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DEPARTMENT OF CIVIL AND ENVIRONMENTAL ENGINEERING

FOUNDATION RECOMMENDATION FOR LIGHTWEIGHT STRUCTURES
(A CASE OF LEGETAFO TOWN)

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List of Abbreviations and Definitions

AASHTO	American Association of State Highway and Transportation Officials
a.m.s.l	Above Mean Sea Level
ASTM	American Society for Testing and Materials
DS	Direct Shear
Km	Kilometer
KN	SI unit of load (KN = Kilo Newton)
KPa	SI unit of pressure (KPa = Kilo Pascal)
LL	Liquid Limit
m	meter
MDD	Maximum Dry Density
mg	Milligram
mm	Millimeter
NMSA	National Meteorological Services Agency
OMC	Optimum Moisture Content
PI	Plasticity Index
PI	Plasticity Index
PL	Plastic Limit
SI	System International
UCS	Unconfined Compression Strength

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Abstract

The selection of safe and economical types of reinforced concrete foundation is one of the challenges for professionals in current designing practice, and this issue is more complex for foundation design for different buildings that are intended to be constructed on expansive soils of Legetafo.

In this project work, an attempt is to made investigate some of the engineering properties of Legetafo soil and to recommend the feasible type of foundation for lightweight structures constructed in the Legetafo town (Chemese). To achieve this objective, soil samples were collected at some remote locations around Chemese and the grain size analysis test, Atterberg limit test, specific gravity test, unconfined compression test, consolidation tests, and swelling pressure test has been carried out.

Laboratory test based on ASTM standard were conducted to characterize the soil at the specified location.

Analysis based on laboratory test result helps to tackle the problem related to expansive soil witnessed in this area and remedial measures has been set by researcher. This include providing Styrofoam at expansion joint between grade beam with ground slab, replacing masonry wall foundation by grade beam noising having maximum diameter 8 reinforcement with maximum embedment depth of one meter below natural ground level, avoiding garden sprinkler and providing micropile in areas where the swelling pressure escape the 310 kPa range.

Finally the compatibility of the recommended methods using Styrofoam between grade beam with slab, and the replacement of masonry wall by grade beam nosing with superstructure shall be checked for other type of foundation in the future.

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CHAPTER ONE - INTRODUCTION

1.1. General

In the civil engineering project, It is major role of geotechnical engineers to propose and recommend an appropprite foundation type through the investigation, analysis and design based on ground condation to keep the constant stability for superstructure.The successes or failure of a foundation depends essentially on the reliability of the various soil parameters obtained from the field investigation and laboratory testing and used as an input into the design of a foundation.

Foundation is an integral part of a structure. The stability of a structure depends upon the stability of the supporting soil. Two important factors that are to be considered are

- The foundation must be stable against the shear failure of the supporting soil.
- The foundation must not settle beyond tolerable limit avoid damage to the structure.

The other factors that require consideration are the location and depth of the foundation. In deciding the location and depth, one has to consider the erosions due to flowing water, underground defects such as root holes, cavities, unconsolidated fills, groundwater level, Presence of expansive soils (Murthy,1990).

Hence, for most structures, it is usually necessary to investigate both the bearing capacity and the settlement of a footing. Whether footing design is controlled by the bearing capacity or the settlement limit rests on a number of factors such as soil condition, footing dimensions, and loads. The ultimate load is the maximum load a foundation can support without shear failure and within an acceptable settlement. In practice, all foundations should be designed and built to ensure a certain safety against bearing capacity failure or excessive settlement.

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In general, any foundation design must meet three essential requirements: according to (Chai, 2000).

- Providing adequate safety against structural failure of the foundation
- Offering adequate bearing capacity of soil beneath the foundation with a specified safety against ultimate failure
- Achieving acceptable total or differential settlements under working loads.

The aim of this project is intended to investigate some of the engineering properties of Legetafo soil and to recommend the possible type of foundation for lightweight structures constructed in the legetafo town. Legetafo is a one special zone of Oromia regional state to administer the eight small towns around the territories of the Addis Abeba city administration which located at elevation of 2388 meter above sea level, and It coordinates are 9^0 4′0′′ N and 38^0 52′60″ E. A vast number of real state is being constructed due to its suitable weather condition. These may lead to the construction of infrastructures such as road, building, different electric and teletransmission tower. However, the engineering properties of the soil in the town are not yet been studied fully in a constructed manner. Therefore, this paper addresses the aforementioned problem.

After visiting the proposed site, four test pit points were selected. Representative disturbed and undisturbed soil samples were collected from open pits by direct manual excavation with a depth of two-meter. The laboratory tests that were carried out include index properties, one-dimensional consolidation, shear strength tests; based on that recommend the feasible type of foundation for lightweight structure in the town.

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1.2. Background of the Problem

Many types of research were done in most part of Ethiopia including Addis Ababa, but there was no project made independently on the investigation of engineering properties of Legetafo soil and on the appropriate types of foundation for the investigated geotechnical data. Today Legetafo is the best real state zone, which is found in Oromia region. Now a day developing itself as a town within the federal state, these may require better refined geotechnical investigation data in order lead recommend the feasible type of foundation for future construction infrastructure and other related structure with geotechnical engineering. Therefore, this study will address better refined geotechnical data and feasible type of foundation recommendation around the town.

1.3. The objective of the Study

The objectives of this project work are the following:

- To recommend the feasible type of foundation for lightweight structure found in Legetafo town.
- To investigate index properties, the shear strength of soil and consolidation characteristics.
- To determine the range of index properties of soil found Legetafo town.
- To classify the soil.
- To give insight into the geotechnical and engineering properties of soil found in legetafo town.
- To study different foundation options for lightweight structures

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1.4. Methodology

1.4.1. Literature Review

1.4.2. Sample Collection

In the field GPS readings will take to locate the coordinates of sampling area, the sampling areas were selected from specific parts of the town (**Chemese**) and four pits were excavated to a maximum depth of three meters (2m). From the excavated pits both disturbed and undisturbed samples were collected for laboratory testing.

1.4.3. Testing and Interpretation

Then, from disturbed and undisturbed samples collected the following laboratory tests were done. All the laboratory tests would be performed according to the American Society for Testing Materials (ASTM) standard.

- Natural moisture content
- Specific gravity
- Atterberg's limit
    - ✓ Liquid limit
    - ✓ Plastic limit
- Grain size analysis
- Unconfined compressive strength
- One-dimensional consolidation
- Swelling pressure

1.4.4. Foundation Recommendation

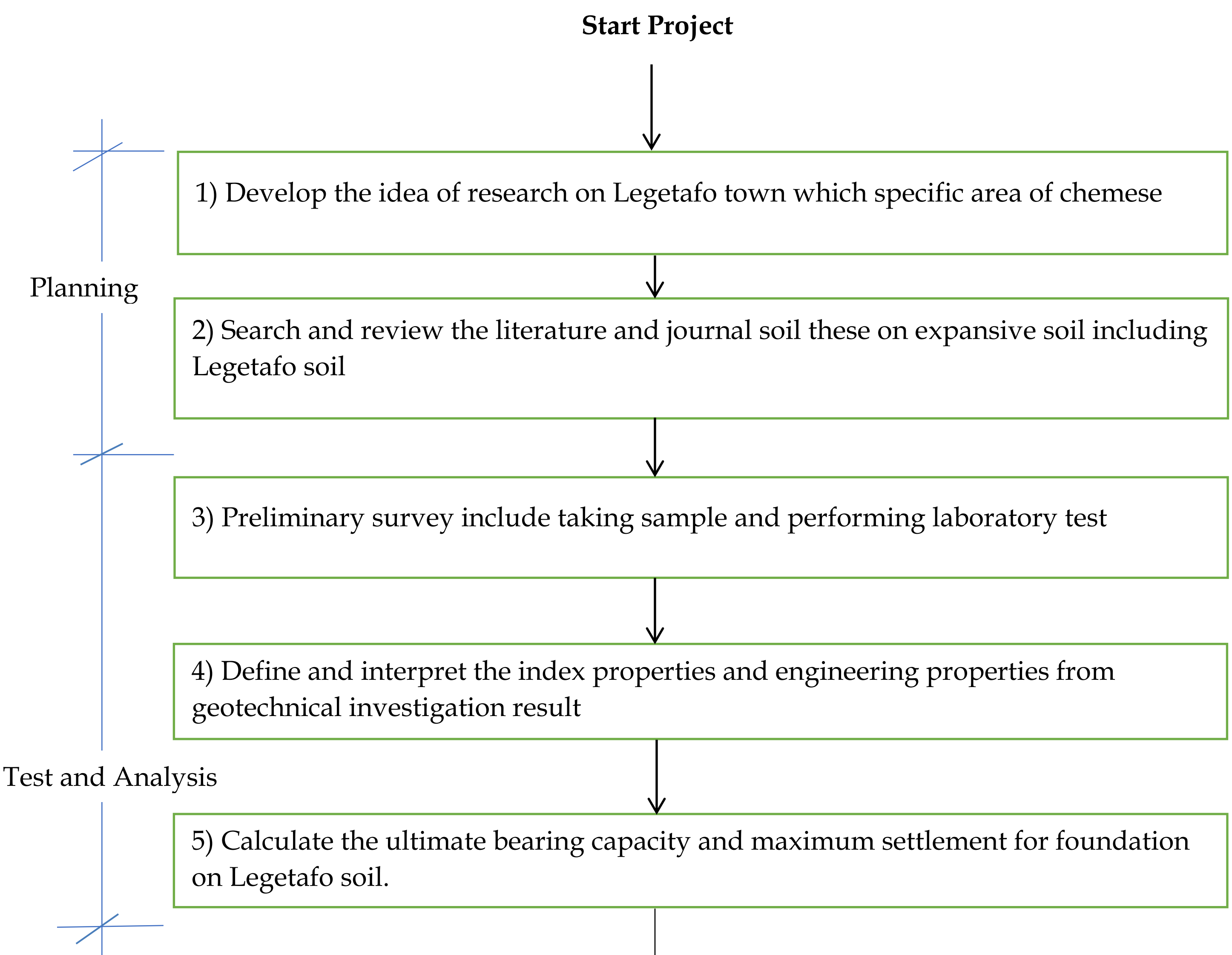
Based on the above test result formulation of the recommendation of the type foundation for lightweight structure (G+2) to build in the study having area according to the European code of standard and EBCS.

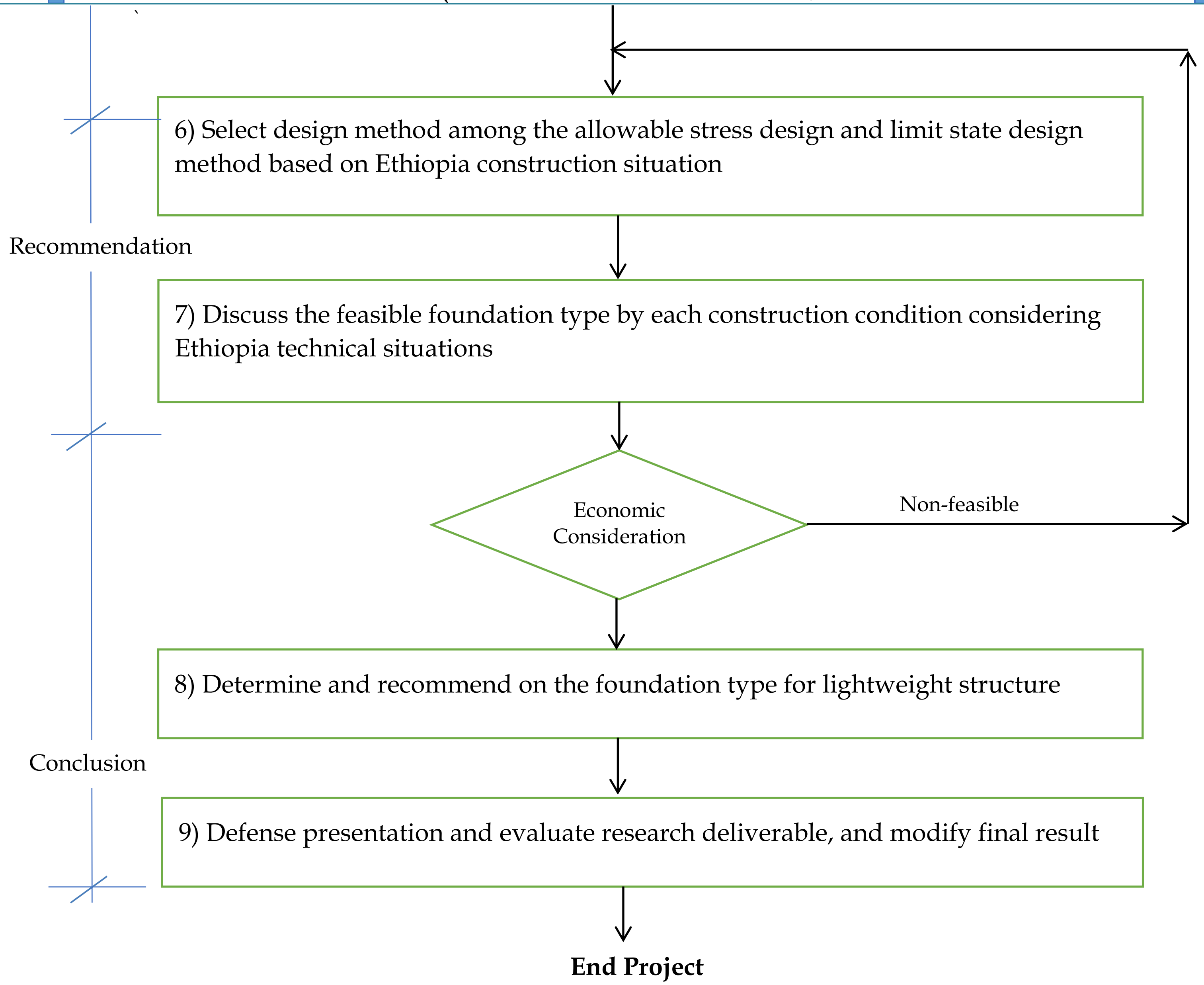
Recommendation of the foundations is based on the following comparison point.

- economical point of view,
- safety point of view (including settlement characteristics and bearing capacity)
- Construction time and method of construction point of view.

1.4.5. Result Presentation

The test result obtained will be presented using chart and tables. The conclusion and the recommended foundation type will be presented in table and discussion.





1.5. The scope of the Study

Eight samples of soil from four pits are collected. The scope of this study is limited to investigating the index properties, shear strength, consolidation characteristic and recommendation the type of foundation only lightweight structure. Due to budget and time constraint, the depth of investigation in this project is limited to the maximum depth of two meters and their foundation recommendation only lightweight structure.

CHAPTER –TWO - LITERATURE REVIEW

2.1. Soil Formation

The soil is defined as a natural aggregate of mineral grains, with or without organic constituents that can be separated by gentle mechanical means such as agitation in water. By contrast, rock is considered to be a natural aggregate of mineral grains connected by strong and permanent cohesive force. The process of weathering of rock decreases the cohesive force binding the mineral grains and leads to the disintegration of bigger masses to smaller and smaller particles. Soils are formed by the process of weathering of the parent rock (Arora, 1997).

Soils are formed from the physical and chemical weathering of rocks. Physical weathering involves reduction of size without any change in the original composition of the parent rock. The main agent responsible for these processes is exfoliation, unloading, erosion, freezing, and thawing. Chemical weathering causes both reductions in size and chemical alteration of the original parent rock. The main agents responsible for chemical weathering are hydration, carbonation, and oxidation. Often chemical and physical weathering takes place in concert.

Chemical weathering is much more important than physical weathering in soil formation. Soils at a particular site can be residual (that is weathered in place) or transported (moved by water, wind, glacier, etc.) and the geologic history of a particular deposit significantly affects its engineering behavior.

Chemical decomposition of rocks results in the formation of clay minerals. These clay minerals impart plastic properties to soils. Clayey soils are formed by chemical decomposition (Murthy, 1990).

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2.2. Expansive Soils

Expansive soils are found extensively in tropical areas. The presence of expansive soils greatly affects the construction activities in many parts of southwestern United States, South America, Canada, Africa, Australia, Europe, India, China, and the Middle East. More and more expansive soil regions are being discovered each year with an increase in the number of constructional activities, particularly in the underdeveloped nations. These soils are characterized by the presence of a large proportion of highly active clay minerals of the montmorillonite group which is responsible for the pronounced volume change capability of the soils (Sridharan, and Prakash, 2005, p. 1). Moreover, they are commonly black or gray in color are clay soils with high plasticity.

Most of the structural damages due to expansive soils result from the differential rather than the total movement of the foundation soil as a result of swell. Damage 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 equilibrium (Kibrom,2005). Such Light structures can't exert the necessary counter load to overcome the swelling pressure.

A number of soils such as volcanic ash soils and diatomaceous earth cannot be grouped by using the existing classification system only; however, their distribution is restricted to certain areas.

Unlike these, the expansive soils are distributed geographically vary widely, covering large areas. Hence, identification and classification of such soils are essential (Sridharan et al, 2005, p.1).

Many criteria are available to identify and characterize expansive soils, such as liquid limit, plasticity index, shrinkage limit, activity, and percent free swell. There is an evaluation of the swelling potential of the soils by single index method and those are expressed by the correlation between the swelling potential with those of index properties (Chen, 1975).

2.2.1. Index and Engineering Properties of expansive soil

Essentially expansive soil is one that changes in volume in relation to changes in water content. Here the focus is on soils that exhibit significant swell potential and in addition shrinkage potential also exists. There are a number of cases where expansion can occur through chemically induced changes (e.g. swelling of lime treated sulfate soils). However, many soils that exhibit swelling and shrinking behavior contain expansive clay minerals, such as smectite, that absorb water, the more of this clay soil contains the higher its swell potential and the more water it can absorb. As a result, these materials swell, and thus increase in volume, when they get wet and shrink when they dry. The more water they absorb the more their volume increases, for the most expansive clays expansions of 10% are not uncommon (Chen 1988; Nelson and Miller, 1992). It should be noted that other soils exhibit volume change characteristics with changes in water content, e.g. collapsible soil

The amount by which the ground can shrink and/or swell is determined by the water content in the near-surface zone; significant activity usually occurs to about 3m depth, unless this zone is extended by the presence of tree roots (Driscoll, 1983; Biddle 1998). Fine-grained clay-rich soils can absorb large quantities of water after rainfall, becoming sticky and heavy. Conversely, they can also become very hard when dry, resulting in shrinking and cracking of the ground. This hardening and softening are known as 'shrink-swell' behavior. When supporting structures, the effects of significant changes in water content on soils with a high shrink-swell potential can be severe.

Swelling and shrinkage are not fully reversible processes (Holtz & Kovacs, 1981). The process of shrinkage causes cracks, which on re-wetting, do not close-up perfectly and hence cause the soil to bulk-out slightly, and also allow enhanced access to water for the swelling process. In geological time scales shrinkage cracks may become in-filled with sediment, thus imparting heterogeneity to the soil. When material falls into cracks the soil is unable to move back, thus resulting in enhanced swelling pressures.

2.2.2. Swelling potential and swelling pressure

Seed et al. (1962) defined swelling potential as the magnitude of heave of the 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. The swelling potential in percent (Sp) is given by:

$$Sp = (\delta h/h)*100 \qquad \text{eqn-2.1}$$

Where: δh- Amount of vertical swell as shown in fig.2.1 (b) h- Initial height

2.2.3. Swelling pressure

Swelling pressure is a very useful index of the troubling potential of 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.2.1(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).

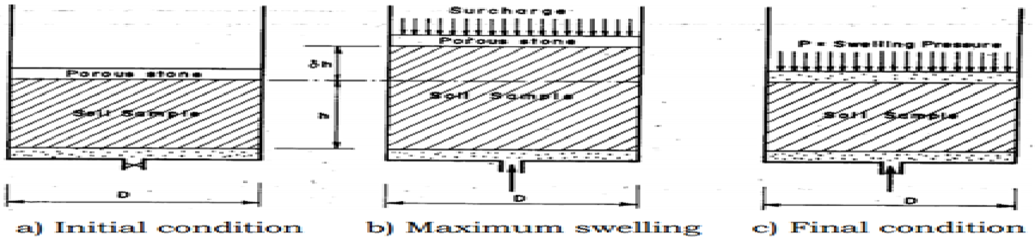

Figure 2. 1 diagrammatic description of swelling potential and swelling pressure

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2.2.4. Factors affecting swelling characteristics and swelling potential of expansive soil

The mechanism of shrink-swell in expansive soil is complex and influenced by a number of factors. The 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 (Alemayehu & Lekikun , 1997).

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.

- **Clay mineralogy:-** 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 (Alemayehu, 1997).
- **Dry density;-**Higher density usually indicates closer particle spacing, which may mean greater repulsive forces between particles and larger swelling potential.
- **Soil structure and fabric:-**The term fabric refers to the arrangement of particles, particle groups and pore spaces in the soil. The structure has a broader meaning of the combined effect of fabric, composition and inter-particle force. The unique relationship between the water content of a soil and matric suction is influenced by soil fabric which in turn affects the swelling potential of the soil.

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- **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 interparticle force field which is dependent on the negative surface charge and electrochemistry 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. Environmental Factors: - Environmental factors influence the changes that may occur in the internal Force system (Chen,1975). These factors are mostly associated with moisture. They are:

- **Initial Moisture Content: -** A desiccated expansive soil will have a 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.
- **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.
- **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.

3. Drainage and broken utility:- Surface drainage features and broken water pipeline stagnant around a poorly graded house foundation, provide a source of water at the surface; leaky plumping can give the soil access to water at greater depth (Chen 1975).

- **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.
- **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.
- **Temperature; -** Increasing temperatures cause moisture to diffuse to cooler areas beneath pavements and buildings.

4. Stress Condition: - Volume change is directly related to change in the state of stress in the soil. A reduction in total stress due to the 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 (Alemayehu & Lekikun , 1997).

2.3. Depth of Heaving

The depth of soil that is contributing to heave at any instant of time depends on two factors. These are the depth to which water contents in the soil have increased since the time of construction and the expansion potential of the various soil strata. As water migrates through a soil profile different strata become wetted, some of which may have more swell potential than others. Consequently, the zone of soil that is contributing to heave varies with time.

In order to determine the amount of heave that will occur at a particular time, it is necessary to know what zone of soil is being wetted and the expansive nature of that soil. This, in turn, depends on the manner in which the groundwater migrates in the soil. Furthermore, the effect of soil heave on a structure will depend on the particular member of the foundation that is being affected by the heave, and where that member is located in the soil profile (Technical manual, Departement of army, 1983).

The term "Active Zone" has been in common usage in the field of expansive soils. However, the usage of that term has taken different meaning at different times and in different places. Therefore, for purposes of clarity and consistency, the following four definitions have been put forth (Nelson, Overton, and Durkee, 2001).

1. **Active Zone** is that zone of soil that is contributing to heave due to soil expansion at a particular point in time. The depth of the active zone will vary as heave progresses, and therefore varies with time. Even though the soil may have the potential to swell and shrink below the depth of the active zone, volume changes will not take places because the water content of the soil is constant.

2**. Zone of Seasonal Moisture Fluctuations** that zone of soil in which water contents change due to climatic changes.

3. **The depth of Wetting** is the depth to which water contents have increased due to external factors. Such factors could include capillary rise after the elimination of evapotranspiration from the surface, infiltration due to irrigation or precipitation, or introduction of water from off-site.

Underground sources may include broken water lines, perched water tables, flow through more permeable strata that are recharged at distant locations, or a number of other factors.

4. **The depth of Potential Heave** is the depth to which the overburden vertical stress equals or exceeds the swelling pressure of the soil. This represents the maximum depth of the Active Zone that could occur.

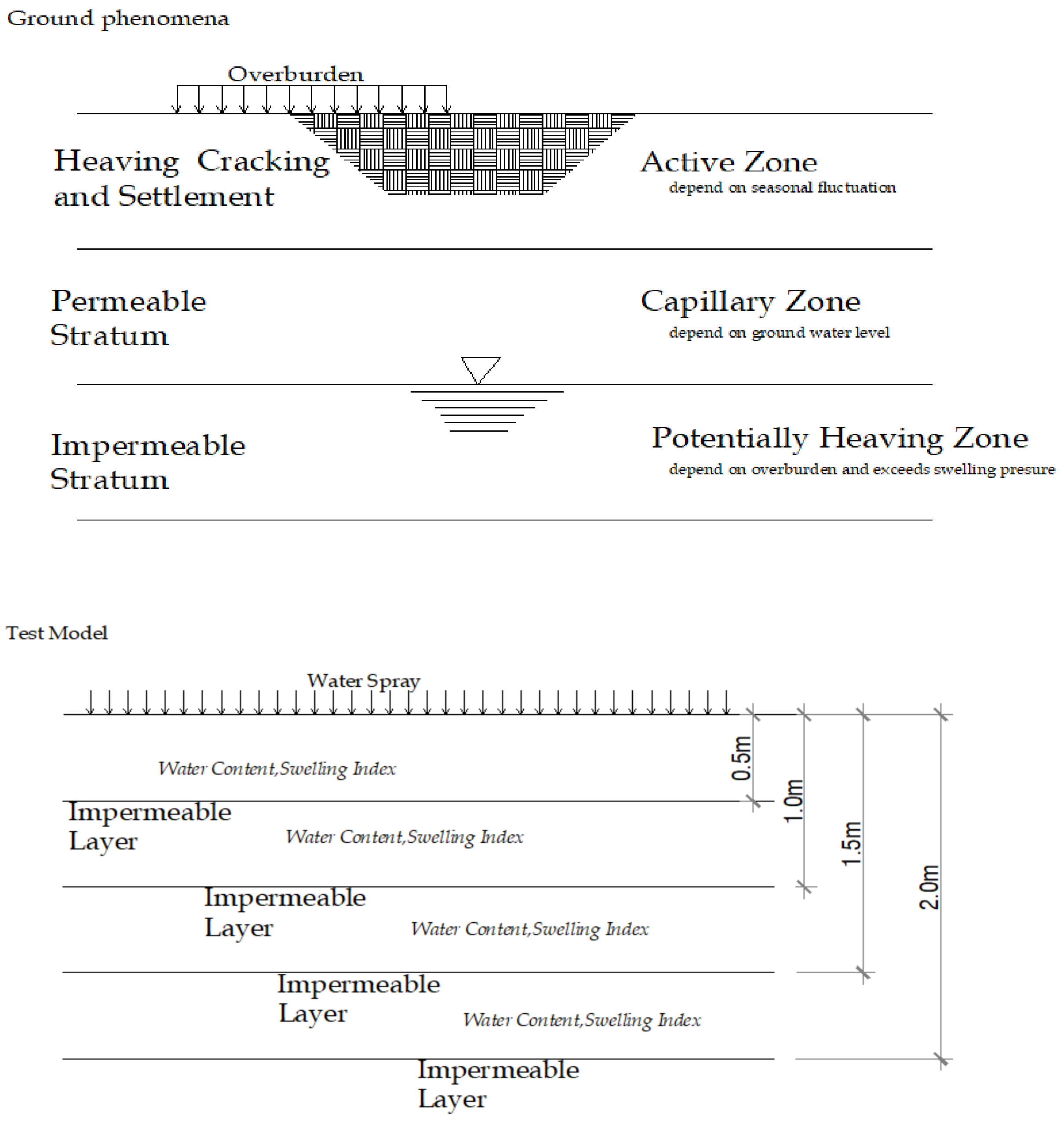

Figure 2.2 model for investigation of expamsive soil

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2.4. Techniques for preventing heave damage

In order to minimize the danger of damages to buildings because of heave and shrinkage (Alemayehu, 1992 and Chen, 1988), the Following methods have been used:-

1. Moisture control
2. Soil stabilization
3. Structural measures.

1. **Moisture Control: -** The main cause of heaving and shrinkage is the fluctuation of moisture under and around the structure is the biggest problem. In any site, depending upon the topographical, geological and weather conditions, the natural groundwater fluctuates. In a country like Ethiopia, where there are distinct dry and wet seasons, the fluctuation of the groundwater table during these periods is large. Each site has its own characteristic active zone. This is the zone under the ground surface in which the fluctuation of the moisture content is large. In some areas of Addis Ababa, shrinkage cracks as deep as two to three meters are common occurrences during the dry season. This seasonal fluctuation of the moisture content along the depth becomes minimal from approximately three to four meters downwards. In addition to the fluctuation of the groundwater one should also consider free water which may seep under foundations, or the effect of evaporation which would cause moisture migration these has been preventing by providing by moisture barriers above such as horizontal and vertical barriers around the foundation soil, and adequate surface and sub-surface drainage system

(Alemayehu, 1984).

2. **Soil Stabilization** is made to minimize the swelling potential of expansive soil by one or more of the following methods, namely:-soil replacement, pre-wetting, compaction control, and chemical treatment (Chen, 1988).

- **Soil Replacement Method**, all of the swelling soil is removed and replaced with non-swelling soil. This method is recommended for cases where the thickness of expansive soil is small, less than 2.5m. It is the simplest and easiest solution for slabs and footings founded on expansive soils and it provides the safest method for slab-on-ground construction (Das, 1988).

