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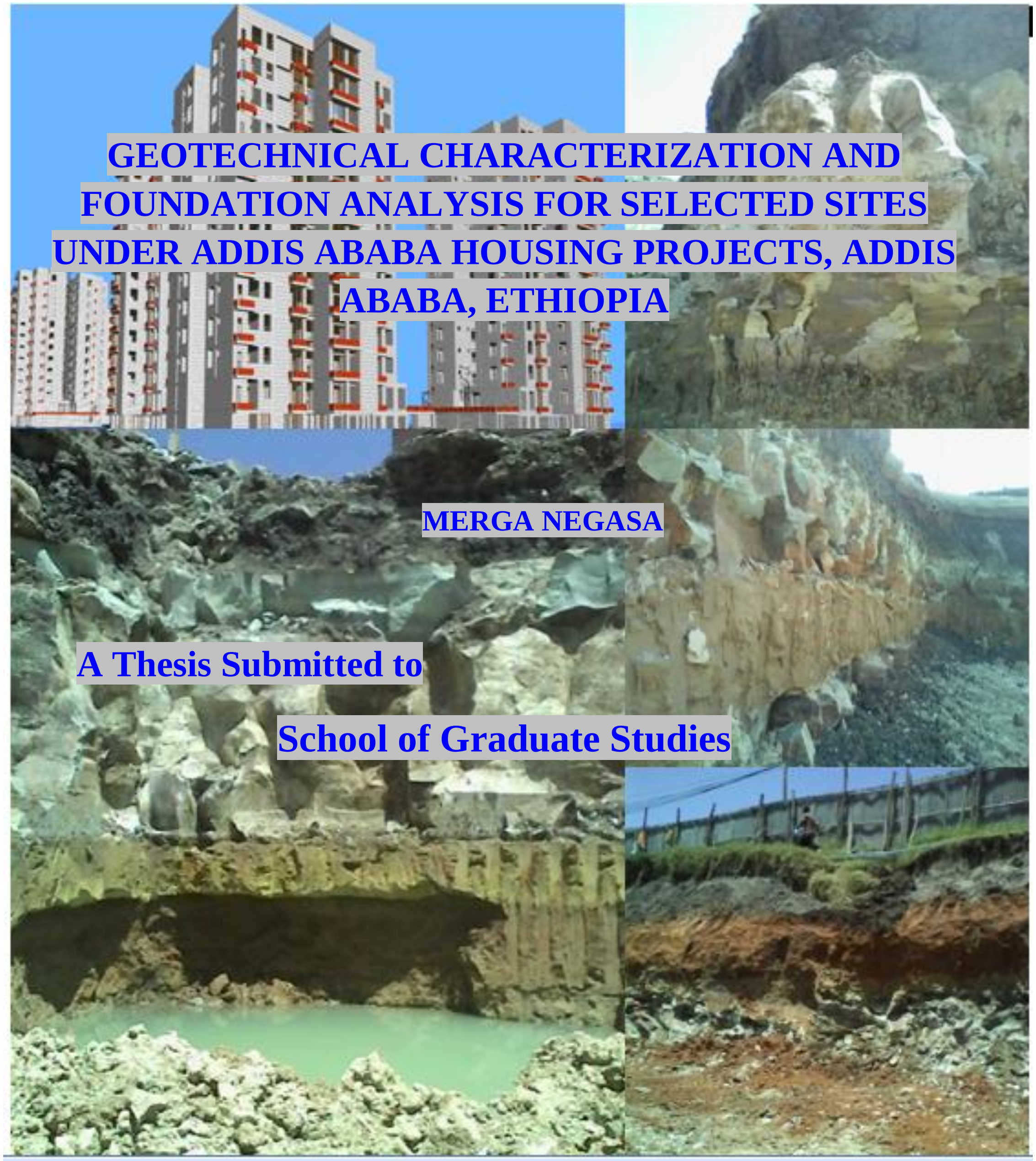

GEOTECHNICAL CHARACTERIZATION AND FOUNDATION ANALYSIS FOR SELECTED SITES UNDER ADDIS ABABA HOUSING PROJECTS, ADDIS ABABA, ETHIOPIA

MERGA NEGASA

A Thesis Submitted to

School of Graduate Studies

Presented In Partial Fulfillment for the Requirements of Degree of Masters in Engineering Geology

Addis Ababa University

Ethiopia

May, 2014

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This is to certify that the thesis prepared by Merga Negasa entitiled: Geotechnical Characterization and Foundation Analysis for Selected Sites under Addis Ababa Housing Projects and submitted in partial fulfillment of the requirements of Degree of Masters in Engineering Geology compiles with the regulations of the University and meets the accepted standards with respect to originality and quality

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Abbreviations

SPT – Standard penetration test

a.s.l – above sea level

Mt. – mountain

MY – Million Years

NE – North east

NNE – North of north east

NW- North west

SW – South west

E-W – East west

EARS – East African Rift System

PH – Hydrogen Power

mm – millimeter

PL – Plastic limit

LL – Liquid limit

PI – Plasticity index

SL – Shrinkage limit

BS – British standard

UCS – Unconfined Compressive Strength

W – Moisture content

Mw – Mass of water

Ms – Mass of solid

NMC – Natural moisture content

FS – Free swell

Symbols

C – Cohesion (KPa)

q - Load (KPa)

B – Width of foundation (m)

D – Depth of foundation (m)

ϒ – Unit weight (Kg/m^3)

Q_{ult} – Ultimate bearing capacity (KPa)

Q_{all} – Allowable bearing capacity (KPa)

N_c, N_q, $N_ϒ$ – Bearing capacity factors

S_c, S_q, $S_ϒ$ – Shape factors

D_c, d_q, $d_ϒ$ – Depth factors

I_c, i_q, $i_ϒ$ – Inclination factors

G_c, g_q, $g_ϒ$ – Ground factors

B_c, b_q, $b_ϒ$ – Base factors

F_1, F_2, F_3, F_4 – Correction factors

Abstract

Geotechnical Characterization and Foundation Analysis for Selected Sites under Addis Ababa Housing Projects, Addis Ababa, Ethiopia
By Merga Negasa

Mass housing projects always paid less attention due to their bulkiness and sometimes failures of buildings occur before their performance stage. It is obvious in such cases the proper ground investigation and characterization will not practically fulfilled, where less and poor routine subsurface investigation is conducted. The city government of Addis Ababa had been practicing construction of massive housing projects and also launched new low cost housing projects to meet urban renewal strategic plan. These projects are vast and construction is booming throughout the city. One of the most recent failures of building in Addis Ababa was occurred at Jemo condominium site where massive housing construction was performed.

This study characterizes subsurface where such urgent construction would be under way to be employed. Three sites were selected to analyze their suitability for the anticipated building types. Soil index tests were used to characterize the subsurface geological formations and data from excavations and borehole logs were used. Index tests were conducted on the samples collected from these sites. Evaluation of bearing capacity and settlement analysis was conducted. Based on the laboratory results and index soil tests Megenagna and Imperial sites are characterized by fine grained, high plasticity SILT and CLAY soils. Asco site is characterized by fill where the most of the top soil layers of the site are highly compressible, coarse grained material. Excessive settlement was resulted in according to Bowels, (1996) estimation and the sites need extensive ground improvement mechanisms before construction begin. The bearing pressure estimated using Meyerhof, (1963) and Hansen, (1970) shows extreme values which is exaggerated. The buildings have double basements and the foundation depth will be around 7m below the natural ground level. The depth factor is responsible for such results of bearing pressure. Thus, Meyerhof and Hansen bearing capacity equations are not valuable for this study.

