ADDIS ABABA UNVERSITY

SCHOOL OF GRADUATE STUDIES
FACULTY OF TECHNOLOGY
DEPARTMENT OF CIVIL ENGINEERING

EXAMINING THE SWELLING PRESSURE OF ADDIS ABABA EXPANSIVE SOIL

A Thesis Submitted to the

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Master of Scienence in Geo-technical Engineering

By

Daniel Teklu

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Abstract

So many damages have been reported as a result of expansive soil. Such damages occur when the pressure exerted by the soil is greater than the foundation pressure. Consequently assessing the swelling pressure is an important step in designing foundation on expansive soil.

Several researchers have developed an empirical relation for the estimation of swelling pressure. The empirical expressions relate the swelling parameters to the geotechnical parameters that are determined by the identification tests. But the expressions should either be reformulated or checked for our local condition.

Therefore in this paper: the various equations already developed are tested for the validity to our condition and new equations are developed that relates swelling pressure with the index and physical properties.

The new formulas are developed, by taking one or more of the four parameters (liquid limit, moisture content, dry density and plastic index) in different combinations. The equations are developed by taking 14 samples and using

SPSS 9.0 for windows software. The equations are then tested for three control samples. The most important equations are proposed, and conclusions and recommendations are made.

Chapter 1

1. Introduction.

1.1 General

Expansive soil is a term generally applied to any soil or rock material that has a potential for shrinking or swelling under changing moisture conditions. Its problem in civil engineering structures was first identified in the later part of 1930's. Since then so many countries reported the problem. These countries are: Argentina, Australia, Burma, Canada, Cuba, Ethiopia, Ghana, India, Israel, Iran, Kenya, Mexico, Morocco, Zimbabwe, South Africa, Spain, Turkey, U.S.A, Venezuela, and a number of other countries.

Expansive soils owe their expansive character mainly to the constituent clay mineral. The most important clay mineral, which is the cause for expansive nature is montmorillonite. Montmorillonite has an octahedral sheet sandwiched between two silica sheets. When this mineral is exposed to moisture, water is absorbed between interlayering lattice structures and exert an upward pressure. This upward pressure, known as swelling pressure, causes most of the damages associated with expansive soils.

Most of the structural damage on expansive soil results from the differential rather than the total movement of the foundation soil as a result of swell. Differential movements distribute equilibrium of a system by redistributing the structural loads causing concentration of loads on portions of the foundation and large changes in moments and shear forces in the structure not previously accounted for in the standard design practice.

Damages can occur within a few months following construction, may develop slowly over a period of about 5 years, or may not appear for many years until some activity occurs to disturb the soil moisture. The probability of damages increases for structures on swelling foundation soils if the climate and other field environment, effect of construction, and effect of occupancy tend to promote moisture changes in the soil.

1.1.1 Structures susceptible to damages.

Types of structures most often damaged from swelling soil include foundations and walls of residential and light (one- or two-story) buildings, highways, canal and reservoir linings, and retaining walls. Lightly loaded one- or two-story buildings, warehouses, residences, and pavements are especially vulnerable to damage because these structures are less able to suppress the differential heave of the swelling foundation soil than heavy, multi-story structures.

1.1.2Type of damages:

Damages sustained by structures due to expansive soils include: distortion and cracking of pavement and on-grade floor slabs; cracks in grade beams, walls, and drilled shafts; jammed or misaligned doors and windows; and failure of steel or concrete plinths (or blocks) supporting grade beams. Lateral forces may lead to buckling of basements and retaining walls, particularly in over consolidated and non-fissured soils. The magnitude of damages to the structures can be extensive, impair the usefulness of the structure, and detract aesthetically from the environment. Maintenance and repair requirements can be extensive, and the expense can grossly exceed the original cost of the foundation.

1.2 Back ground

Expansive soil is known to be widely spread in Ethiopia. Although the extent and range of distribution of this problematic soil has not been studied thoroughly: the southern, south-east and south-west part of the city of Addis Ababa areas, where most of the recent construction are being carried out and central part of Ethiopia following the major trunk roads like Addis-Ambo, Addis-Woliso, Addis-Debre Birhan, Addis-Gohatsion, Addis-Modjo are covered by expansive soils. Also areas like Mekele and Gambela are covered by expansive soils. The distribution is shown in Fig 1.1.

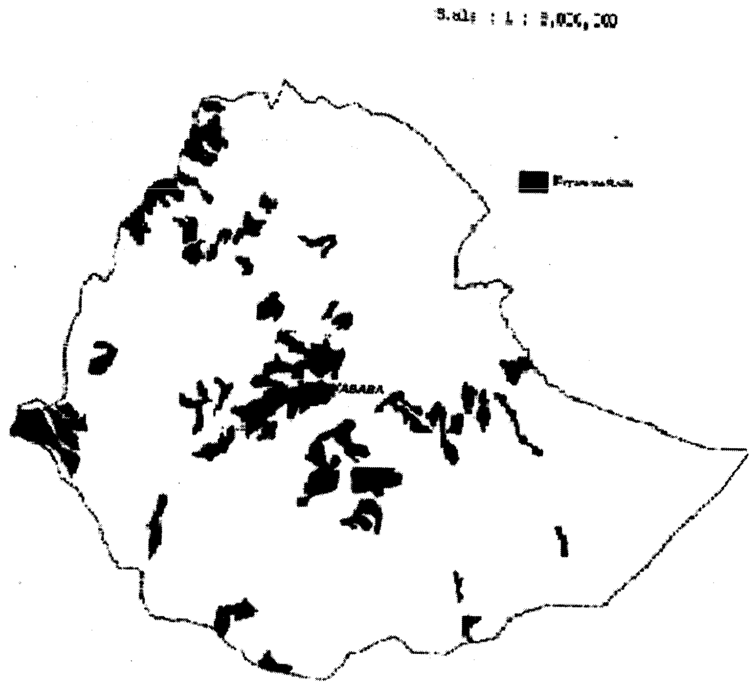

Fig. 1.1 Distribution of expansive soil in Ethiopia

In Ethiopia, a lot of damages have been occurring due to expansive soil. Although there is not an organized economic survey, it is assumed that most of the economic loss due to failures associated with civil engineering construction are attributed to expansive soils. Low level of understanding of the mechanisms of heaving and the measures required to counter such problem by civil engineers and architects, wrong concept of safety associated with the conservative design, improper soil investigation and interpretations, and the use of soil parameters, chart and tables developed for temperate zone soils have contributed for most of the failure caused by expansive soil area in Ethiopia.

1.3 Brief review of previous works

Although expansive soil is the cause for most damages on civil engineering structures in Ethiopia, only limited number of study has been made. Some of the study made on expansive soils are: a case study of building damages due to expansive soils in Addis Ababa (Yohanes, 1982), some characteristics of Addis Ababa soils (Woldemedhen, 1967), investigation on improving the geotechnical properties of " black cotton" soils by blending with red clay soil (Tesfaye, 1989), chemical treatment of black cotton soils to make it usable as a foundation material (Tesfaye, 1994), Investigation of the influence of moisture variation on the characteristics of expansive soils in Addis Ababa (Yeheyis, 1993), a rational design procedure of stiffend mat on swelling soil profile in Addis Ababa (Bejeiga, 1994), Stabilization of potentially expansive sub-grade soil using lime and conead (Nebero, 1994). Except the study made by (Yeheyis, 1993) no detail investigation is made on the swelling pressure of expansive soil of Addis Ababa that gives some correlation between index properties and design parameters as output.

1.4 Purpose

The purpose of this thesis is:

1) To study the swelling pressure of expansive soils of Addis Ababa.

2) To try to make some correlation between index properties and swelling pressure of expansive soils of Addis Ababa.

Chapter 2

General review on expansive soils

2.1 Introduction

Most soil classification systems arbitrary define clay particles as having an effective diameter of two microns (0.002mm) or less. Particle size alone doesn't determine clay mineral. Probably the most important grain property of fine-grained soils is the mineralogical composition (chen,1988). For small size particles, the electrical forces acting on the surface of the particles are much greater than the gravitational force. These particles are in a colloidal state. The colloidal particles consist primarily of clay minerals that were derived from parent rock by weathering.

The three most important groups of clay minerals are Montmorillonite, Illite, and Kaolinite. Montmorillonite is the clay mineral that is mostly present in expansive soil. When these minerals are exposed to moisture, water is absorbed between the inter-layering lattice structures and exerts an upward pressure, which is the cause for most damages associated with expansive soil.

2.2 Origins of expansive soils

Parent material

The parent materials that give rise to expansive soil are classified into two. The first group comprises the basic igneous rocks, which are low in silica, generally about 45% to 25% and rich in metallic base such as pyroxenes, amphiboles, biotitic and olivine. Such rocks include the gabbros, basalts and volcanic glass. The second group includes sedimentary rock that contains montmorillonite as a constituent. These include shales and claystone, and limestone and marls rich in magnesium.

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, of parent rock mineral which generally consists of ferromagnesium mineral, calcic feldspars, volcanic glass, volcanic rocks and volcanic ash. The formation is aided in alkaline environment, presence of magnesium ion and lack of leaching. Such condition is favorable in semi-arid regions with relatively low rain fall or seasonal moderate rainfall particularly where evaporation exceed precipitation. Under these conditions enough water is available for the alteration process but the accumulated cations will not be removed by rainwater.

Clay mineralogy

The clay minerals are classified as follows:

A: Two-layers clays which consists of one tetrahedral layer bounded to one aluminum octahedral layer.

Kaolinite is the most common mineral under this catagory.

B: Three-layer clays which consists of one octahedral layer sandwiched between two tetrahedral layers.

