

**ADDIS ABABA UNIVERSITY  
SCHOOL OF GRADUATE STUDIES**



**INTEGRATED GEOPHYSICAL AND GEOTECHNICAL  
INVESTIGATIONS FOR FUTURE SITE OF PEDIATRIC HOSPITAL  
BUILDING COMPLEX NEAR BISRATE GABRIEL CHURCH,  
SOUTHEAST OF ADDIS ABABA**

**BY:**

**AKLILU GEBREGZIABHER TEKLEAB**

**ADVISOR:**

**DR.SHIMELES FISSEHA**

**JULY, 2008**

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**AKLILU GEBREGZIABHER TEKLEAB**

**ADISOR-DR.SHIMELES FISSEHA**

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## **ABSTRACT**

A study is presented here at the future site of a pediatric hospital building complex near Bisrate Gabriel church, Southeast of Addis Ababa. Effort has been made by combining geophysical and geotechnical investigations to characterize the site for construction of a multistorey building. In this study, two test samples were taken for geotechnical investigation. The results showed that the soil has high content of clay minerals especially montmorillonite. The soil has high degree of expansiveness and high Atterberg limits. The shear strength parameters are very high. However, its shear strength decreases with saturation. Cohesion and angle of friction are 50 KN/m<sup>2</sup> and 17° respectively. The expansive soil has 1.33g/cm<sup>3</sup> maximum dry density 25% optimum dry density. On the other hand, the soil contains high free swell value that is 175%. The investigation was consisting of geophysical survey (2D Electrical resistivity imaging and Vertical Electrical Sounding, VES). The geophysical investigation indicates that soil with respect to their resistive value in their respective profile. Site reconnaissance and soil samples are the methodologies used to characterize the soil.

**Key words:** Geotechnical investigation, Geophysical investigation, test sample, montmorillonite, Cohesion, angle of friction, 2D Electrical resistivity imaging and Vertical Electrical Sounding, VES, and free swell.

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# **1. INTRODUCTION**

## **1.1. General**

This study gives the summary of results of integrated geophysical and geotechnical investigations carried out at the future site of a pediatric hospital building complex near Bisrate Gabriel church, southeast of Addis Ababa.

The integrated investigation was comprised of geophysical survey (2D- Electrical resistivity imaging and Vertical Electrical Sounding, VES), test pits at specified locations are carried out and laboratory soil tests were made based on soil samples collected. In addition, other geo-engineering assessments are also included in this study. Maximum possible effort and assessment were also carried out to collect all necessary geotechnical and geophysical information of adjacent structures including research papers on those areas.

The overall objective of the geo-technical surveys is evaluating the suitability of the site for construction of the stated multi-story building complex. Towards this end, the geophysical survey was, primarily, devised to obtain vivid pictures of the subsurface for better understanding of the soil rock conditions at depth. Test pits on different locations are opened based on the geophysical information obtained, soil samples were taken from the different horizons of the subsurface were analyzed in the geotechnical soil laboratory testing unit of TRANSPORT CONSTRUCTION DESIGN S.Co.(TCDS) for various engineering parameters determinations. Moreover, a review of historical earthquake data in and around Addis Ababa has been made for completeness. Based on these conclusions and recommendations about the site for foundation safety and best foundation type and construction are suggested.

## **1.2.Objective of the Study**

Addis Ababa is located in the western margin of the Main Ethiopian rift and consists of different volcanic rocks that range from basic to acidic composition. Construction of many high rise buildings are now under construction in many parts of the city. Therefore, special attention should be given to the inter relationship between engineering structures and the geotechnical properties of soils and rocks and the response of structures against geological processes.

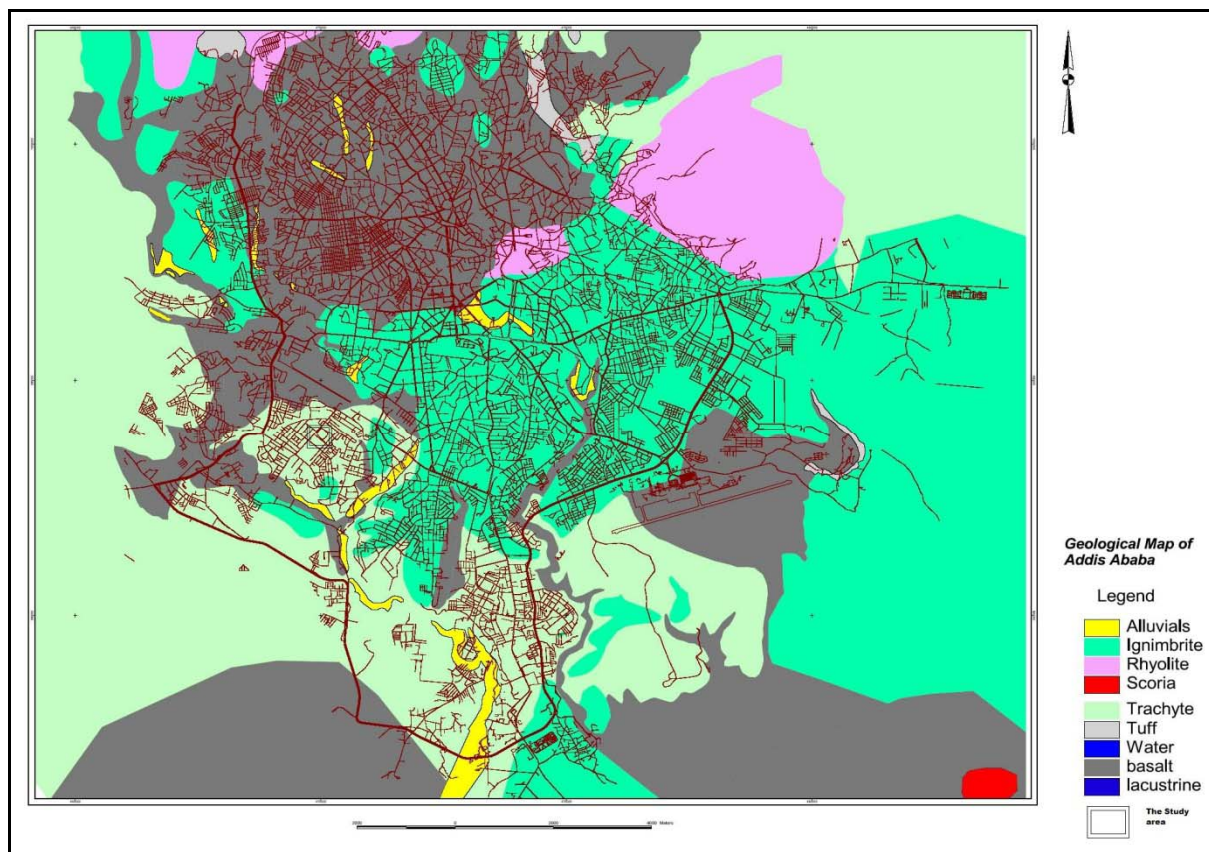
The objective of this study is to show the integrated applications of geophysical and geotechnical investigations of buildings in Addis Ababa, particularly for the future site of a pediatric hospital building complex near Bisrate Gabriel church, Southeast of Addis Ababa. A case study is carried out to evaluate the suitability of the site for construction of a multi-story building complex by integrately applying geotechnical and geophysical investigations. Geophysical investigation conducted is to obtain clear understanding of the subsurface nature of the soil and rock conditions at depth. Soil samples, taken from the different horizons of the subsurface were analyzed in the geotechnical soil laboratory of TCDS for various engineering parameters.

## **1.3.Organization of the Project**

This project is organized into 8 chapters. Chapter 1 presents the general description and geotechnical materials as well as response of engineering structures. Chapter 2 and chapter 3 discuss the geophysical and geotechnical investigations respectively. Experimental study is shown in chapter 6. Review of related literature is compiled in chapter 4 and chapter 5 deals with the methodology used. Chapter 7 presents the the laboratory tests and results on the two test samples. Conclusions and recommendations are given in chapter 8.

## 2. Geology and Tectonics of the Survey Area

Addis Ababa is located in the western margin of the Main Ethiopian rift and consists of different volcanic rocks that range from basic to acidic composition. The vicinity of the city is surrounded by *trachyte* and *rhyolite* hills and mountains. In the northern part, the Entoto mountain chains are composed of *rhyolite* and *trachyte* which are called the Entoto silicic of the Addis Ababa area. They are associated with the Alaji formation and rest on older basalts. Volcanism initiating the Alaji cycle occurred in late Oligocene - early Miocene times.



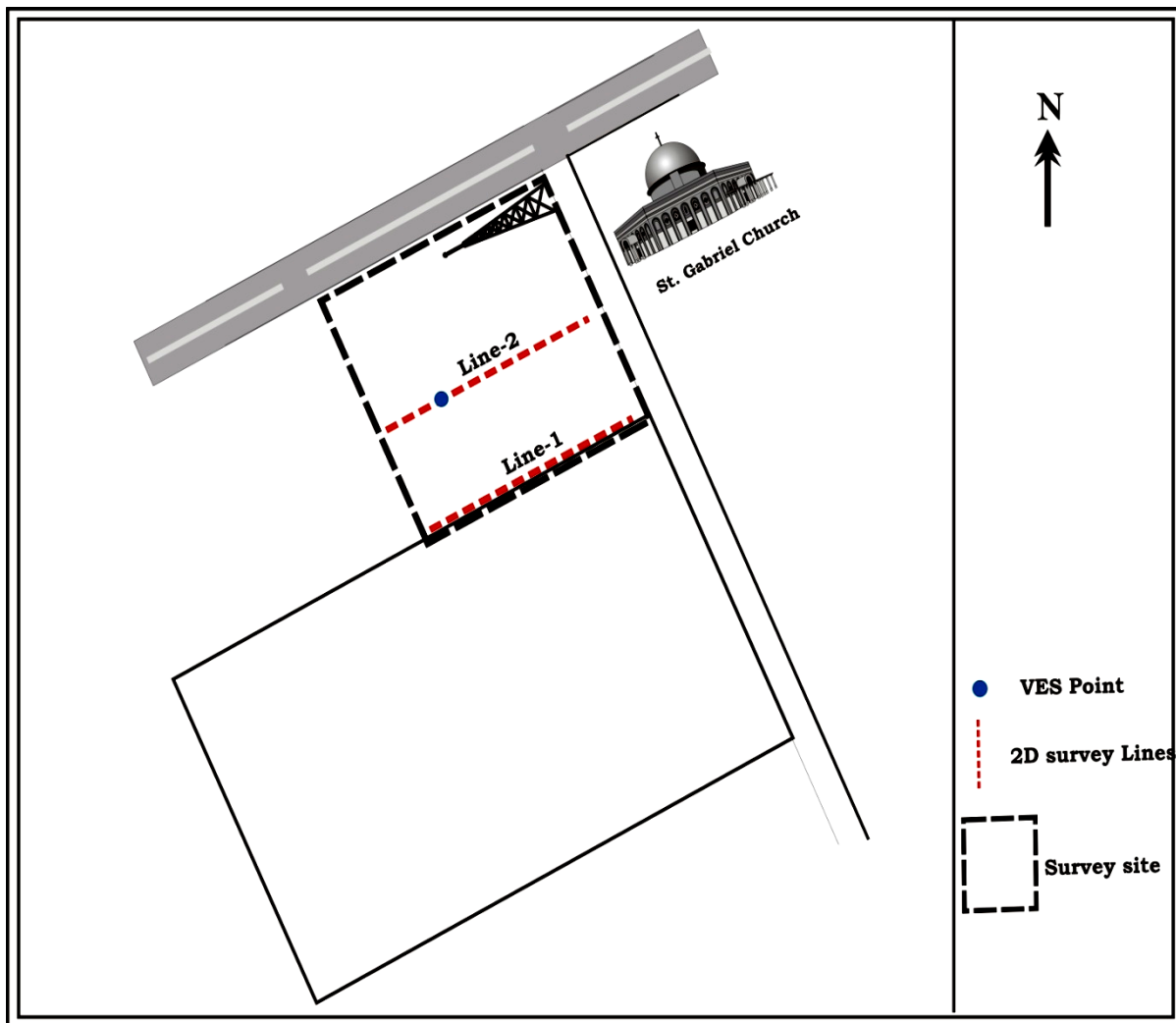
**Figure-1.1: Geological map of Addis Ababa and its surroundings.**

Volcanic mountains such as Wachecha in southwest, Furi in the southern and Yerer in the southeastern parts of Addis Ababa are mainly trachyte in composition. The basalts are outcropping in the central part of Addis Ababa and to the south and north of the Entoto hills some small patches of basalts are capping the Entoto silicic. Geologically, the study area may consist of two major lithologic units. These units (adopted from the geologic map of Addis Ababa), are: *welded tuff (Ignimbrite)*, and *Trachyte*.