- **Pre-Wetting Method**, the soil is flooded to achieve swelling prior to the commencement of construction. This method, however, has a disadvantage in reducing the bearing capacity of foundation soil by reducing the shear strength of the soil. It is successful if the depth of the active zone is not large. It has been effectively used for stabilizing soil beneath floor slabs, pavements. However, its application for building foundations is still questionable and risky (Das, 1988).
- **Compaction Control Method**:- the upper soil is sacrificed and re-compacted to low soil density. This reduces the swelling property and heave of the expansive soil. The main advantage of using this approach is that the swelling potential can be reduced without the negative effects caused by introducing excessive moisture into the soil. The draw-back with this method is that the low-density compaction will result in low bearing capacity of subgrade of foundation soil (Gromko, 1974).
- **Chemical Treatment Method**:- it is the process of mixing additives like lime, cement, and other chemicals to expansive soils, so as to retard their potential expansiveness. Lime will reduce plasticity and hence the swelling potential of the soil, it is often used successfully in the construction of highways and airports. Meanwhile, the action of cement is to reduce the liquid limit, plastic index and the potential volume change, it is mainly used in highway constructions. However, the use of lime and cement stabilization for foundation soils are not advised because one may require to treat considerable depth. Also, its long term durability is not known.

3. Structural Measures The structural measures that should be undertaken in order to minimize or, if possible, to eliminate damages of structures due to heaving are dependent on the architectural and structural design of the structure.in this method, different kind of technique may apply according to (Nelson & Miller, 1992).

- **Providing Rigid Building**: the -A rigid building is one which is free from uneven Displacement which might cause structural damage. The rigidity of a building can be achieved by providing adequate reinforcement to foundation, beams, slabs, and walls in such a manner that all of these will result in a monolithic form of the building. This method is recommended for light and compact buildings. Mat foundations can be used under this category because they can be referred to as structural slab-on-ground and stiffened slab to prevent the possibility of differential heave.
- **Providing Flexibility to the building**: the -A flexible building is one which allows differential movement to occur between its members without itself being damaged. Flexibility can be achieved by providing the building into small rigid compartments with flexible joints. This method is recommended for long buildings.
- **Providing Deep Foundations**;- Pile foundations provide a suitable solution for a variety of structures located on heaving soils. They may prove economical in areas where considerable heave is to be expected, and the additional cost can be balanced against the saving in future maintenance. The piles should be placed well below the active zone where the seasonal fluctuation of the moisture content is minimum (i.e. greater than 3.5m depth). The bottom of piles should be enlarged (under-reamed) to increase bearing and anchoring capacities of the piles. Piles should be protected from tension failure by either decreasing pile diameter or by reinforcing pile for tensile force due to soil heave, or by providing pile with the sleeve (of weak spongy material) in order to isolate pile shaft from the soil.

2.5. Laboratory Tests

A wide variety of laboratory tests can be performed on soils to measure a wide variety of soil properties. Some soil properties are intrinsic to the composition of the soil matrix and are not affected by sample disturbance, while other properties depend on the structure of the soil as well as its composition, and can only be effectively tested on relatively undisturbed samples.

Some soil tests measure direct properties of the soil, while other measure "index properties "which provide useful information about the soil without directly measuring the properties property desired. The main index properties of coarse grained soils are particle size and relative density. For fine grained soils, the main index properties are Atterberg limits and consistency.[Alemayehu,1986]

2.5.1 Index property tests

2.5.1.1 Natural moisture content

For most soils, the water content may be an important index used for establishing the relationship between the way a soil behaves and its properties. The consistency of a fine grained soil largely depends on its water content. The water content is also used in expressing the phase relationships of air, water, and solids in a given volume of soil [Alemayehu,1986].

2.5.1.2 Atterberg limits

Atterberg Limits are defined as water contents at certain limiting or critical stages in soil behavior. They, along with the natural water content, are the most important items in the description of fine grained soils. They are used in classification of fine grained soils, and they are useful because they correlate with the engineering properties and engineering behavior of fine-grained soils [Arora,1997].

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2.5.1.3 Specific gravity

A soil's specific gravity largely depends on the density of the minerals making up the individual soil particles. However, as a general guide, some typical values for specific soil types are as follows. [Arora,1997]

- The specific gravity of the solid substance of most inorganic soils varies between 2.6 and 2.9.
- Sand particles composes of quartz have a specific gravity ranging from 2.64 to 2.66.
- Inorganic clays generally range from 2.7 to 2.9.
- Soils with large amount of organic matter or porous particle (such as diatomaceous earth) have specific gravity below 2.6.Some ranges as low as 2.0.

2.5.1.4 Free swell

To study the swelling property of the soils, the simplest test conducted is free swell test. This test is performed by slowly pouring 10ml of oven dry soil which has passed the No.40 (0.425mm) sieve in to 100 ml graduated cylinder filled with distilled (tap) water. After 24 hours, final volume of the suspension being read. Hence, free swell is defined as:

$$\text{Free swell index (FSI)} = \frac{\text{Final volume} - \text{initial volume of the soil}}{\text{initial volume}} X 100\% \quad \text{................Equation-2}$$

2.5.1.5 Soil classification

A soil classification system is an arrangement of different soils into groups having similar properties. The purpose of soil classification is to make possible the estimation of soil properties by association with soils of the same class whose properties are known and to provide the engineer with accurate method of soils description. The soils under investigation have been classified according to AASHTO M-145 and UCSC [Murthy,1990].

2.5.1.5.1 Unified classification system (USCS)

This system describes a system for classifying minerals and organo-mineral soils for engineering purposes based on laboratory determination of particle-size characteristics, liquid limit, and plasticity index and shall be used when precise classification is required [Murthy,1990].

2.5.1.5.2 AASHTO classification system

The AASHTO system uses similar techniques as that of USC but the dividing line has an equation of the form PI= LL-30. It generally classifies a soil broadly into granular material and silt-clay material. The granular material is further divided into three groups which are called A-1, A-2 and A-3. The silt-clay material is in turn divided into four groups namely, A-4, A-5, A-6 and A-7.

The AASHTO system further uses a group index (GI) to rate a soil qualitatively within a group.

The equation for the GI is;-

$GI= (F-35)\ [0.2 + 0.005(LL-40)] +0.01(F-15)\ (PI-10)$..Equation-3

Where: - F = percent passing No. 200 sieve

LL= Liquid Limit, in percent

PI = Plasticity Index, in percent

2.5.1.6 Classifications of soils based on Activity chart

2.5.1.6.1 Activity Number

The Atterberge limit of a given cohesive soil is controlled by the type and amount of claymineral present in it. Skempton considers that the significant change in volume of a clay soil during shrinking or swelling is a function of plasticity index and the quantity of colloidal clay particles present in the soil. Hence, Activity number has been used as

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an index property to determine the swelling potential of clays. The activity of clay is expressed as: [Arora,1997]

Activity (A) =Plasticity index (PI) /Percentage of clay fraction by weight(C)

2.4.1.6.2 Liquidity Index

The Relative consistency of a cohesive soil can be defined by a ratio called liquidity index LI. It is defined as:

$$LI = \frac{\mathrm{Wn-PL}}{\mathrm{Ip}} \text{ ....Equation-4}$$

Where; - PI = Plasticity Index,

LI = Liquidity index,

PL = Plastic Limit,

Wn = Natural moisture content.

For a natural soil deposit that is in plastic state (i.e., LL > Wn > PL), the value of the liquidity index varies between 1 and 0.A natural soil deposit with Wn > LL will have a liquidity index greater than1.In an undisturbed state, these soils may be stable: however, a sudden shock may transform them in to liquid state .Such soils are called sensitive clay.

2.5.2 Engineering properties

2.5.2.1 Shear strength test

The shear strength of soils is an important aspect in many foundation engineering problems such as the bearing capacity of shallow foundations and piles, the stability of the slopes of dams and embankments, and lateral earth pressure on retaining walls [Murthy,1990].

The main objectives of shear strength test in soil engineering, is generally to determine the shear strength parameters (i.e., the cohesion and angle of internal friction) in terms of total or effective stresses under known test condition.

Some of the inter particle forces which are believed to contribute to soil cohesion includes:

- Valence forces associated with surface
- Ionic forces associated with ions dissociated from polar materials
- Dipole forces and moments associated with polar materials
- Molecular attraction or van der Waal‟s forces.

2.5.2.1.1 Unconfined compression test (UC)

The UC test is the simplest and quickest test used to determine the shear strength of a cohesive soil. An undisturbed or remolded sample of cylindrical shape, about 38 mm in diameter and 76 mm in height is subjected to uni-axial compression until the soil fails ,only cohesive soils can be tested.

The strength of a soil determined by this test is influenced by the length to diameter ratio of the sample and the rate of strain. It is generally accepted that ratios of length to diameter of 2 to 2.5 are satisfactory. Similarly, satisfactory rates of strain are 0.5 to 2.0% per minute [Terzaghi,1943]. For the current investigation, length to diameter ratio of 2 and a strain rate of 2% per minute (about 1.5 mm per minute) is used.

2.5.2.2 Consolidation

When a soil layer is subjected to a compressive stress, such as during the construction of a structure, it will exhibit a certain amount of compression. This compression is achieved through a number of ways, including rearrangement of the soil solids or extrusion of the pore air and/or water. According to Terzaghi (1943), "a decrease of water content of a saturated soil without replacement of the water by air is called a process of consolidation."

The compression of the soil mass due to the imposed stresses may be almost immediate or time dependent according to the permeability characteristics of the soil.The compressibility characteristics of a soil mass might be due to any or a combination of the following factors.

- Compression of the solid matter.
- Compression of water and air within the voids.
- Escape of water and air from the voids.

The compression of a saturated soil under a steady static pressure is known as consolidation. It is entirely due to expulsion of water from the voids (Soil Mechanics and Foundation Engineering by Dr. K.R. Arora).

Consolidation may be due to one or more of the following factors:

- External static loads from structures.
- Self-weight of the soil such as recently placed fills.
- Lowering of the ground water table.
- Desiccation.

2.5.2.2.1 The standard one-dimensional consolidation test

The main purpose of the consolidation test on soil samples is to obtain the necessary information about the compressibility properties of a saturated soil for use in determining the magnitude and rate of settlement of structures. The following test procedure is applied to any type of soil in the standard consolidation test.

Loads are applied in steps in such a way that the successive load intensity, p, is twice the preceding one. The load intensities commonly used being 1/4, 1/2,1, 2,4, 8, and 16 tons/ft2 (25,

50,100,200,400, 800 and 1600 KN/m ^{2}). Each load is allowed to stand until compression has practically ceased (no longer than 24 hours). The dial readings are taken at elapsed times of 1/4, 1/2, 1, 2, 4, 8, 15, 30, 60, 120, 240, 480 and 1440 minutes from the time the new increment of load.

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2.5.2.2.2 Pre-consolidation pressure

A soil in the field at some depth has been subjected to a certain maximum effective past pressure in its geologic history. This maximum effective past pressure may be equal to or less than the existing effective overburden pressure **(P_0)** at the time of sampling. The reduction of effective pressure in the field may be caused by natural geologic process or human processes. During soil sampling, the existing effective overburden pressure is also released. This results in some expansion. When this specimen is subjected to a consolidation test, small amount of compression will occur when the effective pressure applied is less than the maximum effective overburden pressure in the field to which the soil has been subjected in the past (Murthy,1990).

The two basic definitions of clay based on stress history are:

- Normally consolidated; whose present overburden pressure is the maximum pressure that the soil was subjected to it in the past.
- Over-consolidated; whose present effective overburden pressure is less than that which the soil experienced in the past. The maximum effective past pressure is called the pre-consolidation pressure.

There are a few graphical methods for determining the pre-consolidation pressure based on laboratory test data. No suitable criteria exist for appraising the relative merits of the various methods. The earliest and the most widely used method was the one proposed by Casagrande (1936). The method involves locating the point of maximum curvature, B, on the laboratory e-log p curve of an undisturbed sample as shown in Fig. 2.3. From B, a tangent is drawn to the curve and a horizontal line is also constructed. The ngle between these two lines is then bisected. The abscissa of the point of intersection of this bisector with the upward extension of the inclined straight part corresponds to the pre-consolidation pressure Pc.

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The relative amount of pre-consolidation is usually reported as the over-consolidation ratio (OCR) defined as:

$$OCR = \frac{Pc}{Po} \quad \text{……………………………………………………..equation-5}$$

If OCR =1, the soil is normally consolidated soil. If OCR >1 the soil is over consolidated soil. The over-consolidation ratio of soils has been observed to decrease with depth, eventually reaching a value of 1 (normally consolidated state) (Geotechnical Engineering by V.N.S. Murthy).

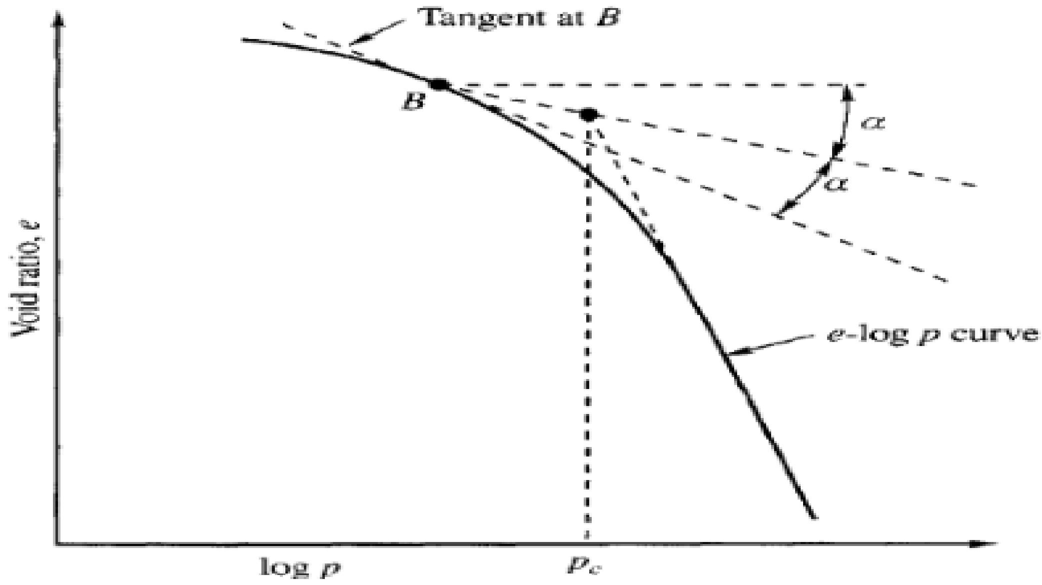

Figure 2.3 method of determining Pc by Casagrande method

2.5.2.2.3 Compression index (C_C) and swelling index (C_S)

The compression index (Cc) and swelling index (C S) can be determined by graphic construction from laboratory results for void ratio and pressure, which is equal to the slope of the linear portion of the void ratio versus log pressure loading curve and unloading curve respectively. Thus:

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$$Cc = \frac{e_1 - e_2}{\log\delta_2' - \log\delta_1'} \quad \text{.........Equation-6}$$

$$Cs = \frac{e_1 - e_2}{\log\delta_2' - \log\delta_1'} \quad \text{.........Equation-7}$$

There are several empirical expressions for compression index and swelling index determination is also used for approximate calculation in the absence of laboratory consolidation data.

Compression index is extremely useful for determination of field settlement caused by consolidation.

2.5.2.3 Swelling pressure

Swelling pressure is defined as the pressure which prevents the specimen from swelling or that pressure which is required to return the specimen back to its original state (void ratio, height) after swelling (ASTM D 4546-96).

Basically, the methods of measuring swelling pressure can be either strain controlled or stress controlled.

Strain controlled method is based on the principle of controlling the strain that is developed as water is added. In such a test, modification to the conventional Oedometer is required to allow the control of strain during testing and measurement of the resulting loads.

For stress controlled tests, the conventional Oedometer is used. The samples are placed in the consolidation ring trimmed to the height of the ring. After loading with a standard load of 7kpa, water is added to the sample. When swelling of the sample is ceased, the vertical stress is increased until the sample is compressed back to its original height. The stress required to compress the sample to its original height is the zero volume change swelling pressure or simply swelling pressure.

Swelling pressure is an integral soil property, hence whether determining it through strain or stress controlled, the result is expected to be the same (Chen, 1988).The swelling pressure is determined graphically from void ratio versus logarithm of pressure curve. Horizontal line was drawn from the end of swell curve towards the loading curve and then the swelling pressure was read directly by projecting downward on the abscissa.

2.6. Foundation option in expansive soil

A foundation is an integral part of a structure. The stability of a structure depends upon the stability of the supporting soil. Two important factors that are to be considered are

1. The foundation must be stable against the shear failure of the supporting soil.

2. The foundation must not settle beyond a tolerable limit to avoid damage to the structure.

The other factors that require consideration are the location and depth of the foundation. In deciding the location and depth, one has to consider the erosions due to flowing water, underground defects such as root holes, cavities, unconsolidated fills, groundwater level, the presence of expansive soils etc (Murthy , 1990).

Hence, for most structures, it is usually necessary to investigate both the bearing capacity and the settlement of a footing. Whether footing design is controlled by the bearing capacity or the settlement limit rests on a number of factors such as soil condition, footing dimensions, and loads. The ultimate load is the maximum load a foundation can support without shear failure and within an acceptable settlement. In practice, all foundations should be designed and built to ensure a certain safety against bearing capacity failure or excessive settlement.

In general, any foundation design must meet three essential requirements, according to (Chai, 2000)

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1. Providing adequate safety against structural failure of the foundation

2. Offering adequate bearing capacity of soil beneath the foundation with a specified safety against ultimate failure

3. Achieving acceptable total or differential settlements under working loads.

A large number of factors influence foundation types and design methods these included aspects such as climate, financial and legal aspects as well as technical issues. Importantly, swell/shrink behavior often does not manifest itself for several months and so design alternatives must take account of this. Other issues such as financial considerations can place a strain on this and so early communication with all relevant stakeholders is essential. Often higher initial costs are offset many times over by a reduction in post-construction maintenance costs when dealing with expansive soils (Nelson and Miller, 1992).

As with any foundation option, the main aim is to minimize the effects of movement, principally differential, and two strategies are used when dealing with expansive soils:

- Isolate structure from soil movements
- Design a foundation stiff enough to resist movements

The major types of foundations used in expansive soils from around the world are pier and beam or pile and beam systems, slab on ground, reinforced raft and modified continuous perimeter spread footing.

2.6.1. Slab on ground

Concrete slabs placed directly on the ground are much less expensive than structural floor slabs or “crawl space” type construction. These slabs are constructed either with or without reinforcement.

The unreinforced slabs are generally constructed in residential houses or where a light floor load is expected. The limits of the length of the unreinforced slab are based upon the amount of shrinkage cracking control desired. Whereas a lightly reinforced slab normally reinforced with temperature control as a prime design factor. The choice between the two types depends upon the subsoil conditions as well as the loading conditions.

Slab-on-ground construction on expansive soil will always pose a cracking and heaving problem unless the subgrade soils are treated or replaced. In the construction of building such as warehouse and storage areas, special design is required to maintain the structural integrity of the building. Slab-on-ground construction cannot be safely used in an area where the subsoil possesses high swell potential (Nelson and Miller, 1992).

2.6.2. Stiffened Rafts

The reinforced mat is often suitable for small and lightly loaded structures, particularly if the expansive or unstable soil extends nearly continuously from the ground surface to depths that exclude economical drilled shaft foundations. This mat is suitable for resisting subsoil heave from the wetting of deep desiccated soil, a changing water table, laterally discontinuous soil profiles, and downhill creep, which results from the combination of swelling soils and the presence of slopes greater than 5 degrees. A thick, reinforced mat is suitable for large, heavy structures (Alemayehu, 1992).

The rigidity of thick mats minimizes distortion of the superstructure from both horizontal and vertical movements of the foundation soil.

Concrete slabs without internal stiffening beams are much more susceptible to distortion or doming from heaving soil. Stiffening beams and the action of the attached superstructure with the mat as an indeterminate structure increase foundation stiffness and reduce differential movement. Edge stiffening beams beneath reinforced concrete slabs can also lessen soil moisture loss and reduce differential movement beneath the

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slab. However, the actual vertical soil pressures acting on stiffened slabs can become very nonuniform and cause localized consolidation of the foundation soil.

Type of Mat foundation stated below:-

1. **Uniform Mat Foundation/Solid**; - Mat is sometimes referred to as structural slab-on-ground. The slab should be designed to resist both the negative and positive moments as a result of structural loading and heaving of the soil (Chen, 1988). Normally mat foundations are reinforced both at the bottom and top (Alemayehu, 1992). This is because the exact contact pressure distribution under the mat is not known. It has, however, limitations as pointed out by previous researchers (Daniel, 2007).

2. **Ribbed Mat Foundation**; - is reinforced concrete mat with a grid of underlying cross beams supporting the slabs that have been successfully used as a foundation for even relatively loaded structures on expansive soils because heavy beams running in both directions to resist column loads and the soil pressure transferred to the beams from the mat. In ribbed foundation systems, the differential swell is the controlling factor governing the success of such construction. If the soil beneath the slab is to swell uniformity, no distortion would be caused in the slab. Distortion occurs when the supporting soil swells non-uniformly or differentially (Chen, 19880 and Daniel, 2007). Ribbed mats are frequently used in practice because they are economical than uniform mats.

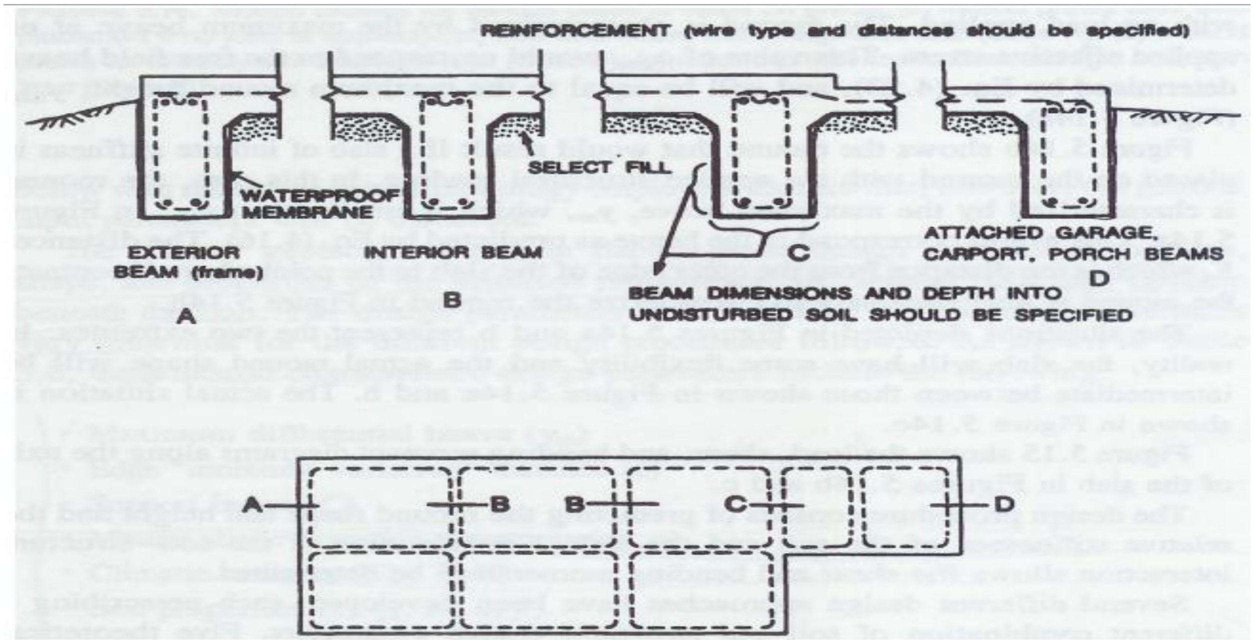

Figure 2. 4 Typical detail of a stiffened raft (Nelson and Miller, 1992)

2.6.3. Modified Continuous perimeter footing

Shallow footing should be avoided where expansive soils are found. However, where they are used a number of approaches can be employed to minimize the effects of swelling/shrinkage. Modifications include:

- Narrowing footing width
- Provide void spaces within the support beam/wall to concentrate loads at isolated points
- Increase perimeter reinforcement taking this into the floor slab stiffening foundations

The use of narrow spread footing in expansive soils should be restricted to soils exhibiting 1% swell potential and very low swell pressures (Nelson and Miller 1992).

NHBC (2011a) suggested that strip and trench fill foundations can be used when placed in a non-expansive layer that overlies expansive soils, provided:

- The soil is consistent across the site
- The depth of non-expansive material is greater than ¾ of the equivalent foundation depth assuming all soil is expansive (guidance provide within NHBC 2011a)
- The thickness of the non-expansive soil below they foundation at least equal to the foundation width

2.6.4. Micropile Technique to Control Heave on Expansive Soils

If the actual overburden or footing pressure is small compared to the estimated vertical swelling pressure, the resulting expansion will be greater than the tolerable value. In this case, micropile reinforcement can be used to compensate for the decreased overburden or footing pressure so that expansion is within tolerance.

The effectiveness of micropiles as a technique to control heave of a lightweight structure over expansive soils can be improved more by using micropiles surrounded with sand than micropile without surrounding sand. As the surface area of micropiles is rough, the reduction in heave is more. Beside that Percentage reduction in heave decreases with increase in number of micropiles.According to (Nusier.O.K and Alawneh. S. 2004, 2007 & 2009)

2.6.5. Using Styrofoam and Grade Beam Nosing Technique

In some expansive soils areas it is almost impossible to design a foundation which will not move. In other areas, because of the presence of a stable or moisture-satisfied material, a simple economical foundation can be designed. When excessive differential movement is anticipated, it becomes necessary to provide for distress relieving materials and features in the design of the structure. (W.P.Jobs, and W.R. Stroman, 1974).

Designing around large movements of the foundation by installing elaborate mechanisms to avoid footings, walls, and slabs crack and break, leading to out-of-level floors, racked doors and windows, and cracking of finished surfaces.

These mechanism are using of Styrofoam between the expansion joints of grade beam and ground floor slab allows the soil freely moves impede crack on the structure.

In addition to that replacing masonry wall by grade beam nosing is effectively minimizing moment reversals caused by differential foundation movement and increasing rigidity of the structure, these has been done extending development reinforcement of grade beam with maximum embedment depth of one meter by 10cm thickness of concrete. (http://www.epsindustry.org/building-construction/below-grade-foundation).

2.7. Review of previous researches

Morin and Perry (1971) studied the origin and mineralogical composition of Ethiopian red clay soils. According to their study, Ethiopian red clay soils are principally residual, derived from the weathering of volcanic rocks. The parent rock for black and red clays in Ethiopia is mainly olivine basalt, basalt, and trachyte. Ethiopian red clay soils have developed where rainfall is plentiful and drainage is good and contains Kaolinite and Halloysite as the principal clay minerals, but Montmorillonite is also frequently present in significant amounts. The red color of the Ethiopian soils indicates the presence of iron.

Samuel (1989) has studied about the investigation into some of the engineering properties of Addis Ababa red clay soils. Based on experimental results from 13 samples around Kolfe, Rufael and Semen Gebeya areas he found out the depth of red clay soil in Addis Ababa ranges from few centimeters to about 10 meters. In the area covered by the study, however, the thickness of the soil is found to be one and a half meter in Semen Gebeya and Rufael areas and more than 3m in Kolfe area and finally index property test result indicates the soil is not potentially expansive.

Mesfin (2004) has studied the investigation on index properties of expansive soils of Ethiopia. Based on experimental results from 125 samples shows high clay content, high to extremely high plasticity ranges. From the test result, the expansive soil of Ethiopia is classified as to extremely high swelling potential. Hence, these soils are unsuitable as construction material and should be considered as problematic foundation soils.

Ayenew (2004) has studied the investigation into shear strength characteristics of expansive soil of Ethiopia. Based on experimental results the shear strength of expansive soil ranges from 30-150kPa in cohesion and 3-25 degree in angle of internal friction in UU test on unsaturated soil. For saturated soil sample in the UU test, the cohesion ranges from 55-94kPa. There is a decrease in strength in saturated samples, which shows that the degree of saturation and the suction pressure can have a major influence on the shear strength of expansive soil.

Hailemariam (2016) guideline for the selection of safe and economical types of reinforced concrete foundation for different buildings constructed on expansive soil of Addis Ababa. According to he developed simple software using programming language, MATLAB 2011, and visual basic ultimate studio 2015 for handling the above issues by applying seven main reinforced concrete foundations such as footing foundations, slab foundations such as slab-on-ground, uniform mat and ribbed mat foundations, straight bored pile, single and double-under-reamed pile foundation for different buildings ranging from one story to sixteen stories under four grouping. The expected input for this software is superstructure loads, swelling pressure, allowable/safe bearing capacity (based on UCS values and N-values from SPT readings), depth of moisture fluctuation and foundation determining factors like aspect ratio for dimensional factor and aspect ratio of load factor.

CHAPTER THREE - DESCRIPTION OF STUDY AREA

3.1. Topography of the area

Legetafo is a one special zone of Oromia regional state to administer the eight small towns around the territories of the Addis Ababa city administration which located at elevation of 2388 meter above sea level, and It coordinates are 9^0 4′0′′ N and 38^0 52′60″ E. However, most of the areas stretch over a predominantly flat land with imperceptible slope changes.