Illite, montmorillonite and vermiculite are the common mineral under this catagory.

C: Mixed-layer clays which consists of interstratifications of the two- and three-layer clay minerals previously described. The mixing may be regular or random. Common mineral under these classes are chlorite, montmorillonite-chlorite.

The clay mineral Kaolinite exhibits very minor interlayer swelling. This is explained by the virtual absence of ionic substitution in either the tetra- or octahedral layers which results in more or less complete electrical neutrality and the absence of compensating cations. Also, the individual two layer structures are more tightly bonded together by the opposing electrical charges on the adjuacent octa- and tetrahedral layers.

Illite also a three-layer clay mineral, it exhibits very minor interlayer swelling. This result from the presence of nonhydrated K+ ions in the interlayer positions within the hexagonal openings of the tetrahedral layer. The K+ satisfies charge deficiencies residing mainly on the tetrahedral layer and is thus tightly bondede. The clay mineral responsible for the most damage of expansive soil is montmorillonite.

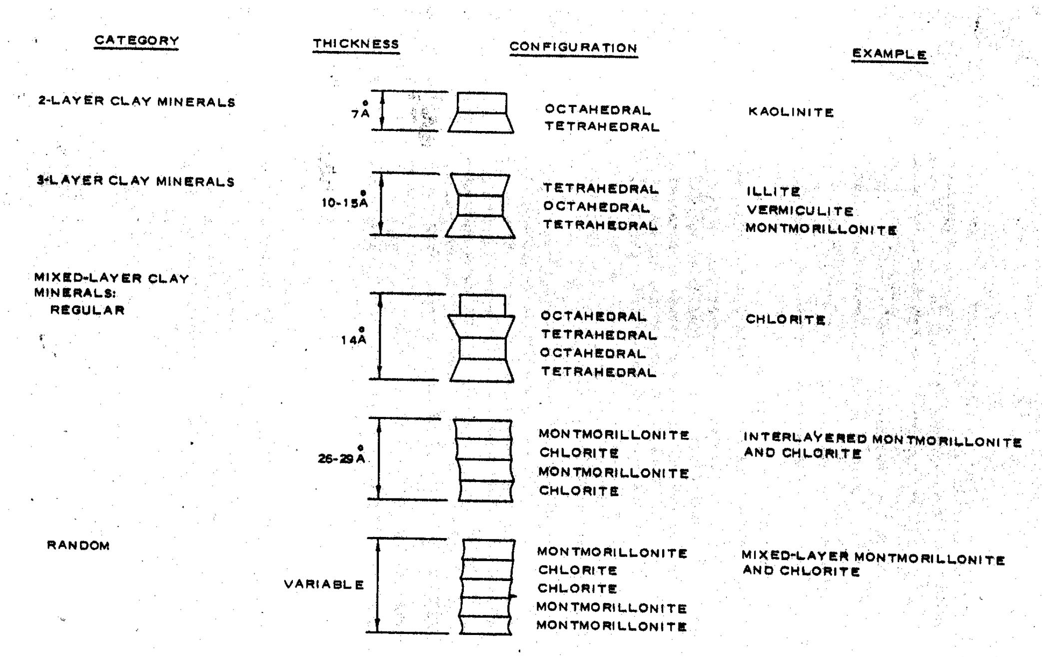

Fig. 2.1 Typical structural configuration of clay minerals (Grim, 1962)

Montmorillonite is a diocthedral and usually contains some magnesium substituted for aluminum in the octahedral layer. This substitution results in a lattice charge deficiency that is neutralized by the presence of cations such as Na+, Ca++, or Mg++ on the interlayer positions. Although these ions posses ionic radii that would permit occupancy of the space within the hexagonal opening at the surface of the tetrahedral layers. The ions are hydrated and as a result of increased ionic radii must occupy space on and above the tetrahedral layers. Such a position props adjacent layers apart and permit access of more water to interlayer positions. (Grim, 1962)

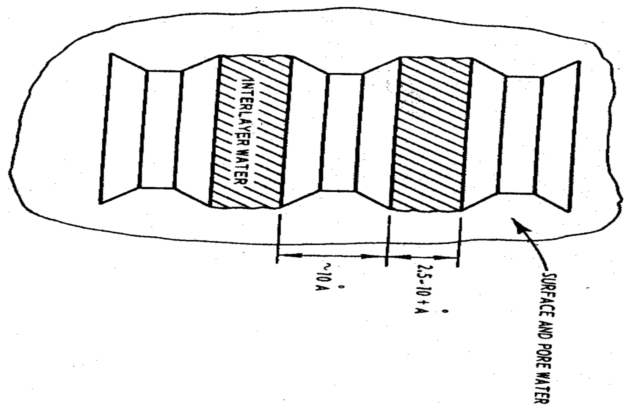

Fig. 2.2 Deflocculated clay mineral showing surface and interlayer water.

2.3 Identifications of expansive soil

2.3.1. Field identification

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

Some of the important field identification method that indicate the potential for expansiveness of a soil are the following:

- A shiny surface is easily obtained when a partially dry piece of the soil is

Polished with a smooth object such as the top of a finger nail

- The wet samples 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

Generally, there are three different method of identifying expansive soil in the laboratory

2.3.2.1 Mineralogical identification

This method is used for identifying the mineralogy of clay particles such as characteristic crystal dimensions, characteristic reaction to heat treatment, size and shape of clay particles and change deficiency and surface activity of clay particle. These properties are a fundamental factor controlling expansive soil behavior.

The various techniques under these methods are

- -X-ray diffraction
- -Differential thermal analysis
- -Dye absorption
- -Electron microscope
- -Base exchange capacity, etc

But these methods are not suitable for routine tests because of the following reason; they are time consuming, require expensive test equipment and, the results are interpreted by specially trained technicians.

2.3.1.2. Indirect methods

These methods include simple soil property test that a practicing engineer resort to use for identifying expansive soil. Such tests are easy and can be performed in most soil mechanics laboratory, and yield an excellent indices of expansive properties. The various tests under these methods are:

i) Atterberg limits:

In this method, measurement of the plasticity and the shrinkage characteristics of the soil are conducted for identification of all engineering soils and provide a wide acceptable means of rating. Especially when they are combined with other tests they can be used to classify expansive soils. The different types of limits under this method are Liquid limit, Plastic limit and Shrinkage limit.

ii) Linear shrinkage tests

Linear shrinkage, is the change in length of a soil sample as it dries to the shrinkage limit, SL, expressed as a percentage of the original length. This test also in combination with other tests is used for classification of expansive soil.

iii) Free swell tests

The free swell test may be considered as a measurement of volume change in clay upon saturation and is one of the most commonly used simple tests to estimate the swelling potential of expansive clay. The test is performed by pouring, 10cc of dry soil passing a sieve size of .425mm(No. 40), into a 100cc graduated jar filled with water. Then the swelled volume of the soil after the material settles is measured. The free swell is then given by:

$$FS = (V - V_o)/V_o * 100\%$$

Where FS= free swell, %

V= soil volume after swelling, cm^3

Vo=volume of dry soil, $10cm^3$

iv) Colloid content test

This test is used to determine the quantity of material in a soil sample that is smaller than a selected size, expressed as a percentage by weight of the total sample. Sizes used are 2μm(.002mm) and 1μm(.001mm); the upper limit of the clay range is generally considered to be 2 to 5 μm. The test usually requires hydrometer analysis.

2.3.2.3. Direct measurement

The most accurate and dependable method of determining the swelling potential and the swelling pressure of expansive clay is by direct measurement. The method quantitatively evaluates the volume change characteristics of expansive soil. The test can be done using consolidometer but care should be taken on the test procedure. A standardized procedure that consider the factors that affect the shrink swell potential as well as simulate the expected loading condition should be adopted. The discussion on the different techniques will be given in chapter 3 section 3.2.1.

2.4 CLASSIFICATION OF EXPANSIVE SOIL

The parameters determined from expansive soil identification tests have been combined in a number of different classification schemes. But before using any soil classification system, engineers should understand the data base from which it was derived and establish its limitations; otherwise, poor reliability and lack of confidence in the system may result. The different classification system are categorized into two:

1. General classification systems which have evolved over many years and are based largely on correlation with actual performance
2. Those devised specifically for classification of expansive soils. Theses system are based on indirect and direct prediction of swell potential, as well as combinations, to arrive at a rating.

2.4.1 General classification system

The most widely used general classification systems are

i) **Unified soil classification systems**

In these classification system a correlation is made between swell potential and unified soil classification as follows

Category	symbol	soil classification in unified system
Little or no expansion	1	GW, GP, GM, SW, SP, SM
Moderate expansion	2	GW, SC, ML, MH
High volume change	3	CL, OL, CH, OH
No rating		PT

The above classification system can be summarized as follow:

1. All clay soil and organic soils exhibit high volume change.
2. All clayey gravels and sands and all silts exhibit moderate volume changes.
3. All sands and gravels exhibit little or no expansion.

In the above classification soils rated as CL or OH may be considered as potentially expansive.

ii) **AASHTO classification**

As shown in the Fig.2.3 soils that lies on A6, A7 and borderline soils A-4, A-6, and A-7 may be considered as potentially expansive.