### 3.GEOPHYSICAL INVESTIGATION

The Geo-electrical investigation is comprised of a Vertical Electrical Soundings (VES) and 2D Electrical resistivity imaging, along selected traverse lines within the land plot allocated for the future pediatric hospital building complex.

The Vertical Electrical Soundings (VES) was carried out at about the center of the survey site.



**Figure-2.1: Schematic Diagram Of The Survey Site.**

It is a focused single, point probing and primarily aims to map the depth to the top of a potential foundation rock, determine the thickness of the overburden soil and to correlate the result with that of the 2D resistivity image of the subsurface. In addition, it is the basis for performing economical and more reliable geotechnical investigation.

The 2D electrical resistivity mapping of the subsurface, on the other hand, gives more detail information that help to:-

- Construct the subsurface layer stratification,
- Determine the depth extent and relief of the foundation rock and,
- Reveal any concealed geologic structures, which may impose post construction hazards.

Table 1 gives the GPS coordinates, in UTM, of the sounding point and the start- and end-points of the 2D resistivity imaging survey lines.

**Table-1:GPS coordinate of the sounding point and Survey Lines.**

<b>E</b>	<b>N</b>	<b>Elev.(m)</b>	<b>Description</b>
469729	993618		VES Pt.
469744	993585	2314	Beginning of Line-1, 2D
469785	993632	2316	End of Line-1, 2D
469710	993605	2321	Beginning of Line-2, 2D
469771	993662	2323	End of Line-2, 2D

### **3.1.Data Acquisition**

The location of the Vertical Electrical Soundings (VES) point is at the center of the survey site. It primarily aims at mapping the depth to the top of a potential foundation rock. The resulting vertical section is used for determining the thickness of the overburden soil and correlating it's with that of the 2D resistivity image of the subsurface. The maximum half-electrode separation,  $(AB/2)$  attained within the area of investigation was 60m. This enabled to reach the top few layers that could have direct relevance for the engineering characterization of the survey site. However, the actual location the VES point and the directions of the survey advancement were determined as a compromise between available open spaces, for maximum extension of the electrode spreads towards guaranteeing adequate depth of penetration, and staying within the intended site of construction.

The measurement was done using PASI Earth Resistivity Meter and with Shlumberger electrode configuration.

The 2D Electrical resistivity imaging, on the other hand, provides all crucial parameters mentioned above and confirms their lateral continuity over the area under consideration.

For the 2D surveys, the Dipole-Dipole configuration was employed. This electrode array was chosen for its superiority in delineating lateral resistivity heterogeneities as compared to other electrode configurations.

In order to study the lateral resistivity variations in sufficient detail, the survey were carried in an arrangement that involves Seventy-two electrodes placed one meter apart and attached to multi-electrode consoles.

The profile lines run in ESE – WNW direction. With 648 data points, each profile lines cover the area deemed to be interesting for the survey. The actual position

of the traverse line is entirely in the premises of construction site.

The instrument used was a Syscal resistivity-imaging unit from IRIS instruments.

### **3.2.Data Analysis**

The resistivity interpretation software, WINRESIST, was used to determine the layer parameters from the measured apparent resistivity values of VES data. It can be used either in an interactive (user guided) or automated mode and handles both forward modeling and inversion of the measured data. The Measured curves are entered as apparent resistivity as a function of  $AB/2$  for Schlumberger soundings. Starting from a user provided initial guess, RESIST utilizes a ridge-regression inversion procedure and gives the formation resistivities and thicknesses of the subsurface electrical layers beneath the sounding point.

The software used in the electric data processing, *RESIST*, is a user-friendly modeling and inversion program and is based on the above concepts. It is an interactive, graphically illustrated, forward and inverse modeling program for interpreting resistivity-sounding data in terms of a layered earth (1-D) model. The Measured curves are entered as apparent resistivity as a function of  $AB/2$  for Schlumberger soundings.

Inversion models done by RESIST can have up to 7 layers while, forward models can have up to 10 layers. Parameters can be resistivity and thickness of layers, or resistivity and depth to bottom of layers. Processing and interpretations of the 2D resistivity imaging data was done using *RES2DINV* software. *RES2DINV* is a computer program that automatically determines the true resistivity model for the subsurface from the measured data.

### 3.3.Presentation and Interpretation of Results

#### 3.3.1.VES -Vertical Electrical Soundings

The measured apparent resistivity data is presented in the form of apparent resistivity curve on Figures 3.2. The depth section given in Figure 3.3 presents the results of the data analysis from the VES investigation as a model response.

Referring to Figure 3.3, the subsurface, in the study area, is represented by four distinct geo-electric layers. The topmost layer, with resistivity range of 119Ωm, is interpreted as the response of the dry sandy soil fill cover. The depth is about 0.5m.

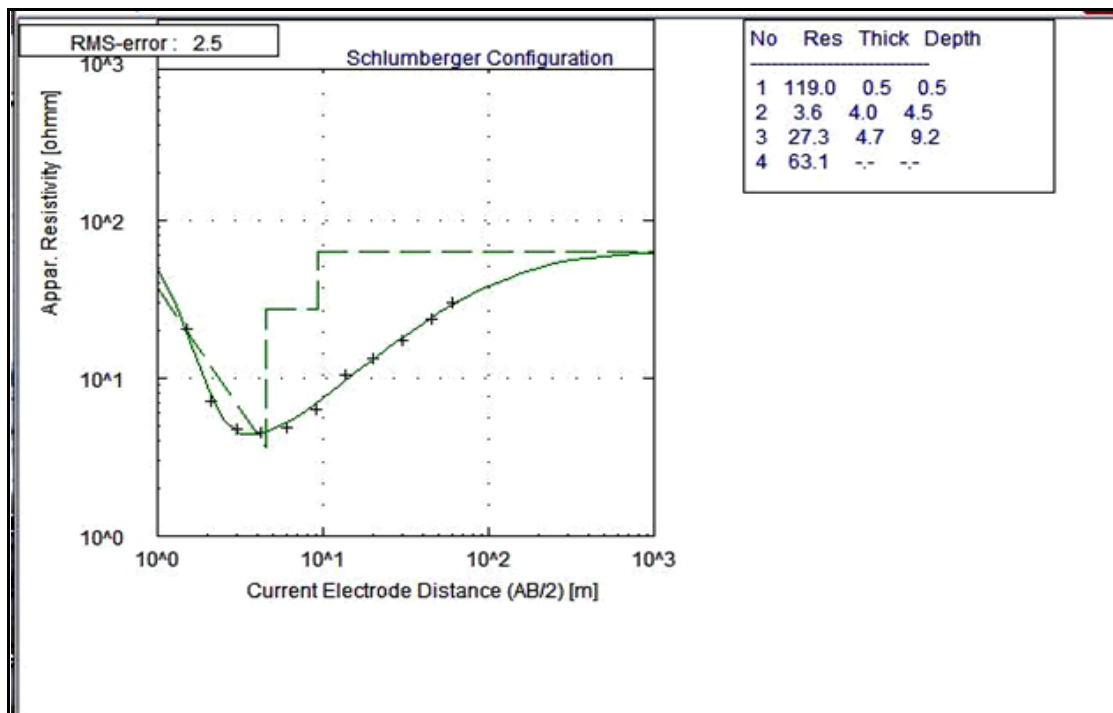
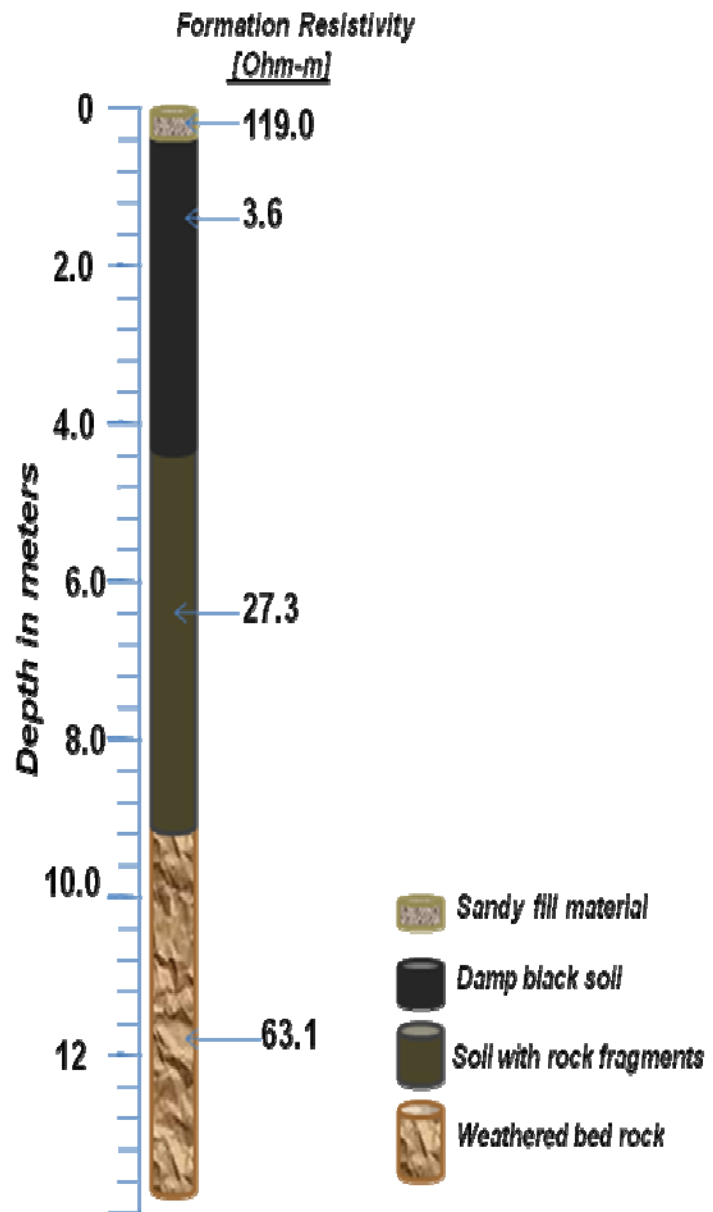


Figure-3.2: Vertical Electrical Soundings VES-curve

### ***Depth section from VES.***

**Site: Future site of Pediatric Hospital  
near Bistrate Gabriel, SW Addis Ababa**



**Figure-3.3: Interpreted Depth Section**

The second horizon in the sequence is designated by a resistivity value of  $3.6\Omega\text{m}$  and is attributed to *moist black cotton soil*, which is highly expansive in nature. As shown in the depth section, the depth to the bottom of this conductive soil goes down to 4.5m. This is underlain by a 4.7m thick soil horizon of a different geo-electric nature. The interpreted formation resistivity,  $27.3\Omega\text{m}$ , corresponding to this third layer is suggestive of intercalation soil with *weathered rock fragments*.