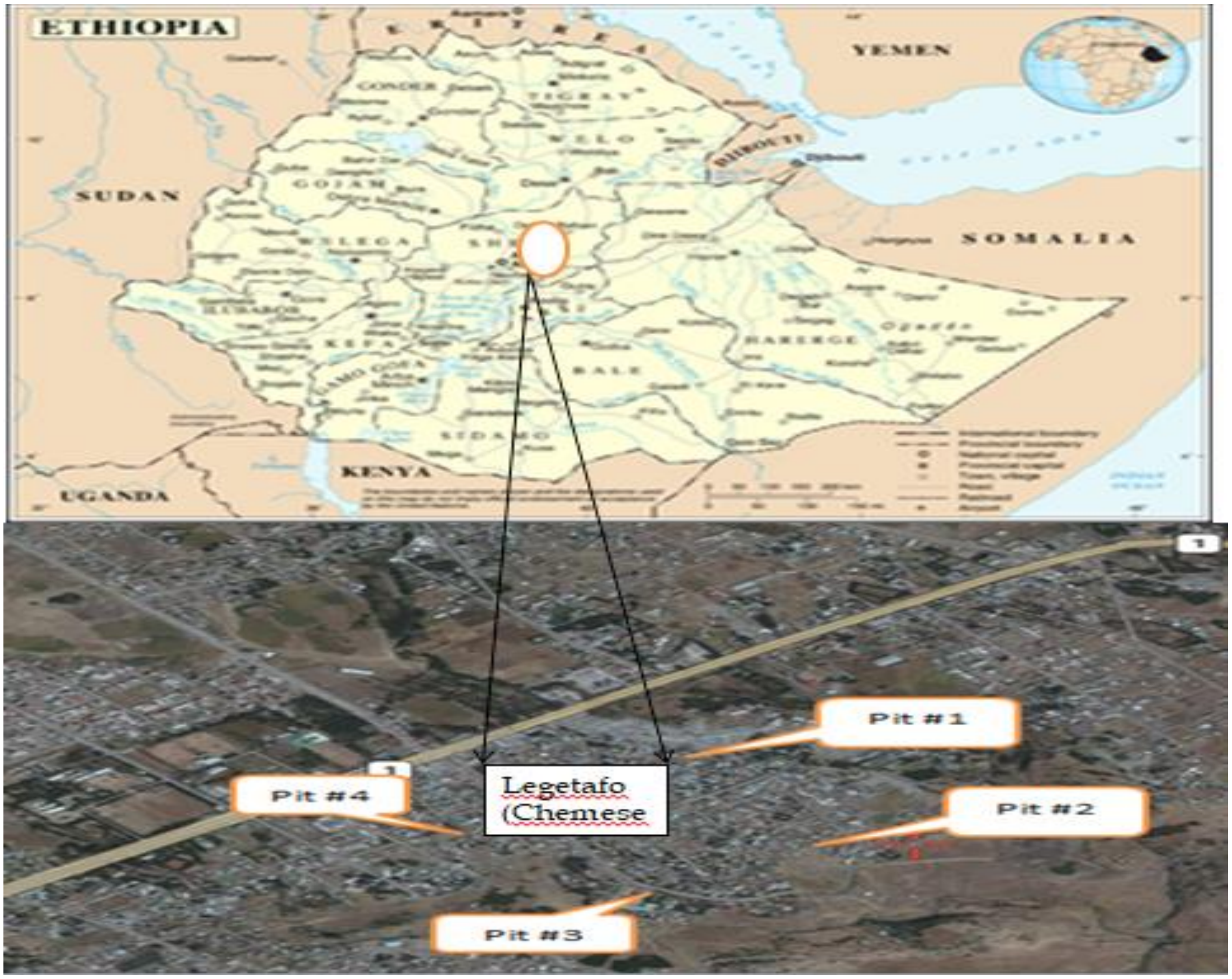

Figure 3. 1 Location of test pit google map to figure

3.2. Meteorological information

3.2.1. Rainfall

Since the project area is located near Addis Ababa city, the rainfall of the city is considered for the project area. Accordingly, the mean annual rainfall of the project area varies in the range of 918mm - 1567mm, and the rainy period is between June through September, although the occasional shower is expected in the months of March, April, May, October, and November. Maximum rainfall is recorded during the months July and August amounting 250mm and 280mm respectively for these months and minimum rainfall is recorded during November and December amounting 8mm and 9mm respectively.

3.2.2. Temperature

The maximum temperature of Legetafo ranges between 20^0c (in wet season) to 25^0c (in the dry season), while the minimum falls between 7 –12^0 c in the year. This indicates that daily variation of temperature is highly pronounced.

3.2.3. Native Surface Soil Type

Before commencement of the field soil sampling, visual surface soil extension survey has been carried out to determine the type and extent of the surrounding materials around the project. According to the soil survey, the predominant soil types around the project are Dark (black) clay soil.

The details of natural sub-grade soil extension around the project are given in on the

Following picture.

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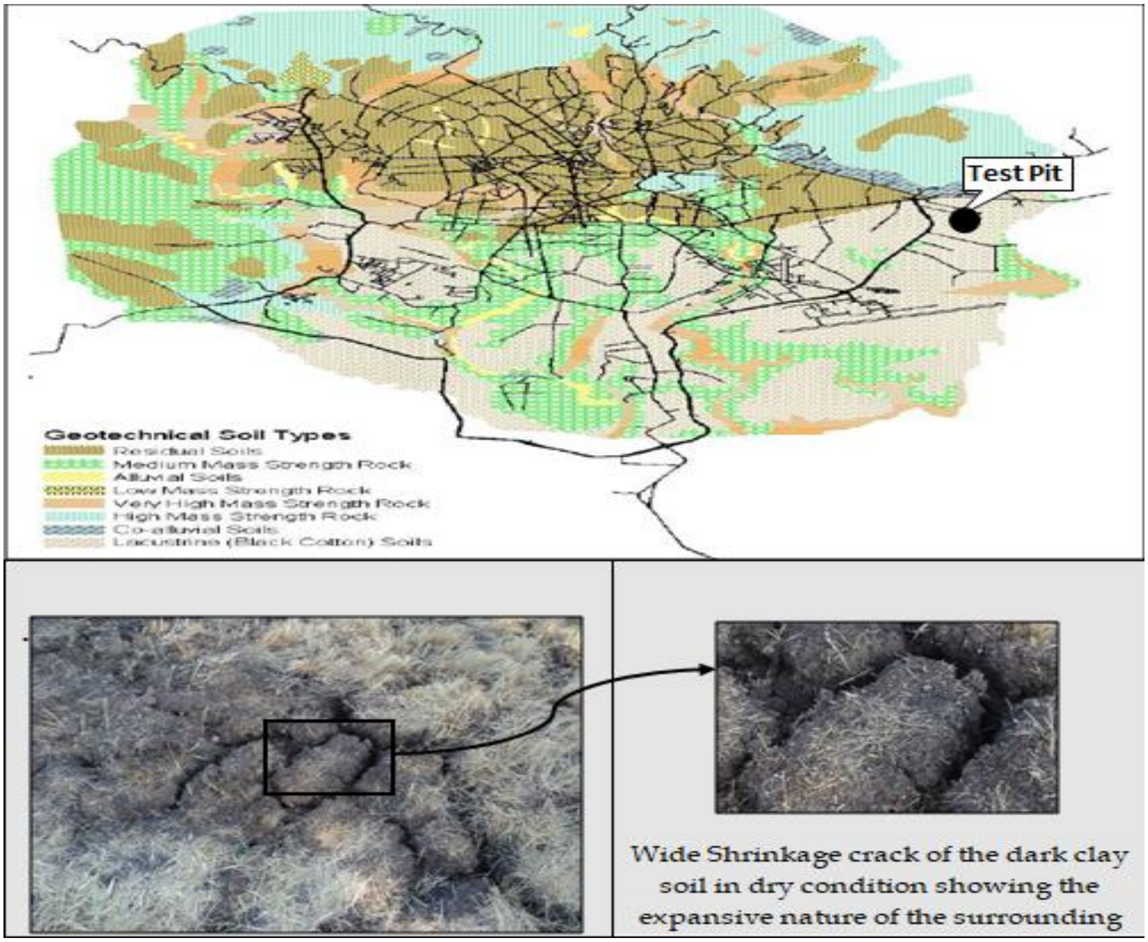

Figure 3. 2: Surface soil type of Legetafo City

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CHAPTER FOUR- LABORATORY TEST RESULTS

4.1. In-situ properties

In order to come up with good soil exploration results and to be economical, the field exploration program should be arranged wisely the actual planning of a subsurface exploration includes the assembly of the available information, reconnaissance of the area, preliminary site investigation and the last was detailed site investigations, in this case, all of them were done.

After preliminary reconnaissance survey was done, Test pits were excavated for detail geotechnical investigation, i.e. sampling of representative foundation soil and geotechnical test pit logging are performed. The exact positions of the test pits and elevation above sea level have been taken using hand GPS. Photographs have also been taken. And primarily, I took both disturbed and undisturbed samples from each depth of the test pit, 1.5m, and 2.0m.

Table 4. 1 coordinate of sample location

Test Pit No	Easting	Northing	Elevation
TP-1	0489718	1003634	2460
TP-2	0489725	1003681	2460
TP-3	0489711	1003689	2460
TP-4	0489748	1003648	2460

4.2. Index properties

Index property is a property, which helps in distinguishing the characteristics of a soil it comprises. Atterberg limit test, grain size analysis, specific gravity, and free swell tests are among the tests which show the index property of soil.

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4.2.1. Results for the Atterberg Limit Test

The results of the Atterberg limit test were given in Table 4.2. ASTM D-4318 is followed for the testing procedure. The detailed Atterberg limit test results are attached in Appendix A.

Table 4. 2: Atterberg Limit Test Result of the Study Area

Test Pit No	Depth	Liquid limit (%)	Plastic limit (%)	PI (%)
TP-1	1.5m	123	30	93
	2.0m	130	31	99
TP-2	1.5m	115	30	85
	2.0m	118	28	90
TP-3	1.5m	121	30	91
	2.0m	119	31	88
TP-4	1.5m	124	29	95
	2.0m	127	33	94

4.2.2. Results for the Liquidity Index Test

Liquidity Index (LI): The index that is used to indicate the consistency of undisturbed soils is called as the liquidity index or water plasticity ratio. The liquidity index is expressed as $LI = \frac{w-PL}{PI}$

Table 4. 3: Liquidity Test Result of the Study Area

Test Pit No	Depth	ω(%)	Plastic limit (%)	PI (%)	LI	Remark
TP-1	1.5m	39.29	30	93	0.103	Plastic state
	2.0m	44.59	31	99	0.137	Plastic state
TP-2	1.5m	39.94	30	85	0.117	Plastic state
	2.0m	42.03	28	90	0.156	Plastic state
TP-3	1.5m	46.16	30	91	0.178	Plastic state
	2.0m	47.08	31	88	0.183	Plastic state
TP-4	1.5m	43.68	29	95	0.155	Plastic state
	2.0m	44.25	33	94	0.120	Plastic state

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4.2.3. Results for the Specific Gravity Test

The specific gravity of soil is the ratio of the unit weight of solids in the soil to the unit weight of water. The test results of the specific gravity of the study area were summarized in Table 4.4.

The detailed Specific gravity test results are attached in Appendix A.

Table 4. 4: Specific Gravity of the Soil of the Study Area

Test Pit No	Depth	Gs
TP-1	1.5m	2.72
	2.0m	2.71
TP-2	1.5m	2.74
	2.0m	2.75
TP-3	1.5m	2.79
	2.0m	2.81
TP-4	1.5m	2.76
	2.0m	2.73

4.2.4. Results for the Grain Size Analysis Test

For a basic understanding of the nature of the soil, the distribution of the grain size present in a given soil mass must be known. Grain size Analysis test is used to determine the percentage of different grain sizes contained within a soil. The mechanical or sieve analysis is performed to determine the distribution of the coarser, larger-sized particles. The procedure followed to run this test is according to ASTM standard with designations D422-63 and D1140-97.

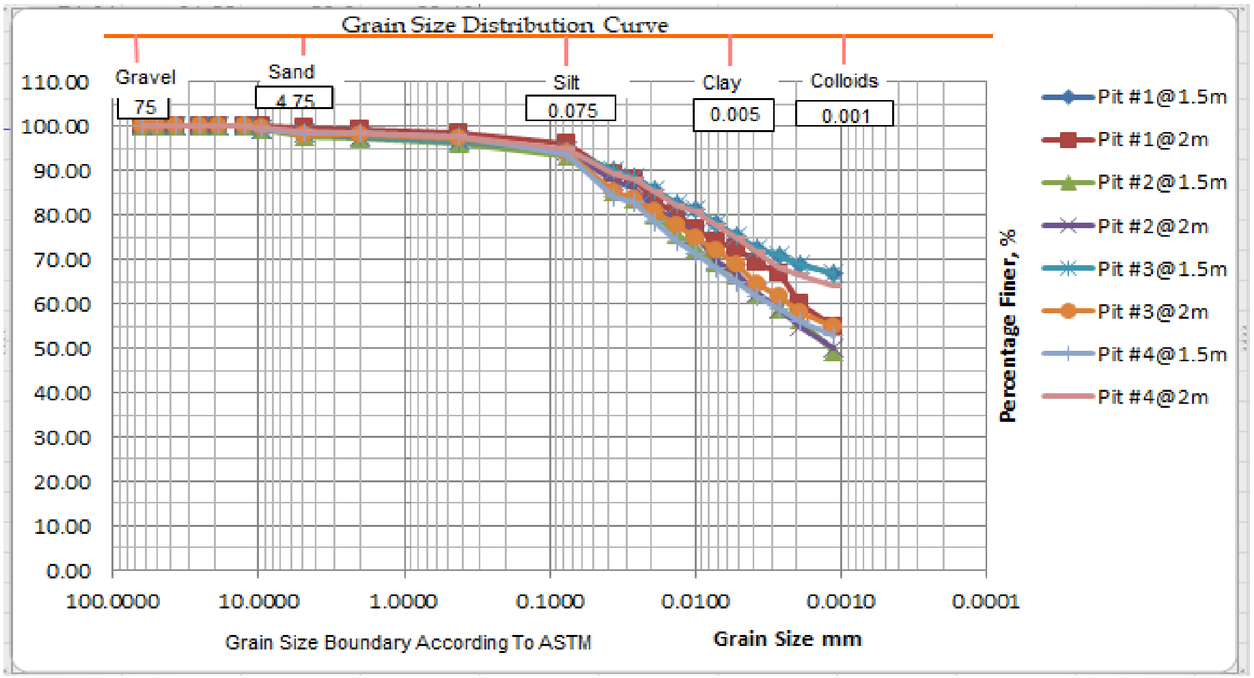

Figure 4. 1: sieve-analysis graph of the study area

Table 4. 5: Grain Size Distribution based on the unified soil classification system of the Study Area

Test Pit No	Depth	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
TP-1	1.5m	0.17	5.78	21.71	72.34
	2.0m	0.27	3.48	19.90	76.35
TP-2	1.5m	2.14	4.62	21.29	71.95
	2.0m	1.68	5.44	18.42	74.46
TP-3	1.5m	1.71	3.87	18.47	75.95
	2.0m	1.76	4.01	18.80	75.43
TP-4	1.5m	1.44	4.78	20.16	73.62
	2.0m	1.26	3.79	17.51	77.44

4.2.5. Free Swell Test Results

The free swell test is one of the most commonly used simple tests for estimating soil swelling potential. Results of the free swell tests of the study area were given in Table 4.5. The detailed free swell test results are attached in Appendix A.

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Table 4. 6: Free swell of the Study Area

Test Pit No	Depth	Free Swell (%)
TP-1	1.5m	97.50
	2.0m	103.75
TP-2	1.5m	108.33
	2.0m	115.42
TP-3	1.5m	111.25
	2.0m	117.50
TP-4	1.5m	109.58
	2.0m	115.83

The soil of the studies area can be classified as highly expansive and the swelling values are greater than 100%

4.2.6. Classification of the Soils

4.2.6.1. Classification of soils based on Unified soil classification (USC) system

This system describes a system for classifying minerals and organic-mineral soils for engineering purposes based on laboratory determination of particle-size characteristics, liquid limit, and plasticity index.

Table 4. 7 :Classifications of soils based on USC Classification system

Test Pit No	Depth	LL (%)	PL (%)	PI (%)	Gravel (%)	Sand (%)	Fine (%)	USC	Remark
TP-1	1.5m	123	30	93	0.17	5.78	94.05	CH	Clay with high plasticity
	2.0m	130	31	99	0.27	3.48	96.25	CH	Clay with high plasticity
TP-2	1.5m	115	30	85	2.14	4.62	93.24	CH	Clay with high plasticity
	2.0m	118	28	90	1.68	5.44	92.88	CH	Clay with high plasticity
TP-3	1.5m	121	30	91	1.71	3.87	94.42	CH	Clay with high plasticity
	2.0m	119	31	88	1.76	4.01	94.23	CH	Clay with high plasticity
TP-4	1.5m	124	29	95	1.44	4.78	93.78	CH	Clay with high plasticity
	2.0m	127	33	94	1.26	3.79	94.95	CH	Clay with high plasticity

4.2.6.2. AASHTO classification system

These classification systems are used Grain size distribution, liquid limit, and plasticity index for classification of soil particles. Even though this classification method is mostly used for road subgrade soils, it is applied here for comparison purpose.

Table 4. 8: Classifications of soils based on AASHTO Classification system

Test Pit No	Depth	LL (%)	PI (%)	Percentage passing sieves #200	AASHTO Classification system	Remark
TP-1	1.5m	123	93	94.05	A-7-5	Clayey soil
	2.0m	130	99	96.25	A-7-5	Clayey soil
TP-2	1.5m	115	85	93.24	A-7-5	Clayey soil
	2.0m	118	90	92.88	A-7-5	Clayey soil
TP-3	1.5m	121	91	94.42	A-7-5	Clayey soil
	2.0m	119	88	94.23	A-7-5	Clayey soil
TP-4	1.5m	124	95	93.78	A-7-5	Clayey soil
	2.0m	127	94	94.95	A-7-5	Clayey soil

4.2.6.3. Classification based on activity

Skempton (1953) showed that for soils with particular mineralogy, the plasticity indexes linearly related to the amount of the clay fraction.

Table 4. 9: Activity Range

S/N	Activity	Soil type
1	<0.75	In active
2	0.75-1.25	Normal
3	>1.25	Active

$$Activity, A = \frac{Plasticity\ Index, PI}{Percent\ finer\ than\ 2\ micron}$$

Table 4. 10: Classification of clay layers based on Activity

Test Pit No	Depth	PI (%)	Percent finer than 2 micro	Activity A	Soil type
TP-1	1.5m	93	57.88	1.60	Active Clay
	2.0m	99	64.96	1.52	Active Clay
TP-2	1.5m	85	64.37	1.32	Active Clay
	2.0m	90	66.29	1.35	Active Clay
TP-3	1.5m	91	52.83	1.72	Active Clay
	2.0m	88	61.12	1.43	Active Clay
TP-4	1.5m	95	64.38	1.47	Active Clay
	2.0m	94	53.67	1.75	Active Clay

4.3. Result of Unconfined compressive strength

The test is carried out only on the saturated sample which can stand without any lateral support. This test is applicable to cohesive soils only and one of the simplest and quickest tests used for the determination of shear strength of cohesive soils.

Table 4. 11: unconfined compressive strength of study area

Test Pit No	Depth	Unconfined compressive strength (UCS KN/m^2)	Classification based Consistency
TP-1	2.0m	99	Stiff
TP-2	2.0m	89	Medium
TP-3	2.0m	167	Stiff
TP-4	2.0m	175	Stiff

4.4. Result Consolidation and Swelling pressure

The main purpose of the consolidation test on soil samples is to obtain the necessary information about the compressibility properties of saturated soil for use in determining the magnitude and rate of settlement of structures.

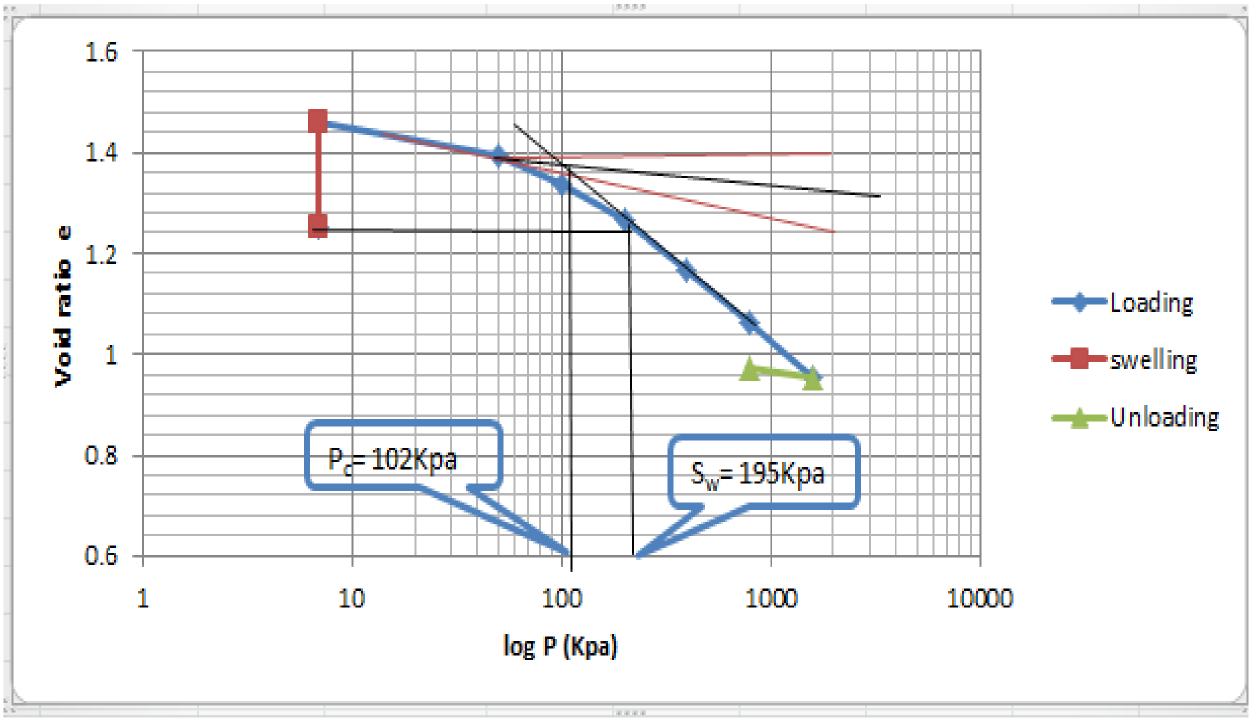

Figure 4. 2 Void ratio Vs. log p curve test pit 1 @2m

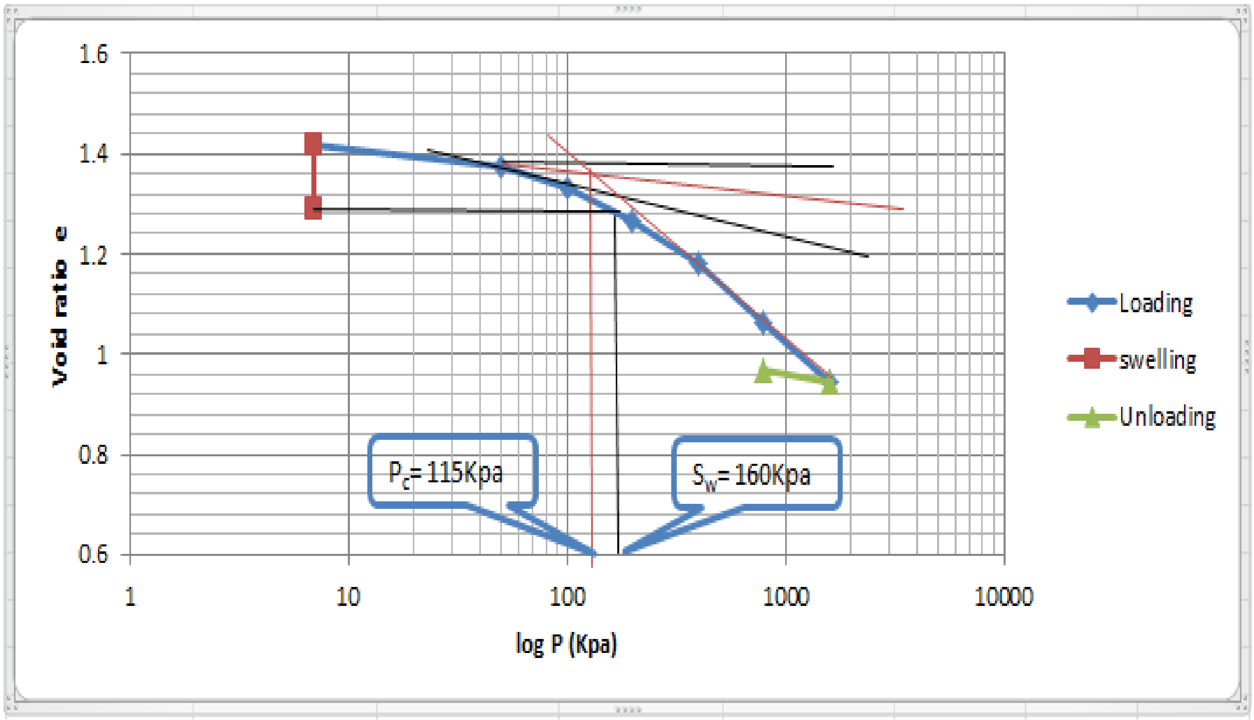

Figure 4. 3 Void ratio Vs. log p curve test pit #2 @2m

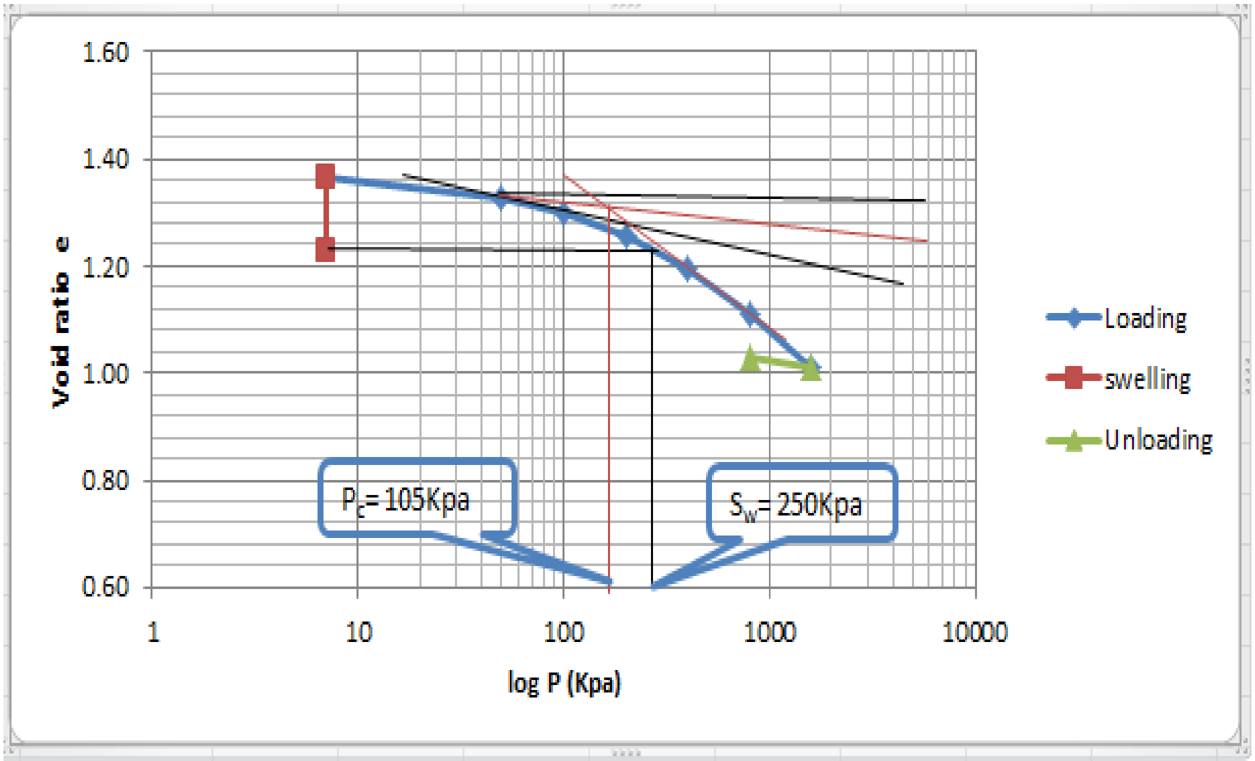

Figure 4. 4 Void ratio Vs. log p curve test pit #3 @2m

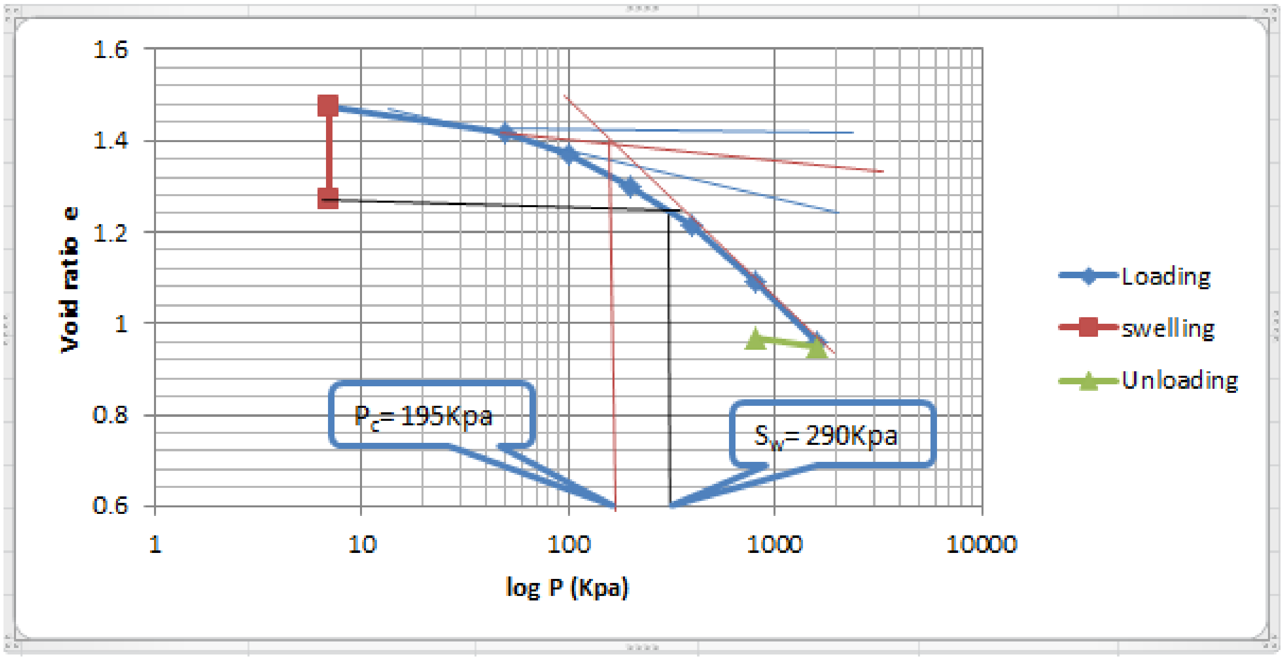

Figure 4. 5 Void ratio Vs. log p curve test pit #4 @2m

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4.4.1. Swelling pressure

The swelling pressure is defined as the vertical pressure required preventing volume change of laterally confined sample when it is allowed to take in water. Swelling pressures of soils of the study area are determined from the swell- consolidation test.