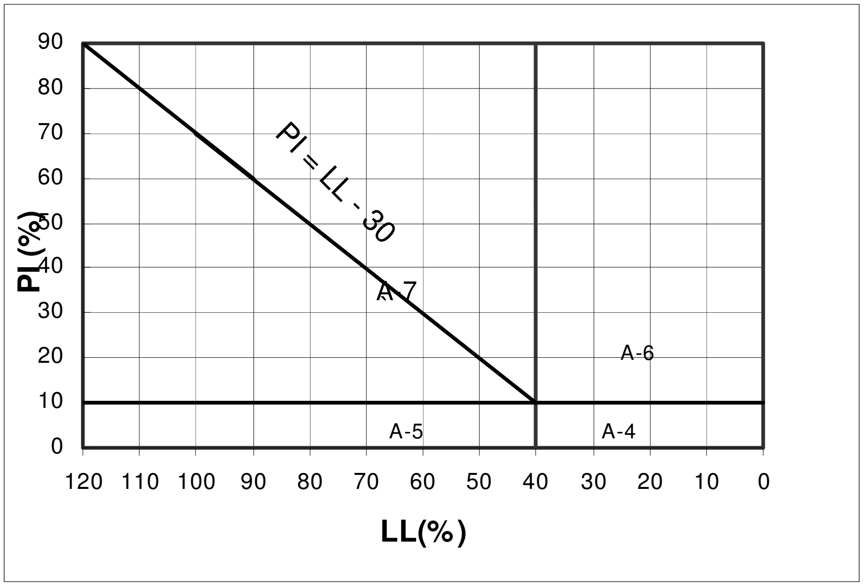

Fig 2.3 Liquid limit against plasticity index to the AASHTO classification method.

2.4.2 Classification specific to expansive soil

The above classification system may give an initial alert that the soil may have expansive character and doesn't provide useful information. 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 (Nelson, 1992). Hence, it is very important to emphasize that design decision has to be based on predicting testing and analysis, which provide reliable information.

2.4.2.1 Classification based on indirect predictions of swell potential

An indirect prediction of swell potential includes correlations based on index properties, swell, physical indicator and a combination of them. Some of such classification systems are:

2.4.2.1.1 Skempton (Mckeen, 1976)

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

Activity, Ac = PI/ percentage by weight finer than 2μm

Skempton suggested three classes of clays according to their activity.

Activity	potential of expansion
Ac<.75	low (inactive)
.75< Ac<1.25	medium (normal)
Ac>1.25	high (active)

2.4.2.1.2 Kantey and Brink (Mckeen, 1976)

In this method expansive soil are recognized by the following criteria.

Liquid limit >30

Plastic limit >12

Linear shrinkage >8

2.4.2.1.3 Altmeyer (Mckeen, 1976)

He suggest rating for degree of expansion based on shrinkage limit (SL) and linear shrinkage:

SL, %	LS, %	degree of expansion

<10	>8	critical
10-12	5-8	marginal
>12	<5	non critical

The classification system developed based on single property alone such as: based on activity (Skempton, 1953), based on shrinkage limit and linear shrinkage (Altmeyer, 1956), based on index property (Kanntey and Brink, 1952), etc (Nelson, 1992) are difficult to use alone as a classification system because they may lead to wrong conclusion.

A better indirect classification system can be developed by combining index property, swell and physical indicator. Good examples for such classification are:

i. Bureau of reclamation method

This method is based on direct correlation of observed volume change with colloid content, plastic index, and shrinkage limit. The classification is as given in table2.4.2.1 (Snethen, 1975).

Table 2.4.2.1 classification based on bureau of reclamation method

Colloid content, %-	PI, %	SL, %	Probable expansion	Degree of expansion
1μm	<18	>15	%	Low
<15	15-28	10-16	<10	Medium
13-23	25-41	7-12	10-20	High

20-31 >28	>35	<11	20-30 >30	Very high

ii. Chen Method

In this method, a correlation is made between swell data and %< No. 200 sieve, liquid limit, and standard penetration resistance. The classification is as given in table2.4.2.2: (Chen, 1988)

Table 2.4.2.2 Chen method of classification of expansive soil

<No200 sieve, %	LL %	Standard penetration blows	Probable Expansion %	Degree of expansion
<30 30-60 60-95 >95	<30 30-4040- 60 >60	<10 10-20 20-30 >30	<1 1-5 3-10 >10	Low Medium High Very High

2.5 Prediction of Heave

As many prediction methods in use today involve direct measurements of swelling pressure with a consolidometer and most of the structural damages occur when the swelling pressure is greater than the foundation pressure, assessing the swelling pressure is an important task in dealing with expansive soil.

Consolidometer Testing.

Evaluation of soil volume changes by consolidometer testing is the most widely used method for predicting heave. The different types of techniques under these methods are:

i. Zero swell test

This test is conducted by applying a small incremental load of 6.9kpa (1psi) on a compacted specimen. Water is the 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. 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. (Mckeen, 1976)

iv. Double-odeometer test

The test involves a pair of nominally identical specimens. The first is loaded in the as-compacted state by incremental vertical pressure with equilibrium deformation recorded at each pressure level. The second specimen is fully inundated with no seating load and the maximum swell is recorded. The difference between the percent changes in specimen height at each stress level is used to define specimen height at each stress level is used to define the swell potential. Furthermore, the stress at which the percent settlement of the firs specimen equals the percent swell of the second specimen i.e. difference equals zero, is used as a measure of swell pressure. (Mckeen, 1976)

2.6 Mechanics of swell

Soil volume change result from an imbalance in internal energy of the system (soil/ water/ plants/ air). Energy imbalances important in engineering result from moisture movement caused by loads, desiccation, and temperature changes. Response to a specific set of conditions is determined by the composition, structures, and geologic history of the soil. The largest component of volume change is that of the clay micelle which surrounds the individual clay particles in the soil. Water is forced out of the micelle by loads, desiccation, or temperature along energy gradient and reduction in volume results. When these influences are removed or reduced, the energy gradients are reversed, the available water is forced into the clay micelle and swell is produced. (Mckeen, 1976)

The natural micro scale mechanisms, which contribute the major portion of volume changes in expansive soils, are (Snethen, 1975)

- Osmotic repulsion: it is a pressure gradient developed in the double-layer water due to variations in the ionic concentration in the double layer.
- Clay particle attraction: as clay particles possess a net negative charge on their surfaces and edges which result in attractive forces for various cations and in particular for dipolar molecules such as water.
- Cation hydration: it is physical hydration of cations substituted into or attached to the clay particles.
- Capillary imbibition: it is a movement of water into a mass of clay particles resulting from surface tension effects of water and air mixtures in the pores of the clay mass.

2.7 Factors influencing swelling and shrinking of a soil

The factors influencing the shrink swell potential of a soil can be considered in three different groups.

1.Soil characteristic that influence the basic nature of the internal force field. These includes

- Clay mineralogy
- Soil water chemistry
- Soil suction
- Plasticity
- Soil structures and fabrics
- Dry density

2. The environment factor that influence the changes that may occur in the internal force system. These include.
    - Initial moisture condition
    - Moisture variation
    - Climate
        * Ground water
        * Drainage and manmade water source
        * Vegetation
        * Permeability
        * Temperature
3. State of stress, which include
    - Stress history
    - Surcharge load

- Soil profile

As swelling pressure is the built in property of expansive soil and will not affected by placement condition or environmental condition, only initial dry density and the amount and the type of clay mineral affect the swelling pressure [Chen, 1975, Yehyese.H, 2001]

Effect of initial dry density

The dry density is an important factor in determining the magnitude of volume change. The swell or the swelling pressure of an expansive soil increases with increasing dry density for constant moisture content. The reason is that higher densities result in closer particles spacing, therefore causing greater particle interaction.

CHAPTER 3

3. Swelling pressure Prediction Model

3.1 Introduction

It is well established that expansion of clays and associated swelling on foundation results in considerable damages to structures (Chen, 1988). Clays that are expansion susceptible, swell as the moisture content of the soil increases. Upon expansion, the soil exerts an upward pressure on foundations. If this pressure is greater than the foundation pressure, then uplift or differential uplift occurs causing floors, walls, beams and columns to crack. Consequently, assessing the swelling pressure is an important step in designing foundation on expansive soils. Several researchers have developed an empirical relation for the estimation of swelling pressure. The empirical expressions relate the swelling parameters to the geotechnical parameters that are determined by means of identification tests. But the expression should either be reformulated or checked for the local condition.

3.2 Swell and swelling pressure

3.2.1 Measurement of swell and swelling pressure

Swell is the process of imbibing available moisture to cause an increase in soil volume until the pore water pressure increases to an equilibrium determined by the environment. The amount of swell to satisfy the new pore pressure equilibrium depends on the magnitude of the vertical loading and soil properties that include soil composition, natural water content and density, and soil structures. The rate of swell depends on the coefficient of permeability (hydraulic conductivity), thickness, and soil properties.

Swelling pressure: ASTM defines swelling pressure as the pressure which prevents the specimen from swelling or that pressure which is required to return the specimen to its original state (void ratio, height) after swell stress controlled (Chen, 1988). The swelling pressure of the soil is independent of the surcharge pressure, the initial moisture content, degree of saturation, and the thickness of the soil stratum. It depends on initial density soil composition and soil fabrics.

Among the techniques used to determine the swelling pressure, discussed in chapter three swell-consolidation tests on one-dimensional oedometer is used. And the following test procedure was adopted for the test.

-A core sample with the size of consolidometr ring is extruded from large size sampling tube. If there is any excess material, it should be trimmed gently with a knife or wire saw. The excess material may be used to calculate the moisture content and the specific gravity.

-Measure the weight of the soil and the ring, and put the sample in the odometer.

-Initial dial reading under the load of 7kpa is taken and then the sample is allowed to absorb water and is put aside to fully expand and reach equilibrium.

Pressure is then applied increasingly at intervals until the sample reaches its initial volume (zero volume change).

The swelling pressure is defined as the pressure that returns the sample to its initial volume.