Key words

Mass house Settlement Bearing capacity Foundation Clay Silt

CHAPTER ONE INTRODUCTION

1.1 Introduction

The city of Addis Ababa, being the capital of the nation, is becoming populous at alarming rate. People, for better life are migrating from rural area to the city (www.cga.gov.et). This has resulted into exponential growth of population and as consequence it is bringing high house demand which has resulted into shelter scarcity in the city. Most of the Addis Ababa population is living in old houses with number of family members and most of the population is living in rental house with poor quality. All these factors have resulted into multiple problems related to socio-economic status and health problems which are directly related to substandard life. Further, the old houses are also affecting the aesthetic value of the city and are not well planned according to this modernized world. On the other hand, majority of the population living in Addis Ababa cannot afford its own standardized house because of low income (www.cga.gov.et).

The city is facing numerous challenges in trying to meet the demands aroused by the ever-increasing population and high rate of urbanization. Consequently, the need for an increase in quantity and quality of shelter, infrastructure, employment and various social services become crucial to sustain the metropolitan life. Accordingly, the city government has planned to address these challenges strategically within the help of nation integrated urban housing development plan along with other plans. The national plan on housing has allocated about 50% of its 2006-2010 plans, which are about housing units to be executed in the city of Addis Ababa (Yohannes Abebe, 2009). The project consists of mainly apartment building.

The Addis Ababa City Government in collaboration with commercial bank of Ethiopia has developed different projects in which citizens can get house by saving monthly. Saving house development enterprise is one of such project which is under implementation stage. The enterprise is planned to construct G+7, G+9, G+10, G+12, G+18, and G+20 and apartment buildings in different areas in Addis Ababa (www.http://aashde.gov.et). The current study sites will be adopted for 2B+G+12typology.

These buildings are designed as common apartments with availability of social services and expected to have long service time. Any foundation problem and incomplete/substandard investigation at building sites will automatically risks the life of residents. Therefore, it is obvious to characterize geologic formations and evaluate the foundation characteristics of each site for existing condition as well as for anticipated adverse conditions. The detailed investigation will not only help to know the foundation problems in the early stage but will also help for safe foundation design (Bell, 2007).

Saving house development enterprise has been developing different sites such as; Sengatera, Bulbula, Ayat, Asco, Megenagna, Mari, Imperial and Crown sites. For the present research Asco, Megenagna, and Imperial sites with areal coverage of 22441m^2, 33454m^2, 34551m^2 respectively were selected. The total area coverage of these sites will be 90,446m^2 (www.http://aashde.gov.et). Hence, the current study will elaborate its findings depending on the data archived from these areas basically data from excavations and boreholes logs. It was intended in the present study that the lithological correlation and cross sections prepared to understand the subsurface conditions. Besides, attempts are also made to evaluate the engineering geological characterization of foundation soils and rocks for each site. Further, it was proposed to conduct the foundation analysis to estimate the bearing capacity and settlement potential of the foundation.

1.2 Construction and Foundation Problems in Addis Ababa

Addis Ababa is experiencing massive buildings and infrastructures in light of renaissance and transformation program. High rise buildings owned by private companies and government agencies; different infrastructures are under construction which elevates construction industry. This industry is led by private owned construction companies and also by government owned agencies. In recent years City Government of Addis Ababa is activating a massive housing program to solve housing problem (second essential need next to food and water) of the city under a program called Addis Ababa Integrated Housing Development Program: A Strategy for Urban Poverty Reduction and Sustainable Socio-Economic Transformation. Many condominiums has been built in different phases of construction and at different part of the city and transferred to more than half million people (www.cgaa.gov.et). Plate 1.1 shows part of condominium buildings in Addis Ababa.

Despite the need of housing there have some problems, where buildings are failed before its performance stage due to poor geotechnical investigations. One of such phenomenon occurred in Jemo condominium site. This site is characterized by many condominium buildings and it is one of the centers of high population in Addis Ababa. Plate 1.2 shows the block is tilted and cracked with mismatching of doors and windows and offsetting of ceilings.

Plate 1.1 Part of condominium buildings in Addis Ababa

Plate 1.2 Jemo condominium buildings

a) The middle building is tilted to the right side

b)Cracks at its base

1.3 Scope Of The Study

This study has been conducted for some selected sites of saving house projects from engineering geologic point of view. Subsurface geology and engineering geological characterization of each site has been produced. Correlation between boreholes and excavations of the same site and engineering property of different strata will be evaluated. Foundation analysis, level and recommendation for each site have been forwarded.

1.4 Objectives

1.4.1 General Objectives

- Geotechnical characterization of selected building foundation sites
- Foundation analysis and suitability evaluation of selected sites for the proposed building structures

1.4.2 Specific Objectives

- To characterize the building foundations based on the engineering geological parameters
- To estimate the bearing capacity and settlement potential of foundation material
- To assess the overall suitability of foundations for proposed buildings
- To suggest suitable foundation type for the buildings

1.5 Outcomes Of The Study

The outcomes of this study are:

- Engineering properties of the sites with engineering characterization of different layers and their possible problem to buildings
- Foundation estimation and recommendation of foundation type
- Subsurface problem to buildings and its mitigation methods will be forwarded

1.6 Limitations of The Study

Availability of secondary data from neighboring sites with specific location is one of the limitation factors in this study. This is because most of the geotechnical works are conducted for simple purposes and not supported with standardized data collection and well organized data archive. On the other hand as a common practice there is poor data recovery system where most of the primary collected data are not restored after construction is accomplished. This may be also another constraint to produce well conducted research.

CHAPTER TWO LITERATURE REVIEW

2.1 Soil and Its Nature

Soil is a complex naturally occurring material which is heterogeneous and difficult to define as entirely a single and uniform unit, and it is quite heterogeneous in its chemical and physical properties. Even though similar soil has some common characteristics, sampling from the same site may show widely varying properties. It requires great ingenuity, therefore, to select representative soil parameters for a natural soil deposit (Ranjan, 1993). At best, soil engineering is an intuitive science where it needs some technical judgments.
The formation of soils structure is governed by several factors. In coarse grained soils the force of gravity is the main factor, while in fine grained soils the surface bonding forces become predominant (Ranjan, 1993).

Geologically soil can be traced back to their origin into two categories, organic soils and parent rock soils. Mechanical disintegration of rocks will result in coarse grained soils where as chemical disintegration will result in fine grained soils. Soil is a leftover of mother rock materials disintegrated from volcanic, sedimentary, or metamorphic rocks. These leftover can be in situ or transported by transporting agents and deposited in other place.