Table 4. 12: Pre consolidation and swelling pressure test results of soils of the study area

Test Pit No	Depth	Total unit weight (kN/m^3)	Overburden pressure (kPa)	Compression Index	Preconsolidation pressure (kPa)	Swelling pressure (kPa)	Over Consolidation Ratio
TP-1	2.0m	17.89	34.04	0.35	102	195	2.99
TP-2	2.0m	18.09	36.16	0.39	115	160	3.18
TP-3	2.0m	18.86	36.96	0.33	105	250	2.84
TP-4	2.0m	17.98	35.90	0.44	110	310	3.06

4.4.2. Result of Coefficient of consolidation

The coefficient of consolidation Cv can be evaluated by means of laboratory tests by fitting the experimental curve with the theoretical.The Calculation given appendix E.

Table 4. 13: Consolidation of soils of the study area

Test Pit No	Depth	Pressure (kPa)	Void Ratio	Coefficient of Consolidation, C_v ($10^{-3}cm^2/sec$)
TP-1	2.0m	50	1.1757	0.0510
		100	1.1616	0.0308
		200	1.1426	0.0236
		400	1.1128	0.0152
		800	1.0736	0.0147
		1600	1.0236	0.0072
TP-1	2.0m	50	1.1777	0.0155
		100	1.1587	0.0985
		200	1.1273	0.0620
		400	1.0913	0.0303
		800	1.0489	0.0146
		1600	1.0037	0.0073

`

4.5. Summary of Test Result

1. Geotechnical test show that the soil found in legetafo town black or gray expansive soil having Grain size analysis tests revealed that, starting from few centimeters below the ground level to the depth of investigation which is two meters, the soil in legetafo town is mostly clay and silt soil in which the percentage of clay ranges from 71.95 to 77.44%, silt from 17.51 to 21.71%, and from 3.48 to 5.78% and gravel 0.17 to 2.14%.

2. The specific gravity of legetafo soil ranges from 2.71 to 2.81

3. The samples show a free swell value 97.50% to 115.03%. This show in the studies area ranges from marginal to high expansive soils.

4. From consistency limit test results the liquid limit of the area ranges from 115 to 127%, plastic limit ranges from 28 to 33% and plastic index from 85 to 99%.

5. The unconfined compression strength test conducted for selected undisturbed samples of soils for legetafo is between medium to the stiff state of consistency i.e. 89 to 175 kPa.

6. As determined from the one-dimensional consolidation test conducted from undisturbed soil samples, compression index ranging from 0.33 to 0.44, the coefficient of consolidation Cc, ranges from 0.0072 to 0.0985, and measured swelling pressure range from 160 to 310kPa.

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Table 4. 14: Summary of the test result

Sample No	Depth In meter	Moisture Content	Percentage amount of particle size (%)				Specific gravity	Free Swell	Atterberg Limits			Degree Activity	USC soil classification	UCS q_u (kPa)	Swelling Pressure (kPa)	Pc kPa
			Grave	san d	silt	clay			LL	PL	PI	Active				
Pit 1	1.5	39.29	0.17	5.78	21.71	72.34	2.72	97.50	123	30	93	Active	CH	-		
	2.0	44.59	0.27	3.48	19.90	76.35	2.71	103.75	130	31	99	Active	CH	99	195	102
Pit 2	1.5	39.94	2.14	4.62	21.29	71.95	2.74	108.33	115	30	85	Active	CH	-		
	2.0	42.03	1.68	5.44	18.42	74.46	2.75	115.42	118	28	90	Active	CH	89	160	115
Pit 3	1.5	46.16	1.71	3.87	18.47	75.95	2.79	111.25	121	30	91	Active	CH	-		
	2.0	47.08	1.76	4.01	18.80	75.43	2.81	117.50	119	31	88	Active	CH	167	250	105
Pit 4	1.5	43.68	1.44	4.78	20.16	73.62	2.76	109.58	124	29	95	Active	CH	-		
	2.0	44.25	1.26	3.79	17.51	77.44	2.73	115.83	127	33	94	Active	CH	175	310	110

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CHAPTER FIVE - DESIGN GENERAL ASPECT AND FOUNDATION RECOMMENDATION

5.1. Introduction

An appropriate foundation should economically contribute to satisfying the functional requirements of the structure and minimize differential movement of the various parts of the structure that could cause damages. The foundation should be designed to transmit no more distortion to the superstructure. The amount of distortion that can be reduced depends on the design and purpose of the structure constructed in expansive soil (Nelson and Miller 1992; Lytton 1994).

5.2. Determination of important foundation parameters

Since structural solutions are the most effective ways of minimizing or eliminating heave damages of the structures due to swelling characteristics of the soil underneath the foundations, these solutions start with the determination of some important parameters from soil test in the laboratory or in the field. These are cohesion (c), the angle of internal friction (ϕ), depth of groundwater table, swelling potential of the soil, unit weight of the soil (γ) and N-values of standard penetration test (SPT) value (Alemayehu & Lekikun , 1997). These values, in combination with the reaction from the super-structural load and certainly given a factor of safety, are used to determine the safe/allowable bearing capacity of the soil for both shallow and deep foundations. The bearing capacity determination can be done both for initial and final conditions. But in the case of my studies area, it is dominate land use are real state and other small commercial building. Due to this reason difficult to use structural solution with regarding to increasing the weight of the building to minimize distortion effect expansive soil. But another cost-effective solution must provide to counterbalance the problem arise from heaving.

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Table 5.1 Allowable soil pressure [EBCS-7]

Supporting ground type	Description	Compactness* or Consistency**	Presumed Bearing Value (kPa)	Remarks
Rocks	Massively crystalline igneous and metamorphic rock (granite, basalt, gneiss)	Hard and sound	5600	
	Foliated metamorphic rock (salt, schist)	Medium hard and sound	2800	
	Sedimentary rock (hard shale, siltstone, sandstone, limestone)	Medium hard and sound	2800	
	Weathered or broken-rock (soft limestone)	Soft	1400	
	Soft shale	Soft	850	
	Decomposed rock, to be assessed as soil as below			
Non-cohesive Soils	Gravel, sand and gravel	Dense Medium dense Loose	560 420 280	Width of foundation, b, not less than 1 m
	Sand	Dense Medium dense Loose	420 280 140	Ground water level assumed to be at depth not less than b, below the base of the foundation
Cohesive Soils	Silt	Hard Stiff Medium stiff Soft	280 200 140 70	
	Clay	Hard Stiff Medium stiff Soft	420 280 140 70	

* Compactness: Dense: $N > 30$
Medium dense: $N = 10$ to 30
Loose: $N < 10$
N = Standard Penetration Test Value

** Consistency: Hard: $q_u > 400$ kPa
Stiff: $q_u = 100$ to 200 kPa
Medium stiff: $q_u = 50$ to100 kPa
Soft: $q_u = 15$ to 50 kPa

$q_u =$ unconfined compressive strength

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The value of in Table 5.2 is obtained from laboratory test result in chapter three and calculation unit weight given Appendix A.

Table 5.2 : bearing capacity parameter

Test pit	UCS (q_u) kN/m²	Consistency	Unit weight of soil (KN/m²)
Pit #1	99	stiff	17.89
Pit #2	89	Medium	18.09
Pit #3	167	stiff	18.68
Pit #4	175	stiff	17.98

5.3. Bearing Capacity

Unconfined compression tests were conducted to derive the undrained shear strength value, Cu of the soil. Angle of internal friction, Ø, is taken as zero.

The UCS values for 2 depth are:

- q_u = 99kPa
- C_u = 49.5 kPa
- Ø = 0
- Υ_{sat} = 17.98kPa , Υ' = 7.98kPa

The net ultimate bearing pressure for vertical loads on clay soils is normally computed as a simplification of either the Meyerhof or Hansen equations as follows:

$$q_{ult} = CN_cS_cd_ci_c + \Upsilon D^*Nq^*S_qd_qi_q + 0.5B\Upsilon N_\Upsilon S_\Upsilon i_\Upsilon d_\Upsilon$$

`

Table 5.3: Ultimate Bearing Capacity Calculation

Shape factors	Depth factors	Inclination factors
$S_c=1+0.2k_pB/L$	$d_c=1+0.2(sqrt(k_p))D/B$	$i_c=i_q=1-\alpha/90^\circ$
(i) For $\phi=0^\circ$ **$s_q=s_\Upsilon=1.0$**	(i) For $\phi=0^\circ$ $d_q=d_\Upsilon=1.0$	$i_\Upsilon=(1-\alpha/\phi)^2$
(i) For $\phi\geq10^\circ$ **$s_q=s_\Upsilon=1+0.1k_pB/L$**	(ii) For $\phi\geq 10^\circ$ $d_q=d_\Upsilon=1+0.1(sqrt(k_p))D/B$	α= angle of resultant measured from vertical axis. $K_p=\tan^2(45^\circ+\phi/2)$

For vertical loads: $i_c=i_q=1$ $\quad i_\Upsilon=0$

In undrained condition: $\varnothing = 0$

$$N_c = 5.14 \text{ and } N_q = 1.0.\ N_\Upsilon=0 \qquad Kp = 1$$

$$Sc = 1.2\ ,\ Sq = S_\Upsilon=1,\ dq= d_\Upsilon = d_c=1,\ i_\Upsilon=0$$

When designing a foundation on the basis of ultimate bearing capacity, a suitable factor of safety should be used to determine the allowable pressure so that the foundation system is safe against shear failure. For footing foundations, a factor of safety of 2 to 3 is commonly used under normal loading conditions. Thus, for our condition, we have taken a factor of safety of 2 and assumed square footing geometry.

The ultimate and allowable bearing pressure for different widths of square footing can be calculated as shown in table 5.3.

`

Table 5.4: Bearing capacity of assumed footing for test pit #1

Bearing capacity of proposed footing For test pit #1						**$i_c = i_q$ 1 , $i_\Upsilon = 0$**			**Υ_{sat} = 17.89kPa, Υ' = 7.89Kpa**			
Footing depth	Footing width	Kp	C_u	Sc	d_c	N_c	N_q	N_Υ	$S_q=S_\Upsilon$	$d_q=d_\Upsilon$	Q_u	$Q_{u\ all}$
2.0m	1.5m	1	49.5	1.2	1.267	5.14	1	0	1	1	402.53	201.27
2.0m	2.0m	1	49.5	1.2	1.2	5.14	1	0	1	1	382.18	191.09
2.0m	2.5m	1	49.5	1.2	1.16	5.14	1	0	1	1	369.97	184.98
2.0m	3.0m	1	49.5	1.2	1.133	5.14	1	0	1	1	361.82	180.91
2.0m	3.5m	1	49.5	1.2	1.114	5.14	1	0	1	1	356.01	178.00
2.0m	4.0m	1	49.5	1.2	1.1	5.14	1	0	1	1	351.65	175.82

Table 5.5 : Bearing capacity of assumed footing for test pit #2

Bearing capacity of proposed footing for test pit #2						**$i_c = i_q$ 1 , $i_\Upsilon = 0$**			**Υsat = 18.09kPa, Υ' = 8.09Kpa**			
Footing depth	Footing width	K_p	C_u	Sc	d_c	N_c	N_q	N_Υ	$S_q=S_\Upsilon$	$d_q=d_\Upsilon$	Q_u	$Q_{u\ all}$
2.0m	1.5m	1	44.5	1.2	1.267	5.14	1	0	1	1	363.83	181.92
2.0m	2.0m	1	44.5	1.2	1.2	5.14	1	0	1	1	345.54	172.77
2.0m	2.5m	1	44.5	1.2	1.16	5.14	1	0	1	1	334.56	167.28
2.0m	3.0m	1	44.5	1.2	1.133	5.14	1	0	1	1	327.24	163.62
2.0m	3.5m	1	44.5	1.2	1.114	5.14	1	0	1	1	322.01	161.00
2.0m	4.0m	1	44.5	1.2	1.1	5.14	1	0	1	1	318.09	159.04

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Table 5.6: Bearing capacity of assumed footing for test pit #3

Bearing capacity of proposed footing for test pit #3						**$i_c = i_q$ 1 , $i_\Upsilon = 0$**			**ϒsat = 18.86kPa, ϒ' = 8.86Kpa**			
Footing depth	Footing width	Kp	C_u	Sc	d_c	N_c	N_q	N_Υ	$S_q=S_\Upsilon$	$d_q=d_\Upsilon$	Q_u	$Q_{u\ all}$
2.0m	1.5m	1	83.5	1.2	1.267	5.14	1	0	1	1	669.32	334.66
2.0m	2.0m	1	83.5	1.2	1.2	5.14	1	0	1	1	634.99	317.49
2.0m	2.5m	1	83.5	1.2	1.16	5.14	1	0	1	1	614.39	307.19
2.0m	3.0m	1	83.5	1.2	1.133	5.14	1	0	1	1	600.65	300.33
2.0m	3.5m	1	83.5	1.2	1.114	5.14	1	0	1	1	590.84	295.42
2.0m	4.0m	1	83.5	1.2	1.1	5.14	1	0	1	1	583.48	291.74

Table 5.7: Bearing capacity of assumed footing for test pit #4

Bearing capacity of proposed footing for test pit #4						**$i_c = i_q$ 1 , $i_\Upsilon = 0$**			**ϒsat = 17.98kPa, ϒ' = 7.98Kpa**			
Footing depth	Footing width	Kp	C_u	Sc	d_c	N_c	N_q	N_Υ	$S_q=S_\Upsilon$	$d_q=d_\Upsilon$	Q_u	$Q_{u\ all}$
2.0m	1.5m	1	87.5	1.2	1.267	5.14	1	0	1	1	858.62	429.31
2.0m	2.0m	1	87.5	1.2	1.2	5.14	1	0	1	1	822.64	411.32
2.0m	2.5m	1	87.5	1.2	1.16	5.14	1	0	1	1	801.05	400.53
2.0m	3.0m	1	87.5	1.2	1.133	5.14	1	0	1	1	786.66	393.33
2.0m	3.5m	1	87.5	1.2	1.114	5.14	1	0	1	1	776.38	388.19
2.0m	4.0m	1	87.5	1.2	1.1	5.14	1	0	1	1	768.67	384.34

From the above analysis, one can see that the average allowable bearing pressure of grayish Black, high plastic, clay layer determined from UC test results vary from 175.82kPa to 201.27 kPa for test pit# 1, 159.04kPa to 181.9 kPa for test pit #2 ,291.7kPa to

334.66kPa for test pit#3 and 429.31kPa to 384.34 kPa for test pit#4 below the lowest ground level depending on the foundation width.

In selecting the type of foundation, the following should be considered.

1. Obtain the required information concerning the nature of the superstructure and the loads to be transmitted to the foundation with respect to bearing capacity and settlement

2. Obtain the subsurface soil conditions.

(i) Explore the possibility of constructing any one of the types of foundation under the existing conditions by taking into account

- The bearing capacity of the soil to carry the required load, and
- The adverse effects on the structure due to differential settlements.

(ii) Once one or two types of foundation are selected on the basis of preliminary studies, make more detailed studies. These studies may require more accurate determination of loads, subsurface conditions and footing sizes. It may also be necessary to make more refined estimates of settlement in order to predict the behavior of the structure.

(iii) Estimate the cost of each of the promising types of foundation, and choose the type that represents the most acceptable compromise between performance and cost.

5.4. Settlement Analysis

The settlement is another criterion for evaluating the performance of a building. Excessive settlements will result in poor performance of the building structure. Different building codes set the limiting settlement for the type of structure and foundation. The foundation is expected to meet this criterion.

Different kinds of settlements occur under a building structure in which the major ones are immediate and consolidation settlements. These settlements depend on a number of parameters as well as the type of soils. The consolidation settlement takes place in cohesive soils. The layer selected as a bearing layer is clay. Therefore, settlement analysis should be conducted in addition to the shear failure criteria to determine the safe bearing pressure.

Settlement check for the project is done by considering the dominant CLAY soil layer beneath the foundation. For the soil type under consideration, the major part of the settlement is contributed by immediate and consolidation settlement. Therefore, the calculation of consolidation settlement is presented below.

Consolidation test was conducted on remolded sample retrieved from test pit at depth of 2.0m. We have the following data through the analysis of consolidation test at 100% degree of saturation.

The following formula is used to calculate the settlement.

$$S = \frac{C_r}{1 + e_o} * H * \log \frac{P_o + \Delta P}{P_o}$$

Where

- Cc = Compression index from e vs. log p plot
- Cr = Recompression index from e vs. log p plot

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- e_o = Average in-situ void ratio in the stratum
- e_c = Void ratio at pre-consolidation pressure
- H = Thickness of stratum
- Pc = Pre-consolidation pressure
- Po = Effective overburden pressure at mid-height of clay

  $Po = \gamma * H_1 = 17.89 * 1.0 = 17.89$ kPa
- Δp = The increased stress at the middle of the consolidation layer . $\Delta p = \frac{q_s B}{(B+1.5)^2}$,

  Where q_s = foundation stress at the surface of footing, B= width of the footing

From settlement criteria, an allowable settlement for isolated footing is 50mm the maximum load from structure load for 50mm settlement is 730KPa loads. But in this paperwork only lightweight structure having 50Kpa to 80kpa load from structure so in particular case I take 80kpa load for analysis of settlement.

We assumed the soil stratum under the foundation depth, i.e less than 2m to be clay and the depth of the clay layer to be 2m.

Table 5.8: Settlement analysis for test pit #1

Pit # 1 The value surcharge Load 80kpa for L=B									
Υ_{sat} (KN/m²)	Precosoil dation pressure (Kpa) P_c	Overburden pressure (Kpa) P_o	C_r	$P_c >> P_o$	Prpposed depth(m)	Footing Width (m)	q_s (Kpa)	Δp (Kpa)	ΔH (mm)
17.89	102	17.819	0.073	Over cosolidation	2.00	1.50	53.333	8.889	11.730
17.89	102	17.819	0.073	Over cosolidation	2.00	2.00	40.000	6.531	9.041
17.89	102	17.819	0.073	Over cosolidation	2.00	2.50	32.000	5.000	7.161
17.89	102	17.819	0.073	Over cosolidation	2.00	3.00	26.667	3.951	5.798

`

Table 5.9: Settlement analysis for test pit #2

Pit # 2 The value surcharge Load 80kpa **for L=B**									
Υ_{sat} (KN/m²)	Precosoil dation pressure (Kpa) P_c	Overburden pressure (Kpa) P_o	C_r	$P_c >> P_o$	Prpposed depth(m)	Footing Width (m)	q_s (Kpa)	Δp (Kpa)	ΔH (mm)
18.09	115	18.082	0.075	Over cosolidation	2.00	1.50	53.33	8.889	11.840
18.09	115	18.082	0.075	Over cosolidation	2.00	2.00	40.00	6.531	9.130
18.09	115	18.082	0.075	Over cosolidation	2.00	2.50	32.00	5.000	7.229
18.09	115	18.082	0.075	Over cosolidation	2.00	3.00	26.66	3.951	5.851

Table 5.10: Settlement analysis for test pit #3

Pit # 3 The value surcharge Load 80kpa **for L=B**									
Υ_{sat} (KN/m²)	Precosoil dation pressure (Kpa) P_c	Overburden pressure (Kpa) P_o	C_r	$P_c >> P_o$	Prpposed depth(m)	Footing Width (m)	q_s (Kpa)	Δp (Kpa)	ΔH (mm)
18.86	105	18.477	0.066	Over cosolidation	2.00	1.50	53.33	8.889	10.350
18.86	105	18.477	0.066	Over cosolidation	2.00	2.00	40.00	6.531	7.975
18.86	105	18.477	0.066	Over cosolidation	2.00	2.50	32.00	5.000	6.311
18.86	105	18.477	0.066	Over cosolidation	2.00	3.00	26.66	3.951	5.106

`

Table 5.11: Settlement analysis for test pit #4

Pit # 4 The value surcharge Load 80kpa **for L=B**									
Υ_{sat} (KN/m²)	Precosoil dation pressure (Kpa) P_c	Overburden pressure (Kpa) P_o	C_r	$P_c >> P_o$	Prpposed depth(m)	Footing Width (m)	q_s (Kpa)	Δp (Kpa)	ΔH (mm)
17.98	110	17.951	0.072	Over cosolidation	2.00	1.50	53.33	8.889	11.557
17.98	110	17.951	0.072	Over cosolidation	2.00	2.00	40.00	6.531	8.914
17.98	110	17.951	0.072	Over cosolidation	2.00	2.50	32.00	5.000	7.060
17.98	110	17.951	0.072	Over cosolidation	2.00	3.00	26.66	3.951	5.715

Based on the above calculations the maximum total settlements for the width 1.50m of the Structure will 11.73mm for pit one, 11.840mm for pit two, 10.350mm for pit three and 11.557mm for pit four. These values are within the limit of acceptable settlement. For isolated foundation, the recommended maximum settlement value for clay is below 50mm (EBCS 7).

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5.5. Discussion on Foundation Recommendation for Lightweight Structure

Foundation recommendation refers to the determination of the bearing layer and depth, allowable pressure on the bearing layer and type of foundation that could be adopted safely and economically for lightweight buildings described above.

The allowable bearing pressures for the selected foundation layers shall be discussed based on the correlation of the relative compaction of the in-situ ground as indicated from laboratory determination of UCS tests.

Results show that high plastic inorganic clay soil layers may undergo volume changes upon wetting and drying:

- Causing differential settlement (as the wetting/drying is not going to be uniform throughout the soil mass and time)
- Inducing swelling/expansion pressures on the foundation structure and floor slabs.

Expansive soil problems typically arise as a result of the increase in water content in the upper few meters, which is generally termed as the zone of seasonal fluctuation or the active zone. If excess water is added at the surface or if evapotranspiration is eliminated, the water content will increase and heave will occur. Thus, the foundation and superstructures have to be able to withstand these adverse movements and stresses and should have sufficient flexibility as well as rigidity to avoid cracks and other associated damages and subsequent failure of the structure.

In selecting the appropriate type of foundation on such soils, numerous factors have to be taken into account. Basic design considerations include either eliminating/removing the potentially expansive soil or minimizing the expansion effects. Some of the considerations are:

- Placement of footing below the depth of active swelling could be considered if it has sufficient bearing capacity to carry the superstructure loads,

Proposed isolated footing, Based on laboratory result obtain swelling pressure will have a range from 160kpa to 310kpa so it is difficult to give a structural solution with regarding increasing the weight of structure in order to minimize the deleterious effects of expansive soils on structures when the swelling potential is high improvement techniques have been suggested and practiced by local builders. Those involve using intercepting layers of earthen materials between the natural soil and the foundation material and/or strengthening the structure. These techniques are detailed as follows:

1. For concrete floor slabs placed directly on potentially expansive clays, provide expansion joints (Styrofoam) so that the floor can move freely from the structural frame. Placing the floor slabs and grade beams directly on the expansive soil should be avoided and if possible collapsible formwork/card boxes or sand/gravel fill. The topsoil should be removed for a minimum of 50 cm and replaced with non-swelling, granular and competent material (W.P.Jobs, and W.R. Stroman, 1974).

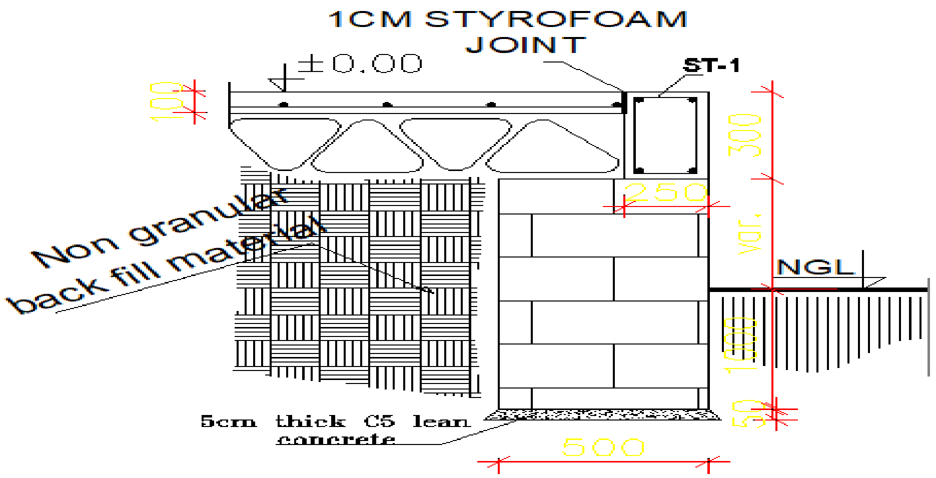

Figure 5. 1 detail layout of Styrofoam

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2. Grade beams should contain sufficient steel reinforcement to resist the horizontal and vertical thrust of swelling soils. If practical, place compressible joint filler or open blocks or boxes beneath grade beams to minimize swelling pressure (Nelson and Miller 1992; Lytton 1994).

3. Provide impervious horizontal and vertical blankets and surface grading around the foundations to prevent infiltration of surface water. The efficient drainage system should be designed and implemented to quickly drain any surface water away from the building structures and without infiltrating into the underlying soils (Alemayehu, 1986).

3. Removing garden sprinkler for vegetation around the building water will not be readily accessible to foundation soils thereby causing damage (Alemayehu ,1986).

4. compacted with low density these approach to compact swelling clays at moisture content slightly above their natural moisture content and low density should give a good result (Gromko,1974). The main advantage of using these approach it that the swelling potential can be reduced without the negative effect caused by introducing excessive moisture into the soil, in which moisture migration to the underlying moisture deficiency soil take place. Even though the required depth of compaction depends on the potential expansiveness of the soil and the magnitude of the structural load.it may, in general, be adequate to compact to a depth of 1.5 to 2m.

5. Replacing masonry foundation wall by grade beam noising because of masonry wall used trachyte stone which highly susceptible to uplift by swelling potential of the soil but grade beam noising play a vital role increasing structural rigidity with a small contact surface and reduce the swelling potential of soil. (http://www.epsindustry.org/building-construction/below-grade-foundation).

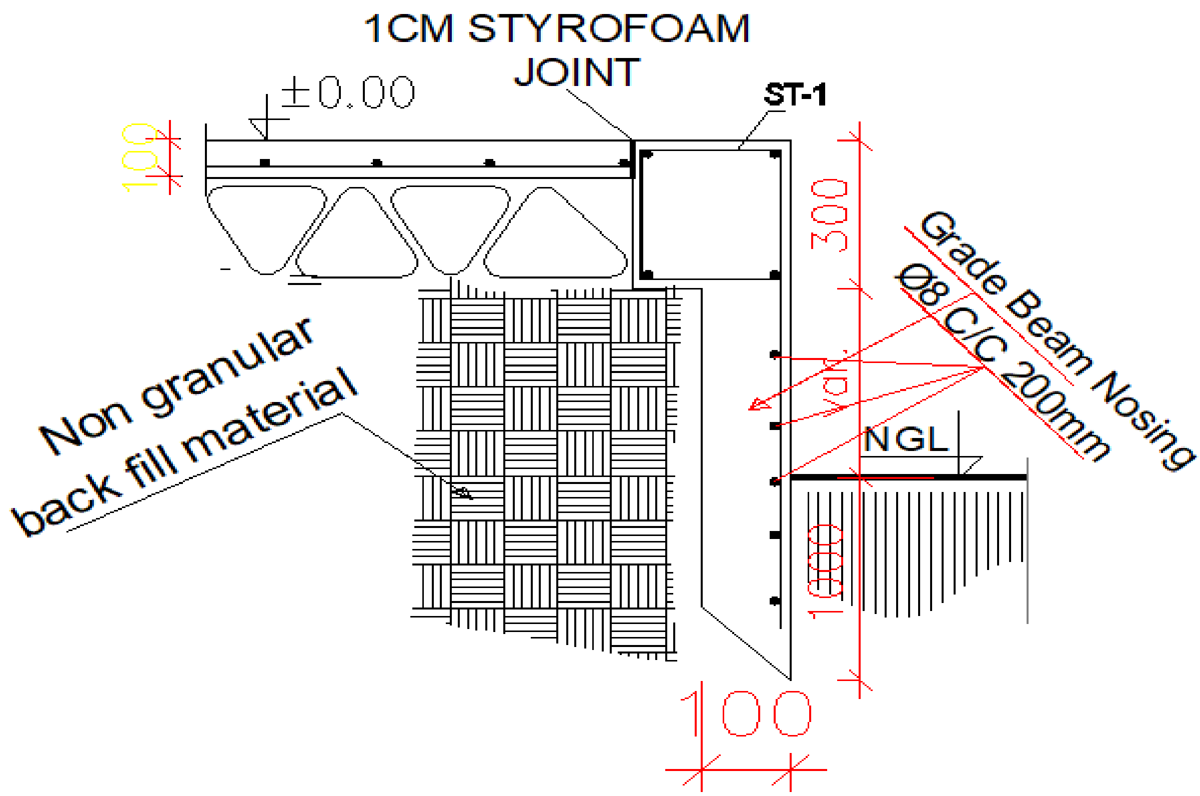

Figure 5. 2: detail grade beam noising

6. Using of micropile technique is based on inserting small-diameter steel piles (75 to 250 mm in diameter) in pre-drilled holes of larger diameter in an expansive soil which are then filled with compacted sand to improve the frictional resistance of the micro piles. The top section of each micropile is fastened to the foundation. In a reinforced-earth experimental study, Sridharam et al. (1989) found that with low-frictional-strength soil (clay) as a bulk material, a 15-mm thickness of sand around the reinforcement (steel strips) is sufficient to increase the interfacial shearing resistance to that of sand as the bulk material.

