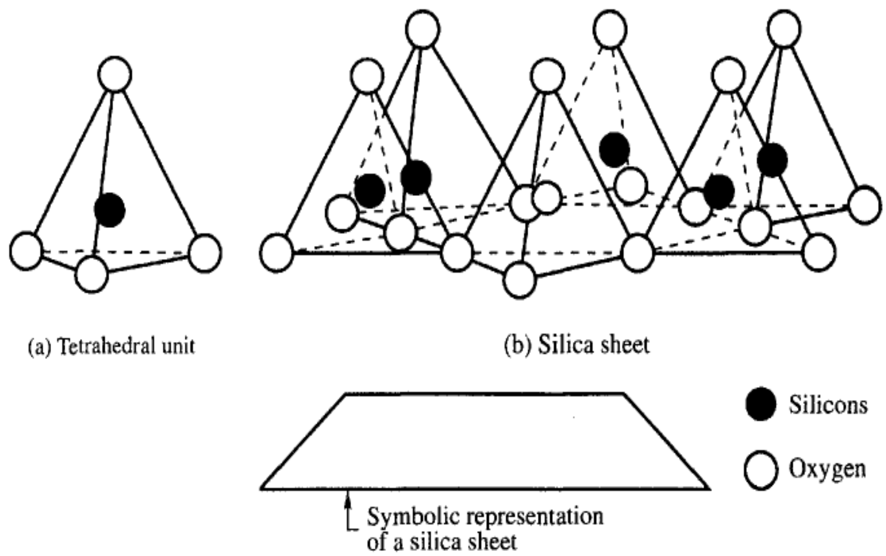


Fig. 2.1 Basic structural units in the silicon sheet [3].)

Each of the oxygen ions at the base is common to two units. The arrangement is shown in Fig.2.1. The oxygen atoms are negatively charged with two negative charges each and the silicon with four positive charges. Each of the three oxygen ions at the base shares its charges with the adjacent tetrahedral unit. The sharing of charges leaves three negative charges at the base per tetrahedral unit and this along with two negative charges at the apex makes a total of 5 negative charges to balance the 4 positive charges of the silicon ion. The process of sharing the oxygen ions at the base with neighboring units leaves a net charge of -1 per unit.

The Octahedral Unit: consists of six hydroxyls forming a configuration of an octahedron and having one aluminum ion at the center. Iron or magnesium ions may replace aluminum ions in some units. These octahedral units are bound together in a sheet structure with each hydroxyl ion common to three octahedral units. This sheet is sometimes called as *gibbsite* sheet. The Al ion has 3 positive charges and each hydroxyl ion divides its -1 charge with two other neighboring units. This sharing of negative charge with other units leaves a total of 2 negative charges per unit [(1/3) x 6]. The net charge of a unit with an aluminum ion at the center is +1. Sometimes, magnesium replaces the aluminum atoms in the octahedral units in this case; the octahedral sheet is called a *brucite sheet.* Fig. 2.2 gives the structural arrangements of the units.

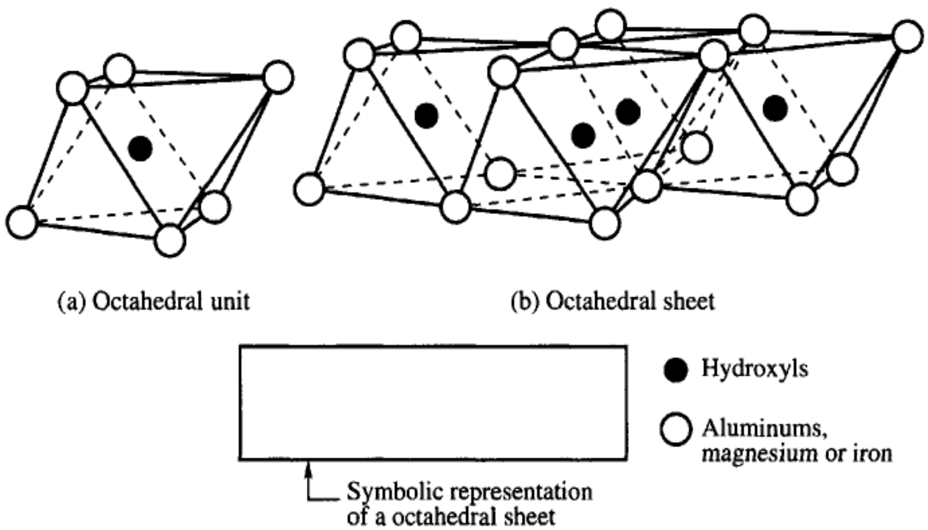


Fig. 2.2: Basic structural units in octahedral sheet [3]

2.5.2 Formation of Clay Minerals

The combination of two sheets of silica and gibbsite in different arrangements and conditions lead to the formation of different clay minerals. In the actual formation of the sheet silicate minerals, the phenomenon of *isomorphous substitution* frequently occurs. Isomorphous (meaning same form) substitution consists of the substitution of one kind of atom for another.

2.5.3 Clay Minerals Classification

Clay minerals are a very distinctive type of particles that give particular characteristics to the soils in which they occur. The most well-known clay minerals are montmorillonite illite and kaolinite.

Montmorillonite: The structural arrangement of this mineral is composed of two silica tetrahedral sheets with a central alumina octahedral sheet. The silica and gibbsite sheets are combined in such a way that the tips of the tetrahedrons of each silica sheet and one of the hydroxyl layers of the octahedral sheet form a common layer. The atoms common to both the silica and gibbsite layer become oxygen instead of hydroxyls. In stacking these combined units one above the other, oxygen layers of each unit are adjacent to oxygen of the neighboring units with a consequence that there is a very weak bond and an excellent cleavage between them. Water can enter between the sheets, causing them to expand significantly. Soils containing a considerable amount of montmorillonite minerals will exhibit high swelling and shrinkage characteristics.

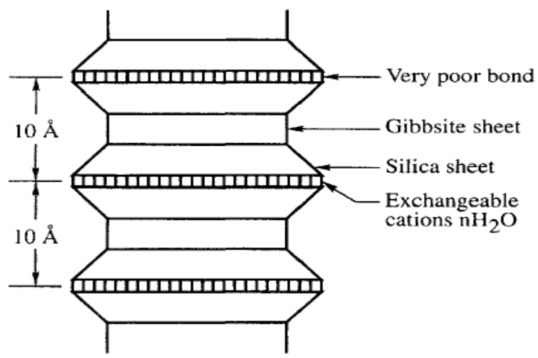


Fig. 2.3: Structure of montmorillonite layer [3]

Illite: The basic structural unit of illite is similar to that of montmorillonite except that some of the silicons are always replaced by aluminum atoms and the resultant charge deficiency is balanced by potassium ions. The potassium ions occur between unit layers. The bonds with the nonexchangeable K^+ ions are weaker than the hydrogen bonds, but stronger than the water bond of montmorillonite. Illite, therefore, does not swell as much in the presence of water as does montmorillonite.

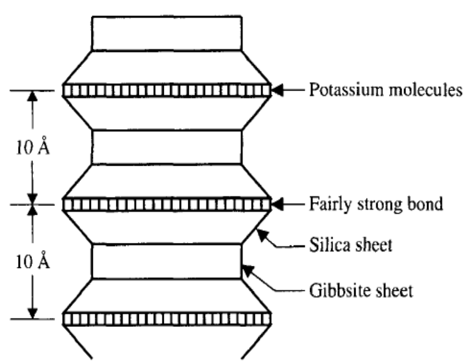


Fig. 2.4: Structure of illite layer [3]

Kaolinite: *The* basic structure consisting of a single sheet of silica tetrahedrons and a single sheet of alumina octahedrons. These combined sheets are then held in a stack fairly tightly by hydrogen bonding. Kaolinite has no or a few exchangeable cation, and the interlayer bonds are relatively strong to prevent any hydration between layers. Kaolinite is relatively stable and water is unable to penetrate between the layers. Consequently Kaolinite shows little swelling on wetting.

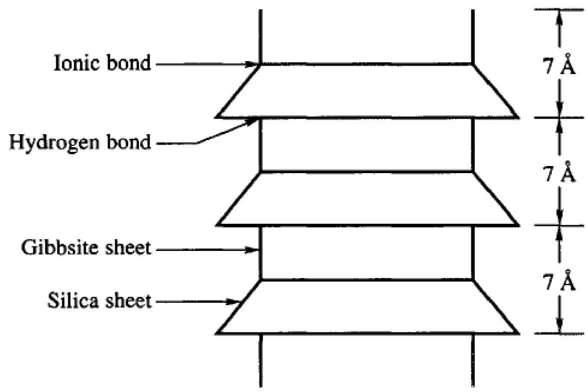


Fig. 2.5: Structure of kaolinite layer [3]

2.6 Swelling Potential and Swelling Pressure

2.6.1 Swelling Potential

The available literature introduced various definitions for the swell potential. Of these, the definition advanced by Seeds et al. (1962) and Holtz and Gibbs's (1956) have been widely used.

Seed et al. (1962) defined swelling potential as the magnitude of heave of a soil of a laterally confined sample on soaking under1 psi (approximately 7 kPa) pressure, after being remolded and compacted at their Standard Proctor MDD (maximum dry density) and OMC (optimum moisture content) values .

Holtz and Gibbs' (1956) definition is based on undisturbed specimens that were inundated under 1 psi.

Swelling potential in percent (S_p) is given by:

Where: $S_p = (\delta h/h)*100$

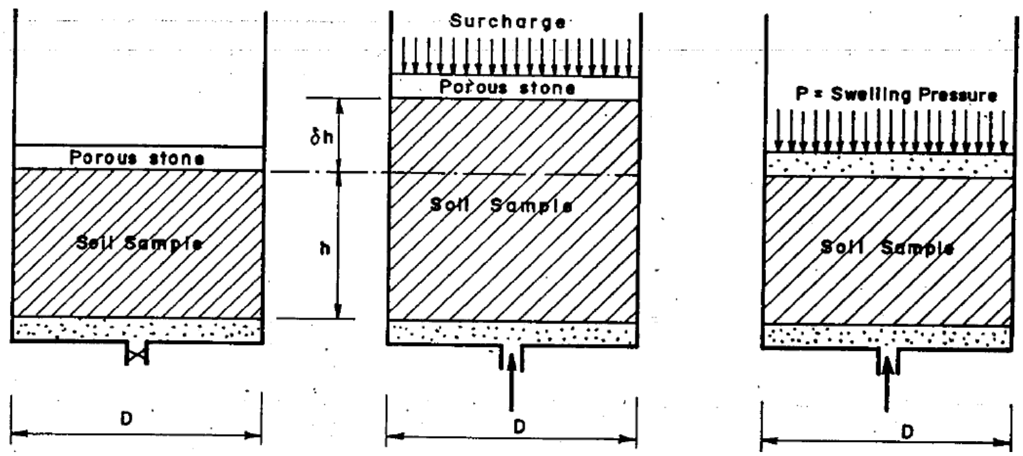
δh- Amount of vertical swell as shown in fig.2.6 (a)

h- Initial height

2.6.2 Swelling Pressure

Swelling pressure is a very useful index of the trouble potential of an expansive soil. This pressure is the maximum force per unit area that needs to be applied over a swelling soil to prevent volume increase as shown in fig.2.6(c)

Specially designed oedometer tests have been found quite useful to determine the magnitudes of these parameters for expansive soils (ASTM D 4546–90 Standard Test Method for One Dimensional Swell or Settlement Potential of Cohesive Soils).



a) Initial condition b) Maximum swelling c) Final condition

Fig.2.6: Diagramatic discription of swelling potential and swelling pressure [12].

2.7 Factors Responsible for Shrink-Swell Phenomena

The mechanism of shrink-swell in expansive soil is complex and influenced by a number of factors. Expansion is a change of particle spacing and this is a result of changes in the soil systems that disturb the internal stress equilibrium. Many of the factors influencing the mechanism of swelling also affect, or are affected by, physical soil properties such as plasticity or density.

The factors influencing the swelling potential of a soil can be considered in three different groups, the soil characteristics that influence the basic nature of the internal force field, the environmental factors that influence the changes that may occur in the internal force system, and the state of stress.

2.7.1 The Soil Characteristics

The soil characteristics influence the basic nature of the internal force field between particles. The following properties are categorized in this group.

2.7.1.1 Clay minerals

Clay minerals of different types typically exhibit different swelling potentials because of variations in the electrical field associated with each other. The swelling capacity of an entire soil mass depends on the amount and type of clay minerals in the soil, the arrangement and specific surface area of the clay particles, and the chemistry of the soil water surrounding those particles [2].