The dry density can also be calculated from the relation

$$\gamma_{bulk} = \frac{weight\ of\ wetsoil}{volume\ of\ the\ ring}$$

-

$$\gamma_{dry} = \frac{\gamma_{bulk}}{1+\omega}$$

Schematically the test procedure is shown in Fig. 3.1

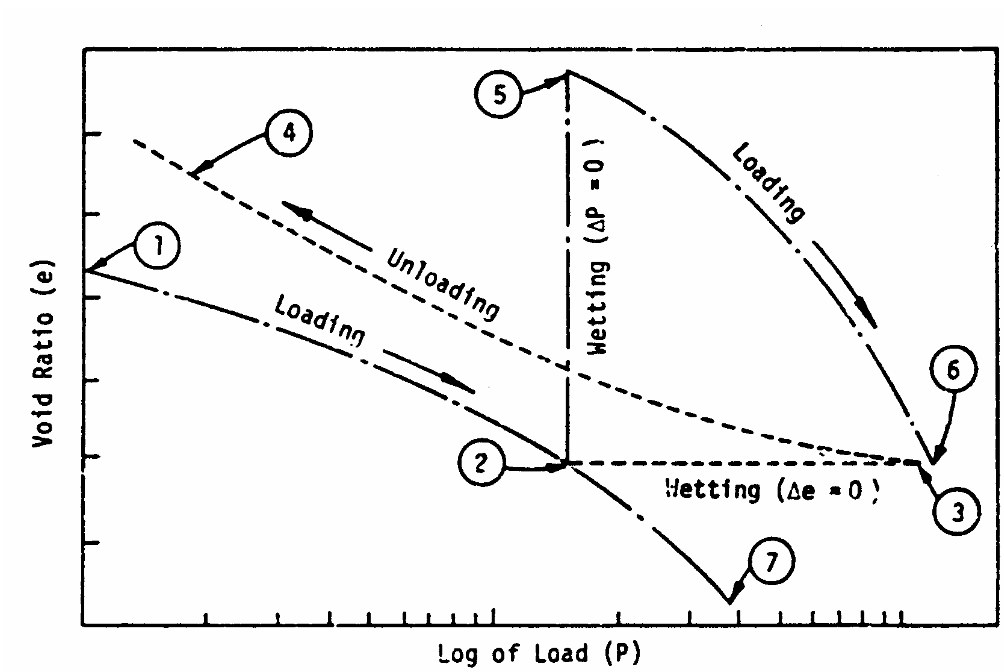

Fig 3.1 Type of swell test data [Mckeen, 1976]

As observed in Fig 3.1, specimen may be loaded to in-situ overburden pressure, (1 to 2). Then it is subjected to a change in moisture condition and maintained at constant volume until equilibrium is reached (3). This pressure is called the swell pressure (no-volume-change state). The pressure is then released to a small arbitrary selected load or to a specific design load, (3 to 4). Another test procedure loads the soil to the overburden pressure (2), allows it to swell under constant load to (5), and loads the sample to the original void ratio (6). With this kind of test procedure, swell may be calculated as follows:

$$S= \frac{\Delta e}{1-e_1}(\Delta H)$$

Where Δe=change in void ratio (final to initial)

e1=original void ratio

ΔH=thickens of soil layer

the curve (1 to 7) illustrates a test in which a soil is loaded to the initial overburden pressure, (7), unloaded to final overburden pressure, (2), and permitted access to water; then the swell is determined (analysis of cut sections). In each situation, events follow a specific sequence. The closer these duplicate in-situ conditions, the better the prediction of soil behavior.

3.2.2 Factor that cause error to the test result of swelling pressure

While using the result of swell test, one should consider the factors that cause error to the test result. Some of them are: (Mckeen, 1976)

1. The moisture gradients produced in the consolidometer are drastically different from those in in-situ conditions.
2. Friction in the measuring apparatus is significant at smaller loads (<.5 kg/cm2 or 7.11psi). At a pressure of .01kg/cm2, the load applied to the sample has been found to be in error by 100% for one type of consolidometer.
3. Compression characteristics of the apparatus are important. Consolidometers should be tested to establish the compression of the loading frame and volume change measuring apparatus. Calibration curves should be prepared and no components should be switched without verifying the compression characteristics of the apparatus. The compression characteristics do not vary significantly from cycle to cycle.
4. Porous disc produce a high degree of compressibility. Smooth grooved thick stones are most desirable.
5. Filter paper used between samples and porous stones produce significant compressibility in swell tests on Bearpaw shale.
6. Sample seating against the porous disc is difficulty to evaluate. As the load increase the significance is reduced.
7. In measuring the swell pressure, very small volume changes result in large differences in measured swell pressure. (All sources of volume changes must be considered in arriving at a measured value.)
8. Lateral confinement of the sample may not duplicate field conditions; a correction factor may be required.

3.3 Swelling pressure prediction models.

For the design of foundation on expansive soils, it is essential to make quantitative estimate of the anticipated pressures that swelling can cause on the structure foundation. The most reliable method for the estimate of swelling pressure would involve utilizing odometer test on undisturbed samples. But such estimate can also be derived from soil properties that are easily determined with a reasonable degree of reliability using simple routine tests.

Many relationships have been established from which swelling pressure can be estimated based on index test and the physical state of the soil. The following relations are tested for their validity in our environmental and climatic condition. They are selected for their simplicity, wide acceptance and practical significances to field application (El-Sohby, 1987)

Komornik and Davvid (1969)

$$\log s_p = 0.132 + 0.0208\left(W_l\right) + 0.0006688\left(\gamma_d\right) - 0.0269(w) \quad \text{----------------------3-1}$$

Vijayvergiya and Ghazzaly (1973)

$$\log s_p = \frac{1}{12}\left(0.4W_l - w + 23.6\right) \quad \text{---3-2}$$

$$\log s_p = \frac{1}{19.5}\left(6.242\gamma_d + 0.65W_l - 100\right) \text{---3-3}$$

El-sohby and Rabba (Proposed equation)

$$\log s_p = K\left(\gamma_d + K_2 w_l - K_1\right) \text{--3-4}$$

Values of K, K1, K2

	K	K1	K2
Sandy-clay soils	2.17	2.00	0.01
Silt-clay soils	2.50	1.83	0.0070

In the above equations

Wl= Liquid limit (%)

Sp=swelling pressure (Kpa) for Equation 3-1,3-2 and 3-3

Sp=Swelling pressure (Kgf/cm^2) for equation 3-4.

γd=dry density (g/cm^3) for equation 3-1and 3-4

γd=dry density (KN/m^3) for equation 3-3.

w=moisture content (%)

Ip=plastic index (%)

4. Laboratory Tests

4.1 Laboratory tests on disturbed sampling

It is not economical, and is time taking to make large number of tests on undisturbed samples to determine swelling pressure. For smaller projects and for preliminary design purposes of any size of building, it is more appropriate to determine swelling pressure from relations developed base on simple tests. Therefore, in this study first index tests were made on 78 disturbed samples taken from 44 different places in Addis Ababa. The test pits locations for the disturbed sampling are shown in fig 4.1. These tests are part of an on going research made in the Civil Engineering Department's laboratory under the project Light House Construction in Expansive Soil sponsored by Ethiopian Consulting Engineers and Architect Association (ECEAA) and GTZ. The purpose of these tests is to classify expansive soil of Addis Ababa based on index properties and to identify those soils that show unique properties. Once they are classified based on index properties grouping will be made and those soils, which showed different properties, will be considered for undisturbed sampling. The summary of the test results are shown in table 4.1.1.

Table 4.1.1 Summary of the laboratory test result of disturbed soil samples

Soil property	Magnitude of the property in (%)
Liquid limit (LL) (%)	79-121
Plastic limit (PL) (%)	25-50
Plastic index (PI) (%)	38-84
% Retained in the 2µm sieve (%)	48-82
% Finer than 75µm (%)	95-99
Free swell (FS) (%)	64-140

Index tests, grain size and specific gravity tests were made by following the ASTM procedure of designation D4318-95 and D427-93, designation D422-63 and designation D845-92 respectively. Based on the different classification system stated in chapter 2 section 2.4, the test results in table 4.1 show that the expansive soil samples taken from different part of Addis Ababa fall in very high swelling potential group by the classification system stated in chapter 2 section 2.4.2. Except for five test pits, the profile of other test pits dug for the study are a black color expansive soil with depth ranging from 0.90m to 1.8m is followed by a grey color expansive soil.

4.2 Laboratory tests on undisturbed samples

The test results in table 4.1 show that all the disturbed samples fall under very high expansive soil catagory, 17 samples from 9 test pits were chosen for undisturbed sampling. The distributions of the test pits were chosen in such a way that an even geographic distribution is attained. The locations of the test pits for undisturbed sampling are shown in fig. 4.2

Swelling pressure test were then made on undisturbed samples following the procedure stated in chapter 3. The purposes of these tests are to make some correlation between index properties and swelling pressure of expansive soil of Addis Ababa. The test results of the swelling pressure and index properties are shown in table 4. 2.1.

Table 4.2.1 Test results of undisturbed samples.