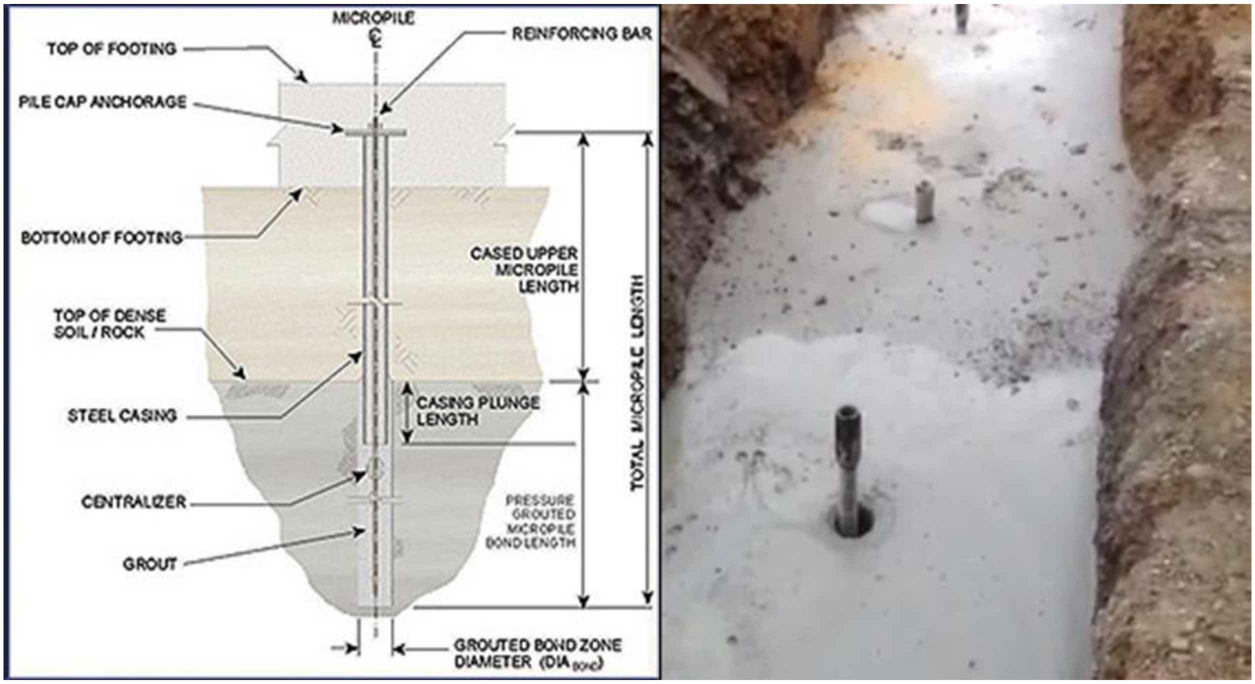

Figure 5. 3: micro pile layout and detail

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CHAPTER SIX - CONCLUSION AND RECOMMENDATION

6.1. Conclusion

The sub-surface geotechnical investigation was conducted for lightweight Building structures. The investigation shows the foundation soil around Legetafo is highly expansive, with an allowable bearing capacity range of 150.04kPa to 429.31kPa and swelling pressure of 160kPa to 310kPa.

A swelling pressure of this magnitude is known to cause countless problems on building structures.

In order to counterbalance the effect of aforementioned problems on structure, remedial measures proposed are providing Styrofoam at expansion joint between grade beam and ground slab, replacing masonry wall foundation with grade beam noising having maximum diameter 8 reinforcement with one meter maximum embedment depth, avoiding garden sprinkler and providing micropile in areas where the swelling pressure escape the 310 kPa range.

6.2. Recommendation

- In this project, the soil samples were collected only from four test pits specific area in Legetafo town, by increasing the number of sampling additional investigation should be done in the future in order to fully characterize the soil properties of the area.
- The compatibility of the recommended methods using Styrofoam between grade beam with slab, and the replacement of masonry wall by grade beam nosing with superstructure shall be checked for other type of foundation in the future.

REFERENCES

1. Arora, K.R. Soil Mechanics, and Foundation Engineering, New Delhi: Standard Publishers Distributors, 1997.
2. Mesfin, L. Basic Soil Mechanics for Civil Engineers Addis Ababa University, Addis Ababa 1984
3. Murthy, V.N.S Principles and Practices of Soil Mechanics and Foundation Engineering, New Delhi: New Delhi, 1990.
4. Alemayehu T. & Solomon Y., Investigation on the expansive soils of Addis Ababa, Zede, a journal of the Ethiopian association of engineers and architects,1986.
5. Alemayehu Teferra, 1984, "Estimation of Settlement of Rigid Footings", Journal of the Ethiopian Association of Engineers and Architects, Zede Volume 6, Pages 1-8, 1984.
6. Alemayehu Teferra, "Principles of Foundation Engineering", Addis Ababa university /AAU/ Press, A.A, Ethiopia, 1stEdition 1992 and 2nd Edition of 2008G.C.
7. Alemayehu Teferra and Leikun M., "Soil Mechanics, Faculty of Technology", AAU, 1997.
8. Samuel.T, Lecture note on soil exploration, Addis Ababa Institute of technology
9. Ministry of urban development and construction (MUDC).government viewed 14 April 2018, from, http/ www.mudco.gov.et
10. Map from Addis Ababa to legetafo Oromiya Ethiopia and legeT'afo(legetafo) map, weather, and photo viewed 14 April 2018, from, http/ www.getamap.net
11. ASTM D5434 Special Procedures for Testing Soil and Rock for Civil Engineering Purpose, U.S. America.2004.
12. Selamawit, M, Investigation on Some of Engineering properties of Soils found in Sebeta Town.Thesis.(2014)

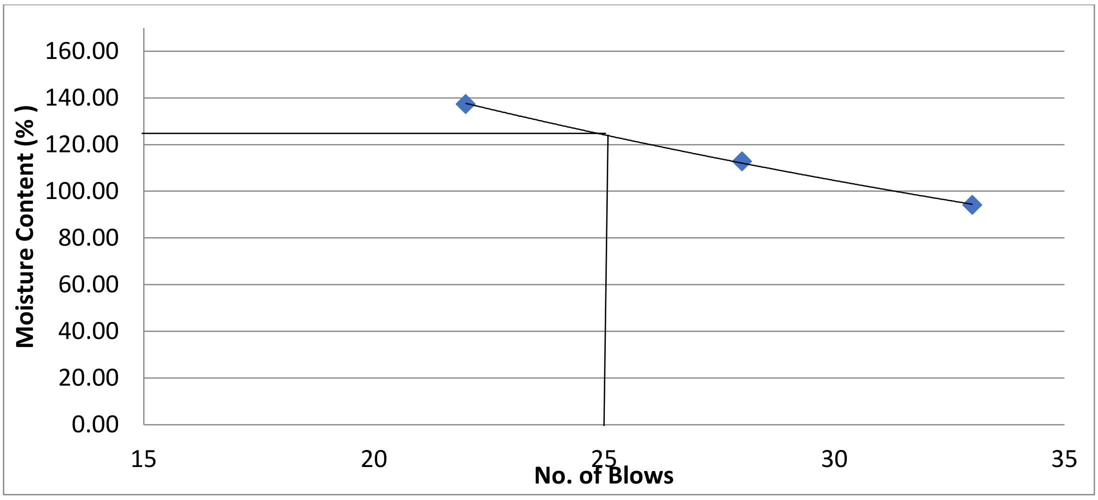
13. Bowles J. E., RE., S.E, "Foundation Analysis and Design", MCgraw-Hill Companies Inc., New York, International Edition, 1988, 5th Edition
14. Chen F.H., "Foundation on Expansive soil", Elsevier Scientific Publishing Company, Amsterdam, 1988 G.C
15. Girma Tessema, -"Foundation on Expansive Soil". Journal of the Ethiopian Association of Engineers and Architects, Zede-Volume 6, pages 13-20, December 1984.
16. Leach, J.A: - "Probabilistic approach to the Design of Residential Foundations in Expansive Clays"
17. John D. Nelson/J.Miller, "Expansive soils, Problems, and Practice in Foundation and Pavement Engineering", 1992.
18. Winterkorn, H.F/Fang H.Y, "Foundation Engineering Handbook". Van Nostrand Reinhold Company, New York, 1975.
19. Nelson, D.J and Debora, J.M. Expansive Soils Problems and Practice in Foundation and Pavement Engineering. New York: John Wiley and Sons, Inc., 1992.
20. Mitchell, J.K. Fundamentals of soil behavior. The Mc-graw hill companies, 1976.
21. Mastewal, G.M, investigation into some of the engineering properties of soils found in mojo town.Thesis. (2016)
22. Samuel.T has studied about the investigation into some of the engineering properties of Addis Ababa red clay soils. Thesis,1989
23. Hailemariam G, guideline for the selection of safe and economical types of reinforced concrete foundation for different buildings constructed on expansive soil of Addis Ababa.Thesis. (2016)

APPENDIX A

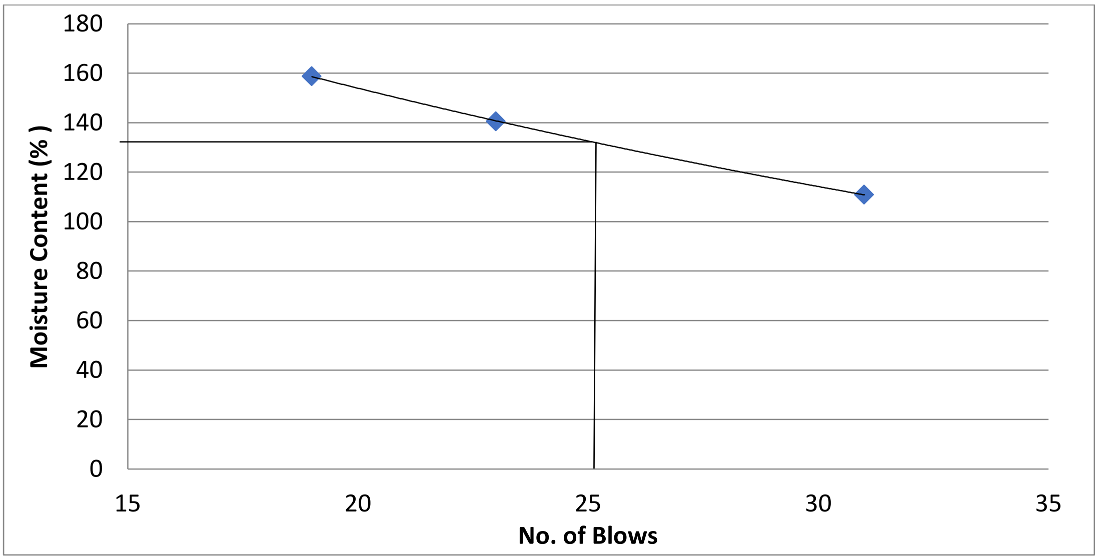
ATTERBER LIMIT AND

SPECIFIC GRAVITY TEST

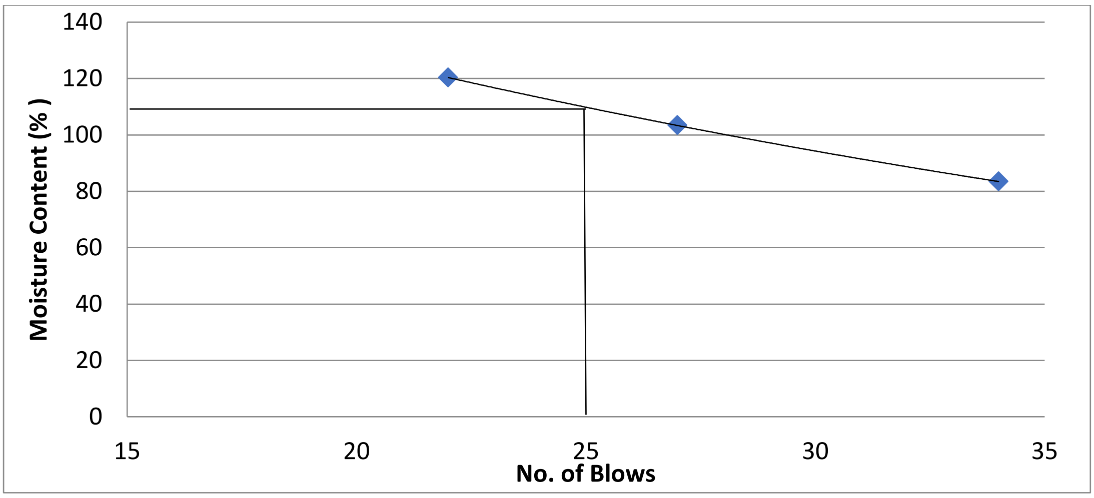
Test pit #1 @1.5m					
Description	Liquid Limit			Plastic Limit	
No. Blows	33	28	22		
Wt. wet soil (g.)	16.5	17.87	20.88	1.05	1.16
Wt. dry soil (g.)	8.5	8.40	8.8	0.83	0.87
Moisture content (%)	94.12	112.74	137.27	26.51	33.33
		Liquid Limit	123	AV.PL (%)	30



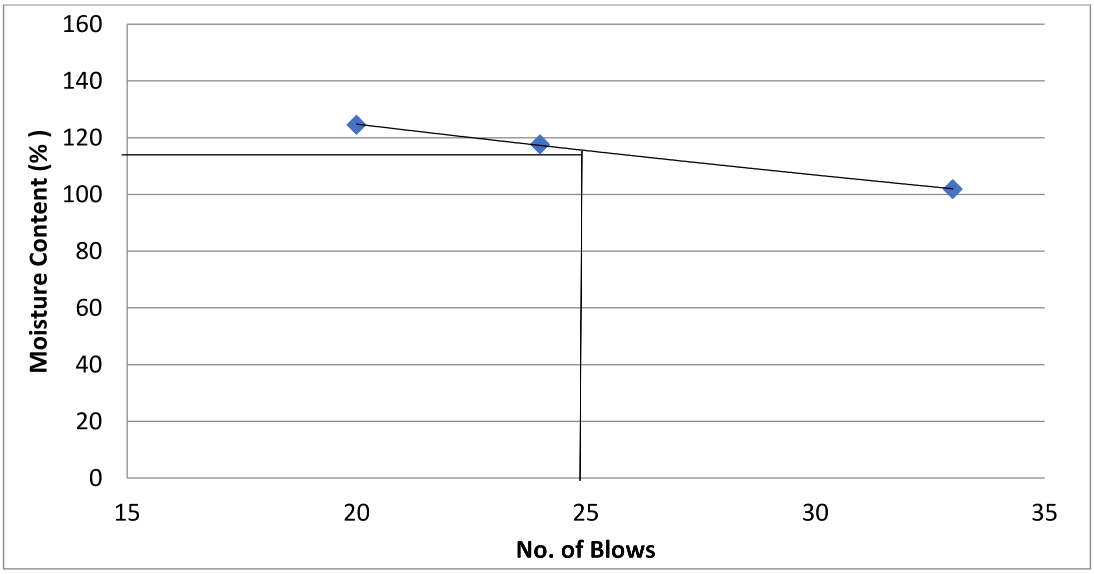
Test pit #1 @2m					
Description	Liquid Limit			Plastic Limit	
No. Blows	31	23	19		
Wt. wet soil (g.)	13.56	13.66	16.02	0.76	0.68
Wt. dry soil (g.)	6.43	5.68	6.19	0.59	0.51
Moisture content (%)	110.89	140.49	158.80	28.81	33.33
		Liquid Limit	130	AV.PL (%)	31



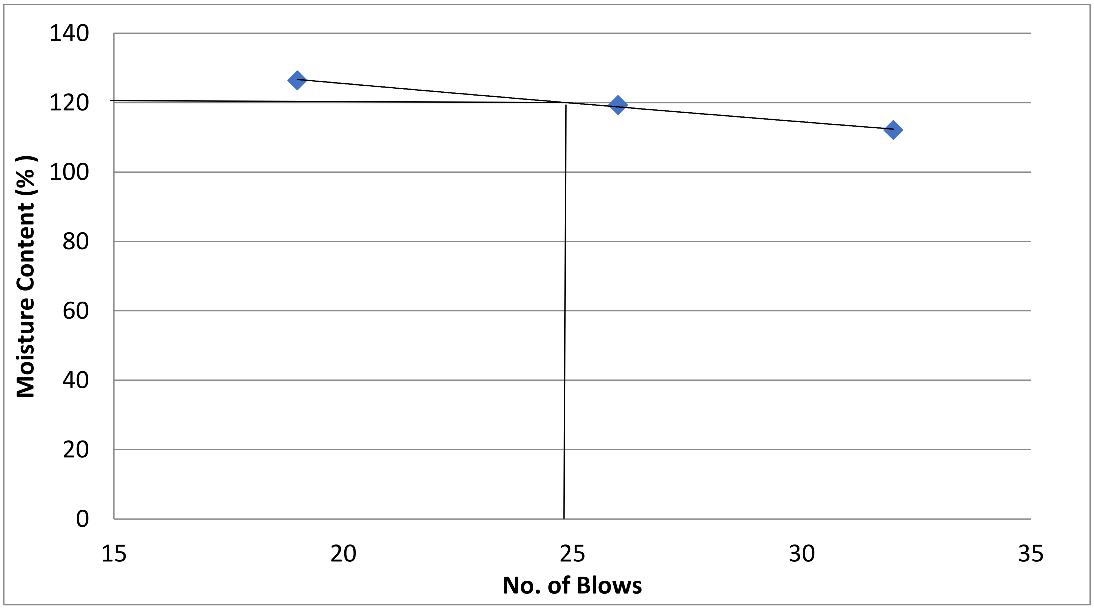
Test pit #2 @1.5m					
Description	Liquid Limit			Plastic Limit	
No. Blows	34	27	22		
Wt. wet soil (g.)	15.83	16.11	18.24	0.98	0.72
Wt. dry soil (g.)	8.63	7.92	8.28	0.69	0.61
Liquid Limit (%)	83.43	103.41	120.29	42.03	18.03
	Liquid Limit		115	AV.PL (%)	30



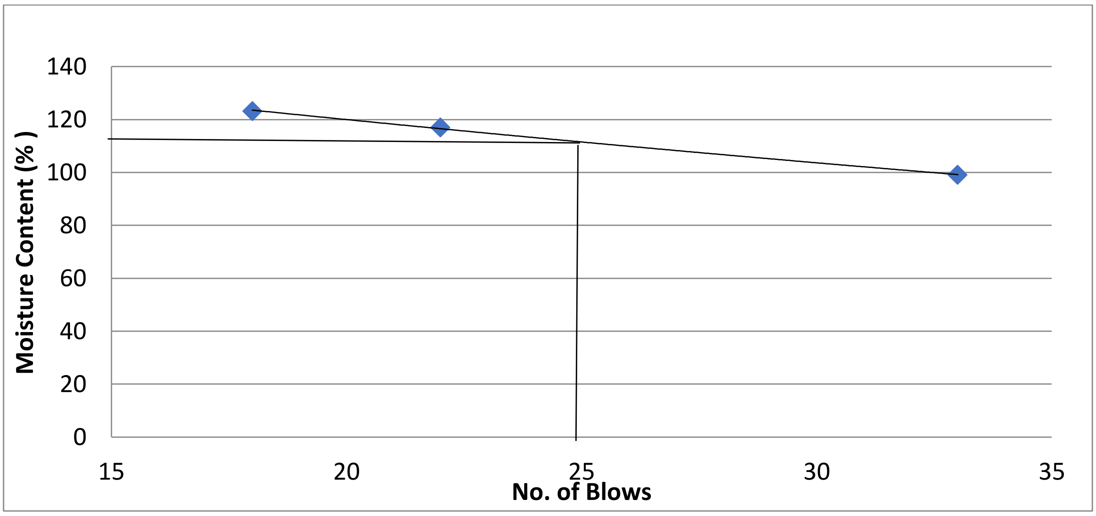
Test pit #2 @2m					
Description	Liquid Limit			Plastic Limit	
No. Blows	31	24	20		
Wt. wet soil (g.)	16.03	15.73	16.32	0.94	0.86
Wt. dry soil (g.)	7.94	7.18	7.27	0.75	0.66
Liquid Limit (%)	101.89	119.08	124.48	25.33	30.30
		Liquid Limit	118	AV.PL (%)	28



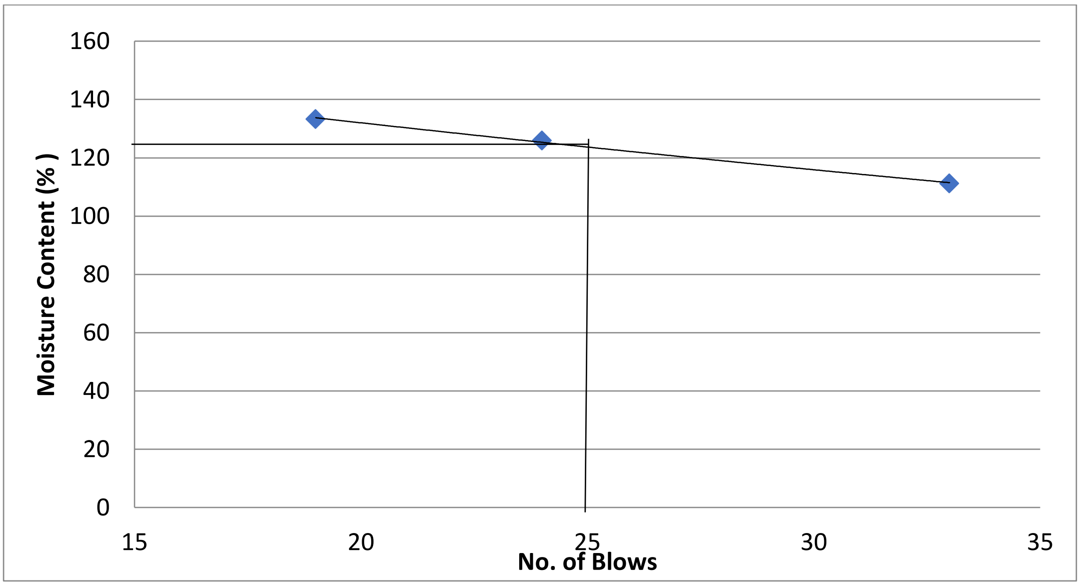
Test pit #3 @1.5m					
Description	Liquid Limit			Plastic Limit	
No. Blows	32	26	19		
Wt. wet soil (g.)	17.67	17.16	18.41	1.04	1.07
Wt. dry soil (g.)	8.33	7.78	8.13	0.84	0.79
Liquid Limit (%)	112.12	120.50	126.45	23.81	35.44
		Liquid Limit	121	AV.PL (%)	30



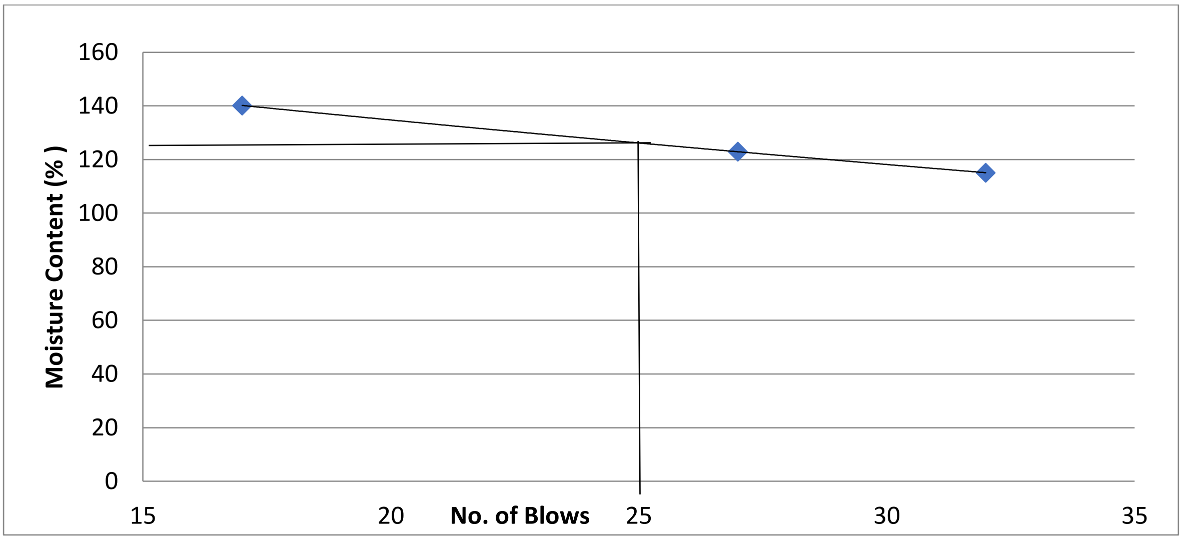
Test pit #3 @2m					
Description	Liquid Limit			Plastic Limit	
No. Blows	32	22	18		
Wt. wet soil (g.)	15.17	14.52	18.21	0.85	0.79
Wt. dry soil (g.)	7.62	6.69	8.16	0.68	0.58
Liquid Limit (%)	99.08	117.04	123.16	25.00	36.21
		Liquid Limit	119	AV.PL (%)	31



Test pit #4 @1.5m					
Description	Liquid Limit			Plastic Limit	
No. Blows	33	24	19		
Wt. wet soil (g.)	21.38	21.31	21.37	1.36	1.28
Wt. dry soil (g.)	10.12	9.43	9.16	1.05	0.99
Liquid Limit (%)	111.26	125.98	133.30	29.52	29.29
		Liquid Limit	124	AV.PL (%)	29



Test pit #4 @ 2m					
Description	Liquid Limit			Plastic Limit	
No. Blows	32	27	17		
Wt. wet soil (g.)	16.73	20.13	18.49	0.65	0.64
Wt. dry soil (g.)	7.78	9.03	7.7	0.46	0.51
Liquid Limit (%)	115.04	122.92	140.13	41.30	25.49
		Liquid Limit	127	AV.PL (%)	33



No	Specific Gravity Determination test at a depth of 1.5m @ Pit #1		
	Determination No.	1	2
1	Weight of pycnometer + soil + water, W_{pws} (g) (m_3)	182	182.1
2	Weight of pycnometer + soil (m_2)	91.9	92
3	Weight of pycnometer + water at T_x , $W_{pw}(atT_x)$ (g) (m_4)	166.1	166.3
4	Tempreture, Tx (^oC)	23.8	22.1
5	K,correction factor for effect of temperature (from table1 of D854)	0.99741	0.99962
6	Weight of Pycnometer (gm) (m_1)	66.9	66.8
7	Weight of Water used (m_3-m_2)	90.1	90.1
8	Wight of soil used (gm) (m_2-m_1)	25	25.2
9	Volume of soil (cm^3) (m_4-m_1)-(m_3-m_2)	9.1	9.4
10	Specific gravity of soil at Tx (^oC). Gs=(m_2-m_1)/ (m_4-m_1)-(m_3-m_2)	2.75	2.68
11	specific gravity of soil at 20^oC .K*Gs(@Tx ^oC)	2.74	2.68
12	Average specific gravity of soil at 20^oC .	2.71	

No	Specific Gravity Determination test at a depth of 2m @ Pit #1		
	Determination No.	1	2
1	Weight of pycnometer + soil + water, W_{pws} (g) (m_3)	160.4	161.1
2	Weight of pycnometer + soil (m_2)	84.9	84.9
3	Weight of pycnometer + water at T_x , $W_{pw}(atT_x)$ (g) (m_4)	144.7	145.2
4	Tempreture,Tx (^oC)	27.40	27.10
5	K,correction factor for effect of temperature (from table1 of D854)	0.9993	0.99962
6	Weight of Pycnometer (gm) (m_1)	59.9	59.9
7	Weight of Water used (m_3-m_2)	75.5	76.2
8	Wight of soil used (gm) (m_2-m_1)	25	25
9	Volume of soil (cm^3) (m_4-m_1)-(m_3-m_2)	9.3	9.1
10	Specific gravity of soil at Tx (^oC). Gs=(m_2-m_1)/ (m_4-m_1)-(m_3-m_2)	2.69	2.75
11	specific gravity of soil at 20^oC .K*Gs(@Tx ^oC)	2.69	2.75
12	Average specific gravity of soil at 20^oC .	2.72	