2.7.1.2 Dry density

Higher density usually indicates closer particle spacing, which may mean greater repulsive forces between particles and larger swelling potential.

2.7.1.3 Soil structure and fabric

The term fabric refers to the arrangement of particles, particle groups and pore spaces in a soil. Structure has a broader meaning of the combined effect of fabric, composition and inter-particle force. The unique relationship between water content of a soil and matric suction is influenced by soil fabric which in turn affects the swelling potential of the soil.

2.7.1.4 Soil water chemistry

Soil water chemistry is important in relation to potential swell magnitude. Salt cations, such as sodium, calcium, magnesium and potassium are dissolved in the soil water and are adsorbed on the clay surfaces as exchangeable cations to balance the negative electrical surface charges.

If the soil water chemistry is changed either by changing the amount of water or the chemical composition, the inter particle force field which is dependent on the negative surface charge and electro chemistry of the soil water will change. This change disturbs the equilibrium and the system tries to adjust itself to the new condition which is manifested as shrinkage or swelling.

2.7.2 Environmental Factors

Environmental factors influence the changes that may occur in the internal force system. These factors are mostly associated with moisture. They are:

2.7.2.1 Initial Moisture Content

A desiccated expansive soil will have high affinity for water, or higher suction than the same soil at higher water content, lower suction. Conversely, a wet soil profile will lose water more readily on exposure to drying influences, and shrink more than a relatively dry initial profile. The initial soil suction must be considered in conjunction with the expected range of final suction conditions.

2.7.2.2 Climate

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

2.7.2.3 Ground Water

Clays beneath the water table have no swelling potential, as they are completely saturated and have no capacity to absorb moisture. Clays above the water table are generally unsaturated and will have capacities to absorb moisture. Generally, saturation levels are high and swelling potentials are low for clays just above the water table, because, due to capillary attraction, they have access to abundant moisture.

2.7.2.4 Drainage and Manmade Water Sources

Surface drainage features, such as ponding around a poorly graded house foundation, provide source of water at the surface; leaky plumping can give the soil access to water at greater depth.

2.7.2.5 Vegetation

Trees, shrubs, and grasses deplete moisture from the soil through transpiration, and cause the soil to be differentially wetted in areas of varying vegetation.

2.7.2.6 Permeability

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

2.7.2.7 Temperature

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

2.7.3 Stress Condition

Volume change is directly related to change in the state of stress in the soil. A reduction in total stress due to excavation of overlying material will result in rebound and heave of the surface. Heave in unsaturated soils is accompanied by imbibition of water and is time dependent [2]

CAHPTER 3

DESCRIPTION OF THE STUDY AREA

3.1 General

Asella is located between $7^0 54'55''N-8^0 00'05''N$, $39^0 06'10''E-39^0 10'00'E$. It is found in Oromia regional national state at a distance of 166kms from Addis Ababa with an area of 46,231,802 m^2.

The foundation of Asella town was related to the pattern of imperial control in the late 19^{th} century and early 20^{th}.As evidenced from different sources, Minilik's expansion to the south, south west and east in the 19^{th} and 20^{th} centuries was motivated by political strategies and economic interests. Therefore, it had been planned that military victories and political control would be followed by collection of tributes, control of trade and trade routes and control of the resources of the area (Abas, 1982; Bahiru, 1991). From the intensity of the campaign to subdue the area and the time frame set for this particular mission Minilik had already realized that the occupation of Arsi would be an essential step for the conquest of Harar which was commercially and strategically crucial.

Therefore, Asela began to take a form of urban settlement during early 1930s, near the custom office and the market place. The Italians had contributed to the development of urbanization during their five years stay in Ethiopia. Asella which originally had begun as a settlement on about 120 hectares in the early 1930s was developed into an urban form sizable village settlement during the Italian occupation period. Various services including houses and small drink houses were established under the Italians.

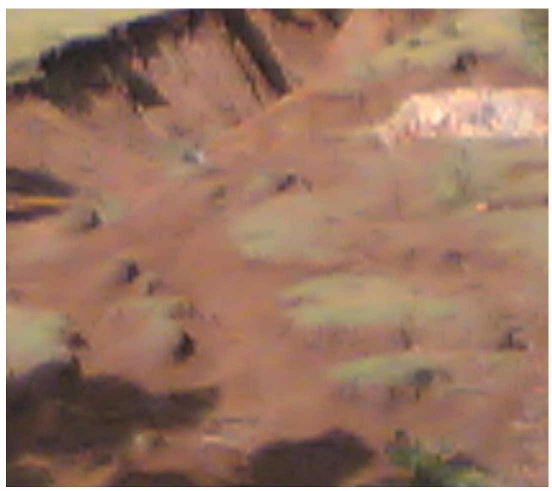
After it became the seat of the governorate general Asella began to witness unprecedented growth. Different people began to come to the town from various areas and permanently settle. In 1947 the town's boundary was delimited and about 230 hectares was allocated to the town. This paved the way for the further urban development this condition continued until 1974 that highly impacted Asella's urban development.

3.2 Topography

The topography of Asella is not uniform. The gradient of the town inclines from east to west direction. There is maximum altitude in the eastern peripheral areas of the town. This rises up to 2700 meters above Sea level in the south east but declines up to 2200 meters above sea level in the west direction of the town. Generally, rugged topographies are common in the eastern parts of the town but relatively flat slopes prevail in the western, southern and northern parts of the town.

3.3 Soil of Asella town

From visual observation the soil of Asella town ranges from red soil which is found in the eastern and southern parts of the town to black cotton soil which is generally observed in the western and some other directions of the town.



Picture 3.1: Soil color of Asella town

3.4 Climatic Characteristics

a) Temperature

Table 3.1: Temperature condition of Asella from 1971 to 2005

Month	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.
Average Temp.(°c)	13.9	14.8	15.2	16.0	16.2	15.8	14.4	14.6	14.2	14.9	13.8	12.8

The average temperature of the town over the past 35 years is 14.7^0C. As is observed from the average temperature of the above table; the lowest temperature of Asella starts in November and becomes lowest in December. The average lowest temperature of the town over the past 35 years is 12.8^0C.

b) Rainfall

In order to analyze the rainfall of Asela the rain fall data of 35 years is used.

Table 3.2: Average rainfall of Asella town from 1971 to 2006

Month	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.
Average rainfall (cm)	4.5	15.4	24.3	35.6	26.2	26.1	29.4	31.6	22.9	20.3	8.0	9.0

Source: *Ethiopian Meteorology Agency, November, 2008*

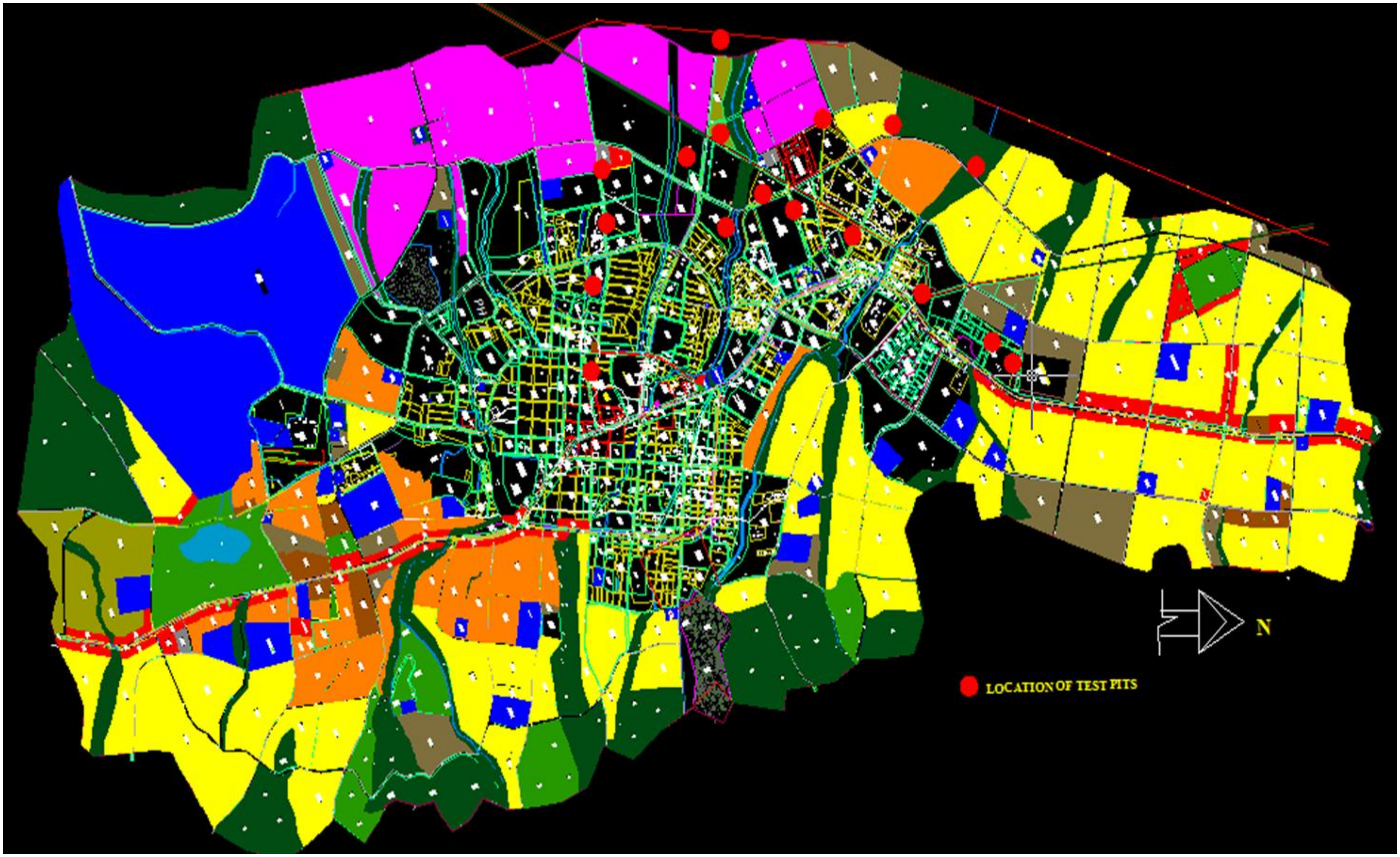


Fig. 3.2 Recent Map of boundary and structural plan of Asella (Source: Asella Municipality

CHAPTER 4

SAMPLING AND LABORATORY TESTS

4.1 Sampling

Soil profile of Asella town varies from place to place. On the southern and eastern side, where mountain Chilalo is found, the soil is generally red in color and is non-expansive. While the soil mainly on the western side of the town, where the topography is relatively wide flat and drainage is much limited, is mainly expansive in nature.

For the laboratory works of this thesis, seventeen disturbed and undisturbed soil samples were taken from different open pits of depth 1.5 – 2.0m at which foundation of lightweight structures are commonly constructed.

Table 4.1: Sample depth, color and location of test pits

Test Pit	Depth (m)	Color	Northing	Easting	Elevation
TP-01	1.7	Black			2284
TP-02	1.5	Black			2259
TP-03	1.5	Black			2257
TP-04	1.5	Black			2252
TP-05	1.8	Black grey			2261
TP-06	2	Black			2325
TP-07	1.5	Black			2243
TP-08	1.7	Black			2260
TP-09	2	Black			2303
TP-10	1.5	Black			2328
TP-11	1.5	Black			2355
TP-12	1.5	Black			2367
TP-13	1.5	Black			227331
TP-14	1.8	Black			2252
TP-15	1.5	Black			2256
TP-16	2	Black			2242
TP-17	2	Black			2261

4.2 Laboratory Tests and Discussions

Laboratory tests are carried out in accordance with the ASTM standard testing methods. The actual test results are presented in the Appendices.

For measuring cation exchange capacity different test procedure is applied.

In order to obtain the intended purpose of the research the following laboratory tests are carried out.

- Particle size distribution (ASTM D422-98)
- Specific gravity of soil solid (Gs) (ASTM D854-98)
- Natural moisture content (ASTM D2216-98)
- Dry density (γ_d) (ASTM D2937-98)
- Atterberg limits (ASTM D4318-98
- Shrinkage Limit (ASTM D427-98)
- Swelling Pressure test (ASTM D4546-96)
- Free swell test
- Cation exchange capacity (CEC)

4.2.1 Grain Size Analysis

This method covers the quantitative determination of the distribution of particle size of the soil in the study area using ASTM D422 - standard test method. The distribution of particle sizes larger than 75 µm (retained on the No. 200 sieve) is determined by sieving, while the distribution of particle sizes smaller than 75 µm is determined by a sedimentation process, using hydrometer 151H.