Site Name	coloure	Moisture content (w)	Liqid limit (LL) (%)	Plastic Limit (PL) (%)	Plastic index (PI)(%)	dry density (g/cm3)	Clay content (%)	Swelling pressure (Kpa)	Specific Gravity
S1	Black	38.4	101	43	58	1.25	76	420	2.77
	Grey	39.6	105	39	66	1.25	81	320	2.8
S2	Black	37.5	110	34	76	1.24	65	300	2.78
	Grey	42	121	37	84	1.17	72	108	2.82
S3	Black	37.6	96	42	54	1.24	60	267	2.79
	Grey	44.3	111	47	64	1.17	78	109	2.82
S4	Black	33.2	96	36	60	1.26	62	210	2.76
S5	Black	42	105	40	65	1.2	72	200	2.85
	Grey	49.6	110	45	65	1.1	78	37	2.81
	Grey*	40	110	45	65	1.24	78	323	2.81
S6	Black	50.8	96	42	54	1.09	67	61	2.79
	Black*	40	96	42	54	1.23	67	199	2.79
	Grey	42.1	101	38	63	1.23	50	248	2.83
S7	Black	38	106	41	65	1.22	65	108	2.78
	Grey	41.5	109	49	60	1.2	75	155	2.81
S8	Black	46	100	41	59	1.14	57	57	2.77
	Black*	32	100	41	59	1.25	57	348	2.77
	Grey	45.5	106	43	63	1.15	68	70	2.8
S9	Black	46	98	26	72	1.13	66	0	2.78
	Black*	35	98	26	72	1.23	66	285	2.78
	Grey	43	101	28	73	1.16	70	0	2.83
	Grey*	37	101	28	73	1.23	70	293	2.83

Note * shows the samples are prepared from disturbed samples.

In table 4.2.1 some of the samples showed small swelling pressure values and zero swelling pressure for sample S9. This can be attributed to the fact that the samples have high natural moisture content during sampling and therefore they have already swelled in situ. Such samples are not considered in the analysis. Instead samples are prepared from the disturbed sample at higher density and small moisture content, and the swelling pressures values are measured and used in the analysis.

Except S4, the black and grey soils are taken from same test pits with an average depth of 1.3m and 2.8m respectively.

4.3 Analysis of test results

4.3.1 comparison of measured swelling pressure with predictive values.

The measured values of swelling pressure for the different sites in Addis Ababa are compared with predictive values obtained by using previous equation suggested by different research works given in chapter 3 section 3.1. The results are tabulated in table 4.3.1.

Table 4.3.1 comparison of measured swelling pressures with predictive equations

T		Laboratory results								Sweling pressure values obtained from already developed emperical relations			
Site name	Colour	W (%)	LL) (%)	PL (%)	PI (%)	Density (g/cm³)	Clay (%)	Sp (KN/m²)	Gs	Eq.1 (KN/M²)	Eq.2 (KN/M²)	Eq.3 (KN/M²)	Eq.4 (KN/M²)
s1	Black	38.4	101	43	58	1.25	76	420	2.77	108.60254	135.93564	173.67751	207.73037
	Grey	39.6	105	39	66	1.25	81	320	2.8	122.11247	146.77993	236.08964	244.06191
S2	Black	37.5	110	34	76	1.24	65	300	2.78	174.00511	322.3542	321.90916	281.83829
	Grey	42	121	37	84	1.17	72	108	2.82	200.21472	316.22777	447.02129	293.42695
S3	Black	37.6	96	42	54	1.24	60	267	2.79	88.444738	107.97752	109.91746	160.32454
	Grey	44.3	111	47	64	1.17	78	109	2.82	107.55386	94.406088	207.4889	196.11012
S4	Black	33.2	96	36	60	1.26	62	210	2.76	119.78653	251.18864	127.37587	179.88709
S5	Black	42	105	40	65	1.2	72	200	2.85	97.445099	92.611873	163.31484	183.02061
	Grey*	40	110	45	65	1.24	78	323	2.81	149.04314	199.52623	321.90916	281.83829
S6	Black*	40	96	42	54	1.23	67	199	2.79	75.062669	68.129207	102.10721	151.35612
	Grey	42.1	101	38	63	1.23	50	248	2.83	83.740201	66.834392	149.87289	185.13989
S7	Black	38	106	41	65	1.22	65	108	2.78	135.06288	215.44347	204.35228	213.79621
	Grey	41.5	109	49	60	1.2	75	155	2.81	121.73347	138.56919	222.00307	215.03047
S8	Black*	32	100	41	59	1.25	57	348	2.77	153.88631	429.86623	160.846	199.52623
	Grey	45.5	106	43	63	1.15	68	70	2.8	76.202637	51.089698	121.98535	142.8894
S9	Black*	35	98	26	72	1.23	66	285	2.78	112.59627	207.33216	119.04827	164.05898
	Grey*	37	101	28	73	1.23	70	293	2.83	114.84815	177.82794	149.87289	185.13989

Note: * samples are prepared from disturbed samples.

In table 4.3.1 the equations are as follows.

Komornik and David (1969)

$$\log s_p = 0.132 + 0.0208\left(W_l\right) + 0.6688\left(\gamma_d\right) - 0.0269(w) \text{ ---------------------------4-1}$$

Vijayvergiya and Ghazzaly (1973)

$$\log s_p = \frac{1}{12}\left(0.4W_l - w + 23.6\right) \text{ ---4-2}$$

$$\log s_p = \frac{1}{19.5}\left(6.242\gamma_d + 0.65W_l - 100\right) \text{ --4-3}$$

Where Sp in KN/m^2, and γ_d in g/cm^3 for equation 4-1 and 4-4 and it is in KN/m^{3}for equation 4-3, and W_l and w are in percentage.

El-sohby and Rabba (Proposed equation)

$$\log s_p = K\left(\gamma_d + K_2 w_l - K_1\right) \text{ --4-4}$$

For the calculation in table 4.3.1 the K-values of silty-clay soil is used.

Where Sp= Kgf/cm^2, and γ_d g/cm^3, and W_l and w are in percentage.

	K	K1	K2
Sandy-clay soils	2.17	2.00	0.01
Silt-clay soils	2.50	1.83	0.0070

In the equations 4-1 - equations 4-4 one or more of the four parameters that affect swelling pressure are used in different combinations. These parameters are: dry density, liquid limit, initial moisture content and plastic limit. All the equations do not predict the swelling pressure for our soil except for the equations developed by komornik and David (1969), which predict well for those samples that have small density. The discrepancies noted might result mainly from variation of the nature of the soil, environmental, climatic condition and geologic formation of the region where the relation is developed to the study area.

Besides these the parameter involved in the equations do not fully explain swelling pressure: for example equation 4-2 misses the important parameter dry density. In equation 4-4 the rate at which dry density and liquid limit affecting the swelling pressure is small so that the equation gives almost similar result for all samples. In equation 4-3 the weight given for liquid limit is greater than that of dry density, which is not true for our soil that is sensitive to dry density.

4.3.2 Development of the new formula.

Swelling pressure is a built-in property varying with the dry density of the soil, the amount and type of clay minerals, and indirectly with the initial moisture content. This may be explained as follows: the higher the dry density result in closer particle spacing, therefore causing greater particle interaction and higher swelling pressure. And also the higher the amount of expansive type clay, the higher the swelling pressure.

The new relations developed therefore incorporate the dry density, Atterberg limits and initial moisture content. The effect of dry density is expressed by directly involving dry density in the developed relations and the type of clay in the soil may be obtained by considering the plastic index in relation to the liquid limit, while the amount of clay present may deduced, within the broad limits, from the value of plastic index.

Then new relations are developed, by taking one or more of the four parameters (liquid limit, dry density, plastic limit and plastic index) in different combination. The relations are developed, by taking 14 samples out of it 4 samples are prepared from disturbed samples, and multiple linear regressions is then made using SPSS 9.0 for windows software.

The multiple regression equations take the form $y=b_1x_1+b_2x_2+\text{-----}+c$. The b's are the regression coefficients, representing the amount the dependent variable changes when the independent changes 1unit. The c is the constant, where the regression line intercepts the y axis, representing the amount the dependent y will be when all the independent variable are 0. The regression coefficients are then calculated using SPSS 9.0 for windows software and tested for three samples, which are used as a control to test the relations. The input and outputs of the software are attached below.

Input file

COLOR	X-COOR	Y-COOR	W (%)	LL (%)	PL (%)	PI (%)	DENSITY Kg/m^3	CLAY (%)	G	Log SP	Sp (Kpa)
Black	476936.00	994652.00	38.40	101.00	43.00	58.00	1250.00	76.00	2.77	2.62	420
Grey	476936.00	994652.00	39.60	105.00	39.00	66.00	1250.00	81.00	2.80	2.51	320
Black	476839.00	993360.00	37.50	110.00	34.00	76.00	1240.00	65.00	2.78	2.48	300
Grey	476839.00	993360.00	42.00	121.00	37.00	84.00	1170.00	72.00	2.82	2.03	108
Black	485893.00	997096.00	37.60	96.00	42.00	54.00	1240.00	60.00	2.79	2.43	267
Grey	485893.00	997096.00	44.30	111.00	47.00	64.00	1170.00	78.00	2.82	2.04	109
Black	470282.00	989330.00	33.20	96.00	36.00	60.00	1260.00	62.00	2.76	2.32	210
Black	474075.00	986145.00	42.00	105.00	40.00	65.00	1200.00	72.00	2.85	2.30	200
Grey	474075.00	986145.00	40.00	110.00	45.00	65.00	1240.00	78.00	2.81	1.57	323
Black	479270.00	995542.00	40.00	96.00	42.00	54.00	1090.00	67.00	2.79	1.78	199
Grey	479270.00	995542.00	42.10	101.00	38.00	63.00	1230.00	50.00	2.83	2.39	248
Black	469889.00	993446.00	38.00	106.00	41.00	65.00	1220.00	65.00	2.78	2.03	108
Black	474235.00	994452.00	35.00	98.00	26.00	72.00	1230.00	66.00	2.78	2.45	285
Grey	474235.00	994452.00	37.00	101.00	28.00	73.00	1230.00	70.00	2.83	2.47	293