The substratum in the section is characterized by a resistivity response of  $63.1\Omega\text{m}$  and may represent *acidic volcanic likely Ignimbrite*.

### **3.3.2. Resistivity Imaging**

Figures 3.4 and 3.5 present, from top to bottom represent the plots of the raw apparent resistivity pseudo-section,

- (a) the calculated model response,
- (b) and the interpreted 2D model
- (c) Both lines run in SW – NE direction.

The plots of the raw apparent resistivity field data of both profiles, on top, represent a realistic picture of both the lateral and vertical variations in resistivity of the subsurface.

#### **Line-1 (Figure 3.4)**

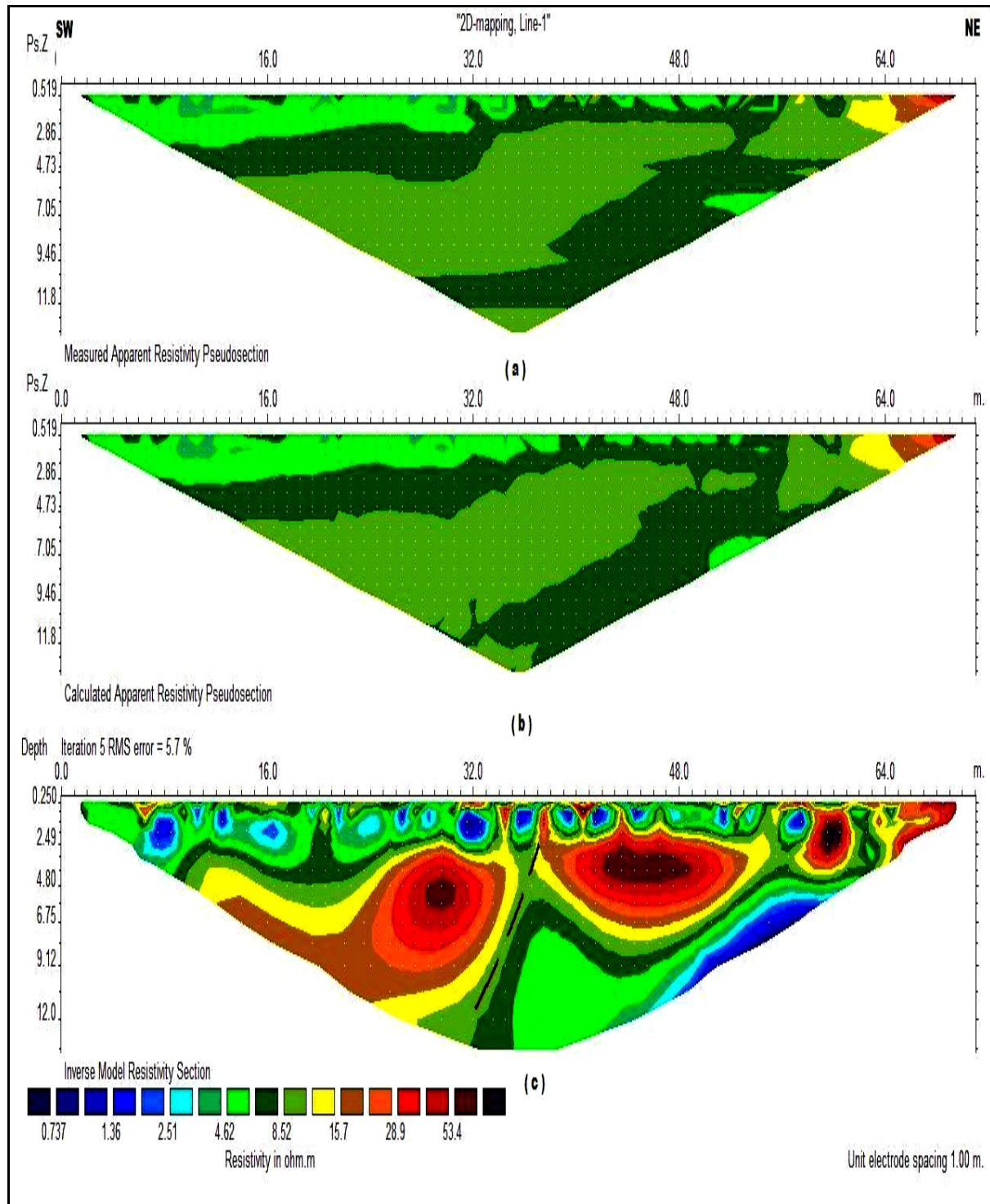
The 2D resistivity imaging survey along Line-1 was done at a level, which is about 2.5m lower than the actual ground surface, at the extreme eastern limit of the site. As there is a huge construction site on the immediate eastern border having excavated ground for foundation, this line was laid, deliberately, at the lower surface so that to facilitate further penetration to deeper horizons.

Referring to the bottom section, the 2D geo-electric model (c), the soil horizons are mapped as a *conductive top cover*, below  $5\Omega\text{m}$ , followed by an intermediate portion with a range of 5 -  $15\Omega\text{m}$ . The overburden soils exhibit a pronounced

thickness variation along the profile.

At the SE end, the depth reaches about 6.5m from the surface of measurement. Thinning out progressively along the survey line, the soil ultimately vanishes at the NE end where the response from the underlying weathered rock crops out at the measurement plain which is about 2m from the ground surface.

With the maximum amplitude of 55Ωm, the *weathered and likely fractured rock* comes into picture beneath the top soil. The rock mass has been mapped to be shallow at the NE end and dips, towards SW, to a depth of about 6m from the level of measurement.



**Figure-3.4: 2D Resistivity Section along Line-1**

The low amplitude in formation resistivity and the discontinuous pattern of its

image may indicate that the rock has been affected by intensive weathering associated with faulting and/or fracturing. The pattern may also suggest that such features could result from cluster of separated boulders rather than a continuous rock bed.

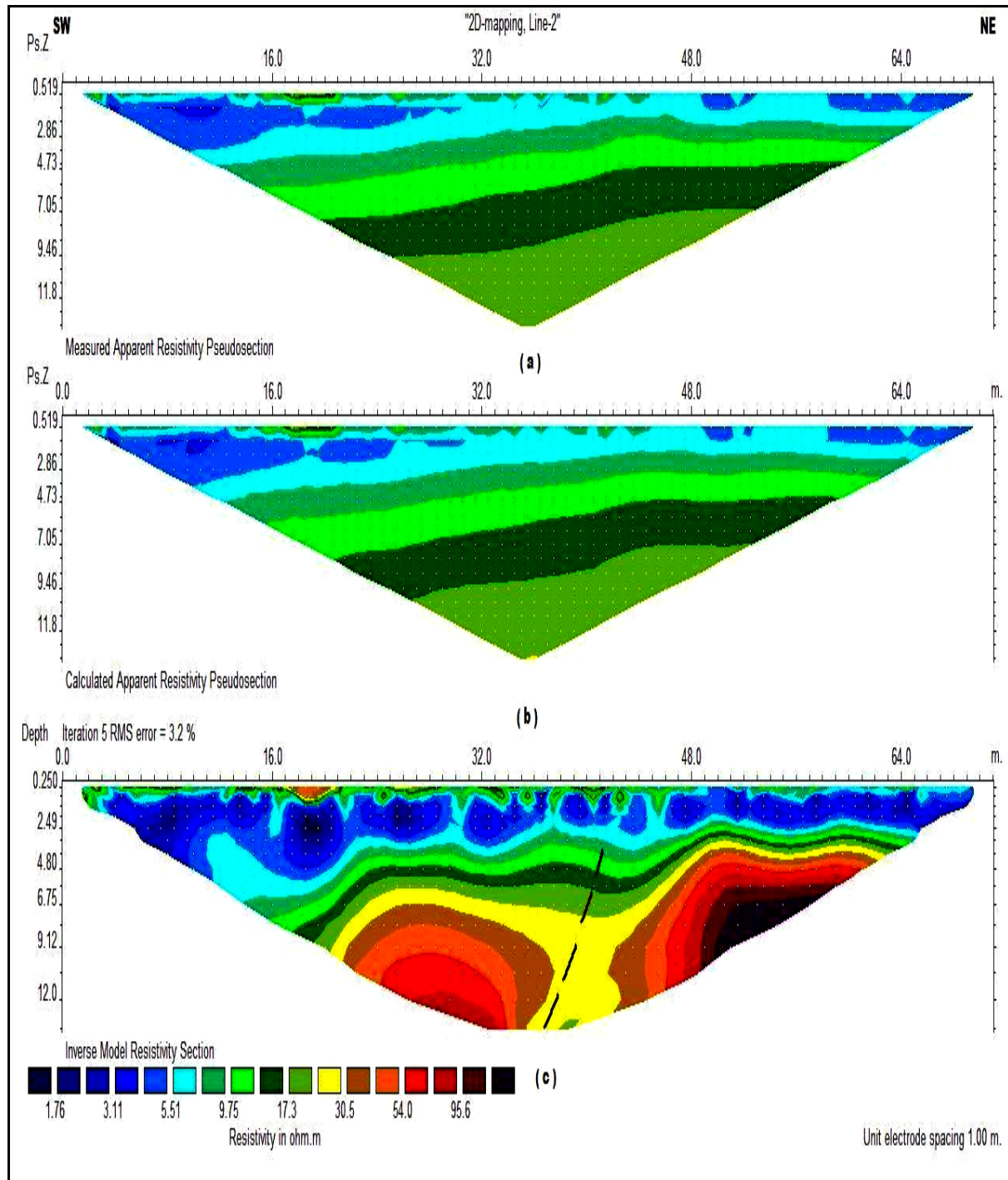
From about the middle of the line (at 35m) towards the NE end, there appear to be another low resistivity ground underneath the rock mass. The very low resistivity values (1 – 5  $\Omega\text{m}$ ) are suggestive of a conductive expansive soil horizon (highly expansive soil), which is possibly water logged.

### **Line-2 (Figure 3.4)**

The subsurface resistivity image along line-2 appears, basically, identical to that resulted from line-1, except few details and depth variation due to measurement level difference, figure 3.5.

The top 3m, from the beginning (SE) point [473210E, 996375N] of the line, the soil horizon is represented by conductive responses, generally below 10 $\Omega\text{m}$ . The measured formation resistivity, then, exhibits a sudden rise in response, beyond station 60m in the NW direction and thought to indicate *some structural feature*.

On figure 3.5, the apparent resistivity profile plot also shows discontinuity between stations 40NW and 50NW, at the 5m depth level. Though it requires further evidence to conclude, the lateral resistivity profiles have given strong indication for the presence of a subsurface structure, possibly, a major fracture.