Since surface force between particles depends upon particle size, for soils of different test pits, grain analysis is carried out to determine the ranges of sizes in which the soil samples fall and their relative proportions.

In this study, hydrometer and sieve analysis is performed on all the samples and percent finer against size of soil particle in millimeter on a semi-log scale is plotted. From this curve the proportion and type of soil grains is determined and Particle size analysis run by this test method is grouped in to sand, silt, clay is summarized in Table 4.2.

Soil particle size limit as per American Association of State Highway and Transportation Officials (AASHTO) classification system is given as:

Sand---- Passing No. 4 sieve and retained on No. 200 sieve

Silt size----- 0.075 to 0.002 mm

Clay size----- smaller than 0.002 mm

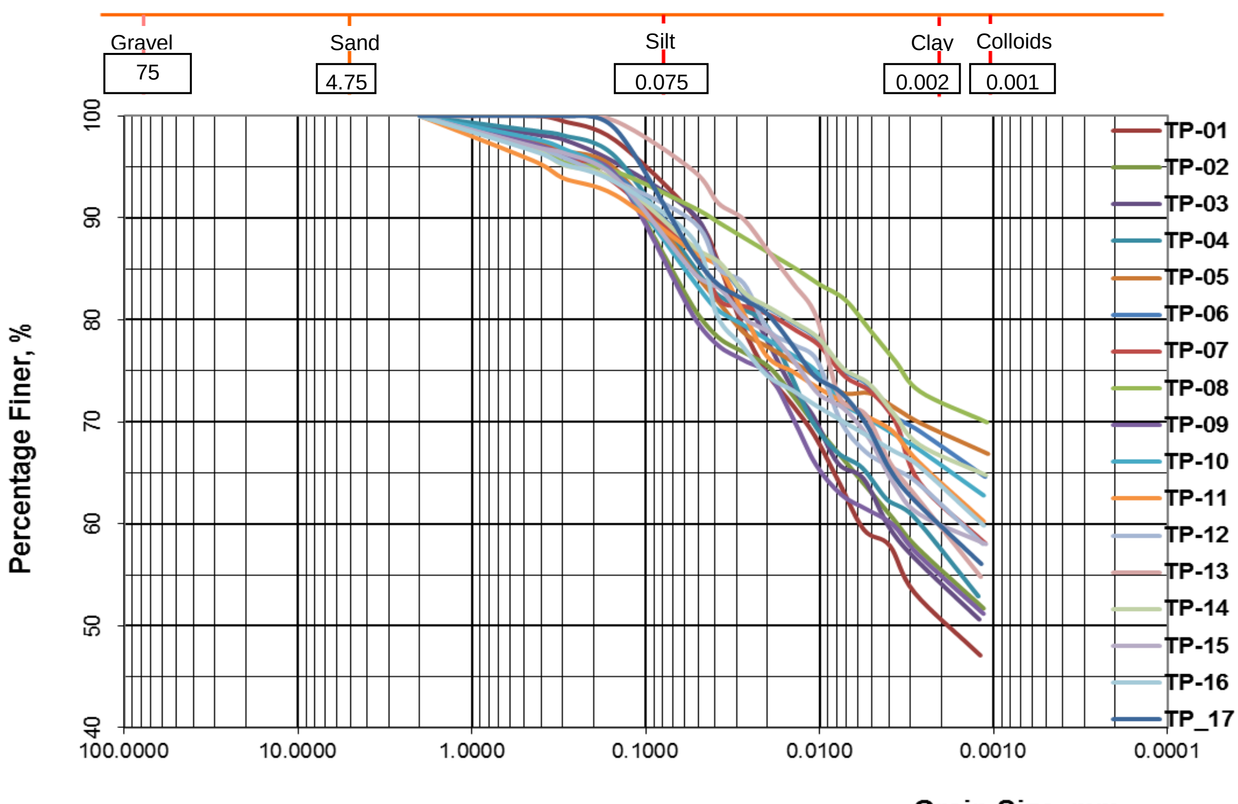


Fig 4.1: Grain size distribution curve

Table 4.2: Grain size distribution

Test Pit	Depth (m)	Sand (%)	Silt (%)	Clay (%)
TP-01	1.7	2.52	47.30	50.18
TP-02	1.5	6.88	38.64	54.48
TP-03	1.5	5.50	40.80	53.70
TP-04	1.5	3.44	37.36	59.20
TP-05	1.8	6.13	24.04	69.83
TP-06	2.0	6.34	26.04	67.62
TP-07	1.5	7.18	31.76	61.06
TP-08	1.7	5.95	22.16	71.89
TP-09	2.0	6.63	39.77	53.60
TP-10	1.5	5.32	29.98	64.70
TP-11	1.5	8.90	25.16	65.94
TP-12	1.5	7.22	31.60	61.18
TP-13	1.5	1.76	41.84	56.40
TP-14	1.8	6.96	26.77	66.27
TP-15	1.5	6.74	30.82	62.44
TP-16	2.0	8.14	27.74	64.12
TP-17	2.0	3.48	37.38	59.14

From the grain size analysis test result, it has been found that sand percentage is between 1.76 and 8.9 %, the values of amount of silt are between 22.16 and 47.30% and the amount of clay is between 50.18 and 71.89 %.

4.2.2 Specific Gravity of Soil Solid (Gs) (ASTM D854-98)

Specific gravity of soil solids, Gs, is the ratio of the unit weight of solids in the soil to the unit weight of water. Specific gravity of soil solids is used for performing weight-volume calculations in soils. The average specific gravity (Gs) values of the soils were found to be 2.67**.**

Table 4.3: Specific gravity of solid of the soil

Test Pit	TP-1	TP-2	TP-3	TP-4	TP-5	TP-6	TP-7	TP-8	TP-9	TP-10	TP-11	TP-12	TP-13	TP-14	TP-15	TP-16	TP-17
Depth (m)	1.7	1.5	1.5	1.5	1.8	2.0	1.5	1.7	2.0	1.5	1.5	1.5	1.5	1.8	1.5	2.0	2.0
Specific gravity (GS)	2.73	2.75	2.60	2.59	2.78	2.65	2.65	2.68	2.75	2.63	2.63	2.68	2.61	2.65	2.73	2.68	2.68

4.2.3 Natural Moisture Content (ASTM D2216-98)

Moisture content measurement is primarily used for performing weight-volume calculations in soils. Moisture content is also a measure of the shrink-swell and strength characteristics of cohesive soils as used in liquid limit and plastic limit testing. The natural moisture content of the test pits are presented in Table 4.4

Table 4.4: Natural Moisture Content (%)

Test Pit	Depth (m)	Natural moisture content (%)
TP-01	1.7	42.6
TP-02	1.5	33.1
TP-03	1.5	45.7
TP-04	1.5	36.3
TP-05	1.8	31.2
TP-06	2	42.8
TP-07	1.5	43.2
TP-08	1.7	39.4
TP-09	2	31.6
TP-10	1.5	33.9
TP-11	1.5	43.4
TP-12	1.5	35.5
TP-13	1.5	36.6
TP-14	1.8	43.4
TP-15	1.5	31.9
TP-16	2	41.7
TP-17	2	28.5

4.2.4 Index Properties

Index property is a property, which helps in distinguishing the characteristics of a soil. Soil grain property and soil aggregate property are two main categories under this term. Soil grain property is based on the individual grains and depends on size, shape and mineralogical characteristics. Soil aggregate property, on the other hand is based on the property of the soil mass as a whole. Atterberg limit test, hydrometer analysis, specific gravity and free swell tests are among the tests which show the index property of a soil.

4.2.4.1 Atterberg Limits and Shrinkage Limit Tests

The liquid limit and plastic limit tests provide information regarding the effect of water content (w) on the mechanical properties of soil. Specifically, the effects of water content on volume change and soil consistency are addressed. The results of this test are used to classify soil in accordance with ASTM D2487, and to estimate the swell potential of soil. For Liquid Limit (LL), Plastic Limit (PL), and Plasticity Index (PI) tests ASTM D4318-98 standard test method is used. Shrinkage limit is measured using a separate standard, ASTM D427-98.

The liquid and plastic limit are water contents at which the mechanical properties of soil changes. They are applicable to fine-grained soils, and are performed on soil fractions that pass the #40 (0.425-mm) sieve.

The volume of fine-grained soil increases with increasing w. This indicates that *PI* is an indicator of the swell potential of a cohesive soil. Certain clay minerals, including bentonite, montmorillonite, and smectite, have a high cation exchange capacity, so their ability to hold water molecules and electrically bind them to their surface is greater. Therefore, they can exist in a plastic state over a relatively wide range of w and soil volume, and have a high swell potential. Shrinkage limit is the water content at which the volume of soil begins to change as a result of a change in w. The three parameters (*SL*, *PL*, and *LL*) are collectively referred to as the Atterberg limits.

Table 4.5: Atterberg and shrinkage limit test results

Test Pit	Depth (m)	Liquid limit, LL (%)	Plastic Limit (%)	PI (%)	Volumetric shrinkage (%)
TP-01	1.7	88.1	40.	48.1	18.4
TP-02	1.5	102.1	43.24	64.9	14.2
TP-03	1.5	91.2	37.7	53.5	16.0
TP-04	1.5	103.9	39.4	64.5	15.1
TP-05	1.8	122.5	35.7	86.8	11.3
TP-06	2.0	111.1	40.2	71.0	14.6
TP-07	1.5	95.6	36.7	58.9	14.8
TP-08	1.7	113.9	37.3	76.7	12.8
TP-09	2.0	103.5	46.1	57.4	19.2
TP-10	1.5	90.6	39.4	63.8	9.9
TP-11	1.5	108.1	40.9	67.2	15.4
TP-12	1.5	105.1	41.5	63.6	16.1
TP-13	1.5	91.6	39.1	52.6	16.8
TP-14	1.8	108.8	40.3	68.5	14.9
TP-15	1.5	90.4	36.7	68.5	7.5
TP-16	2.0	102.9	38.0	64.9	14.5
TP-17	2.0	87.9	38.4	49.5	17.0

The data in table 4.5 shows a liquid limit; Plasticity index and shrinkage limit ranges of 87.9–122.5%, 48.1 – 86.8% and 7.5 – 19.2 % respectively. Relating the liquid limit, plasticity index and shrinkage limit of the soil of the study area with the given range in table 2.8 (United State Bureau of Reclamation) and table 2.10 (Chen, 1983) the test result shows the soil is in the range of high to very high degree of expansion.

4.2.4.2 Liquidity Index of the Soil

Atterberg limits, when compared with the natural water content of the soil, give a valuable indication of the natural state of the soil in the ground. The parameter used for this purpose is the **liquidity index** (LI), which expresses the water content of the soil in relation to the PL and LL. It is defined by the following relationship:

$$LI = \frac{w - PL}{LL - PL}$$

Table 4.6: Liquidity Index of the soil

Test Pit	TP-01	TP-02	TP-03	TP-04	TP-05	TP-06	TP-07	TP-08	TP-09	TP-10	TP-11	TP-12	TP-13	TP-14	TP-15	TP-16	TP-17
LI	0.04	-0.17	0.15	-0.05	-0.05	0.04	0.11	0.03	-0.25	-0.11	0.04	-0.09	-0.05	-0.10	-0.09	0.06	-0.20

LI<0	LI=0	0<LI<1	LI=1	LI>1
w<PL	PL=w	PL< w <LL	w=LL	w>LL

4.2.5 Dry Density (γ_d) (ASTM D2937-98)

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 that higher densities result in closer particles spacing, therefore large amount of swelling clays are packed with in small volume. It is calculated from the relation of $\gamma_{dry} = (\gamma_{bulk} / (1+w))$, where γ_{bulk} = (weight of wet soil/volume of the ring), and w is the moisture content. The values of dry density are presented in table 4.8.

4.2.6 Free Swell Test

This method is suggested by Holtz and Gibbs (1956) to measure the expansive potential of soil. Soils with free swell less than 50% are not likely to show expansive property, while soils with free swell in excess of 50 percent could present swell problems. Values of 100% or more are associated with clay which could swell considerably, especially under light loadings. The actual test result are presented in Appendix 5.