Soil and rock are the ultimate geologic formations that can either sufficiently or insufficiently respond to any load exist on it depending on the amount of stress applied on it. Every civil engineering structure has to be founded on the soil (assuming that a rock stratum is not available) and thus shall transmit the dead and live loads to the soil stratum (Ranjan, 1993). The ultimate goal of any soil material at foundation is to support properly the engineering structure which will be loaded over it. The proper functioning of the structure will, therefore, depend critically on the success of the foundation element resting on the subsoil (Ranjan, 1993).

2.2 Soil Classification

Any system of soil classification involves grouping the different soil types into categories that possess similar properties and, in so doing, providing the engineer with a systematic method of soil description (Bell, 2007). The purpose is to make it possible to estimate a soil's properties or capabilities by its association with soils of the same class whose properties are known and to provide the engineer with an accurate method of soil description. While soil classification in the abstract appears to be simple and logical, it is one of the most confused and controversial subjects in soil engineering in the actual application. There are so many different properties of soil of interest to engineers and so many different combinations of these properties in any natural soil deposit that any universal system of soil classification seems impractical (George, 2009).

According to the USC classification, the coarse grained soils are classified on the basis of their grain size distribution and the fine grained soils (whose behavior is controlled by plasticity) on the basis of their plasticity characteristics. All the soils are classified into four major groups, namely coarse grained, fine grained, organic soils and peat (Ranjan, 1993).

Textural classification- classification based on grain sizes characteristics alone. The entire scale of particle diameters is divided arbitrarily into three ranges or fractions (sand, silt, and clay).

<u>Public roads system</u>- soils according to their stability under wheel loads and developed for the purpose of identifying soils that are suitable for earth road construction. It was found that the different groups could be defined in terms of the results of simple laboratory tests such as grain size, liquid limit, and plastic limit.

<u>State highway department system</u>- several state highway departments have developed classification systems that are adapted to the soil problems that they regularly encounters.

<u>Airfield classification (AC) system</u>- the AC system (U.S Corps of Engineers) was developed by A. Casagrande as a rapid method of estimating a soil's capabilities for airfield construction.

The AC system has proved very useful not only for classifying soils for airfield construction but also as a basis for describing soils for many other engineering purposes. Proper classification of a soil requires only the simple tests for grain size and the liquid and plastic limits; the experienced soil technical, however, is often able to classify many soils correctly without the use of tests.

Civil Aeronautics Administration (CAA) system- The CAA system which was adopted in 1944 classifies soils according to their suitability for airfield sub grades. The soil is divided into groups, E-1 to E-11, on the basis of their grain size and plasticity characteristics. Other tests have been used to aid in the classification, but they are not essential (George, 1989).

2.3 Foundation Estimation

The foundation of a structure is that part which transmits the load of the structure to the ground and static loading implies a pressure from that foundation that does not change with time (George, 1989).

During the long period of time a soil must support a structure it may be changed by many man-made and natural forces (George, 1989). There is a limit to the load ground can carry and this is related to the mechanical properties of the ground and to the size and shape of the engineering structure (George, 2009).

No exact mathematical methods have been derived to define the surface of failure or to determine the failure load. Many approximate solutions have been developed based on the shapes of observed failure zones and on the certain simplifications of the actual soil properties. Some of these methods have been found to give reliable results and have a rational, sensible basis (George, 1989).

The pioneer bearing capacity estimation was developed by Karl Terzaghi, in 1943. Although this estimation was preconditioned with assumptions, it was the break though in soil bearing pressure evaluation. The following formula was derived by Terzaghi.

$$Q_{ult} = CN_c s_c + qN_q s_q + 0.5\gamma BN_\gamma s_\gamma$$

$$N_q = \frac{a^2}{2cos^2(45 + \emptyset/2)}$$

$$a = e^{(0.75\pi - \emptyset/2)\tan\emptyset}$$

$$N_c = (N_q - 1)\cot\emptyset$$

$$N_\gamma = \frac{\tan\emptyset}{2}\left[\frac{K_{p\gamma}}{cos^2\emptyset} - 1\right]$$

Where,
C=cohesion
q=load
B=width of foundation
ϒ=unit weight
$\boldsymbol{\phi}$=angle of internal friction
S_c, S_q, $S_ϒ$=shape factors
D_c, d_q, $d_ϒ$=depth factors
N_c, N_q, $N_ϒ$=bearing capacity factors

Meyerhof, 1963 in Bowels, (1996) also derived an equation for bearing capacity with additional parameters which was not included in Terzaghi's equation. Shape and depth factors are primarily incorporated and bearing capacity constant, $N_ϒ$ was modified.

$$Q_{ult} = cN_c s_c d_c i_c + qN_q s_q d_q i_q + 0.5\gamma BN_\gamma s_\gamma d_\gamma i_\gamma$$

$$N_q = e^{\pi tan\,\emptyset} tan^2(45 + \frac{\emptyset}{2})$$
$$N_c = (N_q - 1)\cot\emptyset$$
$$N_\gamma = (N_q - 1)\tan(1.4\emptyset)$$

Where,
C=cohesion
q=load
B=width of foundation
ϒ=unit weight
$\boldsymbol{\phi}$=angle of internal friction
S_c, S_q, $S_ϒ$=shape factors
d_c, d_q, $d_ϒ$=depth factors
N_c, N_q, $N_ϒ$=bearing capacity factors
I_c, i_q, $i_ϒ$=inclination factors

Hansen, 1970 and Vesic, 1973, 1975, in Bowels, (1996) also made slight modification to the above formula with consideration of some addition parameters.

$$Q_{ult} = cN_c s_c d_c i_c g_c b_c + qN_q s_q d_q i_q g_q b_q + 0.5\gamma B N_\gamma s_\gamma d_\gamma i_\gamma g_\gamma b_\gamma$$

$$N_q = e^{\pi \tan \emptyset} tan^2(45 + \frac{\emptyset}{2})$$

$$N_c = \left(N_q - 1\right) \cot \emptyset$$

$$N_\gamma = 1.5\left(N_q - 1\right) \tan \emptyset$$

Where,

C=cohesion
q=load
B=width of foundation
ϒ=unit weight
ϕ=angle of internal friction
S_c, S_q, $S_ϒ$=shape factors
d_c, d_q, $d_ϒ$=depth factors
N_c, N_q, $N_ϒ$=bearing capacity factors
I_c, i_q, $i_ϒ$=inclination factors
G_c, g_q, $g_ϒ$=ground factors
B_c, b_q, $b_ϒ$=base factors

The above estimation was commonly used for cohesive soils where test result for cohesion and angle for internal friction of representative soil sample was available. Bearing capacity is also calculated based on empirical estimation from SPT values. Bowles, (1996) derived the following estimation method.

$$q_{all} = \frac{N^{'}}{F_2} * \left(1 + {}^{F_3}/_{B}\right)^2 * K_d \qquad B > F_4$$

$$q_{all} = allowable\ bearing\ pressure\ for\ settlement\ limited\ to\ 25mm$$

$$K_d = 1 + 0.33(D/B) \leq 1.33$$
$$F_2 = 0.08$$
$$F_3 = 0.3$$
$$F_4 = 1.2$$
$$B = width\ of\ foundation$$
$$D = Depth\ of\ foundation$$

Bowels, (1996) also recommend estimating bearing capacity for mat foundation using SPT blow counts.

$$Q_{ult} = cN_c s_c d_c + qN_q s_q d_q + 0.5\lambda BN_\gamma s_\gamma d_\gamma$$

According to (George, 2009) civil engineering structures that impose load on the ground can be grouped into two types; mass structures and framed structures.