**Figure-3.5: 2D resistivity section along Line 2.**

#### **4.GEOTECHNICAL INVESTIGATION**

## **4.1. Background**

This section presents the engineering part of an integrated geophysical and geotechnical investigation conducted on the specified site. As discussed in the main introduction, the principal objective of the study is to come up with some of the crucial parameters for final design of foundation and recommendation of construction procedures to be followed. The engineering geotechnical investigation was, therefore, carried out based on the information from the results of the geophysical investigation analysis. This was so done, systematically, in order to optimize the information about the site in an economically viable way.

Accordingly, the geotechnical investigation, along with the geophysical analysis, focuses on the following key factors:

- To assess the suitability of the site for the proposed building,
- To determine the thickness and the engineering characteristics of each dominant layer as this will have significant effect for design,
- Identify the level of ground water level during the investigation time (almost based on geophysical information), if encountered,
- Prepare geotechnical design profile, describing all necessary information,
- possible, recommend the possible type of foundation to be used and the type of construction procedures to be followed,

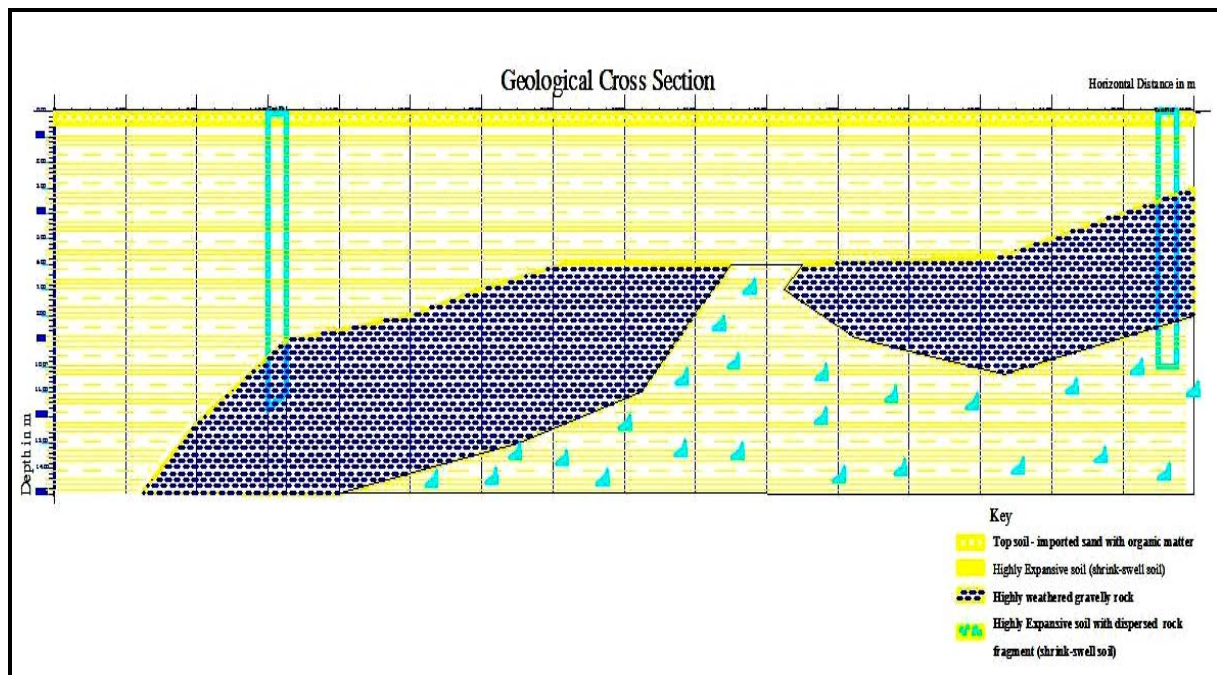
The geotechnical field investigation was conducted one week after geophysical investigation is completed. Accordingly, the methodologies and tests employed described here are used to specify the additional engineering characteristics of the subsurface strata. Moreover, this section presents the analysis and interpretation of results. Finally, recommendations regard to the bearing layer, the suitability of the site, the type of foundation and construction procedures to

be followed will be described.

## 4.2. Geotechnical Characteristics

### 4.2.1. Geotechnical profile

Based on the 2D resistivity sections and the vertical depth section from vertical electrical sounding (VES) together with soil profiling analysis, the following generalized geo-technical layers are identified. Details on the results of the geotechnical soil tests are given in Appendix-1. Soil strata lines, the soil profile and most probable boundaries between different soil types are shown on figure 4.1.



**Figure-4.1: Integrated subsurface profile**

## 4.3. Soil Classification

### 4.3.1. Top soil - Imported sand with organic matter (Central Part)

The upper soil layer is constituted of thin imported sand underlain by lean clay with organic material which is dark brown to black, and some residues from previous demolished construction activities are detected from the study. These layers are very thin and have a maximum thickness of 0.5m. This covers the top of most part of the project site and is expected to be completely excavated and removed during site clearing.

#### **4.3.2.The upper thick highly expansive soil (shrink-swell soil)**

The dominant top portion as shown exposed along the trench wall found at the eastern extreme of the site is found to be black expansive soil, which has a unique property of swelling and shrinking during rainy and dry season's fluctuations. These soils are highly expansive and are too problematic for foundation bearing and construction. The thickness increases from 2.6 m to about 9m in southeast to northwest direction and possibly water logged.

#### **4.3.3.The lower thin seam of highly expansive soil (shrink-swell soil)**

Even though the engineering characteristic might be the same as the black expansive soil. Another soil layers is found below the highly weathered rock, in the SE half of line 1, figure 3.4. Such thin seams which could be water logged, usually makes design and construction more difficult and needs proper consideration.

#### **4.3.4.The highly weathered rock**

Underlying the expansive soil, a highly to completely weathered, decomposed and disintegrated (to gravel and sand size) rock layer. The depth to this rock increases from NW to SE and exhibits discontinuity around the middle portion of the site. This is expected to be the bearing strata even though the layer thickness is inconsistent and exhibits an angle of internal friction of  $17^{\circ}$  and

cohesive strength of 50kPa.

## **5.REVIEW OF RELATED LITERATURE**

### **5.1.General**

Geotechnical investigation includes investigating existing subsurface conditions and materials; determining their physical(mechanical) and chemical properties that are relevant to the project considered, assessing risks posed by site conditions; designing earthworks and structure foundations; and monitoring site conditions, earthwork and foundation construction. A typical geotechnical engineering project begins with a review of project needs to define the required material properties. Then follows a site investigation of soil, rock, fault distribution and bedrock properties on and below an area of interest to determine their engineering properties including how they will interact with, on or in a proposed construction. Site investigations are needed to gain an understanding of the area in or on which the engineering will take place.

Investigations can include the assessment of the risk to humans, property and the environment from natural hazards such as earthquakes, landslides, sinkholes, soil liquefaction, debris flows and rock falls.

A geotechnical engineer then determines and designs the type of foundations, earthworks, and/or pavement subgrades required for the intended man-made structures to be built. Foundations are designed and constructed for structures of various sizes such as high-rise buildings, bridges, medium to large commercial building, and smaller structures where the soil conditions do not allow code-based design. Foundations built for above-ground structures include shallow and deep foundations.

In geotechnical engineering, soils are considered a three-phase material composed of rock or mineral particles, water and air. The voids of a soil, the spaces in between mineral particles, contain the water and air. The engineering

properties of soils are affected by four main factors: the predominant size of the mineral particles, the type of mineral particles, the grain size distribution, and the relative quantities of mineral, water and air present in the soil matrix. Fine particles (fines) are defined as particles less than 0.075 mm in diameter.

## **5.2. Soil**

The term Soil has various meanings, depending upon the general field in which it is being considered. In Geology, soil represents the products of past surface processes where as in engineering, it is a material that can be:

- ▶ built on: foundations to buildings, bridges.
- ▶ built in: tunnels, culverts, basements.
- ▶ built with: roads, runways, embankments, dams.
- ▶ supported: retaining walls.

Engineers are primarily interested in soil's mechanical properties: strength, stiffness, permeability. These depend primarily on the nature of the soil grains, the current stress, the water content and unit weight. It is necessary to adopt a formal system of soil description and classification in order to describe the various materials found in ground investigation.

### **5.2.1. Basic characteristics of soils**

Soils consist of grains mineral grains, rock fragments, etc. with water and air in the voids between grains. The water and air contents are readily changed by changes in conditions and location. soils can be perfectly dry (have no water content) or be fully saturated (have no air content) or be partly saturated (with both air and water present). Although the size and shape of the solid (granular) content rarely changes at a given point, they can vary considerably from point to point.

First of all, consider soil as an engineering material and is a particulate material. It is important to understand the significance of particle size, shape and composition, and of a soil's internal structure or fabric.

### **5.2.2. Soil Properties**

The following properties of soils are used by geotechnical engineers in analysis of site conditions and design of earthworks, retaining structures, and foundations. Soils contain three components, which may be characterized as solid, liquid, and gas.

The solid components of soils are weathered rock and (sometimes) organic matter. The liquid component of soils is almost always water (often with dissolved matter), and the gas component is air. The volume of water and gas is referred to as the void (<http://physics.uwstout.edu/geo/sect6.htm>).

### **5.2.3. Soil Characterization**

Most soil classification systems used in construction classify soils based upon two experimental characterizations of soil. These two measurements are:

- (1) a grain-size distribution curve or gradation curve, and
- (2) the Atterberg limits (or soil consistency).

The grain-size analysis can be either mechanical or with a hydrometer analysis.

### **5.2.4. Soil classification**

Soil classification deals with the systematic categorization of soils based on distinguishing characteristics as well as criteria that dictate choices in use. Soil classification is a dynamic subject, from the structure of the system itself, to the definitions of classes, and finally in the application in the field.

Soil classification can be approached from the perspective of soil as a material and soil as a resource. Engineers, typically Geotechnical engineers, classify soils according to their engineering properties as they relate to use for foundation support or building material. Modern engineering classification systems are designed to allow an easy transition from field observations to basic predictions of soil engineering properties and behaviors (Baldwin, M., Kellogg, C. E., & Thorp, J. , 1938).

#### **5.2.4.1.Unified Soil Classification System**

The most common engineering classification system for soils in North America is the Unified soil classification system(USCS). The USCS has three major classification groups:

- (1) coarse-grained soils (e.g. sands and gravels);
- (2) fine-grained soils (e.g. silts and clays); and
- (3) highly organic soils (referred to as "peat").

The USCS further subdivides the three major soil classes for clarification. This system of soil (i.e., clastic sediment) classification was developed by A. Casagrande (1948) and utilized by the U.S. Army Corps of Engineers (1953) and the U.S. Bureau of Reclamation (1974). Today, it is commonly used by engineers in many countries, including Canada. Coarse textured soils (gravels, sand) are classified according to their particle sizes and particle size distribution. Finer soils are classified as "silt" or "clay" according to their plasticity-compressibility characteristics as determined by consistency tests. Preliminary estimates of soil characteristics for many engineering purposes can be made from tables keyed to the unified soil classes (Van Horn, 1968,). A guide to the field classification of soils (after U.S. Army Corps of Engineers, 1953) is

also presented in Costa and Baker (1981, p. 210-211); laboratory procedures for classification are given in ASTM (1980).