Free swell is given by:

$$Free\ swell\ (\%) = \frac{Final\ volume - Initial\ volume}{Initial\ Volume}\ x100$$

Table 4.7: Free swell test result of the soil

Test Pit	Depth (m)	Free swell (%)
TP-01	1.7	90.00
TP-02	1.5	90.00
TP-03	1.5	100.00
TP-04	1.5	90.00
TP-05	1.8	140.00
TP-06	2.0	120.00
TP-07	1.5	90.00
TP-08	1.7	140.00
TP-09	2.0	90.00
TP-10	1.5	90.00
TP-11	1.5	110.00
TP-12	1.5	90.00
TP-13	1.5	100.00
TP-14	1.8	120.00
TP-15	1.5	90.00
TP-16	2.0	110.00
TP-17	2.0	80.00

4.2.7 Swelling Pressure Test (ASTM D4546-96)

For this test undisturbed soil samples are taken from seventeen different test pits at a depth ranging from 1.50m to 2.0m. The swelling pressure is determined in the laboratory using an oedometer consolidation cell.

In this test the sample under a 6.9kPa applied load is wetted and allowed to fully swell. After swelling, the sample is further loaded by applying incremental loads starting with 50kPa till the initial specimen height is obtained. The pressure required to revert the specimen to its initial void ratio (height) is determined from graph plotted change in specimen height as ordinate and applied pressure as abscissa as indicated in Appendix 9.

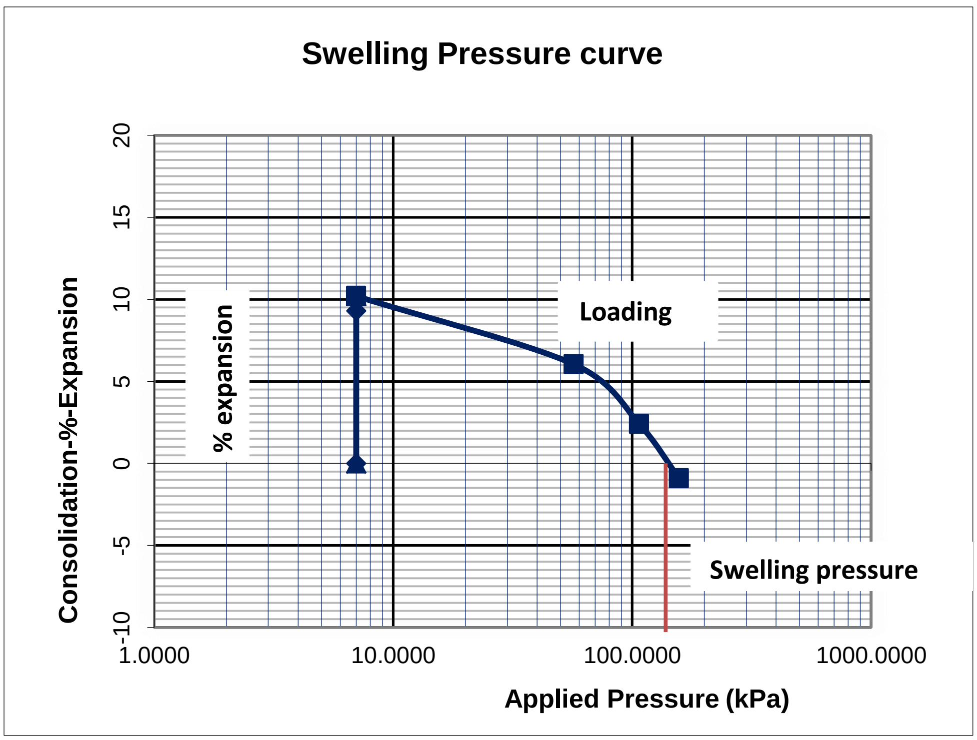


Fig. 4.1: Typical swelling pressure test result

Table 4.8: values of Swelling Pressure, Initial dry density and Initial moisture content test results of the soil

Test Pit	Depth(m)	Dry density γ_d(gm/cc)	Swelling Pressure (kPa)	Natural Moisture content (%)
TP-01	1.7	1.16	107.23	42.6
TP-02	1.5	1.31	135.04	33.1
TP-03	1.5	1.23	-	45.7
TP-04	1.5	1.28	100.20	36.3
TP-05	1.8	1.32	141.20	31.2
TP-06	2.0	1.31	37.41	42.8
TP-07	1.5	1.26	118.03	43.2
TP-08	1.7	1.29	104.03	39.4
TP-09	2.0	1.29	127.52	31.6
TP-10	1.5	1.29	132.00	33.9
TP-11	1.5	1.03	69.50	43.4
TP-12	1.5	1.31	132.00	35.5
TP-13	1.5	1.08	94.50	36.6
TP-14	1.8	1.28	130.11	33.3
TP-15	1.5	1.29	137.50	31.9
TP-16	2.0	1.29	119.50	41.7
TP-17	2.0	1.26	104.45	28.5
TP-03*		1.31	134.50	19.9
TP-06**		1.39	182.00	29.6
TP-11**		1.35	194.50	30.3

* Air dry undisturbed specimen

** Remolded at optimum moisture content

Note: Swelling pressure test for Test Pit-03 (TP-03) is conducted on air dry undisturbed sample. Compaction tests were conducted for soil samples TP-06 and TP-11 to determine their optimum moisture content (Appendix-10) for remolding the soil samples and determine their swelling pressure of the soil.

4.2.8 Cation Exchange Capacity (CEC)

CEC is the quantity of exchangeable cations required to balance the negative charges on the surface of clay particle. A high CEC value indicates a high surface activity and a higher swell potential.

Soil CEC is normally expressed in units of charge per weight of soil. Two different, but numerically equivalent sets of units are used: meq/100 g (milliequivalents of charge per 100g of dry soil) or cmolc/kg (centimoles of charge per kilogram of dry soil).

Cation exchange capacity can be determined by direct analysis (using 1 M ammonium acetate (NH_4OAc)) or indirect calculation from routine analytical results. The sum of equivalent charges from exchangeable bases and acidity from routine analysis provides a value for CEC that is similar to CEC from the neutral NH_4OAc method. Two problems with this method of estimating CEC occurs with the presence of soluble salts or solubilization of calcium or magnesium carbonates. Bases extracted from soluble salts or carbonates are not exchangeable bases and thus will result in false high estimates for CEC.

For the study area the CEC is determined by Ammonium Acetate method at Debirezeyit Agricultural Research Center soil and water testing laboratory. CEC of the study area are found to be in the range of 42.04 – 69.08meq/100gm as shown in table 4.9. These values are compared to the ranges given in Table 2.4 and considering the Atterbeg limit test results, the clay minerals of the study area are put in the Illite clay minerals group.

Another test is also conducted to determine the concentration of the exchangeable cations. Na^+, k^+, Ma^{2+} and Ca^{2+} ions have their own role in the shrink-swell property of expansive soil. More swelling would occur in a soil having exchangeable sodium (Na^+) cations than same soil with calcium (Ca^{2+}) or magnesium (Mg^{2+}) cations.

Table 4.9: CEC and Exchangeable cations of the study area

Test Pit	Depth(m)	CEC (meq/100gm)	Exchangeable cations, cmol(+)/kg			
			Na^+	K^+	Ca^{2+}	Mg^{2+}
TP-01	1.7	42.04	1.00	2.73	122.73	20.10
TP-02	1.5	44.54	2.32	2.23	94.28	9.37
TP-03	1.5	51.14	1.27	2.20	102.93	8.69
TP-04	1.5	55.34	0.90	2.00	140.95	9.49
TP-05	1.8	56.94	1.68	2.57	87.94	9.58
TP-06	2.0	59.04	1.74	1.99	97.83	10.34
TP-07	1.5	50.74	1.72	2.53	77.82	7.43
TP-08	1.7	46.84	1.91	2.00	96.90	8.40
TP-09	2.0	48.14	0.91	2.04	103.90	9.10
TP-10	1.5	60.64	1.76	1.97	98.34	9.64
TP-11	1.5	67.51	1.10	2.44	171.96	11.58
TP-12	1.5	69.08	2.04	2.33	114.46	12.10
TP-13	1.5	46.24	1.10	3.00	135.00	22.11
TP-14	1.8	55.27	2.25	2.36	114.34	9.91
TP-15	1.5	57.34	1.94	2.86	87.94	8.40
TP-16	2.0	67.31	1.95	2.19	109.16	10.70
TP-17	2.0	47.66	2.48	2.39	100.88	10.03

CHAPTER 5

DISCUSSIONS ON TEST RESULTS

5.1 Discussions on Test Results of the Geotechnical Soil Properties

5.1.1 Grain Size Analysis

Wet sieving and hydrometer tests were performed to obtain the grain size distribution of fine particles. As it has been pointed out before, the tests were performed according to the standard test method given in ASTM D422 and presented in Table 4.2.

From the grain size analysis test result, it has been found that the Percentage (%) of sand content is in between 1.76 and 8.9%, the percentage of silt content is in between 22.16 and 47.30% and the percentage of clay content is in between 50.18 and 71.89%. Moreover, the percentage of colloidal particles (less than 0.001 mm in diameter) content is in between 47.10 and 69.95%.

According to Chen (1988), soils containing appreciable quantities of colloidal particles greater than 28% have very high degree of expansion. Soils containing 23% – 15% have medium to high degree of expansion while those containing colloidal particles less than 15% have low degree of expansion. The soils under consideration have high degree of expansion and agree with the above statements of Chen.

5.1.2 Natural Moisture Content and Initial Dry Density Test

The natural moisture content (NMC) of expansive soils controls the amount of swelling. Dry clays with low natural moisture content easily absorb moisture and cause the soil to swell. While, for clays with high moisture content swelling has already taken place and further expansion will be small. The natural moisture content of the soil in the study area ranges between 28.5% and 45.7%. Accordingly, the swelling potential of the soil sample at NMC of 45.7% (TP-03) is null while the swelling potential of the soil sample at NMC of 28.52% (TP-17) is 11.2%. We know that clays may desiccate due to different factors and upon subsequent wetting will again exhibit swelling. For example, lowering the

NMC of the undisturbed soil sample of TP-03 by exposing it to air, the swelling potential increases to 12.7%.

Density is commonly used as an indicator of relative compactness of soil. In expansive soils, the density decreases with water content increase. The increase in soil moisture with the decrease in soil density causes the soil to swell or heave upward against structure on it.

The initial dry densities of the study area ranged between 1.03gm/cc and 1.32gm/cc (Table 4.8). A soil having dry density in excess of 1.76gm/cc would be suspected of a very high potential for shrinkage or swelling [1]. However, the initial dry density test results of the samples in the study area are much less than 1.76gm/cc but the swelling characteristics of the soil were found to be still high.

5.1.3 Atterberg Limit Tests

Atterberg limits (liquid limit, plastic limit, shrinkage limit) were determined according to ASTM D4318-98 standard test method. The results of the Atterberg's limits are presented in Table 4.5. The Plasticity chart, (Figures 5.1 and 5.2) and Activity chart (Figure 5.3) were used to define the category of fine-grained materials. The measured liquid limit was found to be in the range of 87.9-122.5%. The plasticity index and shrinkage limit were found to be in the range of 47.7- 86.8% and 7.5-19.2% respectively. Based on the Atterberg limit test results classification of soil of the study area is discussed as follows:

Using Unified Soil Classification System (USCS) the soils of the study area are classified by using the Casagrande Plasticity Chart (Figure 5.1). Based on this chart, most of the soils were grouped as CH (inorganic clay with high plasticity) and only two of the seventeen soil test samples are on MH group (inorganic silt of high compressibility)

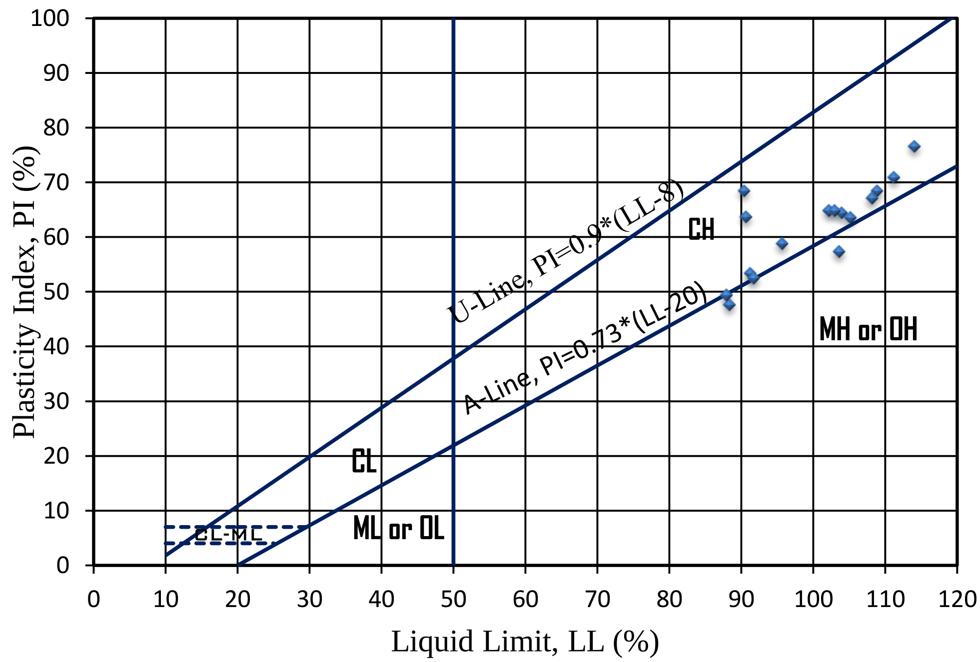


Fig 5.1: Pasticity Chart

Also after classification was done by using **AASHTO Classification System** (Figure 5.2), the soils in the area are put in soil groups A-7-6 and A-7-5. These soil groups' materials have high liquid limits and are highly plastic as well as subject to considerable volume change up on moisture change.