Mass structures made of soil, rock or mass concrete include such constructions as embankments, dams, mining waste tips and breakwaters. These are in direct contact with the ground and the magnitude and distribution of stress imposed upon the ground is related to the size and shape of the constructed mass, and the density of the material from which it is composed.

Framed constructions are buildings containing hollow spaces for dwellings, storage or industrial use. The weight of the upper parts of the structure is transmitted to the ground via the load bearing members of the structure. These load bearing members are composed of steel, concrete, bricks and wood, whose strength is generally much greater than that of the ground immediately beneath them. Contact between these load bearing elements and the ground is made via a foundation, which generally distributes the load of the structure so that the stresses on the ground are not excessive. The stress imposed on the ground due to foundation loading diminishes with depth below the foundation. Mathematical analysis, based on the theories of Boussinesq, has shown that the stresses due to building loads fall within a '*bulb of stress*' beneath a foundation (George, 2009).

There is a maximum load that can be applied to clays, silts, sands, gravels and mixtures of these, beyond which the ground will fail; this is the "*ultimate*" condition; something less than that will be the "*safe*" condition. However, safe loads may still generate a settlement that is excessive, so the safe load may have to be reduced yet further to an "*allowable*" condition (George, 2009).

The bearing capacity of cohesion less soil such as sand or gravel may be analyzed for strip loading in a similar manner to that for saturated clay. Three factors are of importance in the bearing capacity of sand; b, (width), gamma, (unit weight) and phi, (angle of internal friction). Bearing capacity increases directly with the width of the loaded area; in practice this means that small foundations on sand may be dangerous while large foundations are usually safe, capacity increases directly with unit weight (George, 1989).

To determine ultimate bearing capacity there must be adequate data on ground properties and trust in the appropriateness of the calculation theory. Since neither is commonly to be found the calculated ultimate bearing capacity is divided by a factor of safety to give the *Safe Bearing Capacity*. This factor of safety is commonly between 3 and 5, and is chosen on the difficulty of the ground conditions and the importance of the structure (Bell, 2007).

Having made the necessary calculations and determined the safe bearing capacity of the ground with regard to the proposed foundation it may be concluded that the ground strength is adequate *relative to the foundation pressure* (George B., 2009).

2.4 Settlement in Soil Layers

Soil is a non-homogeneous porous material consisting of three phases: solids, fluid (normally water), and air. Soil deformation may occur by change in stress, water content, soil mass, or temperature (Johnson, 1996). Elastic or immediate deformation caused by static loads is usually small, and it occurs essentially at the same time these loads are applied to the soil. Consolidation settlement is the reduction in volume associated with a reduction in water content, and it occurs in all soils (George, 1989). Consolidation occurs quickly in coarse-grained soils such as sands and gravels. Once construction commences, both types of settlement begin to take place simultaneously. However it is convenient in many cases, especially with clay rich soils, to think of immediate settlement taking place during construction and consolidation settlement continuing after completion of construction (George, 1989).

Consolidation in fine-grained soils such as clays and organic materials can be significant and take considerable time to complete. Secondary compression and creep are associated with the compression and distortion at constant water content of compressible soils such as clays, silts, organic materials, and peat. Dynamic loads cause settlement from rearrangement of particles, particularly in cohesion-less soil, into more compact positions (Robert, 1990).

In computing compression settlement it is necessary to determine the vertical stresses acting on each compressible soil stratum. There are only a few instantaneous where stresses in soil caused by surface loads have actually been measured and so the only methods for computing stress at the present time are formulas based on the theory of elasticity. Settlements computed with the aid of these formulas agree fairly well with measured settlements and so many be used until better methods are found (George, 1989).

Computation of the total settlement that the foundation of a structure undergoes is but one part of the solution to the problem of soil compression under structural loads. The second aspect of the problem is the rate of settlement under the applied load. The time required for the settlement to occur vis-à-vis the life span of the structure, is an important consideration. It will give us an idea of how much settlement will undergo after it is constructed and whether such a settlement will impair its functioning or not (Ranjan, 1993). The compression settlement of each soil stratum is computed from the consolidation test data and from the soil stresses before and after construction.

In contrast to contact settlement, compression may be a slow process requiring years to develop fully. Estimates of percent of total compression that will occur in any period may be made from the consolidation test data. The thicker the stratum and the lower its permeability, the longer is the time required (George, 1989).

The basic difference in the compression behavior of a granular soil and that of a fine grained soil can therefore be expressed thus; a granular soil compress almost immediately upon loading the compression is relatively small whereas a fine grained soil exhibits time dependent consolidation and compression is rather large (Ranjan, 1993).

The immediate settlement or distortion settlement occurs almost immediately after the load is imposed as a result of distortion of the soil without any volume change. There is such a negligible flow of water out of partially saturated or extremely pervious. The immediate settlement is usually ermined by using the elastic theory, even though the deformation itself is not truly elastic.

The squeezing out of pore water from the loaded saturated soil causing time dependent decrease in volume is known as primary consolidation. Here the rate of flow is controlled by the pore pressure, the permeability and the compressibility of the soil. With the passing of time, as the pore pressures dissipate, the rate of flow will decrease and eventually the flow ceases altogether, leading to a condition of constant effective stress. This signifies the end of primary consolidation (Ranjan, 1993).

2.5 Foundation of Buildings and Types of Foundation

Soils of different behavior response in different ways to any load imposed on it. This response to load is basically controlled by the engineering property of the soil. A soil that has similar mineralogical content can be different in its texture or consistence or cohesiveness or density or etc. Thus, characterizing soil in its index property can give a clue to the engineering behavior of the soil. The foundation of a structure is that part which transmits the load of the structure to the ground and static loading implies a pressure from that foundation that does not change with time (George, 1989). Every engineering structure has foundation with different types and styles based on typical purpose and size of the structure. All structure foundations have one fundamental characteristic in common; that is, they provide a means whereby service and ultimate loads are transmitted from the structure into the supporting geologic medium (Robert, 2010). Any engineering structure needs proper site to be sit on which can support its load without failure so that it can exist for its purpose. The design of foundations embodies three essential operations, namely, calculating the loads to be transmitted by the foundation structure to the soils or rocks supporting it, determining the engineering performance of these soils and rocks, and then designing a suitable foundation structure (Bell, 2007).

A satisfactory foundation must meet three requirements:

1. It must be placed at an adequate depth to prevent frost damage, undermining by scour, or damage from future construction nearby.

2. It must be safe against breaking into the ground
3. It must not settle enough to disfigure or damage the structure

A foundation must be properly located and founded at such a depth that its performance is not adversely affected by factors such as; Lateral expulsion of soil from beneath the foundation, Seasonal volume changes and Presence of adjoining structures.

During the long period of time a soil must support a structure it may be changed by many man-made and natural forces. These should be carefully evaluated in choosing the location for a structure and particularly in selecting the type of foundation and the minimum depth to which it must extend (George, 2009).