#### **5.2.4.2.AASHTO Soil Classification System**

The AASHTO Soil Classification System was developed by the American Association of state highway and transport officials, and is used as a guide for the classification of soils and soil-aggregate mixtures. A full geotechnical engineering soil description will also include other properties of the soil including color, in-situ moisture content, in-situ strength, and somewhat more detail about the material properties of the soil than is provided by the USCS code. The Unified Soil Classification System (USCS) is a soil classification system used in engineering and geology disciplines to describe the texture and grain size of a soil. The classification system can be applied to most unconsolidated materials, and is represented by a two-letter symbol. Each letter is described below with the exception of Pt. (Hogentogler, C.A. and Terzaghi, K., 1929).

### **5.3.Geology of Addis Ababa Soils**

The origin and mineralogical composition of black and red clay soils of Ethiopia have been studied by (Morin and Parry, 1971). According to the investigation, black and red clay soils have formed over tertiary to recent basaltic volcanic rock which cover the greater part of Ethiopian plateau.

Black clay soils are found in areas with poor drainage and low to moderate rainfall and contain montmorillonite as the principal clay mineral with accessory kaolinite and halloysite. The red clay soils have developed where rainfall is more plentiful and drainage is good, and contain kaolinite and halloysite as the principal clay mineral with accessory montmorillonite.

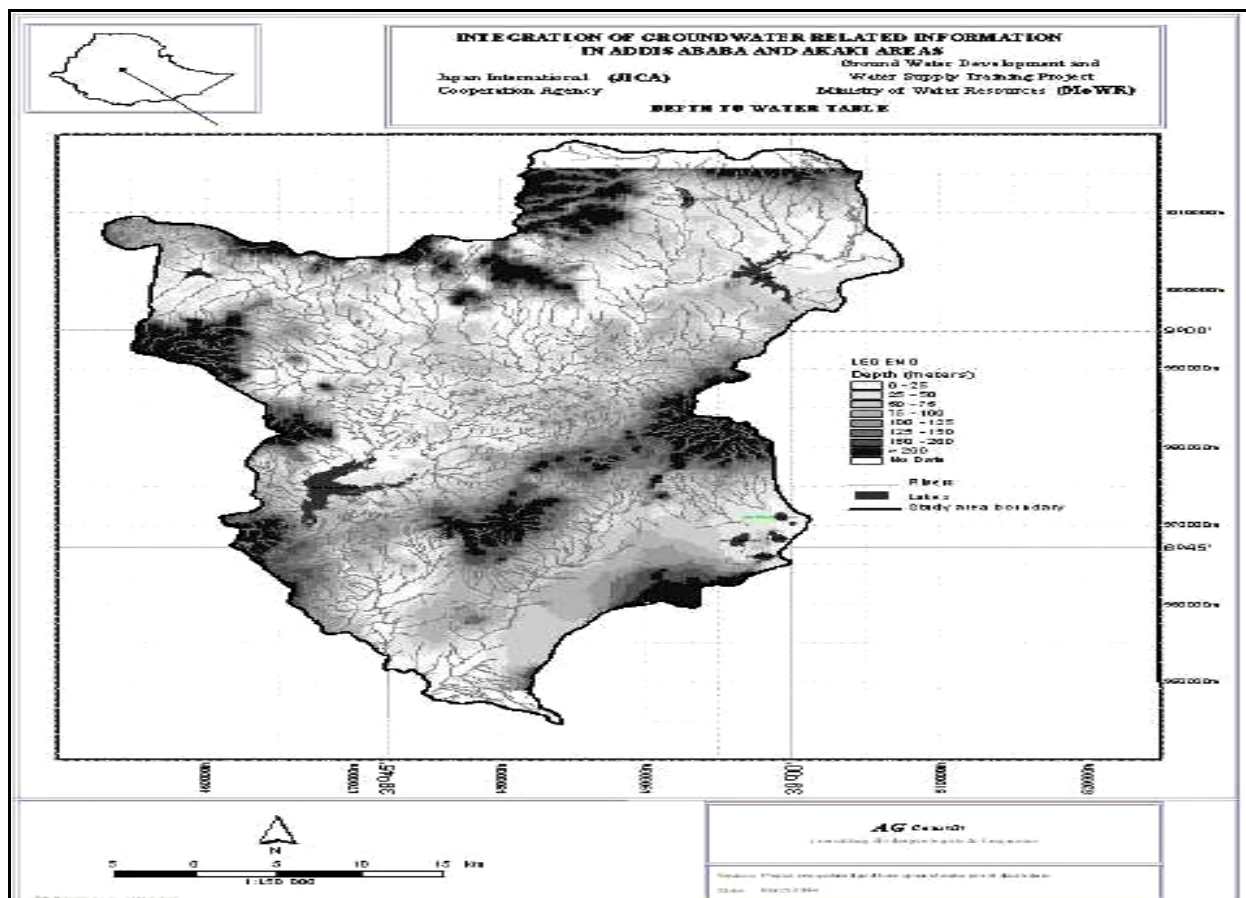
Addis Ababa soils, as part of Ethiopian soils, exhibit the above origins and properties. More specifically the engineering geological soil unit in Addis Ababa

area are grouped into their genetic soil units as alluvial, alluvial fan, colluvial, residual and lacustrine soils (Kenney, T.C, 1967).

### 5.4. Ground water table

A possible ground water indication, based on geotechnical and geophysical investigation studies, may be expected at the expansive clay seam, beneath the weathered rock, mapped on the SE portion of the site along line 1. The resistivity range is highly indicative of waterlogged conductive soil.

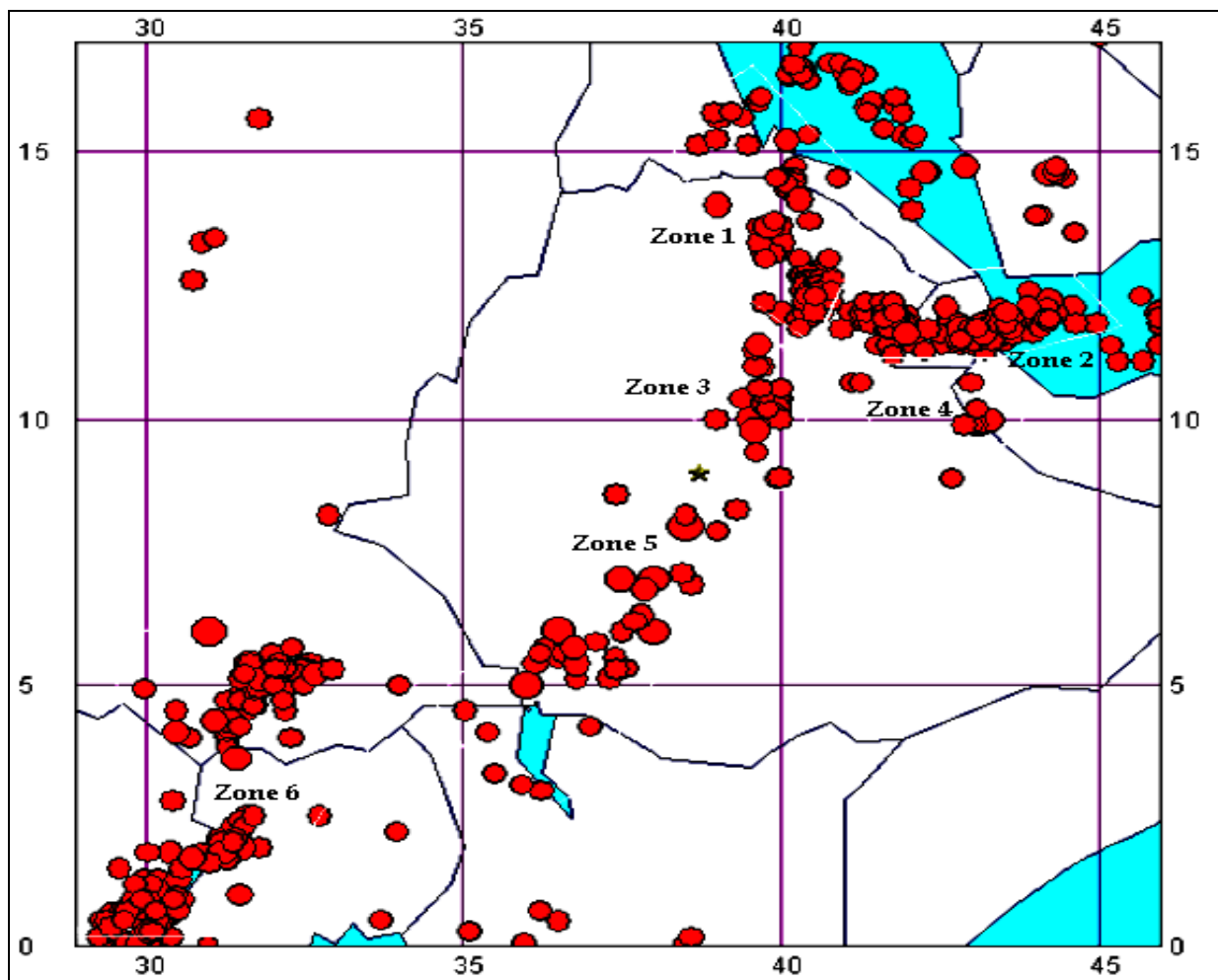
For general reference, a ground water table map of Addis Ababa is given on figure 5.4.1.



**Figure-5.1: Integrated Groundwater map of Addis Ababa.**

## 5.5. Seismic hazard Considerations

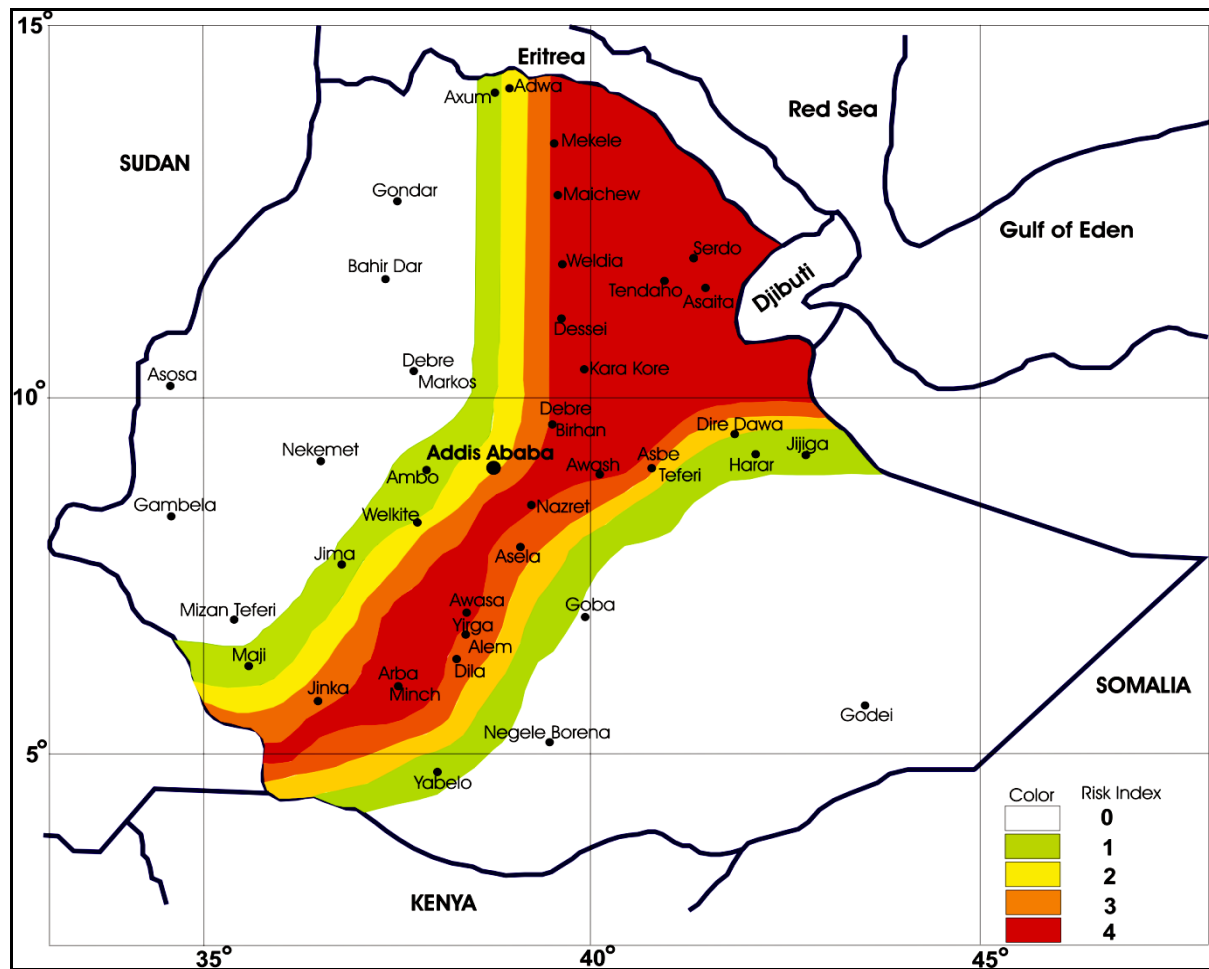
It is possible to say that earthquakes and volcanic eruptions are some of the potential natural disasters in Ethiopia as the Gulf of Aden & the Red Sea oceanic rifts and the East African Rift System meet in the Afar triple junction (Fig. 5.2). The location of Addis Ababa, in general, is at the rift margin toward the western plateau where damaging earthquake activity is possible either within the city or from other active source in the neighborhood.