Regression analysis for Equation 1

Variables Entered/Removed[b]

Model	Variables Entered	Variables Removed	Method
1	LL, W[a]	.	Enter

a. All requested variables entered.

b. Dependent Variable: SP

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.494[a]	.244	.107	.1811

a. Predictors: (Constant), LL, W

ANOVA[b]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	.117	2	5.835E-02	1.779	.214[a]
	Residual	.361	11	3.279E-02		
	Total	.477	13			

a. Predictors: (Constant), LL, W

b. Dependent Variable: SP

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	3.828	.786		4.872	.000
	W	-1.38E-02	.020	-.217	-.682	.509
	LL	-9.02E-03	.008	-.338	-1.064	.310

a. Dependent Variable: SP

Regression analysis for Equation 2

Variables Entered/Removed[b]

Model	Variables Entered	Variables Removed	Method
1	DENSITY, PI, LL[a]	.	Enter

a. All requested variables entered.

b. Dependent Variable: SP

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.791[a]	.626	.514	.1336

a. Predictors: (Constant), DENSITY, PI, LL

ANOVA[b]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	.299	3	9.962E-02	5.579	.016[a]
	Residual	.179	10	1.786E-02		
	Total	.477	13			

a. Predictors: (Constant), DENSITY, PI, LL

b. Dependent Variable: SP

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	-5.000	2.766		-1.808	.101
	LL	-2.06E-04	.009	-.008	-.023	.982
	PI	3.477E-03	.006	.152	.547	.596
	DENSITY	5.827E-03	.002	.841	3.224	.009

a. Dependent Variable: SP

Regression analysis for Equation 3

Variables Entered/Removed[b]

Model	Variables Entered	Variables Removed	Method
1	DENSITY, PI[a]	.	Enter

a. All requested variables entered.

b. Dependent Variable: SP

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.791[a]	.626	.558	.1274

a. Predictors: (Constant), DENSITY, PI

ANOVA[b]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	.299	2	.149	9.204	.004[a]
	Residual	.179	11	1.623E-02		
	Total	.477	13			

a. Predictors: (Constant), DENSITY, PI

b. Dependent Variable: SP

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	-5.045	1.895		-2.662	.022
	PI	3.384E-03	.005	.148	.722	.486
	DENSITY	5.851E-03	.001	.844	4.121	.002

a. Dependent Variable: SP

Regression analysis for Equation 4

Variables Entered/Removed[b]

Model	Variables Entered	Variables Removed	Method
1	DENSITY, PI, W[a]	.	Enter

a. All requested variables entered.

b. Dependent Variable: SP

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.839[a]	.704	.616	.1188

a. Predictors: (Constant), DENSITY, PI, W

ANOVA[b]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	.336	3	.112	7.940	.005[a]
	Residual	.141	10	1.412E-02		
	Total	.477	13			

a. Predictors: (Constant), DENSITY, PI, W

b. Dependent Variable: SP

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	-9.384	3.198		-2.934	.015
	W	2.748E-02	.017	.431	1.628	.135
	PI	6.307E-03	.005	.276	1.334	.212
	DENSITY	8.359E-03	.002	1.206	4.115	.002

a. Dependent Variable: SP

Regression analysis for Equation 5

Variables Entered/Removed[b]

Model	Variables Entered	Variables Removed	Method
1	DENSITY, LL, W[a]	.	Enter

a. All requested variables entered.

b. Dependent Variable: SP

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.809[a]	.654	.550	.1286

a. Predictors: (Constant), DENSITY, LL, W

ANOVA[b]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	.312	3	.104	6.293	.011[a]
	Residual	.165	10	1.653E-02		
	Total	.477	13			

a. Predictors: (Constant), DENSITY, LL, W

b. Dependent Variable: SP

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	-7.232	3.265		-2.215	.051
	W	1.820E-02	.017	.285	1.061	.314
	LL	1.652E-03	.007	.062	.244	.812
	DENSITY	7.096E-03	.002	1.024	3.438	.006

a. Dependent Variable: SP

The following relations are then developed. The relations are developed by taking one or more of the four parameters; liquid limit, plastic index, natural moisture content and dry densities in different combinations.

$$Log\ P_s = -9.139 + 0.04169W - 0.01160LL + 0.01303PI + 0.008331\gamma_d \text{ -------Eq.1}$$

$$LogP_s = -5.00 - 0.0002064\ LL + 0.003477\ PI + 0.005827\ \gamma_d \text{ ---Eq.2}$$

$$LogP_s = -5.045 - 0.003384\ PI + 0.005851\ \gamma_d \text{ --------Eq.3}$$

$$Log\ P_s = -9.384 + 0.02748W + 0.006307PI + 0.008359\gamma_d \text{ ------Eq.4}$$

$$Log\ P_s = -7.232 + 0.01820\ W - 0.001652\ LL + 0.007096\ \gamma_d \text{ ---Eq.5}$$

Where

P_s=Kpa

W, LL and PI are in percentage

γ_d=Kg/m^3

The values of the measured and predicted swelling pressure are shown in table 4.3.2.1

Table 4.3.2.1 values of measured and predicted swelling pressure obtained from the relations.

	Laboratory test results									Predicted by the Relations developed				
Site Name	W (%)	LL (%)	PL (%)	PI (%)	Density (g/cm^3	Density Kg/m^3	Clay (%)_	Sp* (Kpa)	Gs	Eq.1 (Kpa)	Eq.2 (Kpa)	Eq.3 (Kpa)	Eq.4 (Kpa)	Eq.5 (Kpa)
s1	38.4	101	43	58	1.25	1250	76	420	2.77	288.26107	291.48083	118.16205	306.04691	147.92038
	39.6	105	39	66	1.25	1250	81	320	2.8	369.53366	310.17158	111.02122	370.86854	153.20044
s2	37.5	110	34	76	1.24	1240	65	300	2.78	294.50658	293.13792	89.754452	309.73622	116.90148
	42	121	37	84	1.17	1170	72	108	2.82	112.24577	121.54357	32.840821	120.22035	43.134821
s3	37.6	96	42	54	1.24	1240	60	267	2.79	223.35928	247.4355	106.53787	226.45713	123.8135
	44.3	111	47	64	1.17	1170	78	109	2.82	100.33605	104.05238	38.379031	104.00207	49.341006
s4	33.2	96	36	60	1.26	1260	62	210	2.76	257.25747	339.51894	133.11286	274.85016	142.76048
s5	42	105	40	65	1.2	1200	72	200	2.85	173.03342	157.31676	57.047944	162.52306	74.841072
	40	110	45	65	1.24	1240	30	323	2.81	269.14728	268.42626	97.786747	309.25377	129.81354
s6	40	96	42	54	1.23	1230	67	199	2.79	232.14001	216.36749	93.109501	217.44429	116.27544
	42.1	101	38	63	1.23	1230	50	248	2.83	325.5135	231.98238	86.803653	283.02122	124.57991
s7	38	106	41	65	1.22	1220	65	108	2.78	168.41858	205.64067	74.689577	185.42787	87.419436
s9	35	98	26	72	1.23	1230	66	285	2.78	233.68993	249.66979	80.924868	205.81357	93.580207
	37	101	28	73	1.23	1230	70	293	2.83	269.30846	251.31881	80.296756	236.99599	100.60695

Note in table 4.3.2.1 sp* is measured swelling pressure obtained in the laboratory, and Eq.1 to Eq.5 are as shown on page 47.

The five relations developed are then tested for three control samples. The result are shown in table4.3.2.2

Table 4.3.2.2 values of measured and predicted swelling pressure control samples.

	Laboratory test results									Predicted by the Relations developed				
s7(Black)	41.5	109	49	60	1.22	1220	60	155	2.81	187.22119	197.29042	77.656891	215.18897	100.08108
s8(Black)	32	100	41	59	1.25	1250	57	348	2.77	165.0441	293.96322	117.24491	207.12046	113.55336
s8(Grey)	45.5	106	43	63	1.15	1150	68	70	2.8	85.081473	79.116124	29.543232	75.254244	38.140642

A comparison is then made between the measured and the predicted values. The comparison is shown and Fig 4.3.2.1 to 4.3.2.4.

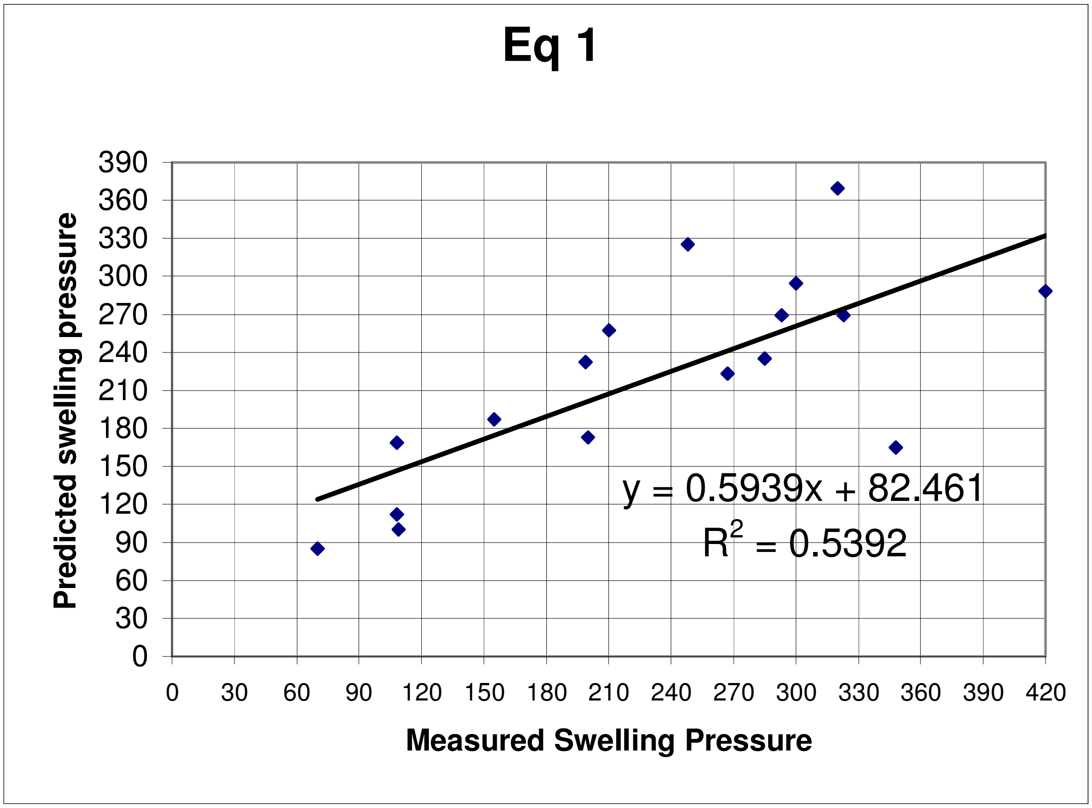

Fig 4.3 The relation ships between the measured and predicted swelling pressure for equation 1.