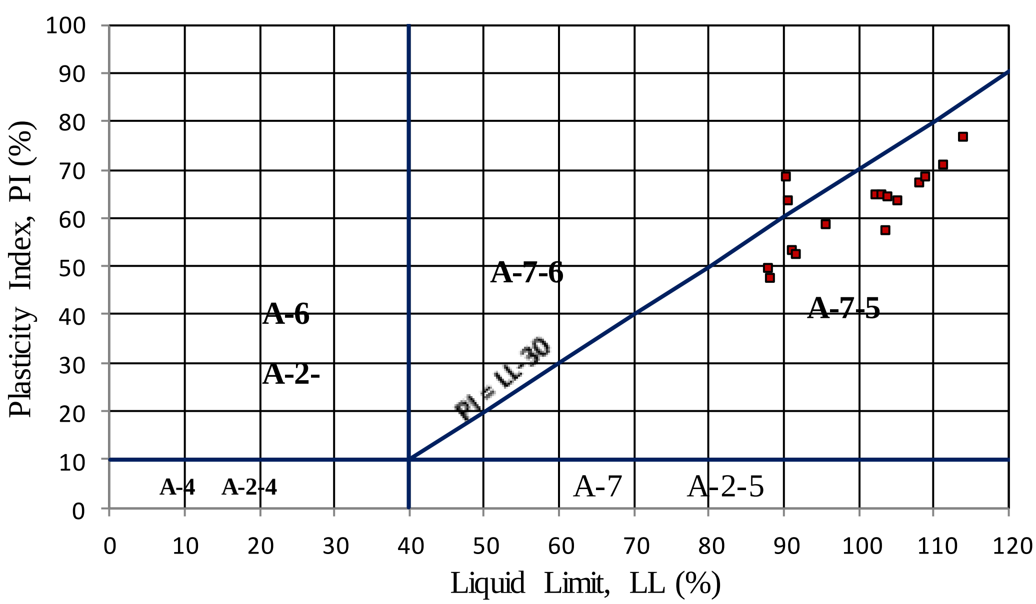


Fig 5.2: Plasticity Chart for AASHTO Classification methods

Another way of identifying the expansive soil is to use the **Activity Chart method** proposed by Skempton. Because the plasticity of soil is caused by adsorbed water that surrounds clay particles, one can expect that the type of clay minerals and their proportional amounts in a soil will affect the liquid and plastic limits. Skempton (1953) observed that the plasticity index of a soil increases linearly with the percentage of clay-size fraction (% finer than 2μm by weight) present.

He proposed three classes of clays according to the activity ratio as follows: Soil with activity less than 0.75 is inactive indicating low potential for volume change, that with activity between 0.75 and 1.25 is normal, and above 1.25 is very active demonstrating very high potential for volume change These values are presented in the form of chart (Figure 5.3). The soil of the study area is compared to these values and it falls in the range of normal.

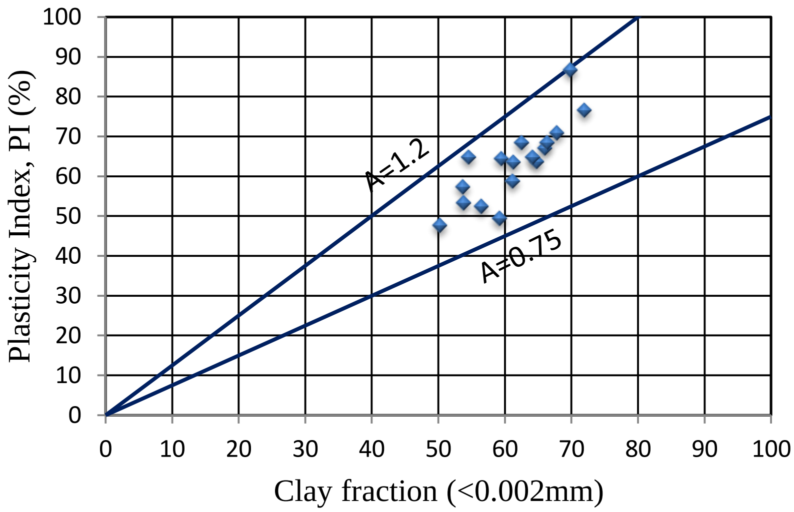


Fig 5.3: Activity Chart

5.1.4 Free Swell Test

The free swell test is a simple test that is widely accepted as a way of getting an estimate of soil swelling potential. This test suggested by Holtz and Gibbs (1956) was carried out on all specimens from the study area. As stated before, the test is performed by slowly pouring 10cm^3 of dry soil passing a 0.425mm sieve into a 100cm^3 graduated jar cylinder with water, and observing the swelled volume of the soil after it comes to rest (Holtz and Kovacs, 1981). In this study, the variation in free swell percent ranged from 80% to 140% (Table 4.7) indicating high swelling potential. This implies that the soils in the area can swell considerably when wet.

5.2 Swell Pressure Test

The swelling pressure and the amount of swell of the soil were measured by means of one-dimensional compression tests using the oedometer apparatus as per ASTM D4546-08 .The results of the oedometer tests showed that the soils can exhibit swelling pressure in the range of 37.41kPa to 141.20 kPa. That means the expansive clay in area under study can exert an upward swelling pressure in this range, which is much greater than pressure exerted by lightweight structures on the subsoil.

5.3 Relationship between Different Soil Properties of the Study Area

5.3.1 Swelling Pressure and Plasticity Index

The dependence of the swell pressure on the plasticity index is shown in Fig 5.4 for samples of all other soil properties are different.

The figure shows that there is a positive relationship between the plasticity index (PI) and swelling pressure. However, the points are highly scattered from the trend line and with small coefficient of regression i.e R=0.3. This shows that the relationship between swelling pressure and plasticity index is weak.

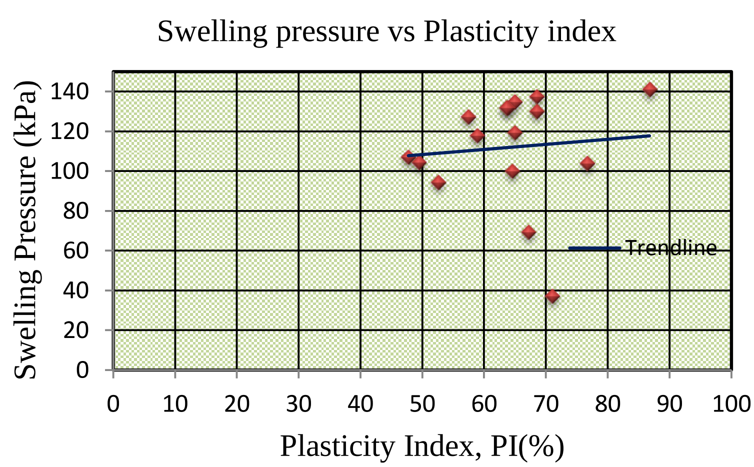


Fig 5.4: Swelling Pressure and Plasticity Index relationship

5.3.2 Swelling Pressure and Natural Moisture Content

Natural moisture content is also another factor which plays a role in swelling characteristics of expansive soils. Graph is plotted to show the relationship between moisture content and swelling pressure for the entire soil samples. The plot shows that there is a tendency of decreasing swelling pressure with increment of moisture content.

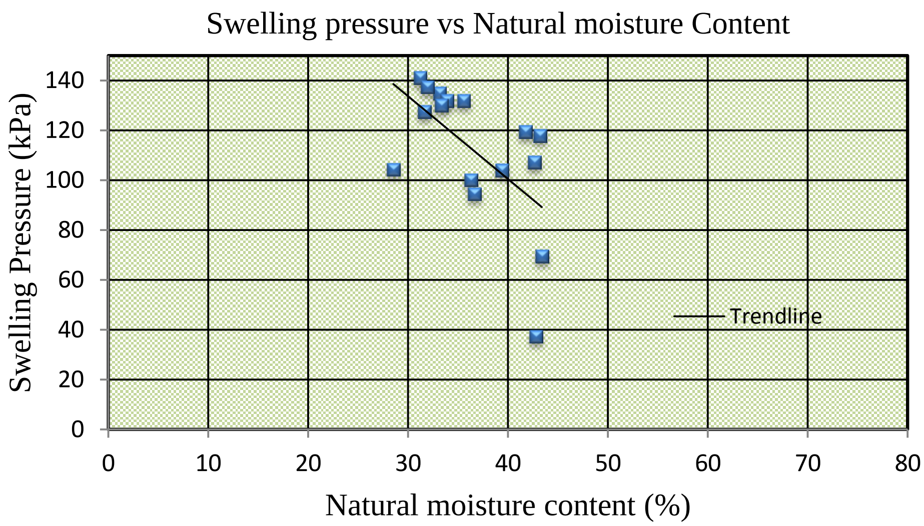


Fig 5.5: Swelling Pressure and Natural moisture content relationship

5.3.3 Swelling Pressure and Dry Density

Dry density, which is a measure of the compactness of soil grains, is also another factor, which plays a role in swelling characteristic of expansive soils. A graph is plotted to see the relationship between the dry density and swelling pressure of the study area. The result shows that there is a high correlation with regression coefficient of 0.85. This shows that there is a tendency of increment of swelling pressure as the dry density increases.

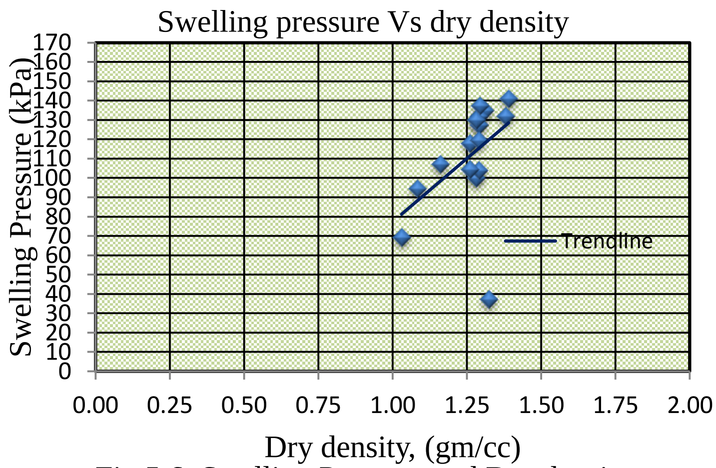


Fig 5.6: Swelling Pressure and Dry density

5.4 Comparison of test results of Addis Ababa expansive soils and the study area

Different laboratory soil tests range of values of the expansive soils of the study area are summarized and compared with range of laboratory test results of Addis Ababa expansive soils that were carried out by various researchers and presented in Table 5.1.Most of the geotechnical properties of the soils of the study area and that of Addis Ababa expansive soils are almost similar. The CEC of Addis Ababa expansive soils is as high as 80 meq/100gm[17], which is much greater than the CEC value of Asella soils. Moreover, the very important characteristic of expansive soils, swelling pressure, of Addis Ababa soils exceeds that of Asella expansive soils.