No	Specific Gravity Determination test at a depth of 1.5m@ Pit#2		
	Determination No.	1	2
1	Weight of pycnometer + soil + water, W_{pws} (g) (m_3)	182	182.1
2	Weight of pycnometer + soil (m_2)	91.6	92
3	Weight of pycnometer + water at T_x , $W_{pw}(atT_x)$ (g) (m_4)	166.1	166.3
4	tempreture,Tx (^oC)	26.3	26.2

5	K,correction factor for effect of temperature (from table1 of D854)	0.99851	0.99854
6	Weight of Pycnometer (gm) (m_1)	66.9	66.8
7	Weight of Water used (m_3-m_2)	90.4	90.1
8	Wight of soil used (gm) (m_2-m_1)	24.7	25
9	Volume of soil (cm^3) (m_4-m_1)-(m_3-m_2)	8.8	9.4
10	Specific gravity of soil at Tx (^oC). Gs=(m_2-m_1)/ (m_4-m_1)-(m_3-m_2)	2.81	2.68
11	specific gravity of soil at 20^oC .K*Gs(@Tx ^oC)	2.80	2.68
12	Average specific gravity of soil at 20^oC .	2.74	

No	Specific Gravity Determination test at a depth of 2m@ Pit #2		
	Determination No.	1	2
1	Weight of pycnometer + soil + water, W_{pws} (g) (m_3)	182.4	183.1
2	Weight of pycnometer + soil (m_2)	91.2	92.3
3	Weight of pycnometer + water at T_x , $W_{pw}(atT_x)$ (g) (m_4)	166.4	166.1
4	tempreture,Tx (^oC)	26.1	26.4
5	K,correction factor for effect of temperature (from table1 of D854)	0.99853	0.99858
6	Weight of Pycnometer (gm) (m_1)	65.8	65.8
7	Weight of Water used (m_3-m_2)	91.2	90.8
8	Wight of soil used (gm) (m_2-m_1)	25	25
9	Volume of soil (cm^3) (m_4-m_1)-(m_3-m_2)	9.4	9.5
10	Specific gravity of soil at Tx (^oC). Gs=(m_2-m_1)/ (m_4-m_1)-(m_3-m_2)	2.70	2.79

11	specific gravity of soil at 20°C .K*Gs(@Tx °C)	2.70	2.79
12	Average specific gravity of soil at 20°C .	2.75	

No	Specific Gravity Determination test at a depth of 1.5m @Pit #3		
	Determination No.	1	2
1	Weight of pycnometer + soil + water, W_{pws} (g) (m_3)	182	182.4
2	Weight of pycnometer + soil (m_2)	91.6	92
3	Weight of pycnometer + water at T_x , $W_{pw}(atT_x)$ (g) (m_4)	166.1	166.3
4	tempreture,Tx (°C)	26.3	26.2
5	K,correction factor for effect of temperature (from table1 of D854)	0.99841	0.99862
6	Weight of Pycnometer (gm) (m_1)	66.9	66.8
7	Weight of Water used (m_3-m_2)	90.4	90.4
8	Wight of soil used (gm) (m_2-m_1)	24.7	25
9	Volume of soil (cm^3) (m_4-m_1)-(m_3-m_2)	8.8	9.1
10	Specific gravity of soil at Tx (°C). Gs=(m_2-m_1)/ (m_4-m_1)-(m_3-m_2)	2.81	2.77
11	specific gravity of soil at 20°C .K*Gs(@Tx °C)	2.80	2.77
12	Average specific gravity of soil at 20°C .	2.79	

No	Specific Gravity Determination test at a depth of 2m @Pit #3		
	Determination No.	1	2
1	Weight of pycnometer + soil + water, W_{pws} (g) (m_3)	183.4	181.1

2	Weight of pycnometer + soil (m_2)	90.8	90.8
3	Weight of pycnometer + water at T_x , $W_{pw}(atT_x)$ (g) (m_4)	166.4	166.1
4	tempreture,Tx (^oC)	26.1	26.4
5	K,correction factor for effect of temperature (from table1 of D854)	0.99753	0.99974
6	Weight of Pycnometer (gm) (m_1)	65.8	65.8
7	Weight of Water used (m_3-m_2)	92.6	90.3
8	Wight of soil used (gm) (m_2-m_1)	25	25
9	Volume of soil (cm^3) (m_4-m_1)-(m_3-m_2)	8	10
10	Specific gravity of soil at Tx (^oC). Gs=(m_2-m_1)/ (m_4-m_1)-(m_3-m_2)	3.13	2.50
11	specific gravity of soil at 20^oC .K*Gs(@Tx ^oC)	3.12	2.50
12	Average specific gravity of soil at 20^oC .	2.81	

No	Specific Gravity Determination test at a depth of 1.5m @Pit #4		
	Determination No.	1	2
1	Weight of pycnometer + soil + water, W_{pws} (g) (m_3)	181.8	182.4
2	Weight of pycnometer + soil (m_2)	91.6	92
3	Weight of pycnometer + water at T_x , $W_{pw}(atT_x)$ (g) (m_4)	166.1	166.5
4	tempreture,Tx (^oC)	26.1	26.5
5	K,correction factor for effect of temperature (from table1 of D854)	0.99873	0.99869
6	Weight of Pycnometer (gm) (m_1)	66.9	66.8
7	Weight of Water used (m_3-m_2)	90.2	90.4

8	Wight of soil used (gm) (m_2-m_1)	24.7	25
9	Volume of soil (cm^3) (m_4-m_1)-(m_3-m_2)	9	9.3
10	Specific gravity of soil at Tx (^oC). Gs=(m_2-m_1)/ (m_4-m_1)-(m_3-m_2)	2.74	2.71
11	specific gravity of soil at 20^oC .K*Gs(@Tx ^oC)	2.74	2.71
12	Average specific gravity of soil at 20^oC .	2.73	

No	Specific Gravity Determination test at a depth of 2m@ Pit #4		
	Determination No.	1	2
1	Weight of pycnometer + soil + water, W_{pws} (g) (m_3)	181.8	182.4
2	Weight of pycnometer + soil (m_2)	91.6	92
3	Weight of pycnometer + water at T_x , W_{pw}(atT_x) (g) (m_4)	166.1	166.3
4	tempreture,Tx (^oC)	26.1	26.5
5	K,correction factor for effect of temperature (from table1 of D854)	0.99873	0.99871
6	Weight of Pycnometer (gm) (m_1)	66.9	66.8
7	Weight of Water used (m_3-m_2)	90.2	90.4
8	Wight of soil used (gm) (m_2-m_1)	24.7	25
9	Volume of soil (cm^3) (m_4-m_1)-(m_3-m_2)	9	9.1
10	Specific gravity of soil at Tx (^oC). Gs=(m_2-m_1)/ (m_4-m_1)-(m_3-m_2)	2.74	2.77
11	specific gravity of soil at 20^oC .K*Gs(@Tx ^oC)	2.74	2.77
12	Average specific gravity of soil at 20^oC .	2.76	

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Unit Weight of studies area soil			
$$\Upsilon sat = \frac{Gs * Pw(1+w)}{1+e}$$			
Test pit No	Water content	Specifc gravity	Υ_{sat} (kN/m²)
1	44.59	2.71	17.89219
2	42.03	2.75	18.08596
3	47.08	2.81	18.86164
4	44.25	2.73	17.98185

APPENDIX B

GRAIN SIZE ANALYSIS TEST

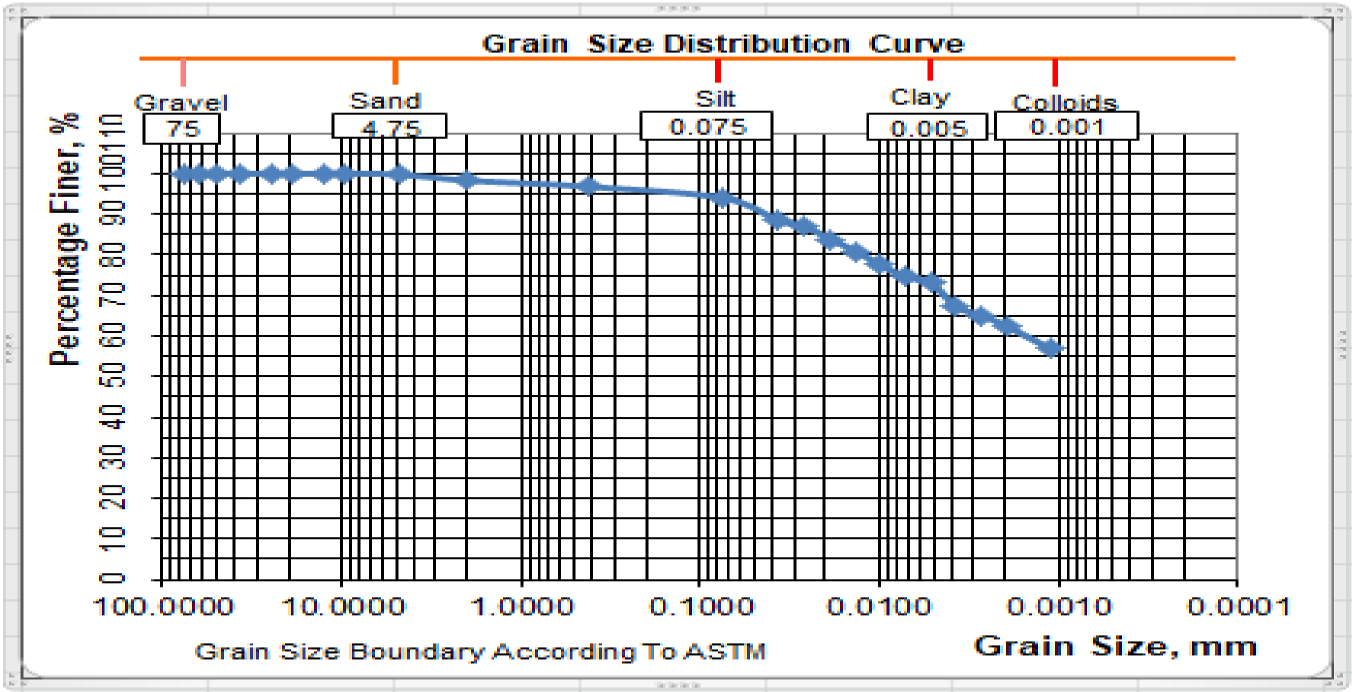
Sieve analysis for Test Result Pit #1 @1.5m

Test Pt 1@1.5m				
Wt of soil	500	gm		
Sieve size mm	Mass Retained (gm)	% Retained	Cumulative Mass Retained	% Passing
75.0	0.00	0.00	0.00	100.00
63.5	0.00	0.00	0.00	100.00
50.0	0.00	0.00	0.00	100.00
37.5	0.00	0.00	0.00	100.00
25.0	0.00	0.00	0.00	100.00
19.0	0.00	0.00	0.00	100.00
12.5	0.00	0.00	0.00	100.00
9.5	0.00	0.00	0.00	100.00
4.8	0.85	0.17	0.17	99.83
2.0	7.25	1.45	1.62	98.38
0.425	7.50	1.50	3.12	96.88
0.075	14.15	2.83	5.95	94.05

Hydrometer Analysis Test Result Pit #1 @1.5m

Specific Gravity of soil = 2.72

Elapsed Time (mm)	Actual hydrometer Reading	Composite correction	Corrected Hydrometer Reading	Effective Depth(Cm)	Tested Temperature C°	Coefficient K	Grain Size (mm)	Perc.Finer (%)	Perc.finer Combined (%)
3/4	1.0360	0.00360	1.0391	6.78	22	0.01312	0.0342	92.63	87.41
1	1.0325	0.00360	1.0356	7.70	22	0.01312	0.0257	89.26	85.86
2	1.0315	0.00360	1.0346	7.97	22	0.01312	0.0185	85.26	82.92
4	1.0285	0.00360	1.0316	8.76	22	0.01312	0.0137	82.57	79.26
8	1.0275	0.00360	1.0307	9.03	22	0.01312	0.0102	79.48	76.65
15	1.0265	0.00360	1.0296	9.29	22	0.01312	0.0073	76.24	73.24
30	1.0260	0.00360	1.0290	9.42	22	0.01312	0.0053	74.53	72.11
60	1.0250	0.00325	1.0281	9.69	22.5	0.01312	0.0051	71.58	69.34
120	1.0240	0.00304	1.0270	9.95	23	0.01312	0.0038	68.58	66.24
240	1.0230	0.00290	1.0259	10.22	23	0.01297	0.0027	65.24	64.26
480	1.0220	0.00290	1.0248	10.48	23	0.01297	0.0019	63.21	61.13
1440	1.0120	0.00290	1.0152	13.13	23	0.01297	0.0012	57.85	55.81



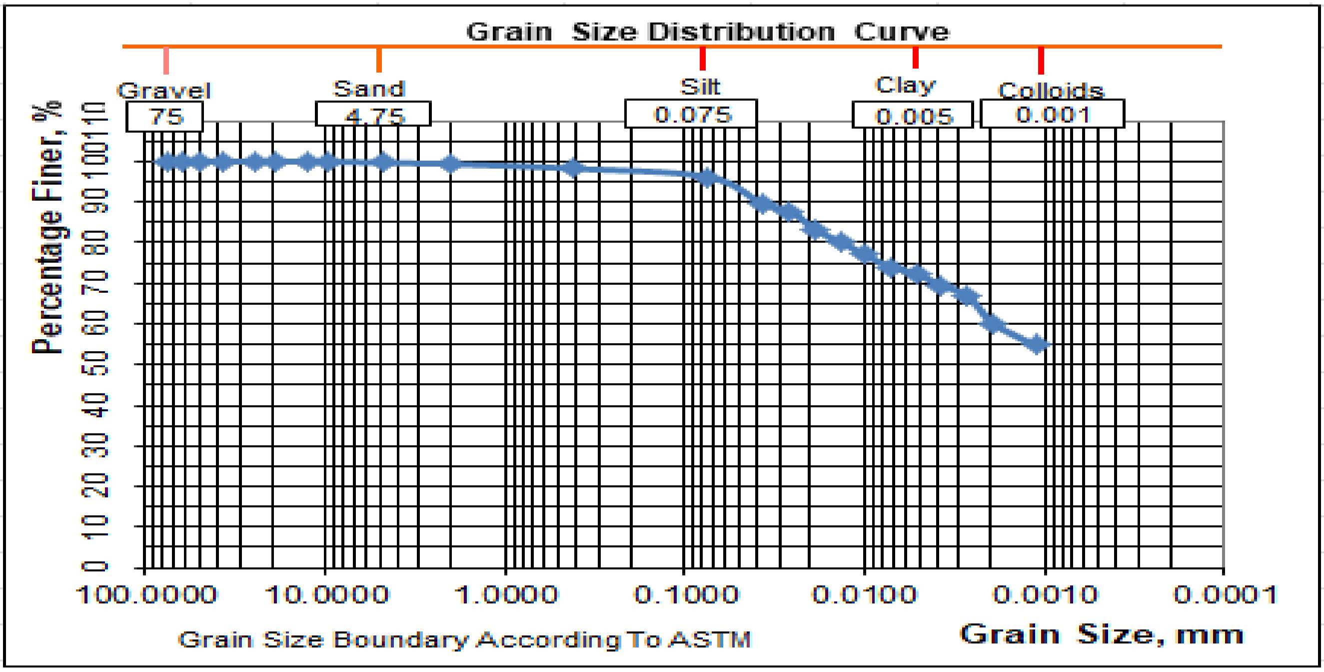
Sieve analysis for Test Result Pit #1 @2m

Test Pt 1@2m				
Wt of soil	500	gm		
Sieve size mm	Mass Retained (gm)	% Retained	Cumulative Mass Retained	% Passing
75.0	0.0	0.00	0.00	100.00
63.5	0.0	0.00	0.00	100.00
50.0	0.0	0.00	0.00	100.00
37.5	0.0	0.00	0.00	100.00
25.0	0.0	0.00	0.00	100.00
19.0	0.0	0.00	0.00	100.00
12.5	0.0	0.00	0.00	100.00
9.5	0.0	0.00	0.00	100.00
4.8	1.35	0.27	0.27	99.73
2.0	1.60	0.32	0.59	99.41
0.425	5.50	1.10	1.69	98.31
0.075	10.30	2.06	3.75	96.25

Hydrometer Analysis Test Result Pit #1 @2m

Specific Gravity of soil = 2.71

Elapsed Time (mm)	Actual hydrometer Reading	Composite correction	Corrected Hydrometer Reading	Tested Temperature C°	Effective Depth(Cm)	Coefficient K	Grain Size (mm)	Perc.Finer (%)	Perc.finer Combined (%)
3/4	1.0330	0.00430	1.0362	21	7.57	0.01297	0.0357	91.11	88.31
1	1.0321	0.00430	1.0353	21	7.81	0.01297	0.0256	89.67	86.75
2	1.0301	0.00430	1.0333	21	8.34	0.01297	0.0187	84.59	82.06
4	1.0295	0.00430	1.0327	21	8.50	0.01297	0.0134	81.38	79.94
8	1.0285	0.00430	1.0317	21	8.76	0.01297	0.0099	78.25	76.49
15	1.0275	0.00430	1.0307	21	9.03	0.01297	0.0071	75.24	72.83
30	1.0260	0.00430	1.0292	21	9.42	0.01297	0.0052	73.64	70.27
60	1.0250	0.00395	1.0281	21.5	9.69	0.01287	0.0050	70.26	68.41
120	1.0240	0.00360	1.0268	22	9.95	0.01287	0.0037	67.54	67.22
240	1.0230	0.00360	1.0258	22	10.22	0.01287	0.0027	64.81	63.76
480	1.0235	0.00360	1.0263	22	10.08	0.01282	0.0019	60.15	58.24
1440	1.0220	0.00360	1.0248	22	10.48	0.01282	0.0011	55.39	53.42



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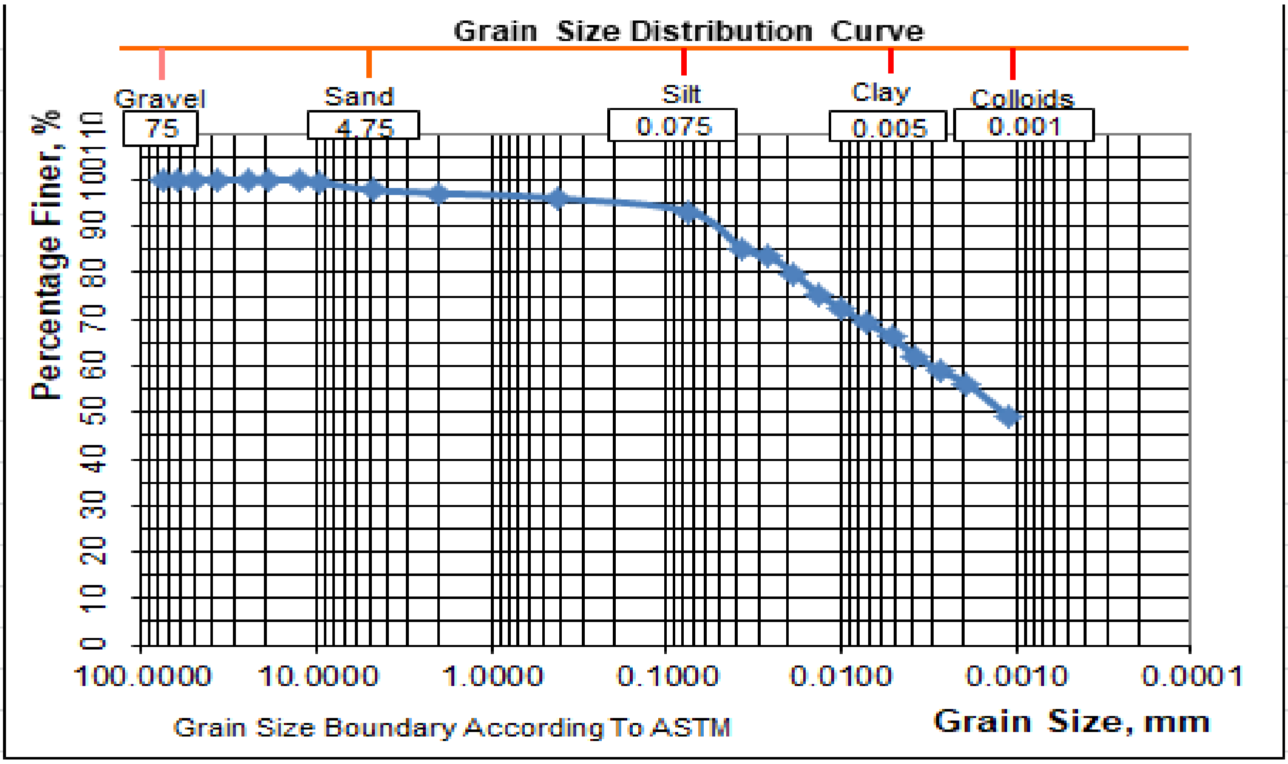
Sieve analysis for Test Result Pit #2 @1.5m

Test Pt 2@1.5m				
Wt of soil	500	gm		
Sieve size mm	Mass Retained (gm)	% Retained	Cumulative Mass Retained	% Passing
75.0	0.00	0.00	0.00	100.00
63.5	0.00	0.00	0.00	100.00
50.0	0.00	0.00	0.00	100.00
37.5	0.00	0.00	0.00	100.00
25.0	0.00	0.00	0.00	100.00
19.0	0.00	0.00	0.00	100.00
12.5	0.00	0.00	0.00	100.00
9.5	0.00	0.00	0.00	100.00
4.8	10.70	2.14	2.14	97.86
2.0	3.20	0.64	2.78	97.22
0.425	5.55	1.11	3.89	96.11
0.075	14.35	2.87	6.76	93.24

Hydrometer Analysis Test Result Pit # @1.5m

Specific Gravity of soil = 2.74

Elapsed Time (mm)	Actual hydrometer Reading	Composite correction	Corrected Hydrometer Reading	Effective Depth(Cm)	Tested Temperature C°	Coefficient K	Grain Size (mm)	Perc.Finer (%)	Perc.finer Combined (%)
3/4	1.0330	0.00360	1.0348	7.57	22	0.01309	0.0360	90.26	84.13
1	1.0325	0.00360	1.0343	7.70	22	0.01309	0.0257	88.25	82.64
2	1.0320	0.00360	1.0338	7.84	22	0.01309	0.0183	84.23	78.87
4	1.0315	0.00360	1.0333	7.97	22	0.01309	0.0131	82.57	74.21
8	1.0310	0.00360	1.0328	8.10	22	0.01309	0.0096	79.27	71.28
15	1.0295	0.00360	1.0313	8.50	22	0.01309	0.0070	76.39	70.08
30	1.0285	0.00360	1.0303	8.76	22	0.01309	0.0054	72.84	67.24
60	1.0270	0.00255	1.0288	9.16	23.5	0.01288	0.0051	68.61	62.84
120	1.0255	0.00220	1.0276	9.55	24	0.01276	0.0036	64.25	59.71
240	1.0245	0.00220	1.0264	9.82	24	0.01276	0.0026	61.47	57.39
480	1.0235	0.00220	1.0254	10.08	24	0.01276	0.0018	58.68	53.81
1440	1.0130	0.00220	1.0149	12.86	24	0.01276	0.0012	54.36	50.08



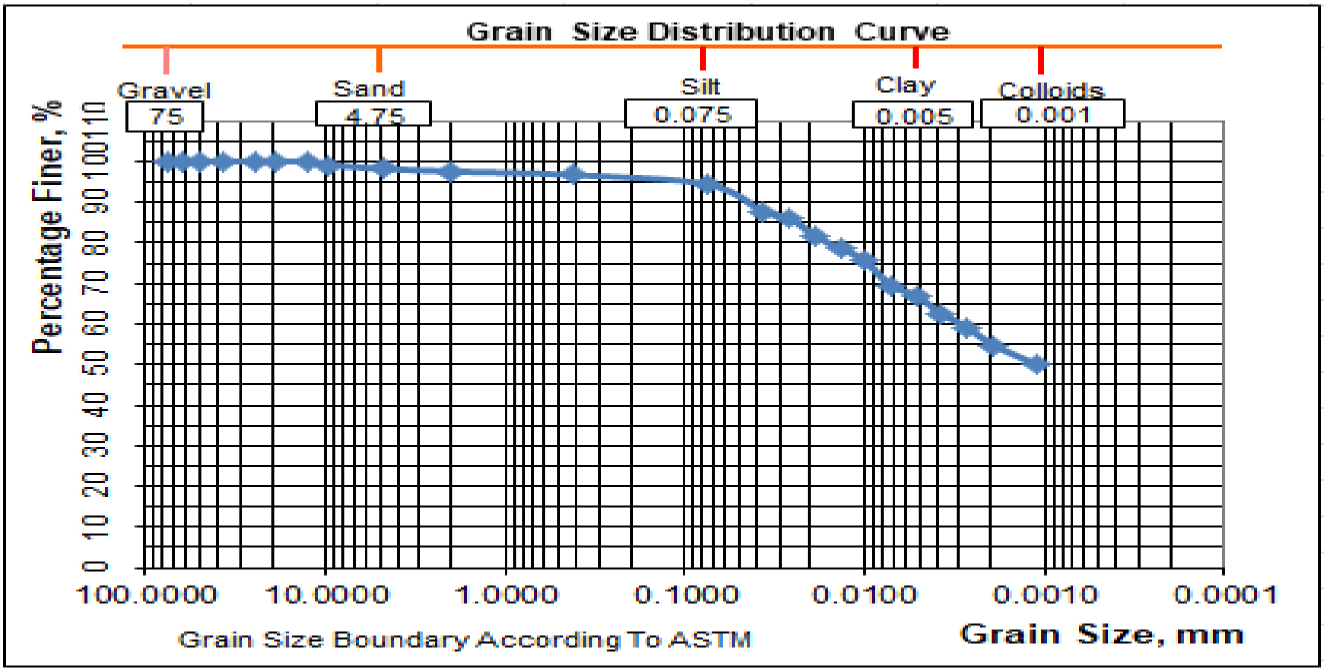
Sieve analysis for Test Result Pit #2 @2m

Test Pt 2@2m				
Wt of soil	500	gm		
Sieve size mm	Mass Retained (gm)	% Retained	Cumulative Mass Retained	% Passing
75.0	0.00	0.00	0.00	100.00
63.5	0.00	0.00	0.00	100.00
50.0	0.00	0.00	0.00	100.00
37.5	0.00	0.00	0.00	100.00
25.0	0.00	0.00	0.00	100.00
19.0	0.00	0.00	0.00	100.00
12.5	0.00	0.00	0.00	100.00
9.5	0.00	0.00	0.00	100.00
4.8	8.40	1.68	1.68	98.32
2.0	3.35	0.67	2.35	97.65
0.425	4.10	0.82	3.17	96.83
0.075	19.75	3.95	7.12	92.88

Hydrometer Analysis Test Result Pit #2 @2m

Specific Gravity of soil = 2.75

Elapsed Time (mm)	Actual hydrometer Reading	Composite correction	Corrected Hydrometer Reading	Effective Depth(Cm)	Tested Temperature Co	Coefficient K	Grain Size (mm)	Perc.Finer (%)	Perc.finer Combined (%)
3/4	1.0315	0.00290	1.0338	7.97	23	0.01279	0.0361	92.21	86.56
1	1.0310	0.00290	1.0333	8.10	23	0.01279	0.0257	89.67	85.11
2	1.0295	0.00290	1.0318	8.50	23	0.01279	0.0186	86.94	80.49
4	1.0285	0.00290	1.0308	8.76	23	0.01279	0.0134	81.23	77.45
8	1.0275	0.00290	1.0298	9.03	23	0.01279	0.0099	79.86	76.49
15	1.0265	0.00290	1.0288	9.29	23	0.01279	0.0071	76.33	73.47
30	1.0255	0.00290	1.0278	9.55	23	0.01279	0.0052	72.26	70.52
60	1.0230	0.00255	1.0253	10.22	23.5	0.01268	0.0050	70.27	68.14
120	1.0215	0.00430	1.0236	10.61	21	0.01309	0.0039	65.61	63.43
240	1.0200	0.00430	1.0221	11.01	21	0.01309	0.0028	61.81	59.87
480	1.0185	0.00430	1.0206	11.41	21	0.01309	0.0020	57.31	55.65
1440	1.0160	0.00430	1.0181	12.07	21	0.01309	0.0012	52.21	51.18