**Figure-5.2: Seismicity of the Horn of Africa.**

On figure 5.2, the red circles represent earthquakes that occurred for the last 100 years in the region. The sizes of the circles are proportional to magnitude and all events with magnitude 4.5 and above are displayed.

The white polygons are area Seismic zones, where zone-1 is Red Sea; Zone-2 Gulf of Aden; Zone-3 Ankober region, Zone-4 the Hargessa or Aisha Block; Zone-5 southern Ethiopia and Zone-6 Southern Sudan. The black star shows the location of Addis Ababa.



**Figure-5.3: Seismic risk zoning in Ethiopia.**

The Ethiopian Building Code Standard (EBCS) seismic provisions are based on a return period of 100 years. However, the Code does not provide for the potential influence of varying predominant periods of the incoming seismic waves, proper consideration of this hazard assessment is highly recommended for foundation design.

## **6.METHODOLOGY**

### **6.1.Site Reconnaissance**

During the site visit made parallel with the geophysical investigation of the site, a long open trench along line-1 near the foundation work of the newly under-construction multistory building, located -2.5m below ground surface of the site under consideration, was found out. In addition the surface of the new site is paved by imported sand to cover the exposed layer, which will turn into mud during the rainy season, for horse playing purpose. Which as it was later revealed by the vertical electrical sounding and 2D resistivity explorations, constitute a total thickness of 0.5m including the soft organic matter found near the surface.

### **6.2.Soil Sampling and Testing**

Due to space limitation, the maximum possible depth of investigation for the 2D resistivity imaging was 14.5m below the ground surface. Based on careful analysis of the results of the geophysical methods, two soil-sampling spots were selected along Line-1 and samples were collected from different vertical interval. The sole purpose of this test is to characterize the engineering characteristics of the two dominantly layered horizons.

Sample-1 was obtained from dominantly expansive soil layers, as indicated on the 2D resistivity sections. Accordingly, the following tests are performed: soil classification and index tests such as: Gradation, Atterberg limit tests: liquid limit, plastic limit, shrinkage limit, free swell, and natural moisture content to characterize the types of soil in the profile was performed. In addition, some previous studies made around these areas for these types of problematic soils were collected, studied and incorporated.

Sample-2, which is a highly fractured rock layers as it is indicated on Figure 3.4 and 3.5, the following tests are performed: Soil classification and index tests

such as: Gradation, Atterberg limit tests: liquid limit, plastic limit, and natural moisture content. In addition direct shear strength test to characterize the types of soil in the profile was performed. The results of the soil test are annexed in appendix-1.

## **7.Experimental Study**

### **7.1.General**

The laboratory work has been performed for the various geotechnical analyses to provide recommendations for construction at the site. Depending on the seismicity of the site, and the specific characteristics of the site, various seismic hazards may be evaluated; frequently, local regulations set forth requirements for which hazards must be examined in different areas.

After the seismic and geologic hazards of a site are evaluated, the geotechnical engineer must prepare recommendations for construction of earthwork, grading, and building foundations. Frequently, the earthworks and foundations are already partially specified before the geotechnical investigation; the geotechnical engineer provides recommendations to make possible the desired construction so that geotechnical distress does not occur(Kavand A., 2006).

To determine the engineering properties of the soil in that particular area, such as swelling property, Atterberg limits, dry density, shear strength, and soil gradation were measured in the test lab in TCDE (Transport Construction Design Enterprise). This is for soil characterization purposes. Swell, shrinkage and collapse properties of geo-materials are among the significant hazards to geotechnical engineering structure in areas of tropical climates. Thus, it was very crucial to be able to quantify the swell, shrink and collapse strains, differential and excessive settlements, moisture content, gradation and shear strength and its real time behaviour for mitigation of geohazards. This helped to predict the pre and post construction performances of geotechnical

structure mode of failure. The laboratory set-up allowed for the free swell of soil to be quantified with respect to the real time strain. The choice of test method was decided based on the durability of the soil. A geophysical survey (2D Electrical resistivity imaging and Vertical Electrical Sounding, VES), is also carried as a pre-construction activity.

## **8:LABORATORY TESTS AND RESULTS**

### **8.1.General**

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 others measure index properties which provide useful information about the soil without directly measuring the property desired (Kavand A. ,2006).

For the design and construction of building structures a thorough geotechnical investigation should be made for the specific site.that is for the determination of the engineering properties of soil so that the geotechnical engineer can be able to decide the suitability of the site for buildngs.two test samples were taken from the site and they were disturbed soil samples.

Tests were carried out at the geotechnical laboratory of transport construction and design enterprise(TCDE).the tests are soil classification,free swell,shear strength,atterberg limit,and procter test.

### **8.2.Soil sampling**

Soil samples are obtained in either disturbed or undisturbed condition. However, undisturbed samples are not truly undisturbed. A disturbed sample is one in which the structure of the soil has been changed sufficiently that tests of structural properties of the soil will not be representative of in-situ conditions, and only properties of the soil grains can be accurately determined.

An undisturbed sample is one where the condition of the soil in the sample is close enough to the conditions of the soil in-situ to allow tests of structural properties of the soil to be used to approximate the properties of the soil in-situ.

All the soil samples taken from the site are disturbed ones. From this soil sample all the tests namely, soil classification, free swell, shear strength, Atterberg limit, and Proctor test were determined.

### **8.3. Geotechnical Tests Conducted On The Soil Samples**

#### **8.3.1. General**

Soil samples are necessary for the description of the soil profile. The index property tests provide physical and engineering properties of soils which are of greater relevance to engineers. Using the experimental results general and localized trends could be assessed. Based on the scope and the objective of this research, more emphasis was given to the laboratory tests which greatly help in identifying and classifying soils including some selected tests for soil characterization.

In this particular study, the following laboratory tests were undertaken:

**Sample one:**

- Sieve analysis
- Soil classification
- Free swell
- Atterberg limits

**Sample two:**

- Sieve analysis
- Soil classification
- Atterberg limits
- Compaction test(standard)
- Shear strength.

### **8.3.2.Soil Index Property Tests**

#### **8.3.2.1.General**

In this section, effort has been made to explain that laboratory tests on the two samples from the specific site with the basic description and information for the specified tests.All the results of the laboratory tests and discussion on the results are clearly presented.

#### **8.3.2.2.Particle Size Distribution**

It is a screening process in which coarse fractions of soil are separated by means of series of sieves. Particle sizes larger than 0.074 mm (U.S. No. 200 sieve) are usually analyzed by means of sieving. Soil materials finer than 0.074 mm (-200 material) are analyzed by means of sedimentation of soil particles by gravity (hydrometer analysis). A sieve analysis or gradation test is a practice or procedure used to assess the particle size distribution of granular material.

#### **Sample 1:**

The soil classification was made based on the UNIFIED classification system. According to the test result, it is shown that about 98.6% of the soil passes

through IS sieve 0.075mm. So, much of the soil proportion dominates fine-grained particles since more than 50% of the soil sample passes through sieve no. 200 based on the UNIFIED soil classification system, probably clay fractions. Therefore, due to this, the soil's classification is CH-FAT CLAY. This soil is highly inorganic and composed of highly plastic clays. The laboratory classification criteria suggests that the soil plots on or above A-line of the plasticity chart.

### **Sample-2:**

Likewise sample 1, The soil classification was made based on the UNIFIED classification system. The percentage passing through IS sieve 0.075mm is 1%. Therefore, the soil is categorized as coarse-grained soil according to the UNIFIED soil classification system. The soil classification shows that the test sample is GW well-graded gravels or gravel sand mixture little or no fines. The laboratory classification criteria suggests that less than 5% is fine.

### **8.3.2.3. Free Swell Test**

#### **Sample-1:**

The dominant clay minerals present in fine-grained soils control the physical and engineering behavior of the soils.

A user-friendly approach based on free swell ratio, defined as the ratio of the equilibrium sediment volume of 10-g oven dried soil in distilled water to that in carbon tetra chloride, has emerged as a simple methodology to predict the clay minerals present in the soil as either montmorillonitic or kaolinitic type quite satisfactorily. The validation of the proposed free swell ratio method has been done with exhaustive experimental data. Free swell or differential free swell, also termed as free swell index, is the increase in volume of soil without any external

constraint when subjected to submergence in water(prakash and sridharan,2004).

The free swell test for the the sample was carried out by placing a known volume (10ml) of soil passing a 425 micrometer sieve in to graduating cylinder containing 100ml. Of water.After the soil comes to settle at the bottom of the cylinder the expanded volume is measured and it is defined aas the difference between the final and the initial volumes,expressed as a percentage of the initial volumes(Holtz and Gibbs,1956).

$$FSI = \frac{\text{Final volume}(v_f) - \text{initial volume}(v_o)}{\text{Initial volume}(v_o)}$$

Initial volume( $v_o$ )

According to the degree of expansion of soils(Mohan,1977),soils are grouped as

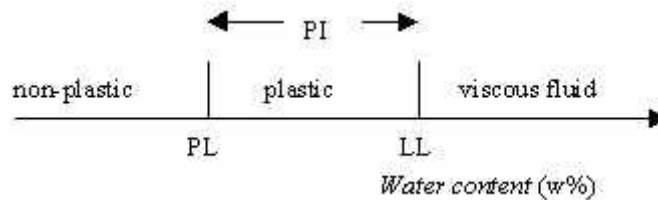
- Soils having free swell index > 200 ..... Very high
- free swell index between 100 and 200 ..... High
- free swell index between 50 and 100 ..... Medium,and
- free swell index < 50 ..... Low.