Table 5.1: Comparison of test results of Addis Ababa expansive soils and the study area

Study area/ Researcher	LL (%)	PL (%)	PI (%)	Clay Content (%)	Sand Content (%)	Free Swell (%)	Swelling Pressure (kPa)	Shrinkage Limit (%)	Specific Gravity
Asella/ Abraham M.	87-122	35-46	48-88	50-70	1.76-8.9	80-140	37-141	7.5-19.2	2.59-2.78
Addis Ababa/ Tewold. E [16]	91-106	24-29	64-80	55-68		80-110	205-388	5-13	2.62-2.72
Addis Ababa/ Daniel T. [16]	96-109	24-28	55- 77	45-80	1-4	75-140	108-420	3-10	2.76-2.85

CHAPTER 6

DEVELOPMENT OF REGRESSION MODELS

6.1 Previous Investigations

Previously various forms of empirical equations were proposed which relate swelling pressure to certain physical properties of soils, such as consistency limits, clay content, initial moisture content and initial density which do not require sophisticated laboratory procedure and apparatus. These empirical equations are easy to apply and give satisfactory results when applied to the particular soils for which they were developed.

However, equations developed for soils in one place may not work at all if tested on soils of other place of the same region. Hence specific models have to be developed for specific areas in order to obtain good result.

Investigations carried out by Daniel Teklu and Dagimawi Nugusse on undisturbed soils of Addis Ababa and Bahardar soils respectively suggested that the existence of strong correlations between the swell pressure and the soil properties. However, these equations should be checked if they apply for the soil of this research area.

6.2 Models Developed by Various Researchers

As mentioned previously, there are different empirical equations developed to determine the swelling behavior of a soil. Some of these are presented below.

a) Daniel Teklu (Addis Ababa expansive soil)

$$\text{Log Sp} = \text{LogP} = -5.00 - 0.0002064LL + 0.003477PI + 0.005827\gamma d$$

b) Dagimawe Nigussie (Bahirdar Expansive soil)

$$\text{Log Sp} = 7.042-1.92\,\gamma d-0.046wi -0.609A$$

c) Komornik and David (1969) proposed the following correlation for swelling pressure of undisturbed clays

$$\text{Log Sp} = 2.132 + 0.0208LL + 0.00665\gamma d - 0.0269wi$$

Where: LL= Liquid limit (%),

Ps=swelling pressure (kg/cm^2)

wi=moisture content (%)

γd =dry density (gm/cm^3) and A=activity

In these equations index properties that are believed to have significance for swelling are used as independent variables. Obviously the proposed equations might have served their purpose in areas where they have been specifically developed. At this point it is worthwhile to test these equations for the soil of the study area and to examine the outcome.

6.3 Evaluation of Previously Developed Models for Soils of the Study Area

Table 6.1:Comparison of previously developed equations with measured values

Test Pit	Depth (m)	Measured Swelling Pressure (kPa)	Daniel Teklu	Dagimawe Nigussie	Komornik and David
TP-01	1.70	107.23	81.20	184.92	66.21
TP-02	1.50	135.04	688.74	185.94	231.94
TP-03	1.50	0.00	222.04	91.21	62.90
TP-04	1.50	100.20	458.62	176.56	207.86
TP-05	1.80	141.20	929.20	148.69	692.85
TP-06	2.00	37.41	629.40	77.15	196.22
TP-07	1.50	118.03	336.52	110.35	91.08
TP-08	1.70	104.03	574.96	125.02	277.64
TP-09	2.00	127.52	504.10	281.20	272.38
TP-10	1.50	132.00	524.76	250.53	127.48
TP-11	1.50	69.50	16.33	276.53	162.84
TP-12	1.50	132.00	680.64	130.87	230.70
TP-13	1.50	94.50	29.79	506.19	112.72
TP-14	1.80	130.11	472.48	260.11	315.72
TP-15	1.50	137.50	570.23	259.72	142.17
TP-16	2.00	119.50	526.46	104.99	141.32
TP-17	2.00	104.45	313.28	621.98	155.90

From the Table 6.1, one can observe that none of the previously developed empirical correlation can precisely predict the swelling pressure of the study area.

6.4 Development of New Correlation

Detail statistical analyses of soil index properties and swelling pressure of expansive soil of the study area are carried out using various datasets to determine suitable correlations for estimating swelling pressure.

To develop this new linear correlation to predict, swelling pressure(SP) in kPa (as output), plasticity index (PI), Liquid limit (LL), dry density (γ_d),initial moisture content(w),clay content(cc), CEC, Na^+, activity(A), Liquidity index(LI), and Shrinkage limit(SL) are taken as inputs. For this analysis, 15 data points were used for development of new model. Measured swelling pressure values of two tests pits (TP-03 and TP-06) are small. Hence, these 2 data points are ignored in the development of the model.

The analysis is carried out by using computer software SPSS 20 data editor. Here in this statistical analysis some non-index properties such as CEC and Na^+ are also used as an input to provide an equation for any user who could have an access to conduct CEC and Na^+.

Using laboratory results obtained for the research area eleven new correlations are selected among many and the predicted values are compared with the measured values of swelling pressure. Developed correlations and correlation coefficient are shown as follows:

Eqn.#1: $LogSp=0.71\gamma_d-0.001PI-0.002w+1.318$, $R^2=0.7097$

Eqn.#2: $LogSp=0.676\gamma_d-0.002w-0.001CEC+1.342$, $R^2=0.7107$

Eqn.#3: $LogSp=0.717\gamma_d-0.002w-0.001CEC-0.001PI+1.325$, $R^2=0.7059$

Eqn.#4: $LogSp=0.704\gamma_d-0.001PI-0.002w+0.003Na^++1.320$, $R^2=0.7097$

Eqn.#5: $LogSp=0.615\gamma_d-0.003w+0.053A+1.331$, $R^2=0.7147$

Eqn.#6: $LogSp=0.656\gamma_d-0.001w-0.052LI+1.285$, $R^2=0.7106$

Eqn.#7: $LogSp=0.756\gamma_d-0.007S_L-0.003PI+1.387$, $R^2=0.7418$

Eqn.#8: $LogSp=0.676\gamma_d-0.005CC-0.001w-0.008S_L+1.694$, $R^2=0.8116$

Eqn.#9: $LogSp=0.726\gamma_d-0.001CEC+1.224$, $R^2=0.6699$

Eqn.#10: $LogSp=0.691\gamma_d+0.006Na^++1.181$, $R^2=0.6951$

Eqn.#11: $LogSp=0.741\gamma_d-0.002w-0.002LL+1.382$, $R^2=0.5008$

Where: Sp, swelling pressure (kPa)

PI, plasticity index in (%)

γ_d, initial dry density (gm/cc)

CEC = meq/100gm, Na^+=cmol/kg

w, initial moisture content (%)

LL, liquid limit (%)

CC, clay content (%),

SL, shrinkage limit (%)

A, activity

LI, liquidity index

6.5 Comparison of Measured and Calculated Values of Swelling Pressure

Using the above new equations the swelling pressure of the study area is compared with the measured values as shown in table 6.2 to see how well the developed equations have predicted the swelling pressure.

Table 6.2: Comparison of Measured value with Calculated Value.

TP	Depth (m)	Measured swelling pressure (kPa)	Calculated Value of Swelling pressure (kPa)										
			Eqn.#1	Eqn.#2	Eqn.#3	Eqn.#4	Eqn.#5	Eqn.#6	Eqn.#7	Eqn.#8	Eqn.#9	Eqn.#10	Eqn.#11
TP-01	1.7	107.23	102.07	99.81	95.94	101.61	92.73	100.31	98.20	109.04	105.79	97.47	143.43
TP-02	1.5	135.04	130.89	130.85	122.62	130.03	126.02	131.88	121.19	144.80	135.06	125.93	120.84
TP-03	1.5	0.00	107.32	107.73	98.89	106.73	105.04	106.90	98.66	121.44	116.78	109.64	105.00
TP-04	1.5	100.20	122.96	120.06	112.30	122.20	116.68	123.29	113.55	127.36	125.30	117.73	112.24
TP-05	1.8	141.20	127.65	130.31	116.23	126.79	130.35	132.56	111.07	130.61	133.47	126.82	112.93
TP-06	2.0	37.41	121.46	119.17	110.02	120.68	114.19	123.90	113.44	118.71	128.46	122.95	109.00
TP-07	1.5	118.03	116.79	113.94	107.76	116.10	106.53	115.53	114.67	119.91	122.47	115.34	109.17
TP-08	1.7	104.03	119.80	122.58	111.59	119.05	115.54	123.15	110.25	116.02	129.92	121.29	107.45
TP-09	2.0	127.52	130.07	126.93	120.80	129.25	122.21	129.90	113.96	130.00	129.81	119.87	117.11
TP-10	1.5	132.00	126.58	121.80	114.22	125.78	118.85	126.77	126.28	134.66	125.86	121.05	122.73
TP-11	1.5	69.50	78.58	76.56	69.51	78.37	77.32	82.30	71.84	78.36	80.22	79.31	69.54
TP-12	1.5	132.00	129.85	122.30	114.95	128.99	121.68	129.93	118.46	128.56	127.64	125.44	117.89
TP-13	1.5	94.50	91.43	90.07	85.00	91.12	86.40	91.46	85.15	93.91	92.04	86.28	84.71
TP-14	1.8	130.11	123.50	121.73	112.81	122.74	118.33	124.93	110.77	118.86	125.32	119.95	111.24
TP-15	1.5	137.50	127.04	124.49	115.52	126.23	122.70	127.70	127.85	145.95	127.54	122.00	124.70
TP-16	2.0	119.50	121.76	115.68	108.20	120.99	112.94	122.04	116.43	122.42	123.95	121.37	111.84
TP-17	2.0	104.45	127.67	122.76	118.64	126.92	116.06	124.00	117.97	121.66	123.34	116.56	121.01
TP-03*	1.5	134.50	135.71	135.40	125.21	134.82	137.79	126.89	112.99	144.66	133.02	124.12	133.52
TP-06**	2.0	182.00	151.39	147.44	137.35	150.22	144.04	147.71	133.78	142.14	150.50	142.96	136.78
TP-11**	1.5	194.50	140.02	133.05	124.48	139.03	132.18	137.00	125.12	132.37	136.66	131.69	126.76

TP-03*- air dry undisturbed sample

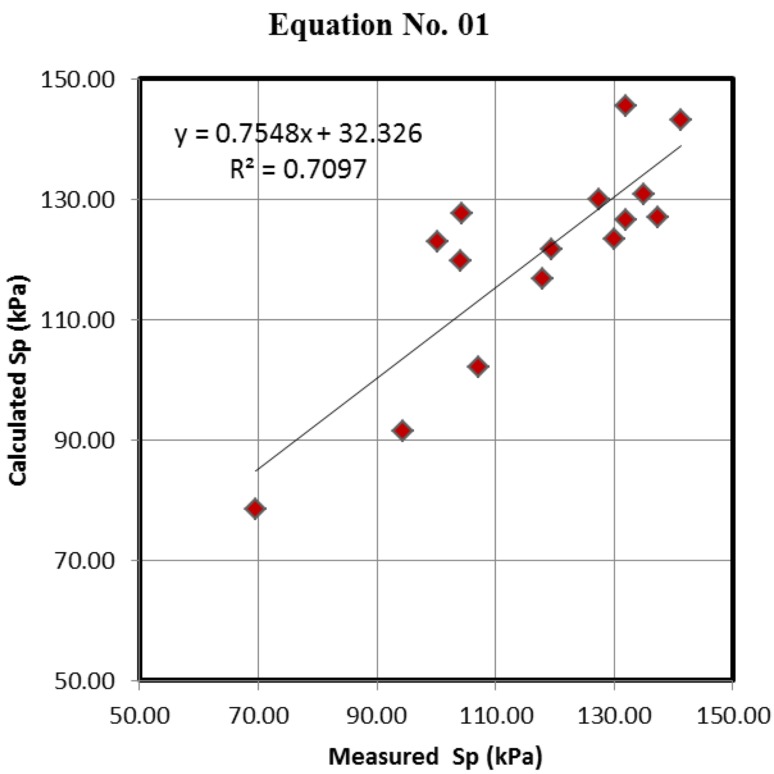
Tp-06** and TP-11**-Remolded sample at optimum moisture content

Here TP-03* (air dry undisturbed sample) and TP-06** (remolded sample at optimum moisture content) can be used as a control sample.