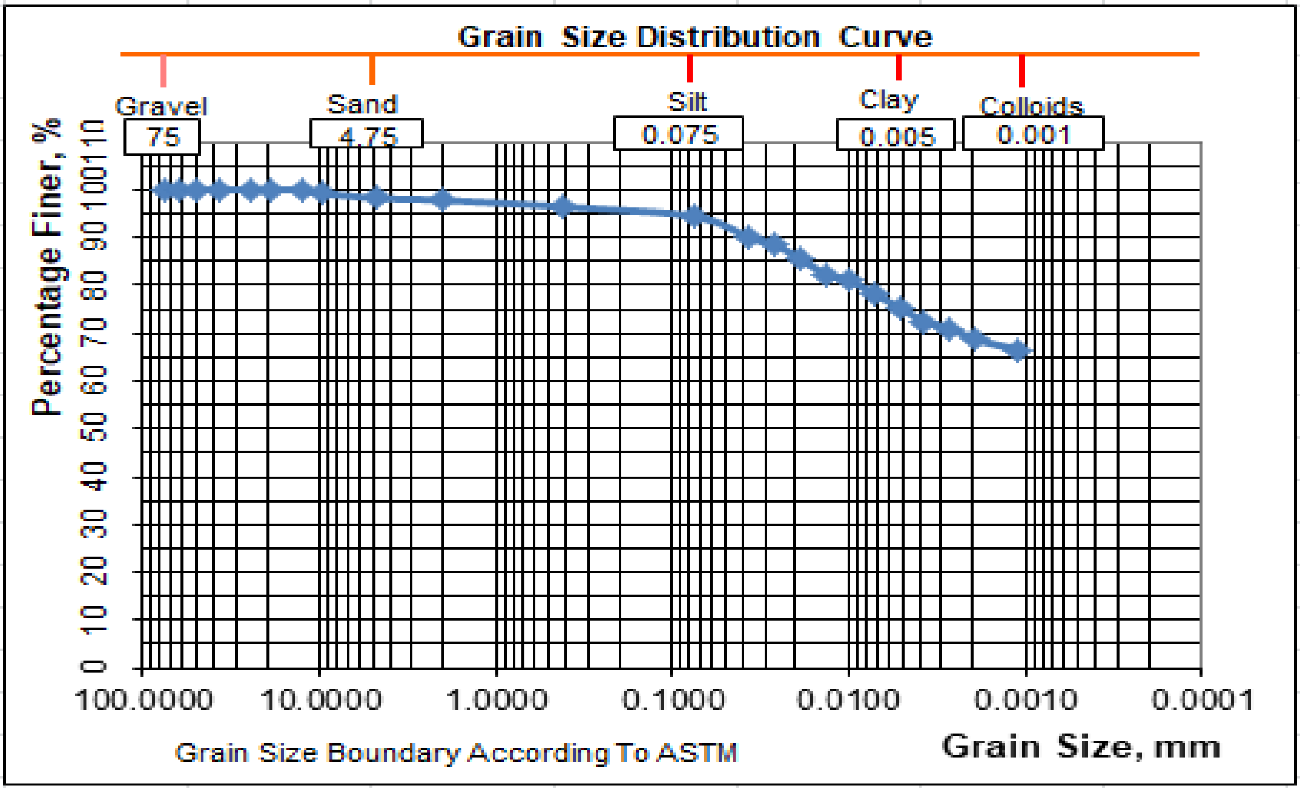
Sieve analysis for Test Result Pit #3 @1.5m

Test Pt 3@1.5m				
Wt of soil	500	gm		
Sieve size mm	Mass Retained (gm)	% Retained	Cumulative Mass Retained	% Passing
75.0	0.00	0.00	0.00	100.00
63.5	0.00	0.00	0.00	100.00
50.0	0.00	0.00	0.00	100.00
37.5	0.00	0.00	0.00	100.00
25.0	0.00	0.00	0.00	100.00
19.0	0.00	0.00	0.00	100.00
12.5	0.00	0.00	0.00	100.00
9.5	3.7	0.74	0.74	99.26
4.8	4.85	0.97	1.71	98.29
2.0	2.20	0.44	2.15	97.85
0.425	6.80	1.36	3.51	96.49
0.075	10.35	2.07	5.58	94.42

Hydrometer Analysis Test Result Pit #3 @1.5m

Specific Gravity of soil = 2.79

Elapsed Time (mm)	Actual hydrometer Reading	Composite correction	Corrected Hydrometer Reading	Effective Depth(Cm)	Tested Temperature C°	Coefficient K	Grain Size (mm)	Perc.Finer (%)	Perc.finer Combined (%)
3/4	1.0325	0.00430	1.0347	7.70	21	0.01298	0.0360	93.17	87.84
1	1.0320	0.00430	1.0342	7.84	21	0.01298	0.0257	91.46	85.37
2	1.0310	0.00430	1.0332	8.10	21	0.01298	0.0185	88.31	82.43
4	1.0300	0.00430	1.0322	8.36	21	0.01298	0.0133	86.20	79.50
8	1.0290	0.00430	1.0312	8.63	21	0.01298	0.0098	82.57	78.11
15	1.0280	0.00430	1.0302	8.89	21	0.01298	0.0071	79.69	75.68
30	1.0270	0.00430	1.0292	9.16	21	0.01298	0.0051	76.84	72.31
60	1.0260	0.00325	1.0282	9.42	22.5	0.01265	0.0049	75.01	69.37
120	1.0250	0.00500	1.0273	9.69	20	0.01316	0.0037	73.25	68.07
240	1.0240	0.00500	1.0263	9.95	20	0.01316	0.0027	71.57	65.62
480	1.0230	0.00500	1.0253	10.22	20	0.01316	0.0019	69.44	63.54
1440	1.0215	0.00500	1.0238	10.61	20	0.01316	0.0011	64.86	59.31



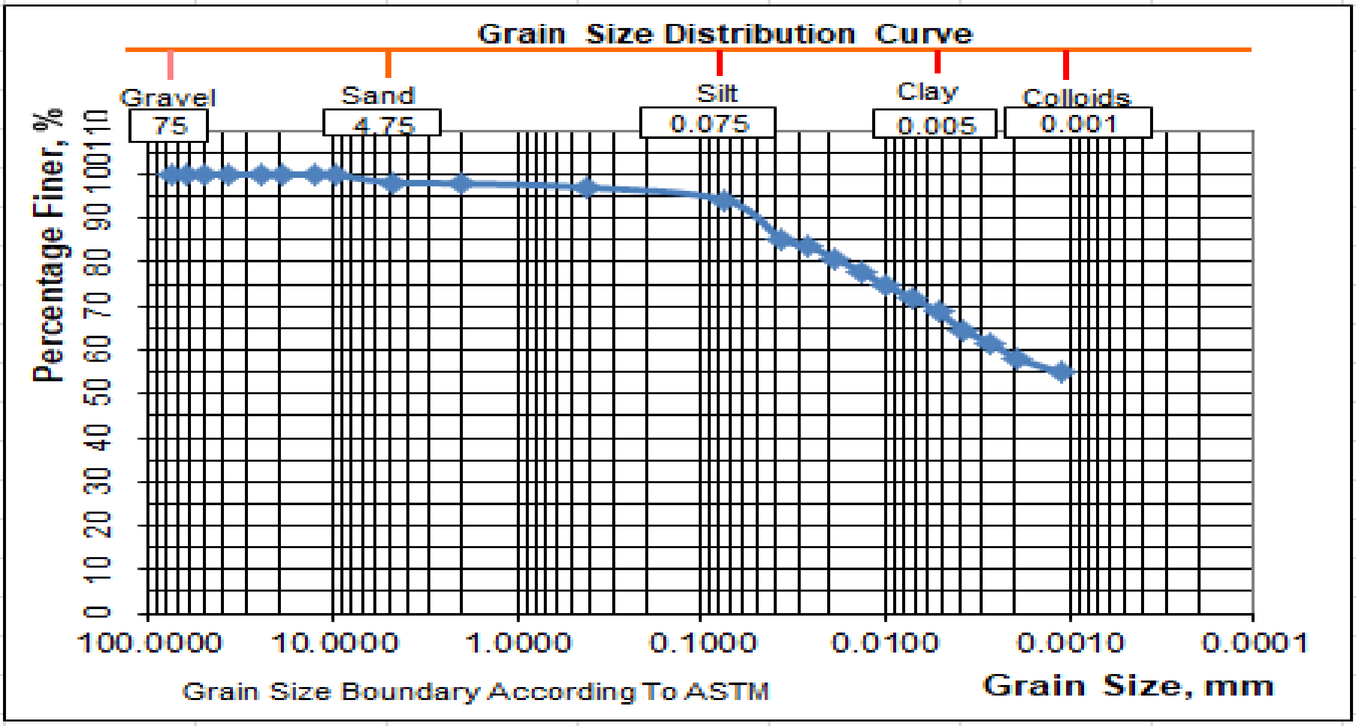
Sieve analysis for Test Result Pit #3 @2m

Test Pt 3@2m				
Wt of soil	500	gm		
Sieve size mm	Mass Retained (gm)	% Retained	Cumulative Mass Retained	% Passing
75.0	0.00	0.00	0.00	100.00
63.5	0.00	0.00	0.00	100.00
50.0	0.00	0.00	0.00	100.00
37.5	0.00	0.00	0.00	100.00
25.0	0.00	0.00	0.00	100.00
19.0	0.00	0.00	0.00	100.00
12.5	0.00	0.00	0.00	100.00
9.5	1.35	0.27	0.27	99.73
4.8	7.45	1.49	1.76	98.24
2.0	1.60	0.32	1.81	97.24
0.425	4.20	0.84	2.65	97.35
0.075	14.25	2.85	5.50	94.50

Hydrometer Analysis Test Result Pit #3 @2m

Elapsed Time (mm)	Actual hydrometer Reading	Composite correction	Corrected Hydrometer Reading	Effective Depth(Cm)	Tested Temperature C°	Coefficient K	Grain Size (mm)	Perc.Finer (%)	Perc.finer Combined (%)
3/4	1.0320	0.00430	1.0342	7.84	21	0.01284	0.0359	90.31	84.08
1	1.0315	0.00430	1.0337	7.97	21	0.01284	0.0256	88.71	83.63
2	1.0305	0.00430	1.0327	8.23	21	0.01284	0.0184	85.57	81.67
4	1.0290	0.00430	1.0312	8.63	21	0.01284	0.0133	82.37	78.50
8	1.0850	0.00430	1.0872	-6.18	21	0.00128	0.0127	79.15	75.60
15	1.0270	0.00430	1.0292	9.16	21	0.01284	0.0071	76.14	72.80
30	1.0260	0.00430	1.0282	9.42	21	0.01284	0.0054	73.61	69.57
60	1.0245	0.00325	1.0267	9.82	22.5	0.01264	0.0050	70.85	65.28
120	1.0230	0.00500	1.0253	10.22	20	0.01291	0.0038	67.17	62.41
240	1.0215	0.00500	1.0238	10.61	20	0.01291	0.0027	63.08	58.84
480	1.0200	0.00500	1.0223	11.01	20	0.01291	0.0020	59.87	55.86
1440	1.0190	0.00500	1.0213	11.27	20	0.01291	0.0011	56.42	51.18

Specific Gravity of soil = 2.81



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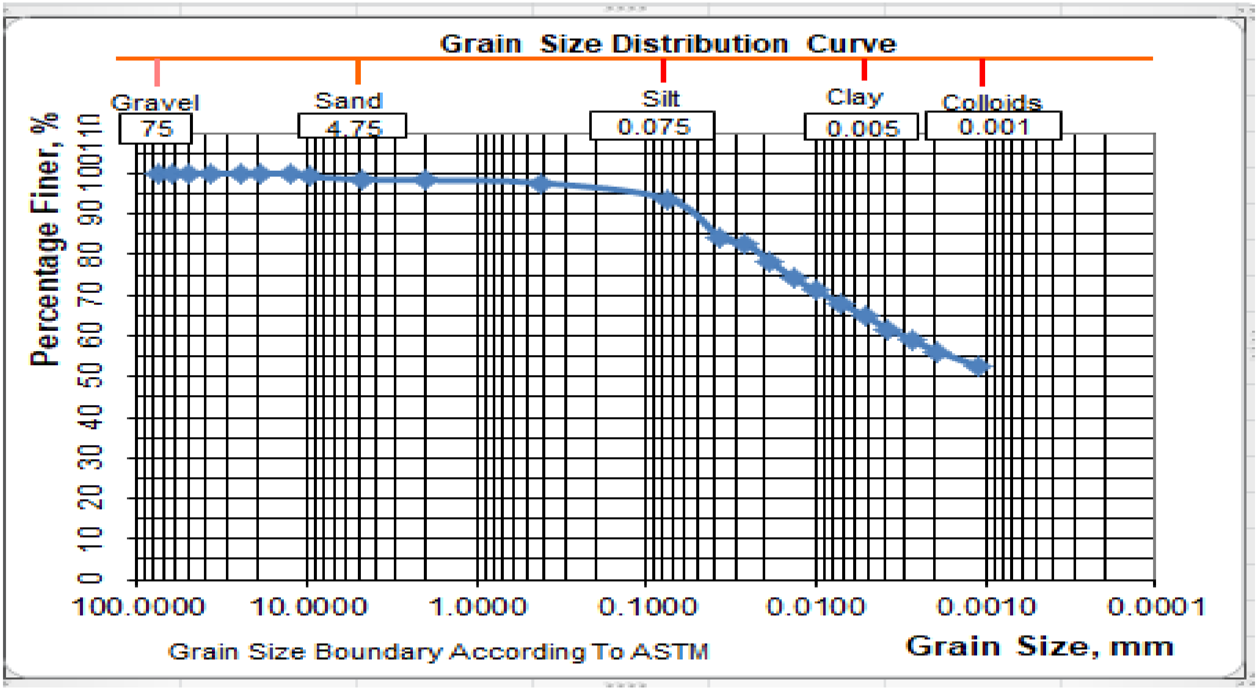
Sieve analysis for Test Result Pit #4 @1.5m

Test Pt 4@1.5m				
Wt of soil	500	gm		
Sieve size mm	Mass Retained (gm)	% Retained	Cumulative Mass Retained	% Passing
75.0	0.00	0.00	0.00	100.00
63.5	0.00	0.00	0.00	100.00
50.0	0.00	0.00	0.00	100.00
37.5	0.00	0.00	0.00	100.00
25.0	0.00	0.00	0.00	100.00
19.0	0.00	0.00	0.00	100.00
12.5	0.00	0.00	0.00	100.00
9.5	3.70	0.74	0.74	99.26
4.8	3.50	0.70	1.44	98.56
2.0	0.70	0.14	1.58	98.42
0.425	3.40	0.68	2.26	97.74
0.075	19.80	3.96	6.22	93.78

Hydrometer Analysis Test Result Pit #4 @1.5m

Specific Gravity of soil = 2.76

Elapsed Time (mm)	Actual hydrometer Reading	Composite correction	Corrected Hydrometer Reading	Effective Depth(Cm)	Tested Temperature C°	Coefficient K	Grain Size (mm)	Perc.Finer (%)	Perc.finer Combined (%)
3/4	1.0310	0.00290	1.0329	8.10	23	0.01265	0.0360	89.56	83.18
1	1.0305	0.00290	1.0324	8.23	23	0.01265	0.0257	88.04	82.65
2	1.0290	0.00290	1.0309	8.63	23	0.01265	0.0186	83.30	78.37
4	1.0275	0.00290	1.0294	9.03	23	0.01265	0.0134	79.57	73.96
8	1.0260	0.00290	1.0279	9.42	23	0.01265	0.0100	76.31	70.22
15	1.0250	0.00290	1.0269	9.69	23	0.01265	0.0072	73.01	68.16
30	1.0270	0.00290	1.0289	9.16	23	0.01265	0.0052	69.47	65.69
60	1.0260	0.00430	1.0281	9.42	21	0.01303	0.0049	66.09	62.57
120	1.0250	0.00220	1.0268	9.69	24	0.01256	0.0036	62.87	59.94
240	1.0240	0.00220	1.0258	9.95	24	0.01256	0.0026	59.26	57.14
480	1.0230	0.00220	1.0248	10.22	24	0.01256	0.0018	55.38	53.25
1440	1.0220	0.00220	1.0238	10.48	`24	0.01256	0.0011	51.21	48.73



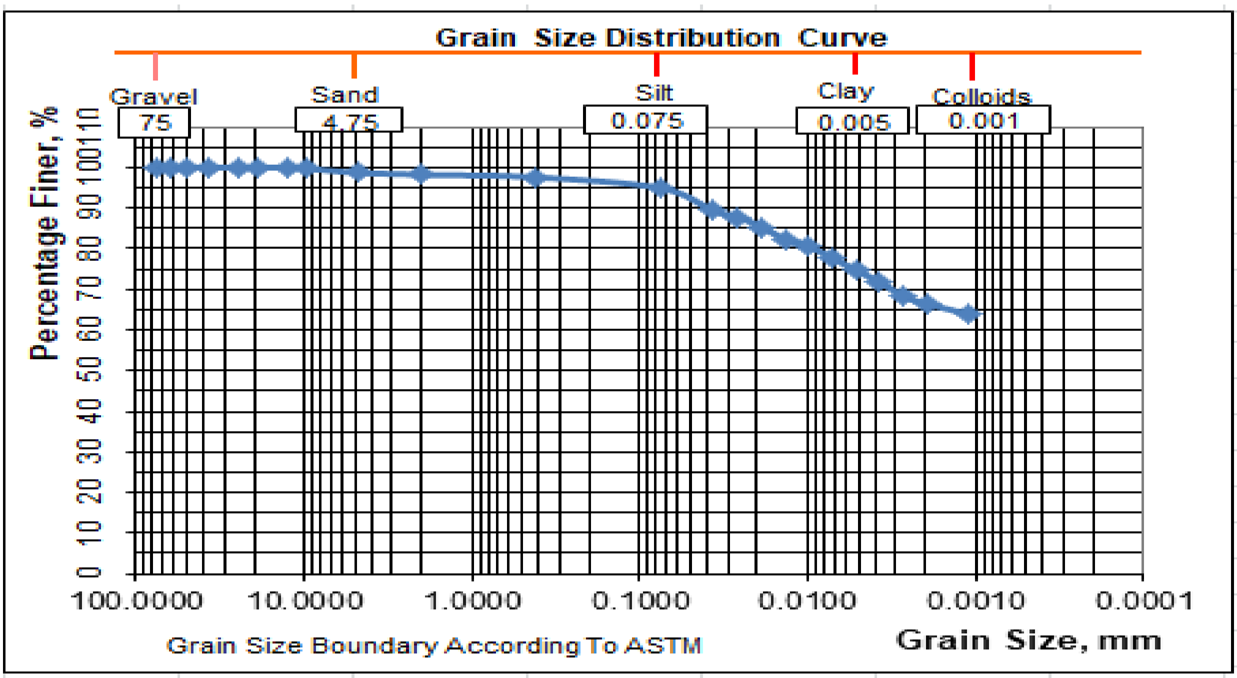
Sieve analysis for Test Result Pit #4 @2m

Test Pt 4@2m				
Wt of soil	500	gm		
Sieve size mm	Mass Retained (gm)	% Retained	Cumulative Mass Retained	% Passing
75.0	0.00	0.00	0.00	100.00
63.5	0.00	0.00	0.00	100.00
50.0	0.00	0.00	0.00	100.00
37.5	0.00	0.00	0.00	100.00
25.0	0.00	0.00	0.00	100.00
19.0	0.00	0.00	0.00	100.00
12.5	0.00	0.00	0.00	100.00
9.5	1.10	0.22	0.22	99.78
4.8	5.20	1.04	1.26	98.74
2.0	2.25	0.45	1.71	98.29
0.425	3.05	0.61	2.32	97.68
0.075	13.65	2.73	5.05	94.95

Hydrometer Analysis Test Result Pit #4 @2 m

Specific Gravity of soil = 2.73

Elapsed Time (mm)	Actual hydrometer Reading	Composite correction	Corrected Hydrometer Reading	Effective Depth(Cm)	Tested Temperature C°	Coefficient K	Grain Size (mm)	Perc.Finer (%)	Perc.finer Combined (%)
3/4	1.0310	0.00290	1.0329	8.10	23	0.01286	0.0366	92.24	87.26
1	1.0305	0.00290	1.0324	8.23	23	0.01286	0.0261	90.41	85.14
2	1.0290	0.00290	1.0309	8.63	23	0.01286	0.0189	87.73	83.21
4	1.0275	0.00290	1.0294	9.03	23	0.01286	0.0137	84.48	80.57
8	1.0260	0.00290	1.0279	9.42	23	0.01286	0.0102	82.84	78.13
15	1.0250	0.00290	1.0269	9.69	23	0.01286	0.0073	79.82	75.62
30	1.0270	0.00290	1.0289	9.16	23	0.01286	0.0052	76.57	72.48
60	1.0260	0.00430	1.0281	9.42	21	0.01310	0.0050	73.45	69.75
120	1.0250	0.00220	1.0268	9.69	24	0.01282	0.0036	69.87	67.24
240	1.0240	0.00220	1.0258	9.95	24	0.01282	0.0026	67.71	65.74
480	1.0230	0.00220	1.0248	10.22	24	0.01282	0.0019	65.26	62.88
1440	1.0220	0.00220	1.0238	10.48	24	0.01282	0.0011	60.48	57.97



APPENDIX C

FREE SWELL TEST

Test Pit -1		
Depth	1.5	
sample no	1	2
Initial volume,ml	10	10
Final volume,ml	19.4	20.1
Free Swell %	94	101
Average FS %	97.50	

Test Pit -1		
Depth	2	
sample no	1	2
Initial volume,ml	12	12
Final volume,ml	24.6	24.3
Free Swell %	105	102.5
Average FS %	103.75	

Test Pit -2		
Depth	1.5	
sample no	A	B
Initial volume,ml	12	12
Final volume,ml	25.6	24.4
Free Swell %	113.333	103.333
Average FS %	108.33	

Test Pit -2		
Depth	2	
sample no	1	2
Initial volume,ml	12	12
Final volume,ml	26.5	25.2
Free Swell %	120.833	110
Average FS %	115.42	

Test Pit -3		
Depth	1.5	
sample no	A	B
Initial volume,ml	12	12
Final volume,ml	26.4	24.3
Free Swell %	120	102.5
Average FS %	111.25	

Test Pit -3		
Depth	2	
sample no	1	2
Initial volume,ml	12	12
Final volume,ml	25.6	26.6
Free Swell %	113.333	121.667
Average FS %	117.50	

Test Pit -4		
Depth	1.5	
sample no	A	B
Initial volume,ml	12	12
Final volume,ml	24.9	25.4
Free Swell %	107.5	111.667
Average FS %	109.58	

Test Pit -4		
Depth	2	
sample no	1	2
Initial volume,ml	12	12
Final volume,ml	26.2	25.6
Free Swell %	118.333	113.333
Average FS %	115.83	

APPENDIX D

UNCONFINED COMPRESSION STRENGTH TEST

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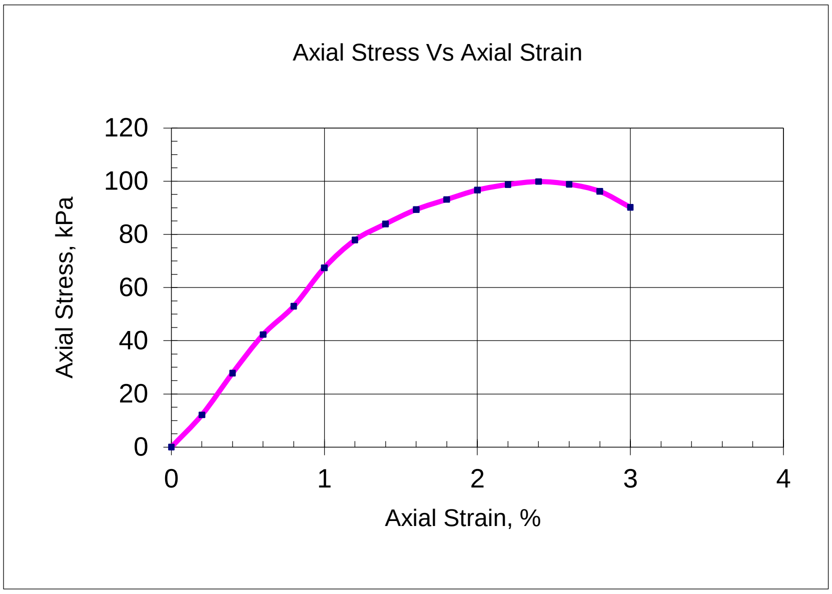
Test Pit #1 Depth , m	**2.00**	Ring Calibration Factor, kN/div	0.00138
Diameter of sample , mm	38	Moisture content, %	39.00
Length of sample , mm	76	Wet unit weight, kN/m3	1.78

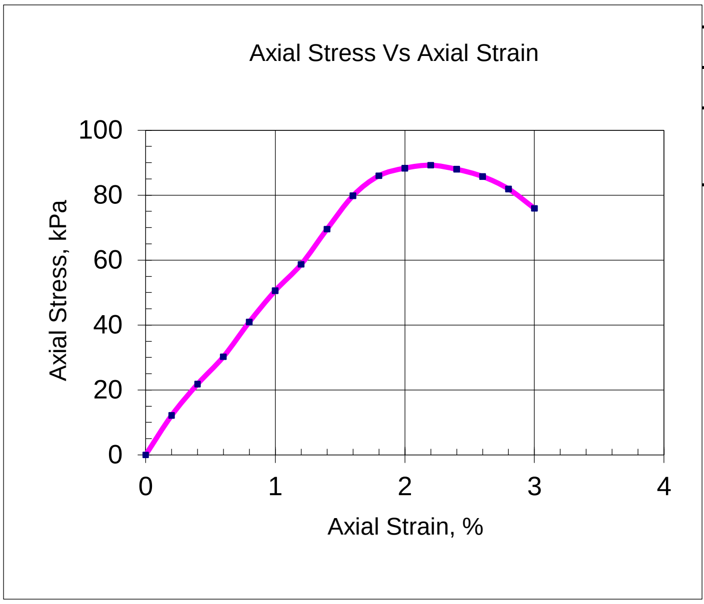
Axial Deformation [mm]	Axial Strain [%]	Proving Ring Reading [div]	Axial Load [kN]	Corrected Area [m2]	Axial Stress [kPa]
0	0.00	0	0.0000	0.00113	0
0.2	0.21	10	0.0138	0.00114	12.14
0.4	0.53	23	0.0317	0.00114	27.84
0.6	0.63	35	0.0483	0.00114	42.32
0.8	1.05	44	0.0607	0.00115	52.98
1	1.05	56	0.0773	0.00115	67.43
1.2	1.58	65	0.0897	0.00115	77.85
1.4	1.47	70	0.0966	0.00115	83.93
1.6	2.11	75	0.1035	0.00116	89.34
1.8	1.89	78	0.1076	0.00116	93.13
2	1.93	81	0.1118	0.00116	96.67
2.2	2.25	83	0.1132	0.00116	97.54
2.4	2.32	84	0.1132	0.00116	97.47
2.6	2.14	83	0.1145	0.00116	98.84
2.8	2.45	81	0.1118	0.00116	96.16
3	2.51	76	0.1049	0.00116	90.17



Unconfined Compressive Strength(q_u),kPa =	99
Undrained Shear Strengh (cu), kPa =	49.5

Test Pit #2 Depth , m	**2.00**	Ring Calibration Factor, kN/div	0.00138
Diameter of sample , mm	38	Moisture content, %	40.19
Length of sample , mm	76	Wet unit weight, kN/m3	1.80

Axial Deformation [mm]	Axial Strain [%]	Proving Ring Reading [div]	Axial Load [kN]	Corrected Area [m2]	Axial Stress [kPa]
0	0.00	0	0.0000	0.00113	0
0.2	0.21	10	0.0138	0.00114	12.14
0.4	0.53	18	0.0248	0.00114	21.79
0.6	0.63	25	0.0345	0.00114	30.23
0.8	1.05	34	0.0469	0.00115	40.94
1	1.05	42	0.0580	0.00115	50.57
1.2	1.58	49	0.0676	0.00115	58.69
1.4	1.47	58	0.0800	0.00115	69.54
1.6	2.11	67	0.0925	0.00116	79.81
1.8	1.89	72	0.0994	0.00116	85.96
2	1.93	74	0.1021	0.00116	88.31
2.2	2.25	75	0.1035	0.00116	89.22
2.4	2.32	73	0.1007	0.00116	86.77
2.6	2.14	72	0.0994	0.00116	85.74
2.8	2.45	69	0.0952	0.00116	81.91
3	2.51	64	0.0883	0.00116	75.93



Unconfined Compressive Strength(q_u),kPa =	89
Undrained Shear Strengh (cu), kPa =	45

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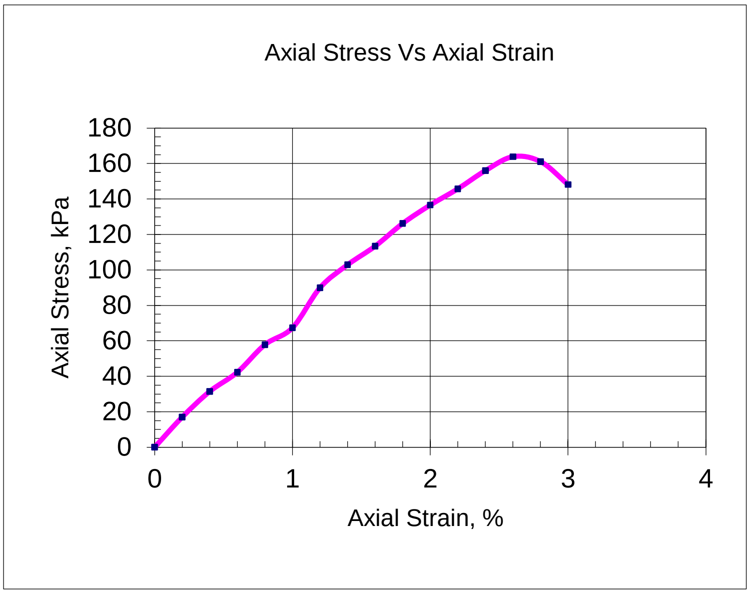
Test Pit #3 Depth , m	**2.00**	Ring Calibration Factor, kN/div	0.00138
Diameter of sample , mm	38	Moisture content, %	36.24
Length of sample , mm	76	Wet unit weight, kN/m3	1.84