According to (Robert, 2010) structure foundations can generally be classified in the following categories: (1) footing foundations (frequently referred to as spread footings), (2) pile-supported foundations (driven and non-driven piles), and (3) special case foundation types that would include micro-piles, tie-backs and tie-downs. Pier columns were once considered specialty foundation types but their use has become more prevalent over the years as they are thought to behave well seismically. The ultimate purpose of a foundation is to transfer the load of a structure to the ground without causing the ground to respond with uneven and excessive movement (Blyth and Freitas, 1974). These foundation types are basically categorized based on the way the load is transferred to soil or rock.

The choice of particular type of foundation depends upon; the magnitude of loads, the nature of the subsoil strata, the nature of the superstructure and its specific requirement. Different foundation types were adopted worldwide for safe and long lasting of engineering structures. Most buildings are supported on one of four types of foundation, viz. pads, strips, rafts and piles: these may be modified and combined to form a suitable foundation for the ground conditions that exist (Blyth and Freitas, 1974).

Footing foundations transmit design loads into the underlying soil mass through direct contact with the soil immediately beneath the footing (Robert, 2010). Footings distribute the load to the ground over an area sufficient to suit the pressures to the properties of the soil or rock. Their size therefore is governed by the strength of the foundation materials (Bell, 2007). There are two types of footing which are commonly used for light structures. Spread footing or pad footing supports a single column of the structure and a number of footings can be used to support a given structure. Continuous footing/strip footing supports the structure in linear mode prepared under the wall of the structure.

Combined footings are generally required when loading conditions (magnitude and location of load) are such that single column footings create undesirable loading conditions, are impractical, or uneconomical. Combined footings may also be required when column spacing is such that the distance between footings is small or when columns are so numerous that footings cover most of the available foundation area (Robert, 2010).

The amount and rate of settlement of a footing due to a given load per unit area of its base is a function of the dimensions of the base, and of the compressibility and permeability of the foundation materials located between the base and a depth that is at least one and a half times the width of the base (Bell, 2007). Light residential buildings constructed on dense soil use strip footing since it is simple and economically preferable. Footings usually provide the most economical type of foundation structure, but the allowable bearing capacity must be available to provide an adequate factor of safety against shear failure in the soil and to ensure that settlements are not excessive (Bell, 2007).

The reinforced concrete mat foundation is a common type of foundation system used in many buildings. They are a specific type of shallow foundation that uses bearing capacity of the soil at or near the building base to transmit the loads to the soil. Compared to individual spread footings, a mat foundation may encompass all or part of the building's footprint. Compared to an ordinary slab on grade, a reinforced concrete mat is much thicker and is subjected to more substantial loads from the building (George, 2009). A mat foundation is often used where soil and load conditions could cause substantial differential settlement between individual spread footings but where conditions are not so poor as to require a deep foundation system (Robert, 2010). For buildings with significant overturning moments, which can occur in regions of high seismicity or because of irregularities of the superstructure, a mat foundation is commonly used to distribute the bearing pressure over a large footprint and/or to resist significant uplift forces that can develop. Another frequent application for a mat foundation is where individual spread footings would be large and close together. Similarly, where many grade beam ties between footings are required, it may not be economical to excavate and form individual spread footings as compared to building a single mat foundation (Blyth and Freitas, 1974).

For a basement that is below the water table, a mat foundation is often used to create a “bathtub” system to keep the basement dry and to use the weight of the mat to resist hydrostatic uplift forces. Where a mat is supported on deep foundation elements, the mat also functions as the pile cap (George, 2009).

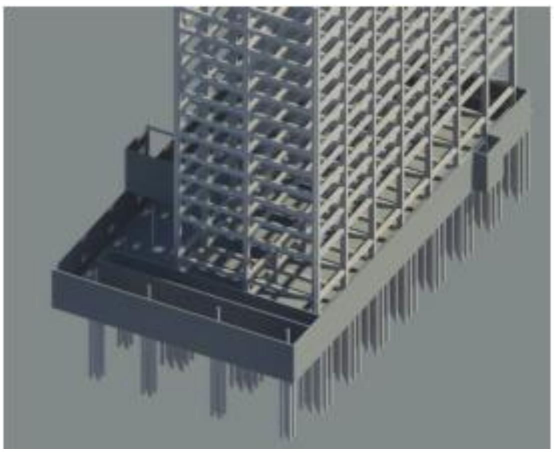


Fig. 2.1 schematic view of deep foundation

In case where the above foundation types are potentially undesirable to support the structure, pile foundation is another option to put structure on it (Fig. 2.1). It is an expensive foundation type where long structural columns are used to support structures above it.

Pile supported foundations transmit design loads into the adjacent soil mass through pile friction, end bearing, or both (Robert, 2010). Massive engineering structures on soft soils are preferably used this type of foundation.

Various geologic and non-geologic features affect foundation type and selection for its intimate performance. Geologic consideration includes soil profile, location of water table and any potential fluctuation and the potential for scour or undermining. Non-geologic consideration includes size and shape of footing, adjacent structures and existing utilities (Robert, 2010).

The intensity of loading that cause’s shear failure to occur beneath a foundation is termed the *bearing capacity* of the ground. This capacity is governed by the fabric of the rocks and soils beneath a foundation and by the reaction of this fabric to changes in effective stress (Blyth and Freitas, 1974).

Foundation design is concerned primarily with ensuring that movements of a foundation are kept within limits that can be tolerated by the proposed structure without adversely affecting its functional requirements. Hence, the design of a foundation structure requires an understanding of the local geological and groundwater conditions and, more particularly, an appreciation of the various types of ground movement that can occur (Bell, 2007).

Several factors can affect the bearing capacity of a particular soil. They include soil type, relative density or consolidation, soil saturation and location of the water table and surcharge loads. These factors can act individually or in concert with each other to increase or decrease the bearing capacity of the underlying soil (Robert, 2010).

Regarding bearing capacity, Blyth and Freitas (1974), different soil materials exhibit in different ways with change in texture, cohesion, water content and shape of grains. Sands and gravels; when dense have a high bearing capacity and low compressibility, but when loose they have a low bearing capacity.

Stiff clays, have a reasonable bearing capacity and moderate compressibility, but are liable to soften when wetted. Soft clays, and carbonate muds are weak. The clays can be extremely sensitive and their bearing capacity is low and their compressibility is high. According to (Bell, 2007) the shear strength of clay is, in turn, influenced by its consistency. Saturated clays in relation to applied stress behave as cohesive materials provided that no change of moisture content occurs. Thus, when a load is applied to saturated clay, it produces excess pore water pressures that are not dissipated quickly. In other words, the angle of shearing resistance is equal to zero.

Blyth and Freitas (1974) Silts are usually soft and weak, especially when saturated, and have a low bearing capacity. Loess and other cemented, but porous sediments may have a high bearing capacity and low compressibility, but be liable to collapse. Loess is susceptible to collapse when saturated. Organic sediments have a high compressibility and low strength often over-estimated by vane tests when the vane becomes entwined in the organic fibers.