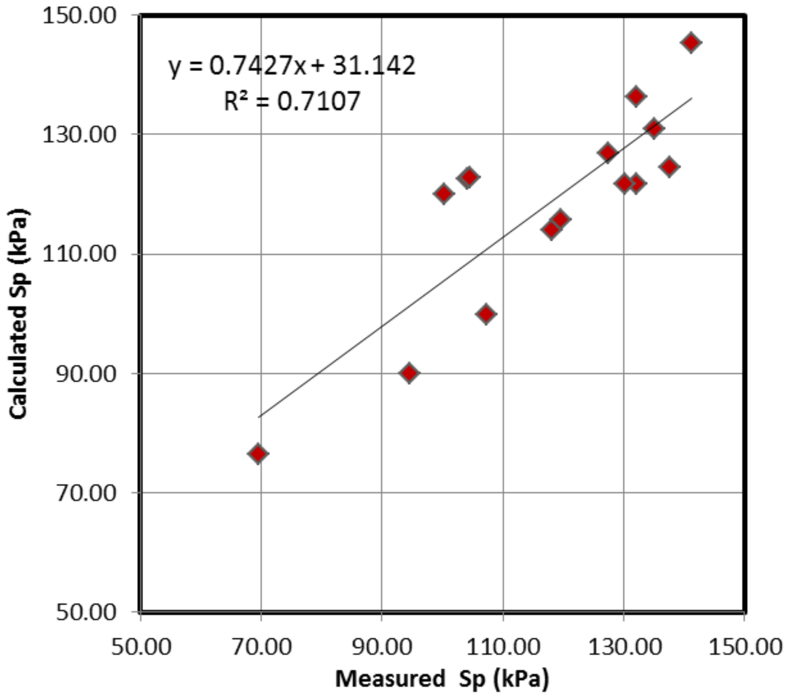
A comparative model assessment of the regression model developed to predict swelling pressure of the studied soil correlating,(a)Sp, γd, w,A (b) Sp, γd, SL, PI (c) Sp, γd, cc, w, SL (i.e Eqn.#5, Eqn.#7 and Eqn.#8 it table 6.2) indicates that the developed models provide a very good agreement with the measured values.

6.6 Selection of Better Equations and Comparison of Calculated and Measured Values

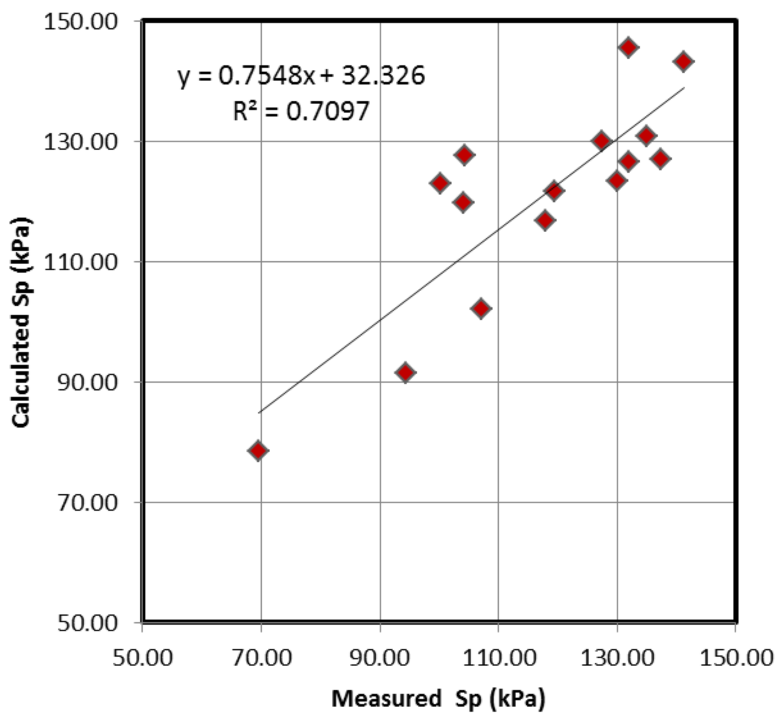
Graphs are plotted between measured and predicted values for comparison (Fig 6.1) regression lines are drawn to observe the gap between the measured and the calculated values.