The Correlation coefficient =0.74

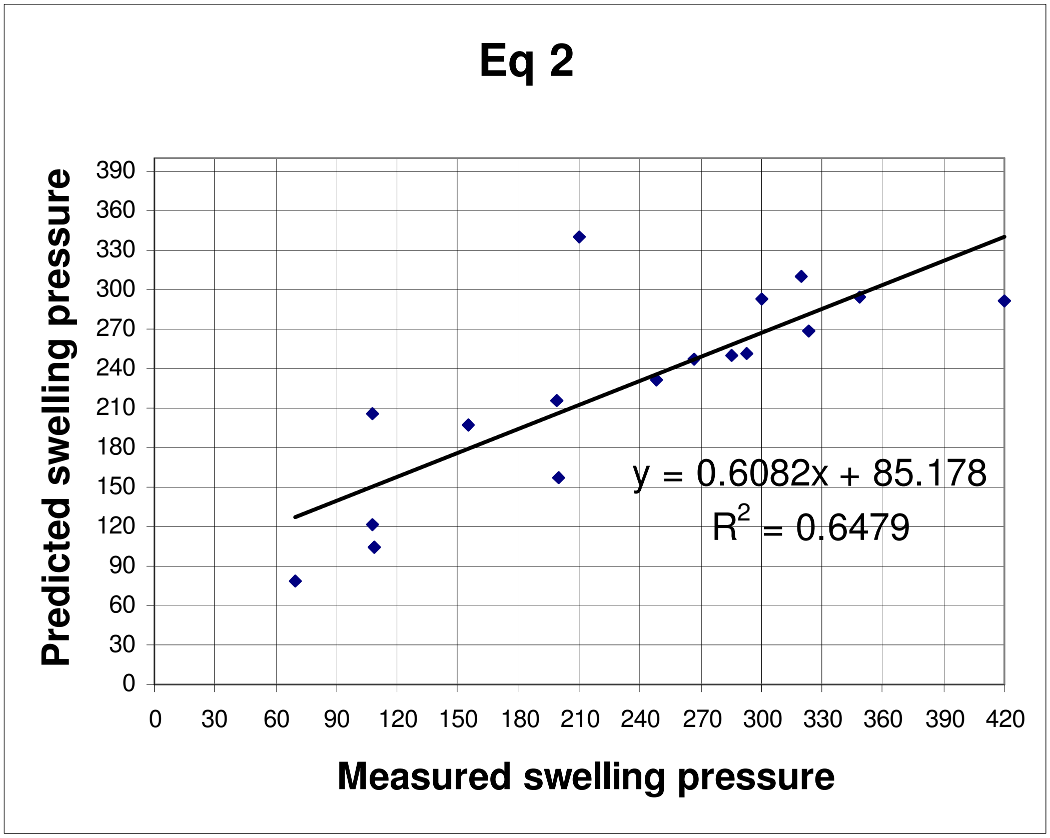

Fig 4.4 The relation ships between the measured and predicted swelling pressure for equation 2.

The Correlation coefficient =0.81

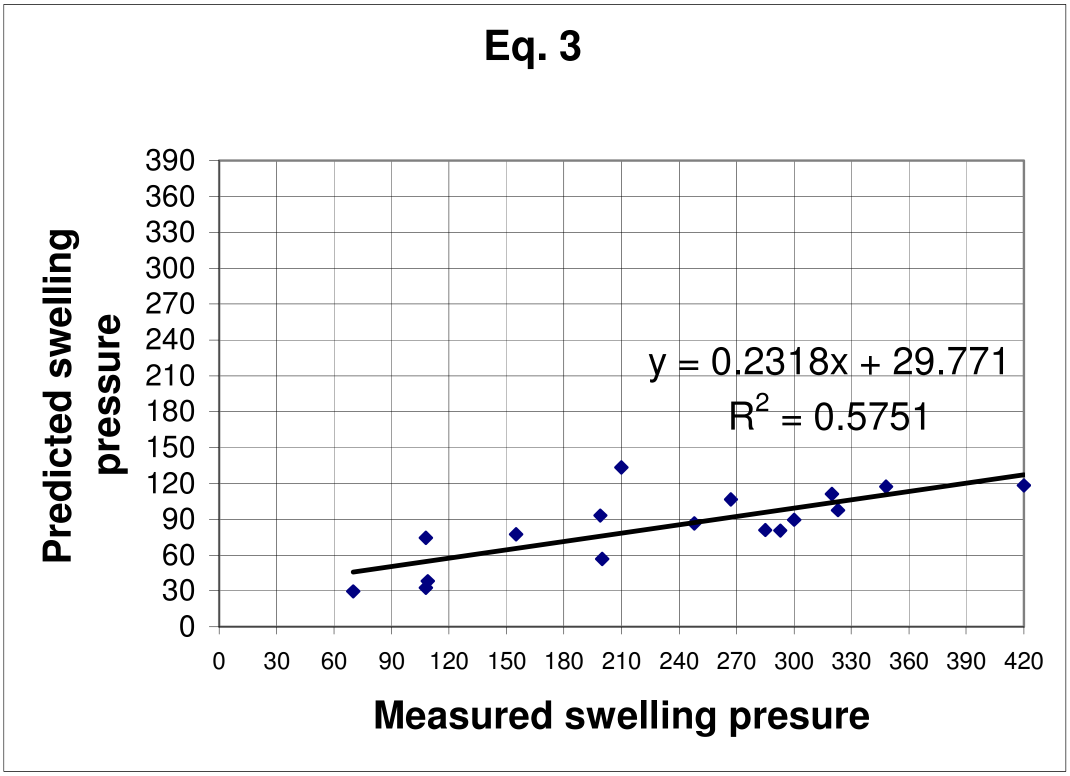

Fig 4.5 The relation ships between the measured and predicted swelling pressure for equation 3.

The Correlation coefficient =0.76

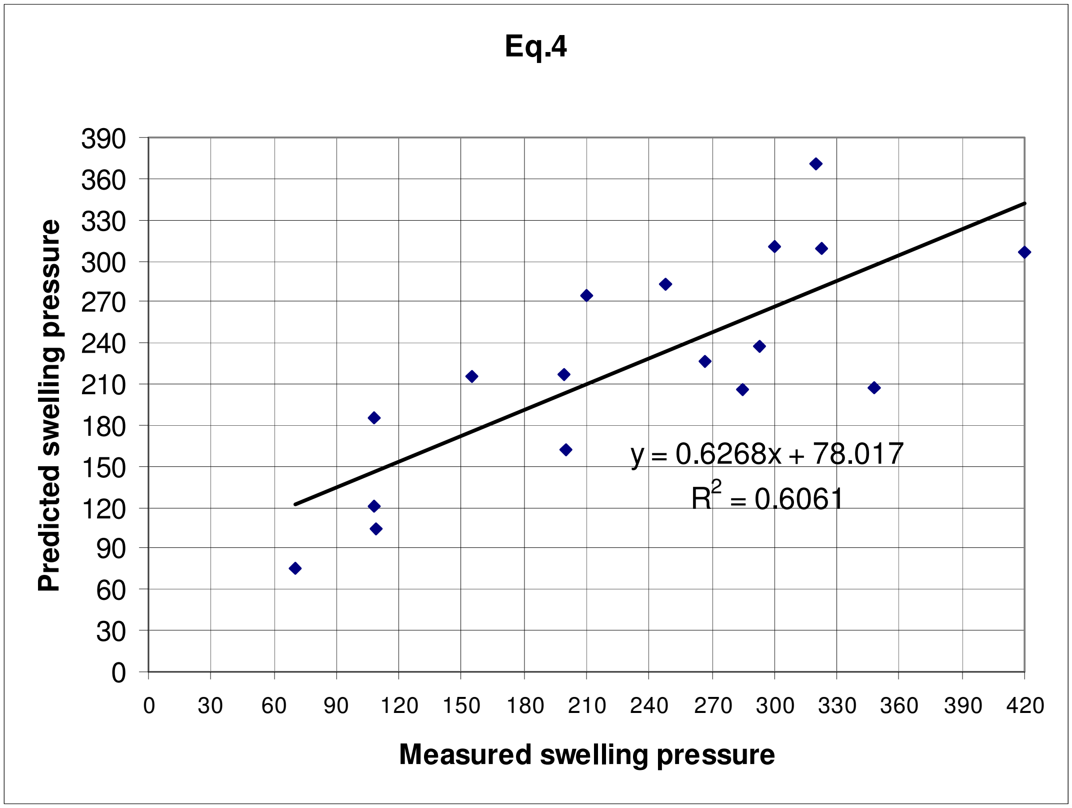

Fig 4.6 The relation ships between the measured and predicted swelling pressure for equation 4.