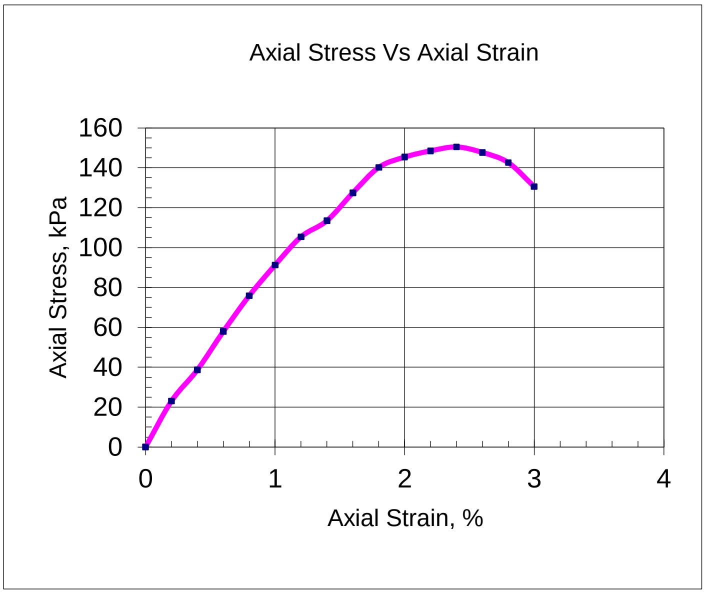
Axial Deformation [mm]	Axial Strain [%]	Proving Ring Reading [div]	Axial Load [kN]	Corrected Area [m2]	Axial Stress [kPa]
0	0.00	0	0.0000	0.00113	0
0.2	0.24	14	0.0193	0.00114	17.00
0.4	0.48	26	0.0359	0.00114	31.49
0.6	0.72	35	0.0483	0.00114	42.29
0.8	0.96	48	0.0662	0.00114	57.85
1	1.20	56	0.0745	0.00115	64.93
1.2	1.45	75	0.0883	0.00115	76.75
1.4	1.69	84	0.1007	0.00115	87.33
1.6	1.93	95	0.1132	0.00116	97.86
1.8	2.17	106	0.1283	0.00116	110.72
2	2.41	115	0.1435	0.00116	123.51
2.2	2.65	123	0.1642	0.00116	140.98
2.4	2.89	132	0.1822	0.00117	155.99
2.6	3.13	139	0.1960	0.00117	167.40
2.8	3.37	137	0.1891	0.00117	161.10
3	2.61	125	0.1725	0.00116	148.15



Unconfined Compressive	
Strength(q_u),kPa =	164
Undrained Shear Strengh (cu), kPa =	82

Test Pit #4 Depth , m	**2.00**	Ring Calibration Factor, kN/div	0.00138
Diameter of sample , mm	38	Moisture content, %	39.65
Length of sample , mm	76	Wet unit weight, kN/m3	1.79

Axial Deformation [mm]	Axial Strain [%]	Proving Ring Reading [div]	Axial Load [kN]	Corrected Area [m2]	Axial Stress [kPa]
0	0.00	0	0.0000	0.00113	0
0.2	0.26	19	0.0262	0.00114	23.06
0.4	0.53	32	0.0442	0.00114	38.74
0.6	0.79	48	0.0662	0.00114	57.95
0.8	1.05	63	0.0869	0.00115	75.86
1	1.32	76	0.1049	0.00115	91.27
1.2	1.58	88	0.1214	0.00115	105.40
1.4	1.84	92	0.1270	0.00116	109.90
1.6	2.11	107	0.1477	0.00116	127.46
1.8	2.37	118	0.1628	0.00116	140.19
2	2.89	121	0.1670	0.00117	142.99
2.2	3.16	123	0.1697	0.00117	144.95
2.4	3.42	126	0.1739	0.00117	148.09
2.6	3.68	128	0.1766	0.00118	150.03
2.8	3.95	128	0.1766	0.00118	149.61
3	4.21	122	0.1684	0.00118	142.22



Unconfined Compressive	
Strength(q_u),kPa =	150
Undrained Shear Strengh (cu), kPa =	75

APPENDIX E

CONSOLIDATION AND SWELLING

PRESSURE TEST

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Table A.56: Consolidation dial reading for test Pit #1 @ depth of 2m

Setting load dial reading=2.1mm							
Time(min.)	50Kpa	100Kpa	200Kpa	400Kpa	800Kpa	1600Kpa	Unloading
	Dial reading((mm)	Dial reading(mm)	Dial reading(mm)	Dial reading(mm)	Dial reading(mm)	Dial reading(mm)	1600Kpa
0	0.916	1.063	1.337	1.698	2.162	2.621	3.158
0.1	0.932	1.120	1.349	1.732	2.195	2.668	
0.25	0.941	1.158	1.356	1.784	2.214	2.704	
0.5	0.941	1.164	1.368	1.811	2.258	2.742	
1	0.963	1.176	1.384	1.862	2.288	2.776	
2	0.969	1.179	1.493	1.891	2.302	2.809	
4	0.988	1.182	1.508	1.908	2.349	2.867	
8	0.993	1.197	1.529	1.937	2.384	2.911	
15	1.006	1.206	1.552	1.962	2.412	2.959	
30	1.019	1.221	1.574	1.991	2.463	2.986	
60	1.029	1.235	1.586	2.018	2.482	3.016	
120	1.038	1.268	1.602	2.068	2.519	3.034	
240	1.046	1.296	1.631	2.094	2.567	3.095	
480	1.052	1.319	1.663	2.123	2.594	3.114	
1440	1.063	1.337	1.698	2.162	2.621	3.158	3.008

Pressure Kpa	ΔH(mm)	Cummula ΔH(mm)	$H_f=H_i-\sum \Delta H$	$H_v=H_f-H_s$	$e_f=H_v/H_s$	$\Delta e=\Delta H/H_s$	$e=e_o-\Delta e$
7	0.00	0.00	20.00	10.86	1.19	0.000	1.250
7	-0.584	-0.584	20.584	11.63	1.27	-0.084	1.461
50	0.147	-0.619	20.619	11.48	1.26	-0.068	1.392
100	0.274	-0.345	20.345	11.21	1.23	-0.038	1.338
200	0.361	0.016	19.984	10.84	1.19	0.002	1.266
400	0.464	0.480	19.520	10.38	1.14	0.053	1.165
800	0.459	0.939	19.061	9.92	1.09	0.103	1.062
1600	0.537	1.476	18.524	9.38	1.03	0.161	0.953
800	0.150	0.687	19.313	8.77	0.96	0.229	0.975

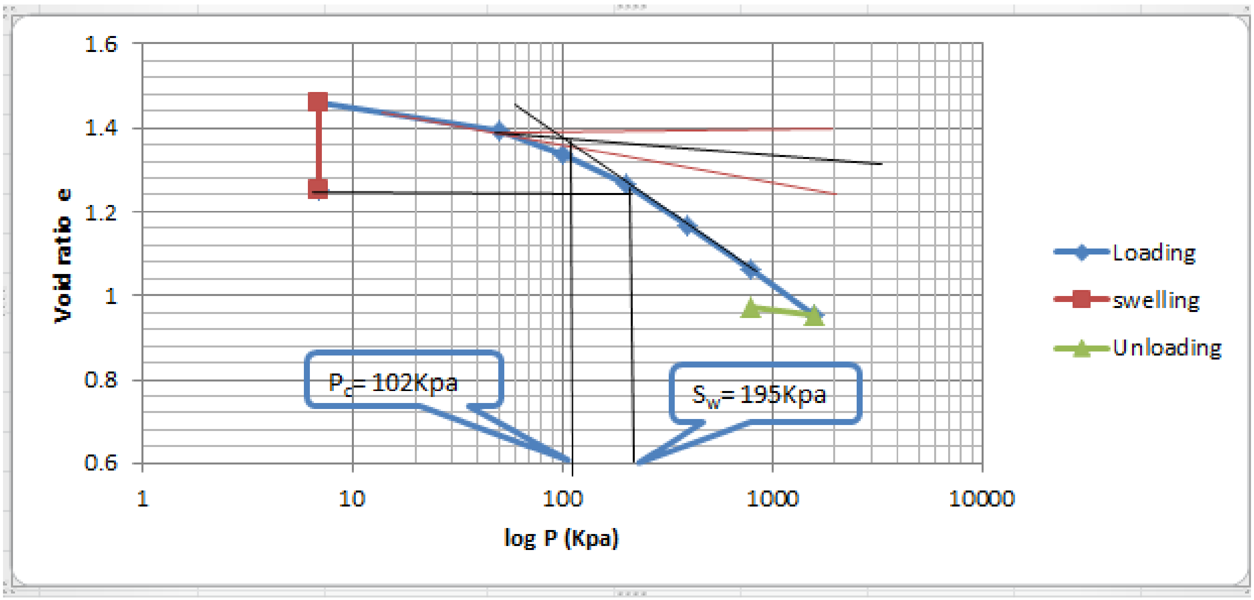


Table A.56: Consolidation dial reading for test Pit #2 @ depth of 2m

Setting load dial reading=mm							
Time(min.)	50Kpa	100Kpa	200Kpa	400Kpa	800Kpa	1600Kpa	Unloading
	Dial reading(mm)	Dial reading(mm)	Dial reading(mm)	Dial reading(mm)	Dial reading(mm)	Dial reading(mm)	1600Kpa
0	1.201	1.445	1.834	2.418	3.121	3.836	4.576
0.1	1.289	1.487	2.019	2.540	3.267	3.881	
0.25	1.324	1.524	2.071	2.684	3.324	4.030	
0.5	1.347	1.587	2.114	2.714	3.361	4.088	
1	1.354	1.592	2.153	2.731	3.394	4.106	
2	1.368	1.618	2.176	2.772	3.418	4.138	
4	1.371	1.654	2.186	2.811	3.442	4.147	
8	1.376	1.697	2.211	2.842	3.472	4.182	
15	1.385	1.709	2.234	2.861	3.512	4.208	
30	1.398	1.746	2.271	2.876	3.586	4.264	
60	1.407	1.762	2.294	2.931	3.621	4.310	
120	1.418	1.779	2.313	2.986	3.694	4.384	
240	1.429	1.792	2.344	3.034	3.724	4.428	
480	1.437	1.814	2.387	3.072	3.787	4.520	
1440	1.445	1.834	2.418	3.121	3.836	4.576	4.184

Pressure Kpa	ΔH(mm)	Cummula ΔH(mm)	$H_f=H_i-\sum \Delta H$	$H_v=H_f-H_s$	$e_f=H_v/H_s$	$\Delta e=\Delta H/H_s$	$e=e_o-\Delta e$
7	0.00	0.00	20.00	11.03	1.23	0.000	1.29
7	-0.899	-0.899	20.899	11.79	1.31	-0.085	1.42
50	0.244	-0.522	20.522	11.55	1.29	-0.058	1.38
100	0.389	-0.133	20.133	11.16	1.24	-0.015	1.33
200	0.584	0.451	19.549	10.58	1.18	0.050	1.27
400	0.703	1.154	18.846	9.87	1.10	0.129	1.18
800	0.715	1.869	18.131	9.16	1.02	0.208	1.06
1600	0.740	2.609	17.391	8.42	0.94	0.291	0.95
800	0.392	1.132	18.868	7.57	0.89	0.311	0.97

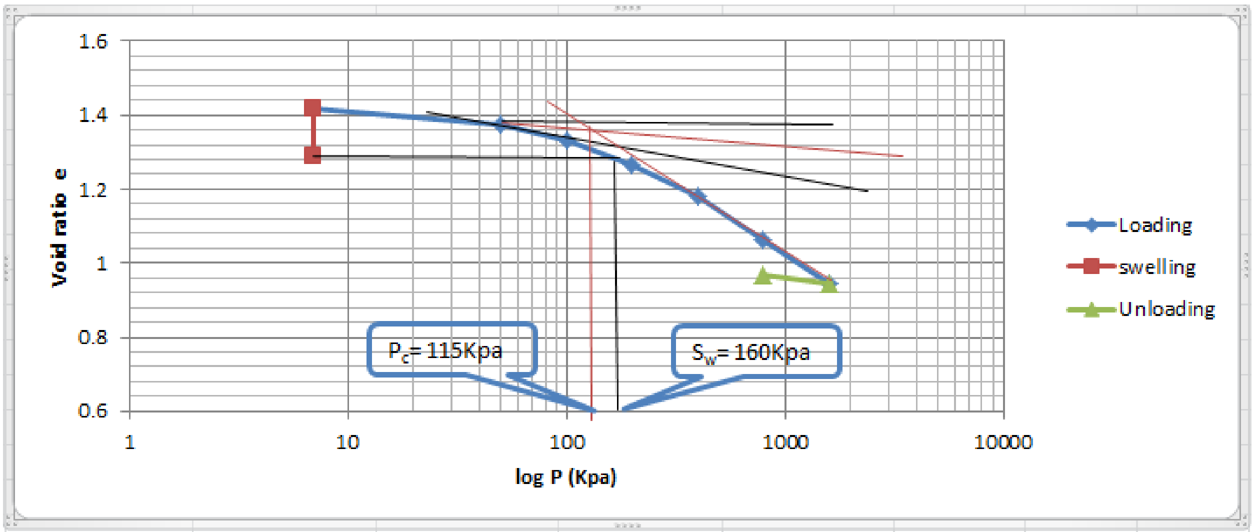


Table A.56: Consolidation dial reading for test Pit #3@ depth of 2m

Setting load dial reading=2.4mm							
Time(min.)	50Kpa	100Kpa	200Kpa	400Kpa	800Kpa	1600Kpa	Unloading
	Dial reading(mm)	Dial reading(mm)	Dial reading(mm)	Dial reading(mm)	Dial reading(mm)	Dial reading(mm)	1600Kpa
0	1.186	1.509	1.749	2.124	2.692	3.442	4.281
0.1	1.254	1.533	1.783	2.172	2.753	3.493	
0.25	1.313	1.557	1.836	2.258	2.816	3.534	
0.5	1.336	1.564	1.871	2.321	2.867	3.572	
1	1.349	1.581	1.894	2.343	2.892	3.596	
2	1.376	1.593	1.908	2.375	2.916	3.625	

4	1.391	1.603	1.947	2.391	2.953	3.652	
8	1.406	1.619	1.971	2.408	2.987	3.694	
15	1.426	1.636	1.996	2.469	3.011	3.739	
30	1.449	1.657	2.006	2.488	3.074	3.792	
60	1.456	1.674	2.031	2.526	3.119	3.856	
120	1.467	1.681	2.059	2.574	3.209	4.021	
240	1.483	1.698	2.088	2.614	3.304	4.134	
480	1.495	1.715	2.093	2.667	3.376	4.263	
1440	1.509	1.749	2.124	2.692	3.442	4.281	4.008

Pressure Kpa	ΔH(mm)	Cummula ΔH(mm)	$H_f=H_i-\sum \Delta H$	$H_v=H_f-H_s$	$e_f=H_v/H_s$	$\Delta e=\Delta H/H_s$	$e=e_o-\Delta e$
7	0	0	20	11.22	1.28	0.000	1.22
7	-1.214	-1.214	20.214	12.43	1.42	-0.138	1.36
50	0.323	-0.891	20.891	12.11	1.38	-0.101	1.33
100	0.240	-0.651	20.651	11.87	1.35	-0.074	1.30
200	0.375	-0.276	20.276	11.49	1.31	0.031	1.26
400	0.568	0.292	19.708	10.93	1.24	0.033	1.19
800	0.750	1.042	18.958	10.18	1.16	0.119	1.11
1600	0.839	1.881	17.119	9.34	1.06	0.214	1.01
800	0.273	1.112	18.888	9.08	0.88	0.287	1.08

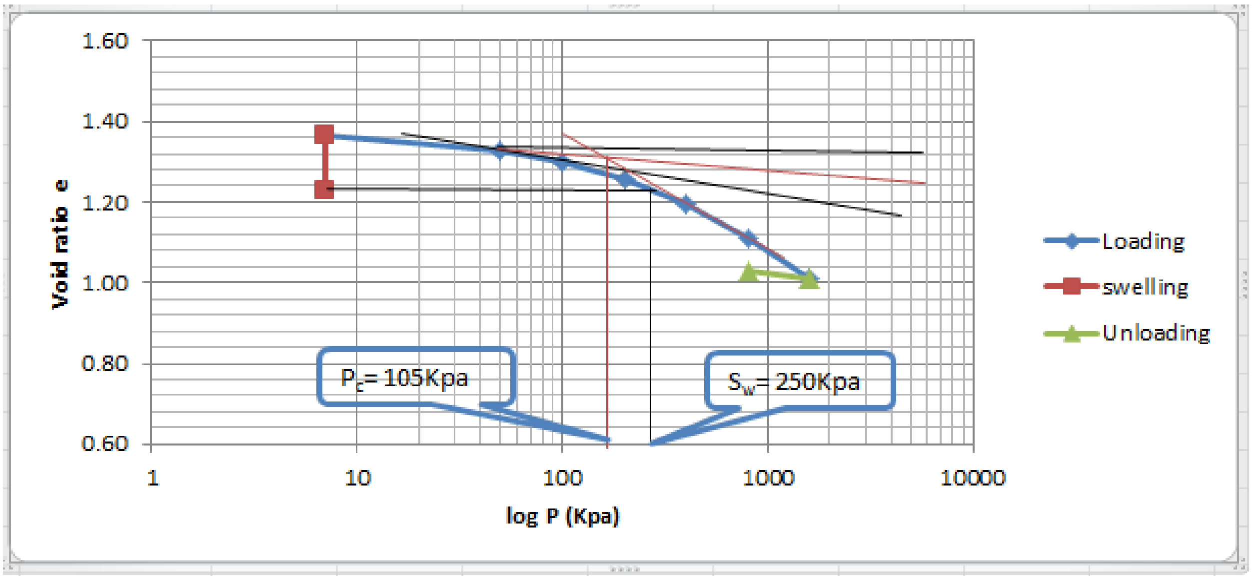
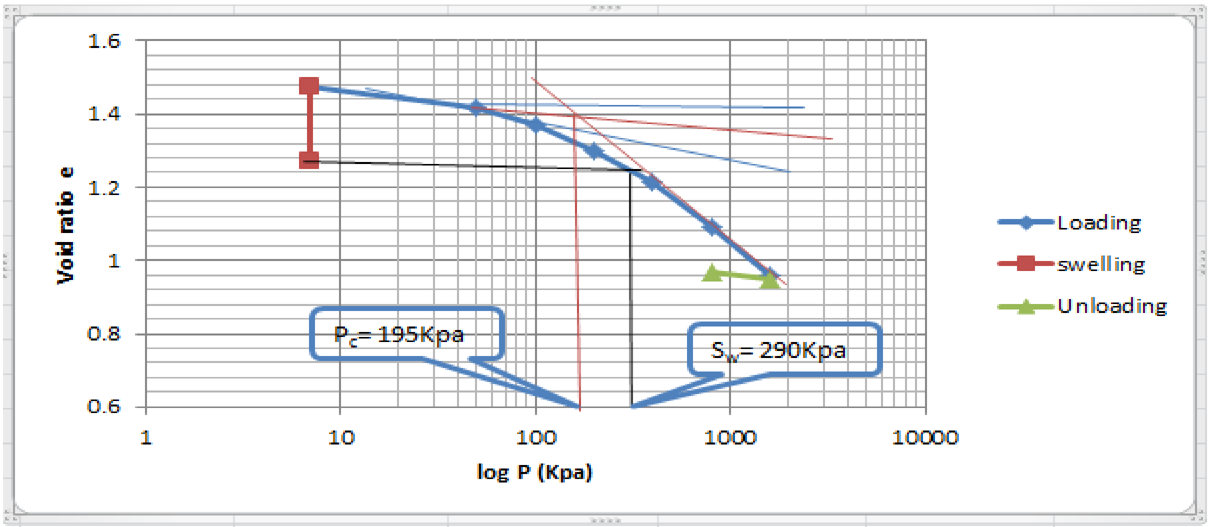


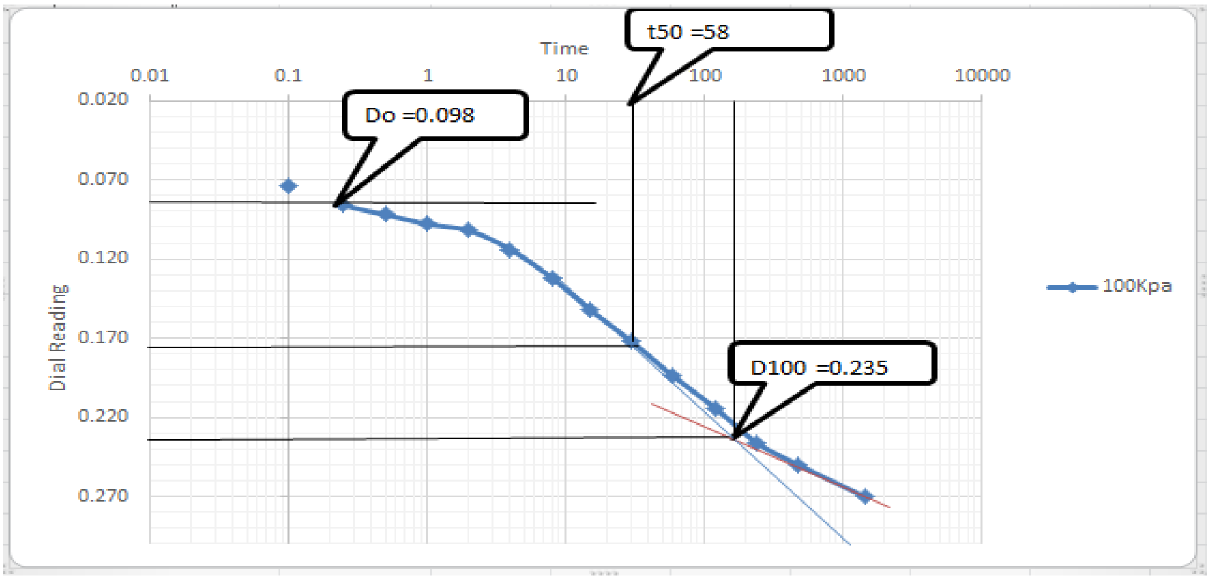
Table A.56: Consolidation dial reading for test Pit #4@ depth of 2m

Setting load dial reading=4.8mm							
Time(min.)	50Kpa	100Kpa	200Kpa	400Kpa	800Kpa	1600Kpa	Unloading
	Dial reading(mm)	Dial reading(mm)	Dial reading(mm)	Dial reading(mm)	Dial reading(mm)	Dial reading(mm)	1600Kpa
0	3.324	3.864	4.147	4.609	5.176	5.741	6.487
0.1	3.384	3.871	4.211	4.681	5.210	5.809	
0.25	3.424	3.889	4.257	4.697	5.273	5.931	
0.5	3.483	3.894	4.283	4.708	5.298	5.974	
1	3.521	3.907	4.311	4.754	5.327	5.996	
2	3.586	3.934	4.342	4.782	5.362	6.009	
4	3.613	3.957	4.361	4.819	5.379	6.031	
8	3.674	3.987	4.387	4.852	5.391	6.084	
15	3.694	3.996	4.410	4.886	5.409	6.110	
30	3.721	4.011	4.453	4.903	5.456	6.192	
60	3.744	4.054	4.492	4.942	5.492	6.267	
120	3.769	4.071	4.524	4.981	5.549	6.284	
240	3.794	4.092	4.573	5.043	5.621	6.321	
480	3.814	4.103	4.592	5.112	5.694	6.384	
1440	3.864	4.147	4.609	5.176	5.741	6.487	6.227

Pressure Kpa	ΔH(mm)	Cummula ΔH(mm)	$H_f=H_i-\sum \Delta H$	$H_v=H_f-H_s$	$e_f=H_v/H_s$	$\Delta e=\Delta H/H_s$	$e=e_o-\Delta e$
7	0	0	20	10.96	1.21	0.000	1.27
7	1.376	1.376	21.376	12.34	1.36	-0.085	1.30
50	0.540	-0.226	20.836	11.80	1.24	-0.025	1.24
100	0.283	0.057	19.943	11.51	1.21	0.006	1.21
200	0.462	0.519	19.481	11.05	1.16	0.057	1.16
400	0.567	1.086	18.914	10.48	1.09	0.120	1.09
800	0.565	1.651	18.349	9.92	1.03	0.183	1.03
1600	0.746	2.397	17.603	9.17	0.95	0.265	0.95
800	0.260	1.006	17.154	8.92	0.84	0.293	1.10

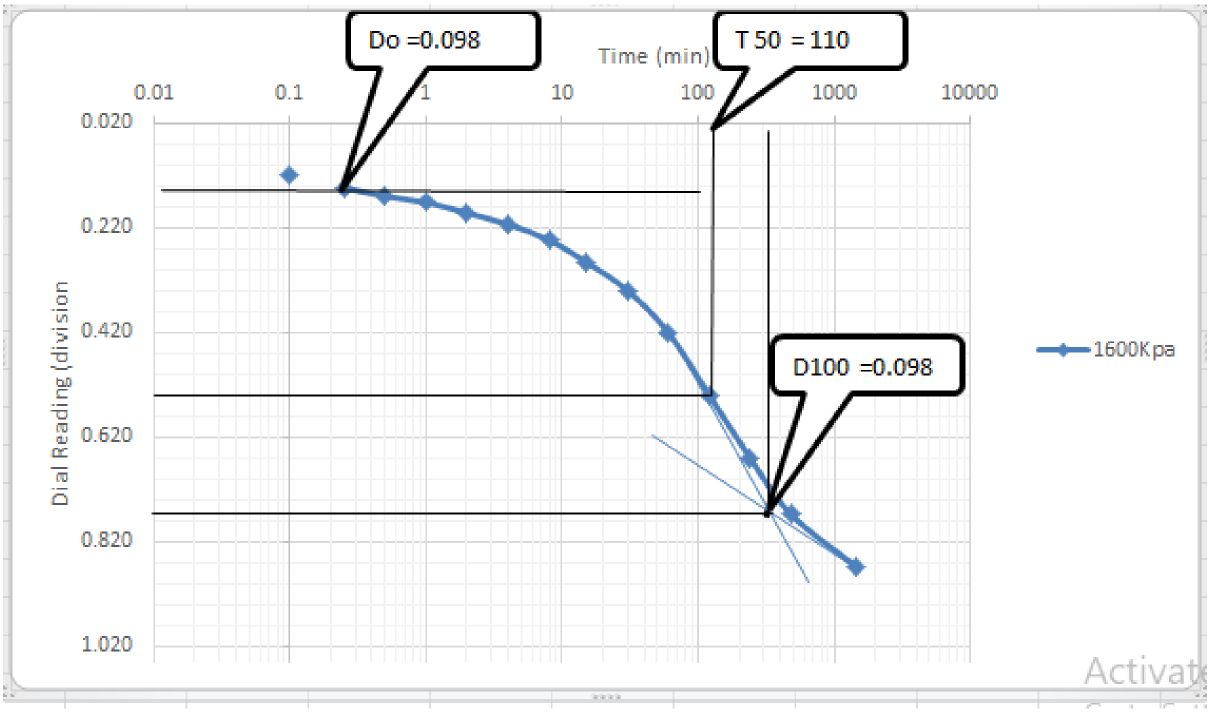


Sample calculation coefficient of consolidation for test pit #1



pressure(kPa)	D-100*0.002mm		Do-*0.002mm		D-50mm	T-50	ΔH	Δe= ΔH/Hs	e= eo-Δe	H=Ho-D50	H/2	CV
50	0.226	0.000452	0.091	0.000182	0.000317	35	0.242	0.01260417	1.1757	19.0415	9.52075	0.0510
100	0.235	0.00047	0.098	0.000196	0.000333	58	0.270	0.0140625	1.1616	19.0335	9.51675	0.0308
200	0.35	0.0007	0.14	0.00028	0.00049	75	0.366	0.0190625	1.1426	18.955	9.4775	0.0236
400	0.501	0.001002	0.2	0.0004	0.000701	115	0.572	0.02979167	1.1128	18.8495	9.42475	0.0152
800	0.68	0.00136	0.214	0.000428	0.000894	117.5	0.752	0.03916667	1.0736	18.753	9.3765	0.0147
1600	0.841	0.001682	0.136	0.000272	0.000977	238.5	0.960	0.05000	1.0236	18.7115	9.35575	0.0072

Sample calculation coefficient of consolidation for test pit #2



pressure(kPa)	D-100*0.002mm		Do-*0.002mm		D-50mm	T-50	ΔH	Δe= ΔH/Hs	e= eo-Δe	H=Ho-D50	H/2	CV
50	0.195	0.00039	0.098	0.000196	0.000293	115	0.204	0.010625	1.1777	19.0535	9.52675	0.0155
100	0.351	0.000702	0.108	0.000216	0.000459	18	0.364	0.018958333	1.1587	18.9705	9.48525	0.0985
200	0.585	0.00117	0.264	0.000528	0.000849	28	0.604	0.031458333	1.1273	18.7755	9.38775	0.0620
400	0.658	0.001316	0.306	0.000612	0.000964	57	0.690	0.0359375	1.0913	18.718	9.359	0.0303
800	0.718	0.001436	0.24	0.00048	0.000958	97	0.814	0.042395833	1.0489	18.721	9.3605	0.0146
1600	0.848	0.001696	0.16	0.00032	0.001008	110	0.868	0.045208333	1.0037	18.696	9.348	0.0073

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