Tropical soils (red sandy soils) normally have a high bearing capacity. Black soils are extremely hard materials when dry and appear as attractive founding material, but when saturated they lose their strength and often expand. The failure mechanisms that can occur in rocks usually depend upon movement along pre-existing surfaces, these being the weakest part of many rocks masses. For all types of foundation structures on clays, the factors of safety must be adequate against bearing capacity failure.

Grim (1952), distinguished two modes of swelling in clay soils, namely, inter-crystalline and intra-crystalline swelling. Inter-crystalline swelling takes place when the uptake of moisture is restricted to the external crystal surfaces and the void spaces between the crystals. Intra-crystalline swelling, on the other hand, is characteristic of the smectite family of clay minerals, of montmorillonite in particular. The individual molecular layers that make up a crystal of montmorillonite are weakly bonded, so that on wetting moisture enters not only between the crystals but also between the unit layers that comprise the crystals. Generally, kaolinite has the smallest swelling capacity of the clay minerals, and nearly all of its swelling is of the intercrystalline type. Illite may swell by up to 15% but intermixed illite and montmorillonite may swell some 60–100%. Swelling in Ca montmorillonite is very much less than in the Na variety, it ranges from about 50–100%. Swelling in Na montmorillonite can amount to 2000% of the original volume, the clay then having formed a gel (Bell F., 2007).

CHAPTER THREE OVERVIEW OF STUDY AREA

3.1 The Study Area

The study area is located within two sub-cities in Addis Ababa city. The first one is found in the northern part of Kolfe Keraniyo sub-city a locality known as Asco, specifically at Asco Brick factory. It is accessible by Piazza-Wingate-Arat Menta-to Asco Addis Sefer road and distance wise it is 10km from Piazza.

The rest of the study areas are found in Bole sub-city which is among the largest sub-cities in Addis Ababa. Ehil Nigid is a specific local name of the site formerly it was used for cereal storage and division center. It is around 8km from Piazza to east direction. The third site is located behind imperial hotel formerly used by construction material supply enterprise. It is accessible through Megenagna-Bob Marley square-Gerji to Unity University road. From Piazza it is about 12km to south east direction. Fig. 3.1 shows the study area and its location.

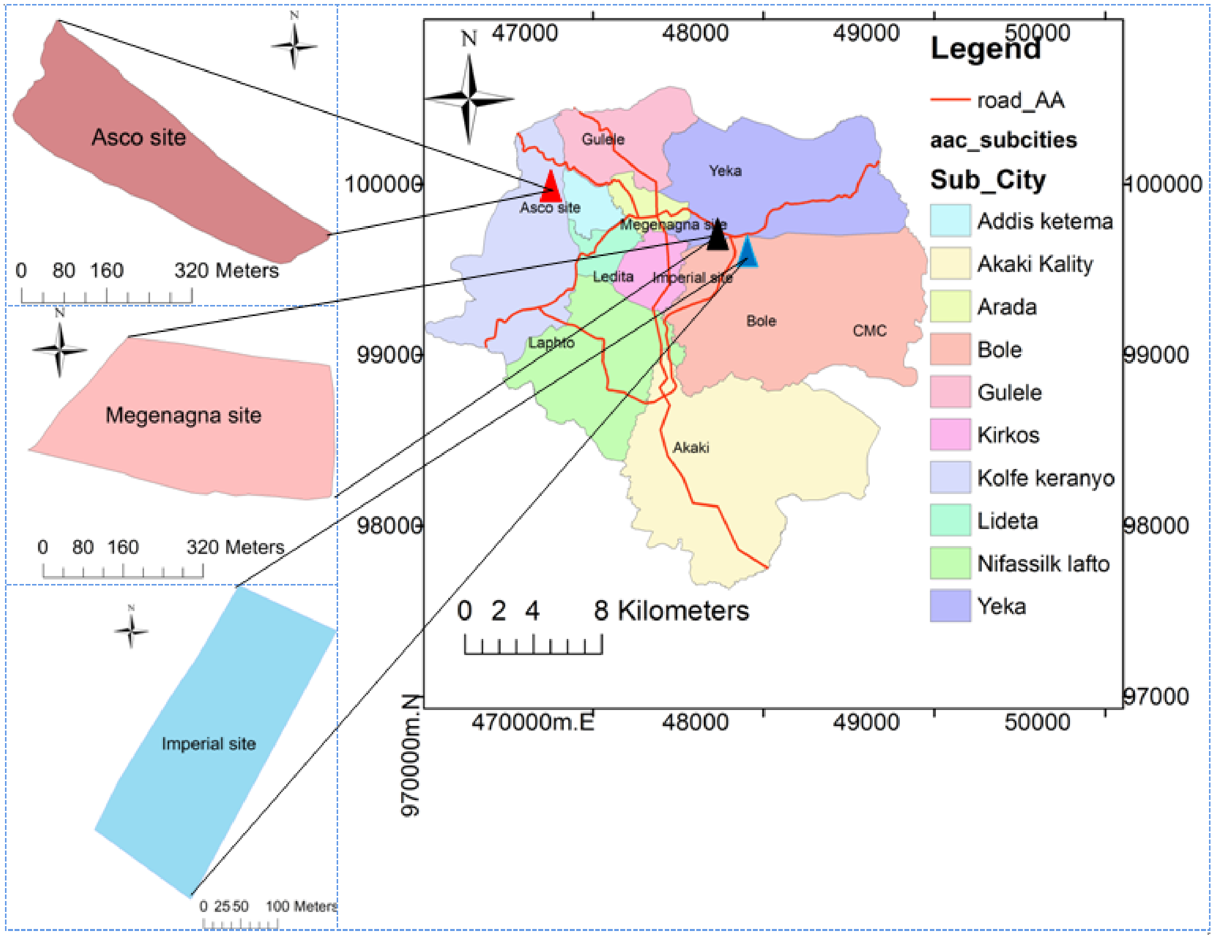


Figure 3.1 location map of the study area

3.1.1 Physiography, Climate and Drainage

The study area is generally characterized by different topography from rugged to flat. In case of Asco site it is rugged topography where as other sites are generally flat. Generally, Addis Ababa is a city with undulating topography close to ridges and relatively flat to the eastern and southern part. The city was founded on the southern flank of Entoto ridge (3199m a.s.l) and expanded in all directions with along the southern alignment of Entoto ridge and extensively to southern part. Ridge marks the northern boundary of the city following the east-west trending major fault, (Ambo-Kessem). Belayneh Desta (2009), marks the prominent volcanic features which mainly built up of acidic and intermediate lava flows such as; Mt. Wechecha (3385m a.s.l) in the west, Mt. Furi (2839m a.s.l) in the southwest and Mt. Yerer (3100m a.s.l) in the southeast are the most elevated areas surrounding Addis Ababa city. Due to topographic difference there are number of drainage lines running from these high ridges to the low elevated land mainly to the south of the city. Most of them are seasonal which has abundant water discharge during summer season and less discharge or no discharge in dry seasons.

Climate of Addis Ababa is categorized as subtropical in which two weather patterns exist. The dry season ranges from mid September to May. In the mid December there is uncommon low rainfall while in March and April there is intermediate rain fall. The rest of the months are characterized by high rain fall (Tamiru et al., 2003) (Table 3.1). Accordingly, the amounts of rainfall during these months are varied from year to year. Tamiru et al., (2003) also presented that the amount of mean annual temperature of the city is 16.02^{0}C.