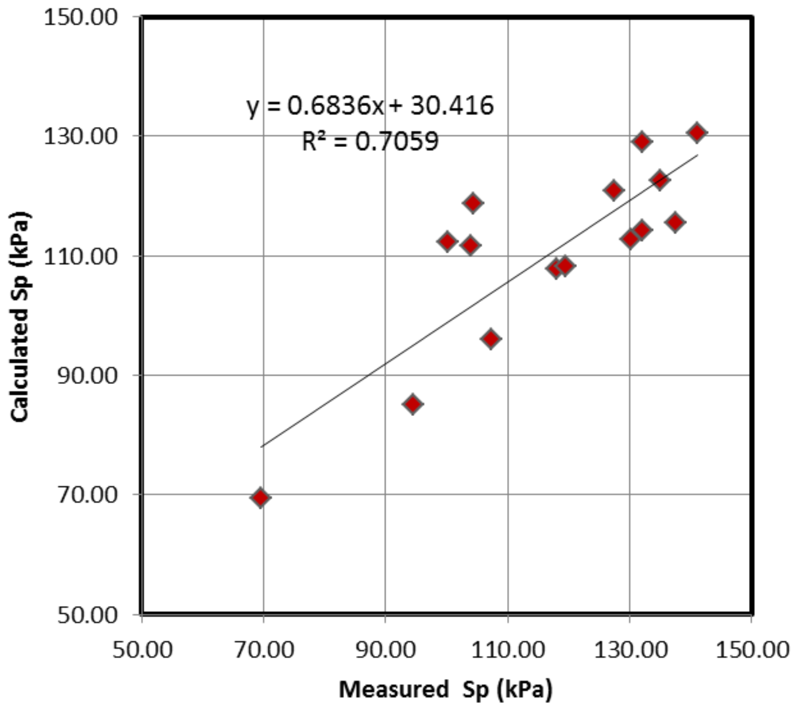
Equation No. 02



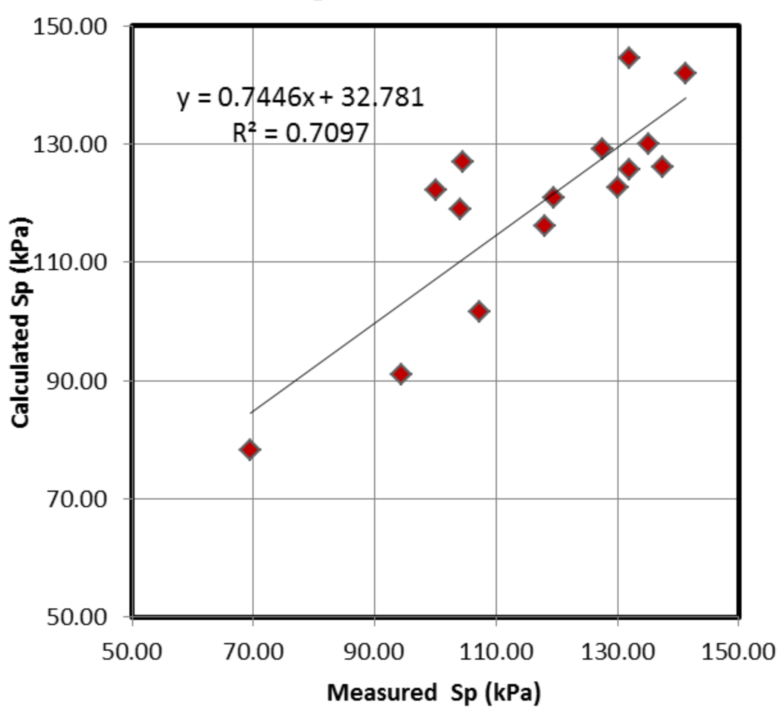
Equation No. 01



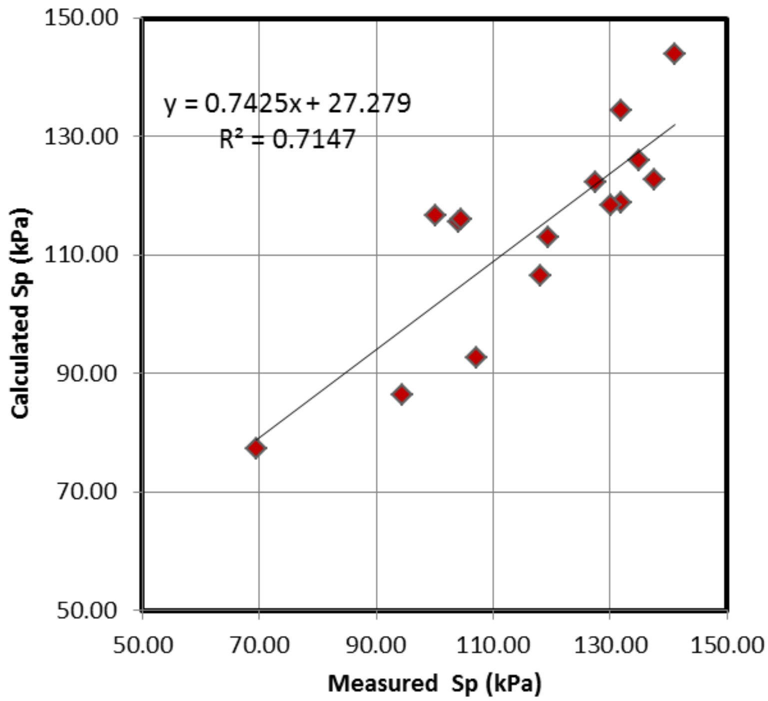
Equation No. 03



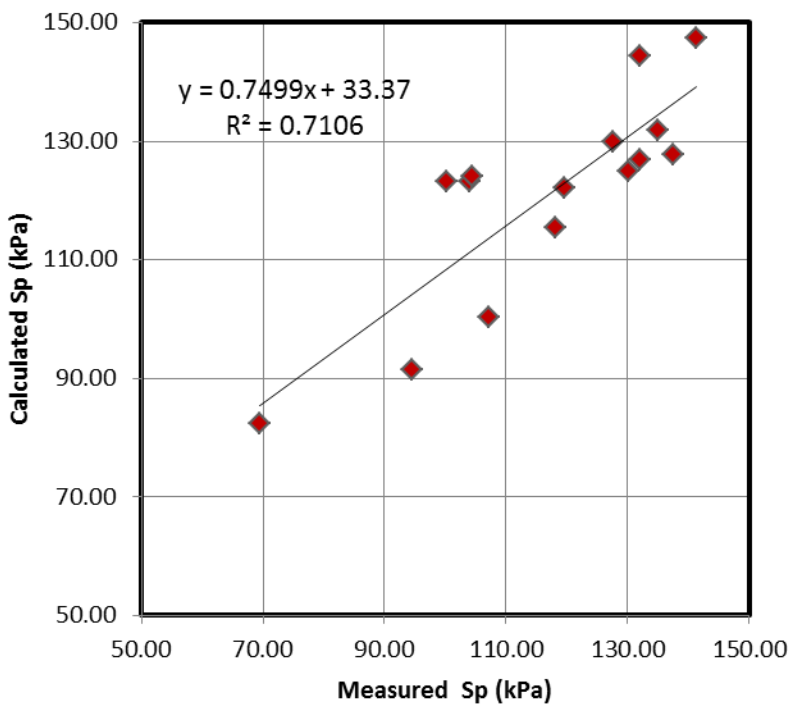
Equation No. 04



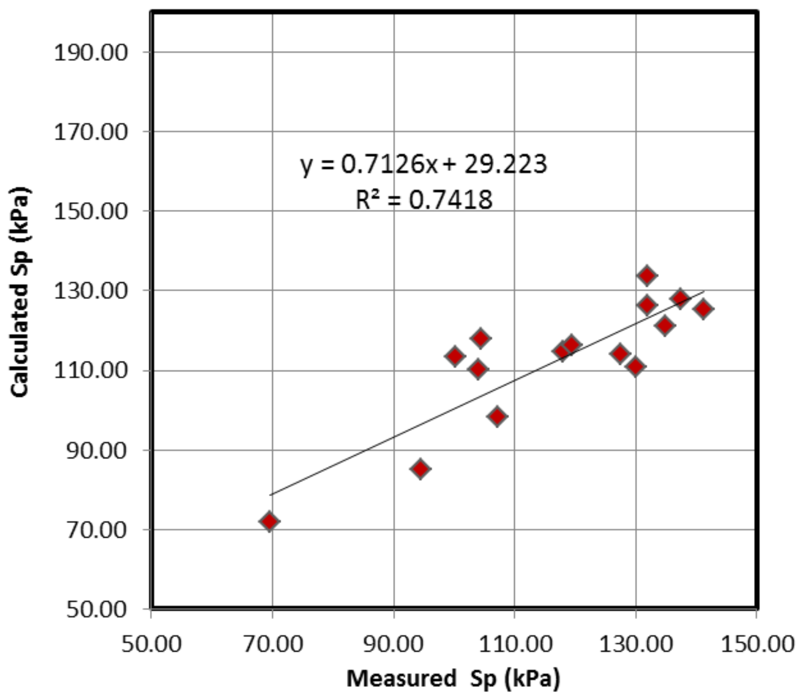
Equation No. 05



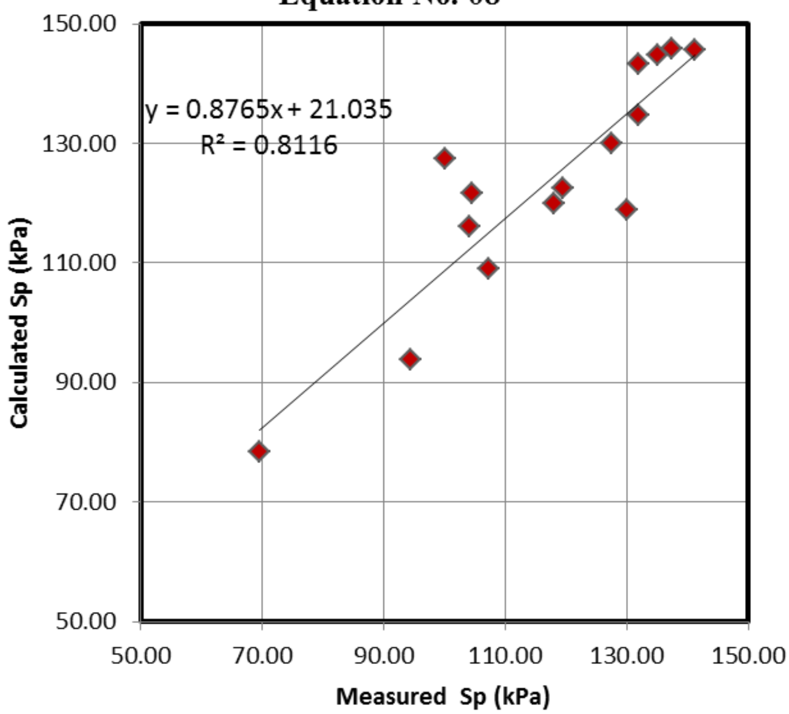
Equation No. 06



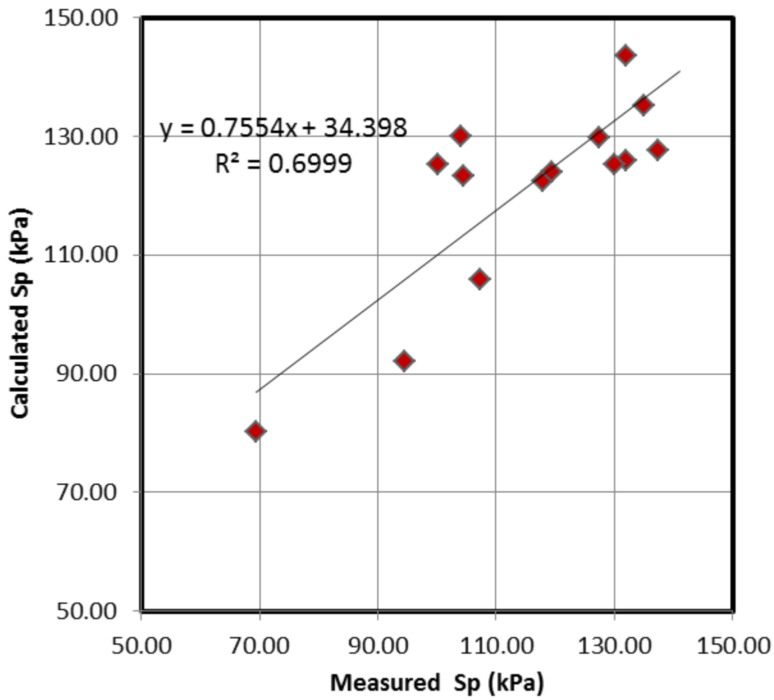
Equation No. 07



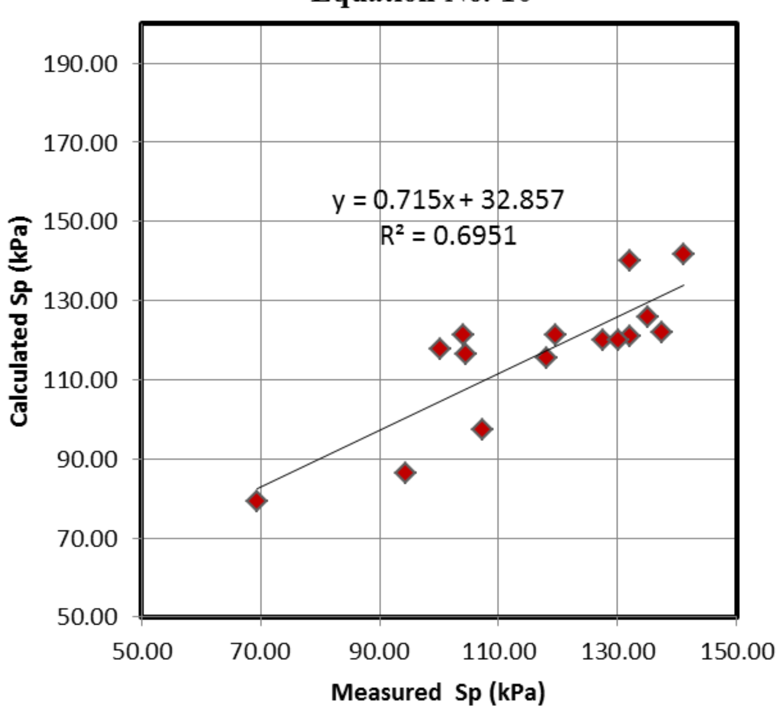
Equation No. 08



Equation No. 09



Equation No. 10



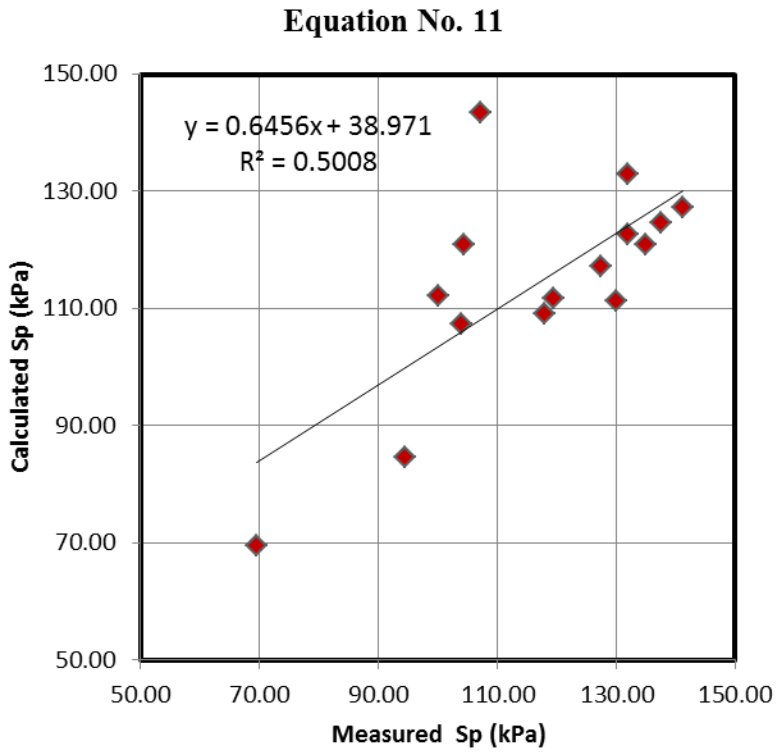


Fig.6.1: Comparison of calculated and measured values

Among eleven regression models developed between predicted and measured values those three equations which have larger R^2 value are selected for this research. The equations are: Eqn. #5, Eqn. #7 and Eqn. #8.

LogSp=0.615γd-0.003w+0.053A+1.331, R^2= 0.7147………. (a)

LogSp=0.756γd-0.007SL-0.003PI+1.387, R^2=0.7418……….. (b)

LogSp=0.676γd-0.005CC-0.001w-0.008SL+1.694, R^2=0.8116…(c)

Either of the above three equations can be used to determine the swelling pressure values of the study area with the help of basic soil properties, which can be easily found out in any laboratory.

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

1. The soils of the studied area are grouped as A-7-6 and A-7-5 as per AASHTO classification system. Soils in this group have high liquid limits and are highly plastic. Moreover, as per USCS soil classification system about 88% of the soils are grouped as CH (inorganic clays with high plasticity.
2. As per some classification systems developed to classify expansive soils, most of the soils of the studied area are in the range of high to very high degree of expansion.
3. Swelling pressure of the expansive soils in the study area ranges from 69.50 – 141.20 kPa, which cannot be prevented from swelling by dead load pressure that could be exerted by ordinary buildings and an embankment of small thickness.
4. Comparison of the very important characteristic of expansive soils, i. e swelling pressure of Addis Ababa expansive soils exceeds that of the swelling pressure of the study area.
5. The equation developed using initial dry density, clay content, initial moisture content and shrinkage limit as an input can predict swelling pressure for the range of data used for developing this equation.
6. A comparative assessment between measured and calculated values of the swelling pressure indicates that the developed equation provides good agreement with the measured values.
7. Finally, since prediction of swelling pressure by empirical relationships cannot be expected to yield accurate results, swelling pressure must be determined from oedometer tests.

7.2 Recommendations

1. To clearly understand the swelling characteristics of the expansive soils of the area it is recommended to increase the number of samples from different test pits and carry out more number of laboratory analyses.
2. It is better to undertake some swelling pressure test on disturbed soil samples that are remolded at maximum dry density to find out the possible maximum swelling pressure of the soils.
3. Since this problematic expansive soil covers only some areas of the town, boundary where this soil is really available must be clearly marked.

REFERENCE

1. F.H. Chen, *Foundation on Expansive Soil*, New York: Elsevier Science, 1998.

2. John D.Nelson, Debora J. Miller, *Expansive Soils Problems and Practice in Foundation and Pavement Engineering*, New York: John Wiley and Sons, Inc.1992.

3. Murthy,V.N.S, *Soil Mechanics and Foundation Engineering*, Fifth Revised Edition, New York: UBS Marcel Dekker Inc,2001.

4. Bowles J.E., *Foundation Analysis and Design*, Fifth Edition, New York: McGraw-Hill Companies Inc, 1996

5. Amer Ali, Al-Rawas and Mattheus F.A. *Goosen, Expansive Soils Recent advances in characterization and treatment,* London: Taylor and Francis, 2006

6. Negussie D., *In depth Investigation Of Relationship Between Index Property And Swelling Characteristic Of Expansive Soil in Bahir Dar*, Unpublished M.Sc. Thesis, Addis Ababa University, 2007.

7. Teklu D., *Examining the Swelling Pressure of Addis Ababa Expansive Soil*, Unpublished M.Sc. Thesis, Addis Ababa University, 2003.

8. Dr.K.R.Arora, *Soil Mechanics and Foundation Engineering*, Delhi: A.K.Jain, 2003

9. Michael E. Kalinski, Ph.D., P.E.(2011), *Soil Mechanics Lab Manual*, 2nd Edition University of Kentucky , Frankfort:John Wiley & Sons, Inc, 2011.

10. American Society for Testing and Material (ASTM), Annual Book of ASTM Standards, Washington DC: 1996.

11. American Association of State Highway and Transportation Officials (AASHTO), *Standard Specification for Testing Transportation Material and Method of Sampling and Testing*, 25th edition, Washington DC:2005

12. Teferra A. and Liekun M., *Soil Mechanics*, Addis Ababa:Addis Ababa University press,1999

13. Robert D. Holtz, and William D. Kovacs, *An Introduction to Geotechnical Engineering*, New Jersey: prentice- Hall Inc,1981.

14. Cheng Liu and Jack B. Evete , *Soils and foundations*, 7th edition , New Jersey: Pearson education Inc, 2008.

15. Das, B.M., *Advanced Soil Mechanics*, Philadelphia: Taylor and Francis Publishers,1997.

16. Sisay A., *Assessment of damage of buildings constructed in expansive soil areas of Addis Ababa*, Unpublished M.Sc. Thesis, Addis Ababa University, 2004.

17. Kassa M., *Relationship between Consolidation and Swelling Characteristics of Expansive Soils of Addis Ababa*, Unpublished M.Sc. Thesis, Addis Ababa University,2005.

APPENDICES