Based on the laboratory test result on sample one,the free swell result is shown to be 175.So,the result suggests that the soil has high degree of expansion as the free swell index is between 100 and 200.This means that the soil sample contains high amount of clay minerals which are responsible for foundation failure.therefore,the soil is classified as high degree of expansion.

### **8.3.3.Atterberg Limits**

The Atterberg limits are a basic measure of the nature of a fine-grained soil. Depending on the water content of the soil, it may appear in four states: solid, semi-solid, plastic and liquid(figure-1). In each state the consistency and behavior of a soil is different and thus so are its engineering properties.

Thus, the boundary between each state can be defined based on a change in the soil's behavior. The Atterberg limits can be used to distinguish between silt and clay, and it can distinguish between different types of silts and clays(Holtz, et.al, 1981).



**Figure-8.1: Illustration of Atterberg Limits**

([http://www.civil.umaine.edu/cie366/atterberg\\_limits/attberg\\_fig1.jpg](http://www.civil.umaine.edu/cie366/atterberg_limits/attberg_fig1.jpg))

### 8.3.3.1.Liquid Limit and Liquidity Index

The liquid limit (LL) is the water content where a soil changes from plastic to liquid behavior. The original liquid limit test of Atterberg's involved mixing a pat of clay in a little round-bottomed porcelain bowl of 10-12cm diameter. A groove was cut through the pat of clay with a spatula, and the bowl was then struck many times against the palm of one hand.(Das and Braja M. 1998).

At liquid state,a soil contains higher water content.It offers no shearing resistance, and can flow like liquids and has no shear deformations.However,as the water content reduces the soil becomes stiffer and starts developing resistance to shear deformation and at some partucular water content,the soil starts to become plastic.the water content at this changing state is the liquid limit of the soil (LL).Liquidity index(IL) is an important parameter and can be given as:

$$\text{Liquidity Index(IL)} = \frac{\text{Natural water content}(W_n) - \text{plastic limit(PL)}}{\text{plasticity index(PI)}}$$

When  $I_L < 0$ , the soil is in the semisolid state and has high strength, sudden brittle is expected,

When  $0 < I_L < 1$ , the soil is in the plastic state and has intermediate strength, soil deforms like a plastic material and,

When  $I_L > 1$ , the soil is found in the liquid state and has low strength, soil deforms like a viscous fluid.

As can be seen from the sieve analysis, sample-1 is inorganic soil. It is not common that, in any case, this soil with volcanic origin to have liquid limit greater than 100. The liquid limit tested for the soil sample is 115. Thus, the soil bears high montmorillonite mineral and high organic matter since the test sample have liquid limit greater than 100.

Sample-2 has lower liquid limit (LL) as compared to sample-1. The soil sample has liquid limit of 61. Its liquidity index (IL) is greater than 1. This implies that the soil is found in liquid state and has low strength and the soil deforms like a viscous fluid. Its linear shrinkage (from Eq.3) shows that it is greater than 8 and its liquid limit (LL) is greater than the permissible limit which is 30. Therefore, the soil is turned out to be expansive.

### **8.3.3.2. Linear Shrinkage and Shrinkage Limit**

The shrinkage limit (SL) is the water content where further loss of moisture will not result in any more volume reduction. This test was conducted to obtain quantitative indication of how much volume change can occur and the amount of moisture necessary to initiate volume change.

Soils that belong to the montmorillonite group shrink more than the soil of the other groups. Shrinkage limit is sufficient in determining the properties of a soil. But, it can also be possible to determine the linear shrinkage limit as:

$$PI = 2.13 \times (L_s)$$

where,  $L_s$  is linear shrinkage. .... (Eq.3)

-According to Kantey and Brink (Mckeen,1976), expansive soils are obtained by the following criteria:

- Liquid Limit > 30
- Plastic Limit > 12
- Linear Shrinkage > 8.

-Altmeyer(Mckeen,1976), suggested that rating for degree of expansion based on shrinkage limit(SL) and linear shrinkage:

<u>SL(%)</u>	<u>LS(%)</u>	<u>Degree of Expansion</u>
<10	>8	Critical
10-12	5-8	Marginal
>12	<5	Non critical

Therefore, based on the above criterias(Mckeen,1976), it can be suggested that the test sample(sample-1) is inactive because the AC value shown is below 0.75. Its liquid limit depicts less than the permissible limit that is 30. Therefore, the soil is expansive soil(using eq.-1). Its degree of expansion is not

critical because its shrinkage limit is greater than 28 but linear shrinkage shows that it is critical.

### **8.3.3.3. Plastic Limit(PL) and plasticity index(PI)**

The plastic limit (PL) is the water content where soil starts to exhibit plastic behavior. A thread of soil is at its plastic limit when it is rolled to a diameter of 3 mm and crumbles. To improve consistency, a 3 mm diameter rod is often used to gauge the thickness of the thread when conducting the test. Plasticity index is very important to determine the plasticity index.

The plasticity index (PI) is a measure of the plasticity of a soil. The plasticity index is the size of the range of water contents where the soil exhibits plastic properties. The PI is the difference between the liquid limit and the plastic limit ( $PI = LL - PL$ ). Soils with a high PI tend to be clay, those with a lower PI tend to be silt, and those with a PI of 0 tend to have little or no silt or clay (Holtz et al., 1981).

Accordingly, sample-1 has high plasticity index, i.e., 68, the test sample is considered to be clay soil. The soil has high clay content.

Sample-2 has lower plasticity index as compared to sample-1. Its clay content is much lower than sample-1. So, its plasticity is less. The shear strength at the plastic limit is about hundred times that at the liquid limit. Both the soils have smaller plasticity index (PI) and plasticity limit (PL) than their respective liquid limit. The soil samples have smaller shear strength.

### **8.3.3.4. Activity of Clays**

Activity of the soil is obtained by combining Atterberg limits and clay content into a single parameter (McKeen, 1976). Activity is defined as:

$$\text{Activity} = \text{PI} / \text{percentage by weight finer than } 2\mu\text{m}.$$

Skempton suggested that there are three classes of clays according to their activity.

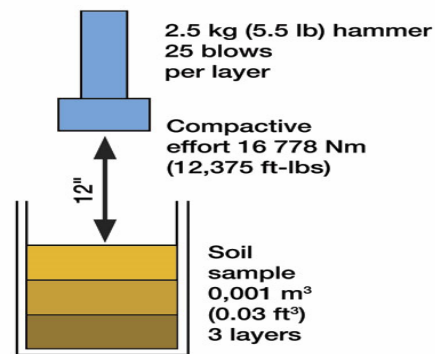
<u>Activity</u>	<u>Potential of expansion</u>
AC < 0.75	Low (Inactive)
0.75 < AC < 1.25	Medium (normal)
AC > 1.25	High (Active)

Active clays are the most expansive soils (Tefera, 1989). Sample-1 has an activity value 0.68 and sample-2 has 3.7.

### **8.3.3.5. Proctor Compaction Test**

Compaction is the process of increasing the bulk density of a soil or aggregate by driving out air. For any soil, for a given amount of compactive effort, the density obtained depends on the moisture content. At very high moisture contents, the maximum dry density is achieved when the soil is compacted to nearly saturation, where almost all the air is driven out. At low moisture contents, the soil particles interfere with each other; addition of some moisture will allow greater bulk densities, with a peak density where this effect begins to be counteracted by the saturation of the soil.

The test consists of compacting the soil or aggregate to be tested into a standard mould using a standardized compactive energy at several different levels of moisture content. The maximum dry density and optimum moisture content is determined from the results of the test. Soil in place is tested for in-place dry bulk density, and the result is divided by the maximum dry density to obtain a relative compaction for the soil in place.



**Figure-8.2:Standard Proctor Test.**

(<http://www.siteprepmag.com/SP/Home/Images/SP0308TestingModifiedProctor.jpg>-  
accessed 18/05/2009)

Standard proctor compaction tests were performed on the second sample soil taken from the area under consideration to determine the compaction parameters:the maximum dry density and optimum moisture content.In the compaction tests,the material was compacted in three equal layers by using a rammer consisting of a 2.5kg mass falling freely through 305mm,each layer receiving 25 blows.By using the data obtained from the compaction tests,compaction curve was plotted,and using this curve value of maximum dry density and optimum moisture content was found to be  $1.39\text{gm/cm}^3$  and 25% respectively.

It is shown that the soil is highly expansive determined by the parameters above.So,expansive soils expand very little compacted at low densities and high

moisture content but expand greatly when compacted at high densities and low moisture content.

### **8.3.3.6. Shear Strength Test**

Shear strength is a term used to describe the maximum strength of soil at which point significant plastic deformation or yielding occurs due to an applied shear stress. There is no definitive "shear strength" of a soil as it depends on a number of factors affecting the soil at any given time and on the frame of reference, in particular the rate at which the shearing occurs. plasticity index of soils can be defined as a range of water contents producing a 100-fold variation in undrained shear strength has been experimentally verified with the help of a large number of tests on soils of diverse nature.

This has led to the redefinition of the plastic limit as the water content at which undrained shear strength is around 170 kN/m<sup>2</sup>. Undrained shear strength of a soil at the liquid limit can be considered to be around 1.7 kN/m<sup>2</sup>(Binu Sharma and Padma K. Bora,2005).

To determine the shear strength properties of a soil samples,laboratory test should be on undisturbed samples.The shear strength is measured in terms of two soil parameters,cohesion or inter particle attraction, and angle of internal friction, $\Phi$ ,the resistance to inter particle slip.This behaviour is well represented Sby Mohr-Coulomb criterion as, is

$$S=C + \delta \tan \Phi \dots\dots\dots (Eq.2)$$

Where, S is shear strength

$\delta$  is normal stress on shear plane

C is cohesion.

For the remoulded sample at its optimum moisture content(OMC=25%) and maximum dry density(MDD=1.39gm/cm<sup>3</sup>),the angle of internal friction and cohesion are 17<sup>0</sup> and 50KN/m<sup>2</sup> respectively.

The result shows big values of angle of internal friction and cohesion which is not expected in clay soil.This is mainly due to the fact that the sample is not pure clay.Such a higher value of friction is due to the presence of granular and fragmented material.

## **9.CONCLUSION AND RECOMENDATION**

### **9.1.Conclusion**

The present research work has mainly to establish based upon the information obtained from the soil test and geophysical studies. The nature and extent of variations across the site may vary over the entire construction site.

The conclusions drawn from the study are:

**1.**the result can not generalize all the geotechnical properties of the soil,but the results on the two test samples show the soil is highly plastic clay and well- graded gravel.

**2.**the soil has high value of shear strength.The shear strength paramrtrs cohesion( $50\text{KN}/\text{m}^2$ ) and angle of internal friction( $17^\circ$ ) at a lower moisture content.Therefore,moisture content has a significant effect on the shear strength property for expansive soil.The shear strength of expansive soil reduces when the moisture content increases.When the shear strength of expansive soil is mentioned,it shold always be referenced with the moisture content.

**3.**free swell value on the second sample shows that it is very high(175).During rainy season, the soil swells at a higher degree and the soil has high clay content.

**4.**the Atterberg limit test results and activity has shown that the soil is expansive and degree of expansion is critical.Hence,expansion potential of sample one is inactive and sample two is active.