Table 3.1 Rainfall coefficients of three stations in Addis Ababa, (after Tamiru et al., 2003)

Station	Months											
	Jan.	Feb.	Mar.	April	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
	1	2	3	4	5	6	7	8	9	10	11	12
AA Obs.	0.17	0.43	0.65	0.93	0.86	1.28	2.57	2.78	1.76	0.39	0.1	0.1
AA Bole	0.17	0.44	0.75	1.02	0.87	1.28	2.58	2.69	1.65	0.38	0.11	0.05
Akaki	0.14	0.45	0.62	0.98	0.69	1.34	2.82	3.16	1.46	0.25	0.14	0.04

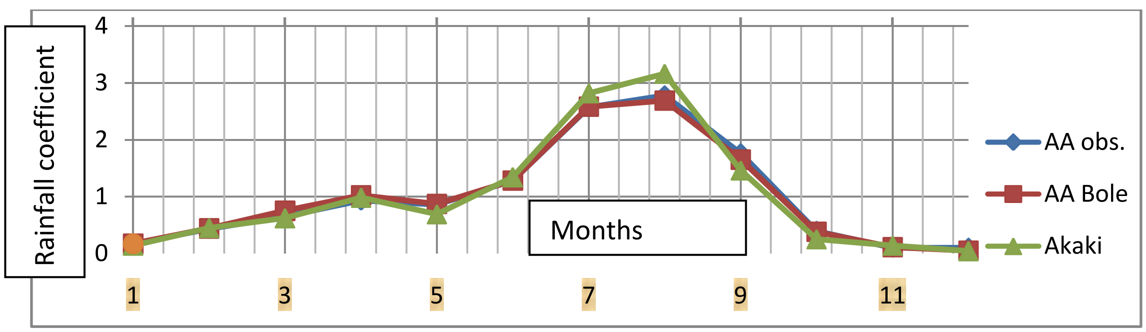


Figure 3.2- Rainfall coefficient (after, Tamiru et al., 2003)

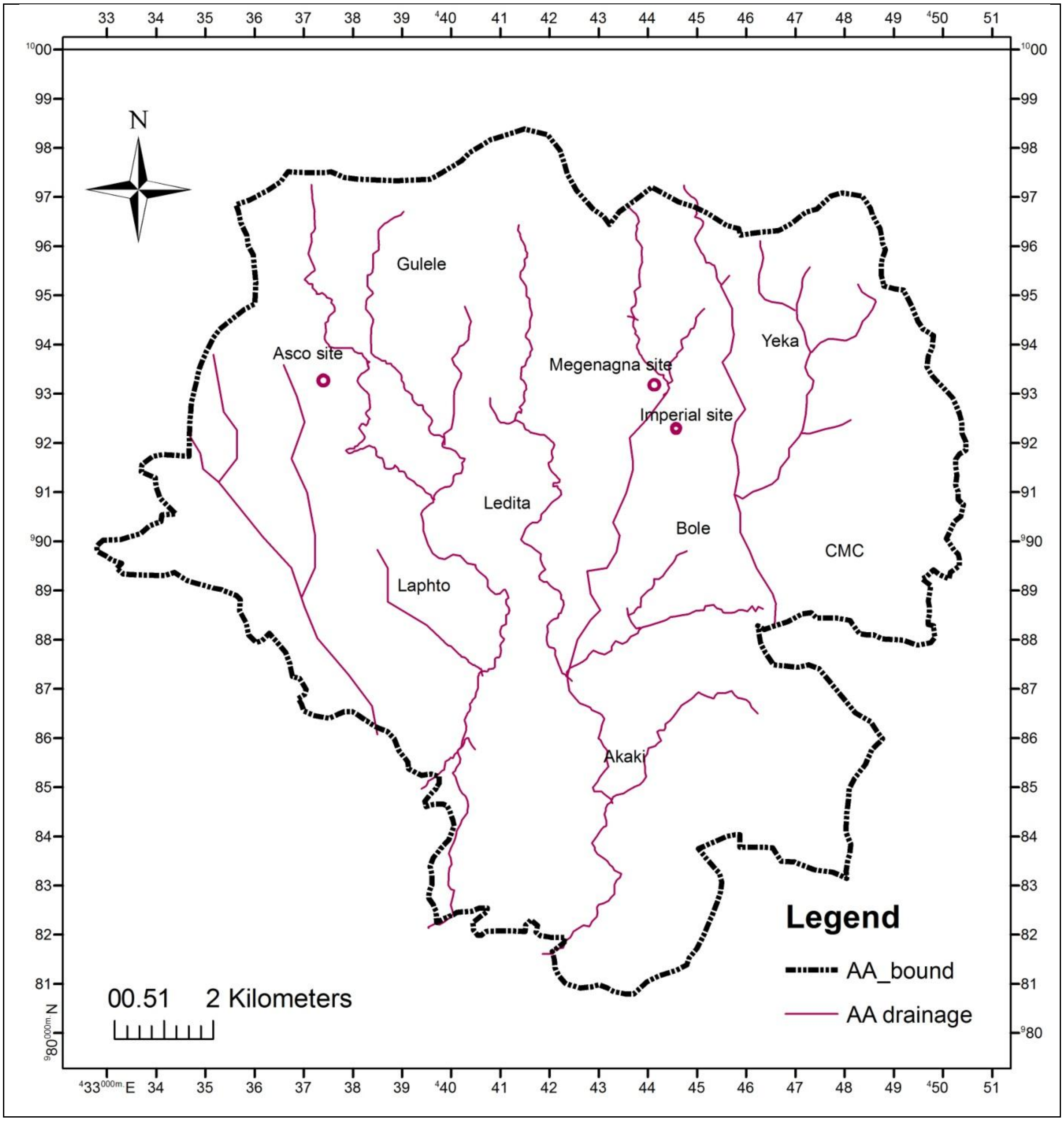


Figure 3.3- Drainage map of Addis Ababa

3.1.2 Regional Geology

The orogenic belts of the Arabian Nubian shield and the Mozambique belt are believed to be more prominent as outcrop in Ethiopia than other countries of Africa (Kazmin, 1972 and Berhe 1990). On the other hand, Mengesha Tefera (1996) has pointed out that the rocks belonging to the orogenic belts are only exposed in few areas which have not been affected by Cenozoic volcanism and rifting.

Cenozoic rocks of Ethiopia are divided into trap and Aden series (Mohr, 1963) in Goshu W/Yohannes. Trap series is used to represent tertiary flood basalt sequences while Aden series represent post rift (middle Miocene-Quaternary) volcanic rocks of the main Ethiopian rift depression.

Tertiary volcanic of central Ethiopia had developed in three stages of volcanism and tectonism; transitional flood basalt, alkali rhyolite and alkali basalt (Zanettin, 1997). During the uplifting of and deep erosion of the outer part of elongated basin occurred by first volcanic, the stage of volcanism died out on the uplifted area. Accordingly, volcanism radically changed to central type (Tarmaber-Megezez basalt) volcano, occurring of extensive crustal movement extended to the escarpments and fissural (Alaji rhyolite and Alaji basalt).