The Correlation coefficient =0.78

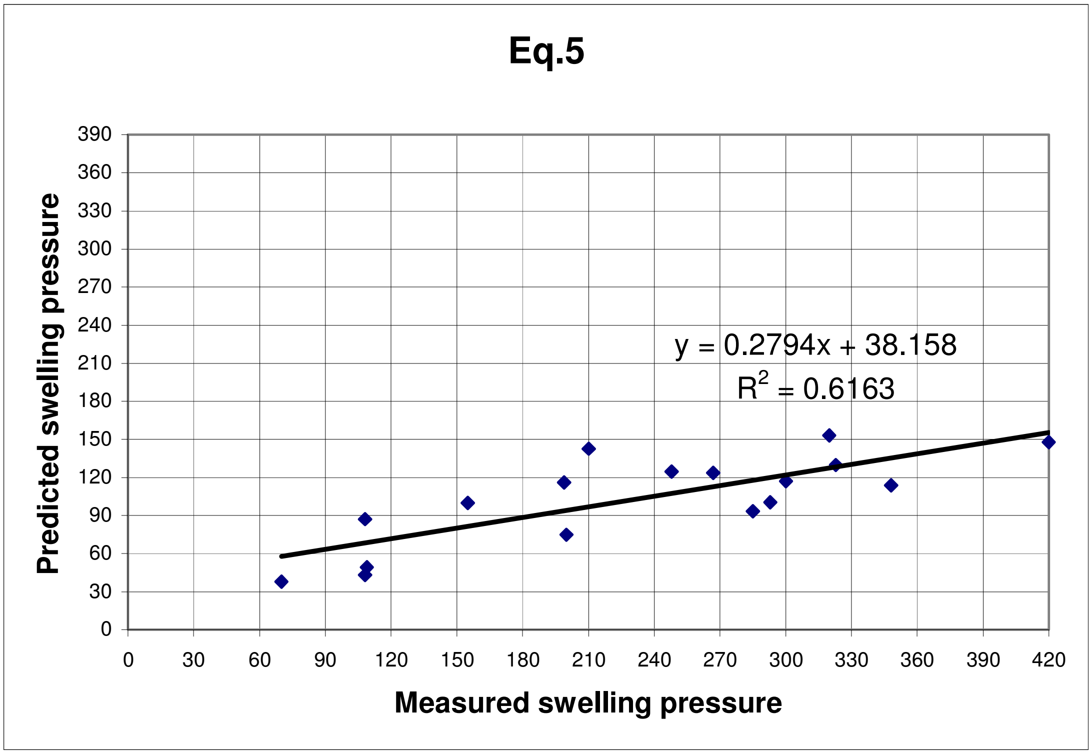

Fig 4.7 The relation ships between the measured and predicted swelling pressure for equation 5.

The Correlation coefficient =0.79

As shown in the fig. 4.3-4.7 all the relations have similar correlation coefficient with 20-26 percent unexplained uncertainty. But the relation described by equation 2 and 4 better closes to the measured swelling pressure. This is because the two equations have the slope of the line that better approaches to one when they compared to the others.

Conclusions and recommendations

1. All the equations stated in chapter 3 section 3.3 do not predict the swelling pressure for the soils used in this study. Only the equation developed by David and Komornik

(1969) predict the swelling pressure reasonably for soils having smaller density and/or swelling pressure.

2. All the developed formulas in this study predict the swelling pressure with various degrees of accuracy; a good approximation is obtained by equations which involves moisture content, liquid limit, plastic index and dry density. Testing for the validity of the newly developed equations gives very good results for equation 2 and 4.

3. The prediction of swelling pressure by empirical relationships cannot be expected to yield accurate results. Therefore, for detail investigation swelling pressure should be determined from oedometer tests on a sample that have an expected initial conditions that could yield maximum swelling pressure. For small projects and for preliminary design purpose of any size of building the equations developed can be used to predict the swelling pressure.

4. The equations developed may be further improved by increasing our database from tests performed on a number of undisturbed samples during the driest season of the year and/or from prepared disturbed samples.

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APPENDIX

Swelling pressure test results

Parameters and formulas used

1. Initial dimensions of the specimen

   Initial height of the specimen, Ho=0.02m

Initial diameter of the specimen, Do=0.07m

Initial cross-sectional area of the specimen, A=0.003848m2

Initial volume of the specimen, V=7.697m3

2. Applied pressure

P=(p/A)*C

Where P=Applied pressure

P= Applied load

A=Cross-sectional area of the specimen

C=5 (lever arm multiplier)

3. The change in height of the specimen (Swell height)

ΔH= (corresponding dial reading –initial dial reading)*calibration factor

4. Total height of the specimen, H=initial height of the specimen at the start of the test plus change in height of the specimen after swelling for each applied load

Swelling pressure test for sample 1 (S1) Black

1. Location of the sample

Bole high school

X- coordinate=476936

Y-coordinate=994652

2 Sample descriptions

Initial moisture content=38.4%

Dry density=1.25g/cc

3. Initial dial reading=700

No	Applied load, p (Kg)	Applied pressure, P (Kg)	Dial reading	Total height, H (mm)	Swell height Sh (mm)	Percent swell
1	0.549	7.137	1026	23.26	3.26	16.3
2	8.234	107.042	886	21.86	1.86	9.3
3	13.234	172.042	850	21.5	1.5	7.5
4	18.234	237.042	820	21.2	1.2	6
5	24.156	314.028	765	20.65	0.65	3.25
6	27.657	359.541	752	20.52	0.52	2.6
7	30.798	400.374	715	20.15	0.15	0.75
8	32.337	420.381	700	20	0	0

Swelling pressure test for sample 1 (S1) Grey

1. Location of the sample

Bole high school

X- coordinate=476936

Y-coordinate=994652

2 Sample descriptions

Initial moisture content=39.6%

Dry density=1.25g/cc

3. Initial dial reading=800

No	Applied load, p (Kg)	Applied pressure, P (Kg)	Dial reading	Total height, H (mm)	Swell height Sh (mm)	Percent swell
1	0.549	7.137	1080	22.8	2.8	14
2	5.538	71.994	928	21.28	1.28	6.4
3	10.538	136.994	880	20.8	0.8	4
4	14.386	187.018	856	20.56	0.56	2.8
5	24.645	320.385	800	20	0	0

Swelling pressure test for sample 2 (S2) Black

1. Location of the sample

Bole in front of Unit College

X- coordinate=476839

Y-coordinate=993360

2 Sample descriptions

Initial moisture content=37.5%

Dry density=1.24g/cc

3. Initial dial reading=500

No	Applied load, p (Kg)	Applied pressure, P (Kg)	Dial reading	Total height, H (mm)	Swell height Sh (mm)	Percent swell
1	0.549	7.137	737	22.37	2.37	11.85
2	5.549	72.137	654	21.54	1.54	9.3
3	8.69	112.97	611	21.11	1.11	7.5
4	13.69	177.97	570	20.7	0.7	6
5	16.831	218.803	550	20.5	0.5	3.25
6	19.972	259.636	519	20.19	0.19	2.6
7	23	299	500	20	0	0.75
8	32.337	420.381	700	22	2	0

Swelling pressure test for sample 2 (S2) Grey

1. Location of the sample

Bole in front of Unit College

X- coordinate=476839

Y-coordinate=993360

2 Sample descriptions

Initial moisture content=43.4%

Dry density=1.17g/cc

3. Initial dial reading=800

No	Applied load, p (Kg)	Applied pressure, P (Kg)	Dial reading	Total height, H (mm)	Swell height Sh (mm)	Percent swell
1	0.549	7.137	891	20.91	0.91	4.55
2	3.69	47.97	847	20.47	0.47	9.3
3	5.229	67.977	820	20.2	0.2	7.5
4	6.768	87.984	812	20.12	0.12	6
5	8.3	107.9	800	20	0	3.25

Swelling pressure test for sample 3 (S3) Black

1. Location of the sample

Ayat

X- coordinate=485893

Y-coordinate=997096

2 Sample descriptions

Initial moisture content=37.6%

Dry density=1.24g/cc

3. Initial dial reading=1100

No	Applied load, p (Kg)	Applied pressure, P (Kg)	Dial reading	Total height, H (mm)	Swell height Sh (mm)	Percent swell
1	0.549	7.137	1290	21.9	1.9	9.5
2	5.549	72.137	1220	21.2	1.2	9.3
3	10.549	137.137	1175	20.75	0.75	7.5
4	15.498	201.474	1148	20.48	0.48	6
5	20.538	266.994	1100	20	0	3.25

Swelling pressure test for sample 3 (S3) Grey

1. Location of the sample

Ayat

X- coordinate=485893

Y-coordinate=997096

2 Sample descriptions

Initial moisture content=44.3%

Dry density=1.17g/cc

3. Initial dial reading=600

No	Applied load,	Applied pressure,	Dial	Total height,	Swell height	Percent swell

	p (Kg)	P (Kg)	reading	H (mm)	Sh (mm)	
1	0.549	7.137	680	20.8	0.8	4
2	3.69	47.97	645	20.45	0.45	9.3
3	8.35	108.55	600	20	0	7.5