**5.**Four distinct geoelectric layers generally represent the subsurface in the study area. The topmost layer is a dry sandy soil fill cover with a depth of about 0.5m. The second horizon in the sequence is *moist black cotton soil*, which is highly expansive in nature.

The depth to the bottom of this conductive soil ranges from few centimeters at the NNW to 4.5m towards the SSE. This is underlain by a 4.7m thick horizon representing intercalation soil with weathered rock fragments.

## **9.2.Recommendation**

**1.**It is recommended to apply all existing available theories and practctices for the design and construction of buildings located in expansive soil areas.great emphasis should be made specially on moisture control measures.

**2.**The result obtained may be used as a basis for further research in the area.

**3.**Soils with organic matter encountered in the investigation include topsoil as well as sandy fill with organic matter.As a cautionary measure, consideration must be given to removing of such soils from any planned construction areas. This is especially crucial in areas designated for structural fill, buildings, other structures, roadways, driveways, and sidewalks around and on the building areas.

**4.**The highly plastic soil, from the study area, tested is designated CH and has been found to be potentially expansive, i.e., its water content altered during drying or wetting. These soils are not reliable to support structural fill, foundations, floor slabs and paved surfaces. Therefore, it is recommended to excavate and remove the top soil, at least, to a depth of 6m below the surface, for the proposed multistory building construction. Furthermore, care must be taken for the design of foundation to be used and construction of basements.

**5.**The two possible types of foundation, appropriate for such cases will be pile or mat foundation. The final decision should depend on the safety and cost consideration after further detailed analysis. Moreover, the bottoms of over-excavated sections should be thoroughly compacted with proper compactive effort.

**6.**Before the erection of a structure or structural backfill in over-excavated areas, a professional Engineer should observe exposed sub-grade soils at the bases of structures and structural backfills. This can determine if exposed soil is suitable and reliable to support the super-imposed imposed load on the proposed grade.

**7.**the fractured rock mass in the bed rock is very dangerous for supporting heavy structures.so, it is recommended that this may be treated with a suitable grouting material (cement grout) for its stability.But, this may be very costly so that other techniques could be used in order to be economical.Or the fractured rock should be excavated and removed.

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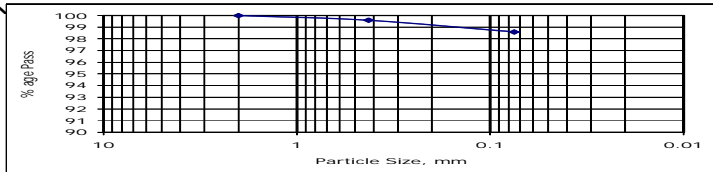
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# **APPENDIX-I**

PROJECT: RESEARCH ON INTEGRATED GEOPHYSICAL AND GEOTECHNICAL INVESTIGATION ON MULTISTORY BUILDING IN ADDIS ABABA  
 CLIENT: ADDIS ABABA UNIVERSITY, DEPARTMENT OF EARTH SCIENCES ON BEHALF OF AKLILU G/EGZIABEHER MATERIAL: SOIL(SAMPLE 1)  
 YOUR REF: SFESD/368/2001/2009 TEST: SIEVE ANALYSIS

**1. SIEVE ANALYSIS**

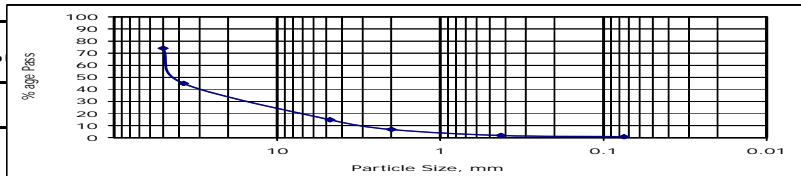


0.075	-	-
98.6	-	-

**2. SOIL CLASSIFICATION**

Method: UNIFIED		Atterberge Limits		SL	FREE SWELL, %	Soil Classification
Sieve Analysis, %PASS, mm	LL	PI				
4.75	115	68	14	175		CH-FAT CLAY

COND



APPROVED BY: \_\_\_\_\_  
 HEAD, GEOTECH DEPT.  
 Sco. LAB No.: 1424/2001  
 1-514231 DATE: 28/4/2008  
 PAGE: 1 OF 1  
 GEOTECHNICAL INVESTIGATION

CLIENT: ADDIS ABABA UNIVERSITY, DEPARTMENT OF EARTH SCIENCES ON BEHALF OF AKLILU G/EGZIABEHER MATERIAL: SOIL(SAMPLE 2)  
 YOUR REF: SFESD/368/2001/2009 TEST: SIEVE ANALYSIS

**1. SIEVE ANALYSIS**

METHOD: WET SIEVE

Sieve Size, mm	75	50	37.5	4.75	2	0.425	0.075
% Passing, %	100	74	45	15	7	2	1

**2. SOIL CLASSIFICATION**

Method: UNIFIED		Atterberge Limits		Soil Classification
Sieve Analysis, %PASS	LL	PI		
4.75MM	61	26		GW-Well graded gravel with sand

BY: \_\_\_\_\_ REPORTED BY: \_\_\_\_\_ APPROVED BY: \_\_\_\_\_  
 CREW HEAD SECTION HEAD HEAD, GEOTECH DEPT.



**TRANSPORT CONSTRUCTION DESIGN Sco.**

Tel No. 011-115 5799 P.O.Box 41726 Fax 251-1-514231

LAB No.: **1424/2001**  
 DATE: 28/4/2009  
 PAGE: 1 OF 1

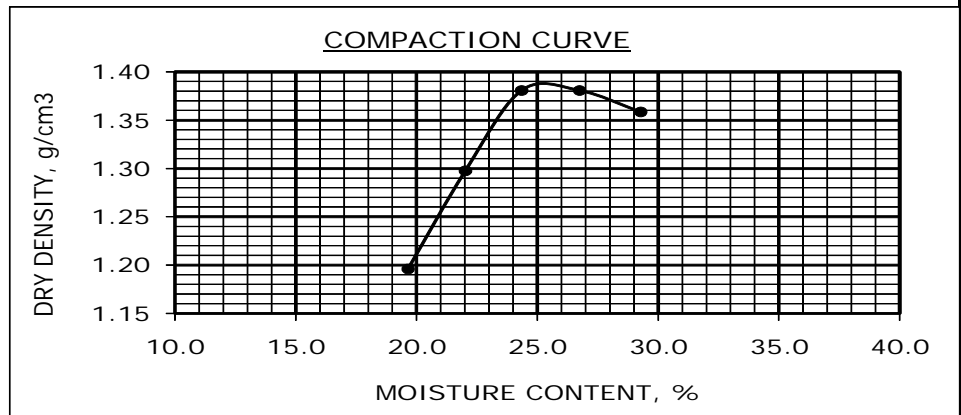
PROJECT: RESEARCH ON INTEGRATED GEOPHYSICAL AND GEOTECHNICAL INVESTIGATION ON MULTISTORY BUILDING IN ADDIS ABABA CITY  
 ADDIS ABABA UNIVERSITY,  
 DEPARTMENT OF EARTH SCIENCES ON

CLIENT: BEHALF OF AKLILU G/EGZIABEHER MATERIAL: SOIL(SAMPLE 2)  
COMPACTION  
 YOUR REF: SFESD/368/2001/2009 TEST: (STANDARD)

**1. TEST DATA**

VOLUME OF MOLD=			944 cm <sup>3</sup>	TYPE OF COMPACTION: STANDARD					
TRIAL No.	Mass of mold + Wet Soil g	Mass of Mold g	Wet Density g/cm <sup>3</sup>	Moisture Content Determination					Dry Density g/cm <sup>3</sup>
				Can No.	Mass of Wet soil + Can g	Mass of Dry Soil + Can g	Mass of Can g	Moisture Content %	
1	5529	4178	1.43	56	524	458	122	19.64	1.20
2	5673	4178	1.58	8	513	439	103	22.02	1.30
3	5799	4178	1.72	AB	511	436	128	24.35	1.38
4	5830	4178	1.75	KM	520	443	155	26.74	1.38
5	5836	4178	1.76	33	551	450	105	29.28	1.36
6	-	-	-	-	-	-	-	-	-
7	-	-	-	-	-	-	-	-	-

**2. COMPACTION CURVE**



**3. COMPACTION RESULT**

OMC, %	MDD, g/cm <sup>3</sup>
25	1.39

CONDUCTED BY: \_\_\_\_\_

REPORTED BY: \_\_\_\_\_  
 SECTION HEAD

APPROVED BY: \_\_\_\_\_  
 HEAD, GEOTECH DEP



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Tel No; 011-115 5799 B.O.Box 41726 Fax: 251-1-5142 31

LAB No.: **1424/2001**

DATE: 27/04/2009

PAGE: 1 OF 1

INTEGRATED GEOPHYSICAL AND GEOTECHNICAL INVESTIGATION for MULTISTORY  
PROJECT: BUILDING NEAR BISRATE GABRIEL CHURCH, SE ADDIS ABABA CITY

ADDIS ABABA UNIVERSITY,  
DEPARTMENT OF EARTH SCIENCES ON

CLEINT: BEHALF OF AKLILU G/EGZIABEHER  
YOUR REF: SFESD/368/2001/2009

MATERIAL: SOIL(SAMPLE 1)  
TEST: ATTERBERGE LIMITS

**1. TEST DATA**

	Liquid Limit		Plastic Limit	
	20	25	17	-
Number of Blows			17	-
Mass of Container	9.51	27.32	25.75	
Mass of Container	0.29	24.12	22.92	
Mass of Water	1.22	3.20	2.83	
Mass of Container	2.41	17.31	16.89	
Mass of Dry Soil	1.88	6.81	6.03	
Moisture Content (%)	7.01	46.99	46.93	



**2. FLOW CURVE**



**3. ATTERBERGE LIMITS**

LL	PL	PI	NATURAL MOISTURE, %
115	47	68	-

70.00									
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INTEGRATED GEOPHYSICAL AND GEOTECHNICAL INVESTIGATIONS ON  
PROJECT: MULTISTORY BUILDINGS IN ADDIS ABABA

ADDIS ABABA UNIVERSITY,  
DEPARTMENT OF EARTH SCIENCES ON

CLEINT: BEHALF OF AKLILU G/EGZIABEHER  
YOUR REF: SFESD/368/2001/2009

MATERIAL: SOIL(SAMPLE 2)

TEST: ATTERBERGE LIMITS

### 1. TEST DATA

	Liquid Limit			Plastic Limit	
	33	26	19	-	-
Number of Blows	33	26	19	-	-
Mass of Cont. + Wet soil, gm	41.90	37.00	37.23	25.26	24.96
Mass of Cont. + Dry Soil, gm	34.43	31.40	31.43	23.16	22.98
Mass of Water, gm	7.47	5.60	5.80	2.10	1.98
Mass of Cont, gm	21.87	22.21	22.21	17.17	17.32
Mass of Dry Soil, gm	12.56	9.19	9.22	5.99	5.66
Moisture Content, %	59.47	60.94	62.91	35.06	34.98

### 2. FLOW CURVE

### 3. ATTERBERGE LIMITS

LL	PL	PI	NATURAL MOISTURE, %
61	35	26	-

CONDUCTED BY: \_\_\_\_\_ REPORTED BY: \_\_\_\_\_

APPROVED BY: \_\_\_\_\_

SECTION HEAD

HEAD, GEOTECH DEPT.