Addis Ababa is part of the western margin of the Ethiopian rift valley and is covered by volcanic rocks ranging from basic to acidic in composition. According to Goshu W/Yohannes (1997), lower and upper volcanic sequences have been recognized, and the onset of volcanic activity of the upper volcanic sequences has been recognized, and the offset of volcanic activity of the upper volcanic sequence was estimated, from their dating and from the available geochronological data, at about 12MY.

According to the geological map of Addis Ababa compiled by Morton (1974) the area consists of older volcanic of Entoto. The contact of welded tuff, dark glassy variety with an overlying welded tuff which has prominent fiamme passes at the western edge of Hilton compound. In the same map, the major Filwoha fault which runs NE-SW and with a down throw on the NW side is indicated to be passing about 500m NW of Hilton hotel.

Kebede Tsehayu and Tadesse H/Mariam (1990) concluded from Justin et al., 1974, Alaji formations (Alaji basalt, Alaji rhyolite and Alaji trachytes) are the oldest with an age of 21.5 MY. They are located in the northeastern part of Addis Ababa forming high topographic ridges. Rhyolite have more areal extension than trachytes, while tuff and agglomerate, which are contemporaneous with rhyolite and trachite of lower volcanic sequences come out of the east-west trending fault during lower Miocene flowing to south and southwest Goshu W/Yohannes (1997).

Goshu W/Yohannes, (1997) also describes the Alaji formation associated with rhyolite and trachyte, which are called Entoto silicic of the Addis Ababa cover, the northern part of the Entoto mountain chains. The basalts out cropping on tops of Entoto silicic are products of the tarmaber basalt, but basalts at the center of the city are called Addis Ababa basalts and they are younger than tarmaber basalts (Meller and Mohr, 1966: Morton and Rex, 1975).

The flat and undulating topography of central of the city has a gradual decrease in topography to the south east consisting of Addis Ababa basalt which erupted during the lower Pliocene after a long period of erosion of the Entoto silicic rocks.

Wechecha (west), Furi (south west) and Yerer (southeast) composed of trachytes are centers of last-phase silicic activities occurred during the upper Pliocene (Mohr, 1966 in Goshu W/Yohannes, 1997)). These young volcanic give intermediate acidic products like trachy-basalt, trachyte, ignimbrite and tuff. Most of faults have sub parallel trend to the main Ethiopian rift faults trending in the direction of NE-SW. there are also some faults in the direction of NW. Trachyte and basaltic dikes are oriented in the direction of NE, NNE and very few in E-W (Goshu W/Yohannes, 1997).

According to Assiged Getahun (2007), the Foota flood basalt is mostly characterized by high weathering effect and locally identified by its spheroidal weathering behavior. It shows alternating layers of vesicular basalt with either porphyritic basalt or aphanitic basalt. It is mainly ridge forming unit with the maximum thickness measure being 480m. The quaternary olivine basalt unconformably overlies the Wechecha-Yerer-Furi ignimbrite and the Wechecha-Yerer-Furi trachite and trachy basalt. On the other hand, the banks of the rivers are covered by tertiary sediments where deposition is maximized by the topography and river channels. It also forms gentle slope and lower topography. The tertiary sediments are overlain by young quaternary basalt and Repi basalt units. The maximum thickness is about 9m which is around Akaki area.

On the other hand, the western and southwestern part of Addis Ababa is covered by Wechecha-Furi-Yerer trachite and trachy basalt. The high topography of Wechecha, Furi and Yerer are characterized by this unit. It is underline by Repi basalt and contact is marked by a thick paleosoil. It is an aphanitic to medium grained in texture with vesicular varies mostly at its lower part. The trachyte and trachy basalt are found alternatively layered with the trachyte being dominant (Assiged Getahun, 2007). The scoria cones are found as either cones or simple domes. Mostly scoria cones/domes are layered and sometimes contain grey vesicular basalt bombs. It is mainly cut by basaltic dykes of different orientations.

Assegid Getahun (2007) also identifies most dominant rock unit and widely exposed in the western part of Addis Ababa. Wechecha-Yerer-Furi ignimbrite contains fragments of ignimbrite, rhyolite and pumice with sanidine phenocrysts. It is fine to medium grained in texture. The top part is identified by columnar jointing where as at its base shows layering and the top part of this unit is covered by pyroclastic deposit which contains phenocrysts of sanidine. The pyroclatic layers are multiple with each layer separated by thin paleosoil. This unit is overlain by Wechecha-Yerer-Furi trachy basalt and trachyte to the central and western part but so the north and eastern part it overlies the lower ignimbrite, Entoto mixed rocks and Repi basalt.

According to Assegid Getahun (2007), the different lithologies in the region lies from Miocene-Pleistocene volcanic succession ranging from older plateau volcanic to younger rift volcanic. Vernier (1985), states that the geology of Addis Ababa is quite complex. Starting from the Entoto ridge, which constitutes the water divide between awash and the Blue Nile basins, steep slopes are present in the northern part of the town giving rise to a very rough morphology characterized by a severe drainage pattern of dendritic, radial and rectangular behaviors. Deep gorges, frequent rapids and waterfalls clearly show evidences of young stage morphology.

Some of the lithologies in sequence from bottom to top are; Alaji basalt, Entoto silicics, Addis Ababa basalt, Lower welded tuff, Aphanitic basalt, Upper welded tuff.

A. **Alaji basalt**: - According to Zanettin (1974) Alaji group volcanic rocks (Alaji rhyolite and basalt) were outcropped from the end of Oligocene until middle Miocene. Hailesillassie Girmay and Getaneh Assefa (1989) also reported the existence of this unit extending from crest of Entoto (ridge bordering the northern part of Addis Ababa) towards the north. Mohr (1967) proved that the Entoto trachyte overlies the Alaji basalt. This unit is underlain by tuff and ignimbrites; on the other hand its stratigraphic relationship with the Entoto silicics difficult to determine as they occur in a fault contact. The age of the rock is 22.8MY and (Morton et al, 1979).

B. **Entoto mixed rocks**: - Entoto mixed rocks are composed of trachyte, ignimbrite, pyroclastic rocks and sediments (Assiged Getahun, 2007). Age wise these rocks are Miocene age and represent localized terminal episodes to massive Oligocene fissure-basalt activity in the Addis Ababa region (Morton, 1979). Assiged, (2007) the high weathering effect characterizes these units and forms thick soil. Zanettin and Justin-Visenstin (1974), marks the lava make up a thick pile of flows accumulated along east west fissures (east-west fault running from Kessem River to Ambo) and uplifted northwards. The unit is unconformably overlain by Addis Ababa basalt on the foothill of Entoto and underlain by Alaji basalt. Assiged Getahun (2007), identifies the commonly occurrence of patches of quaternary basalts in this unit. It is highly affected by joints trending E-W and N290E and forms high mountain chains trending E-W.

Hailesillassie Girmay and Getaneh Assefa (1989) intended to identify the Entoto silicics as compositionally rhyolite and trachyte with minor amount of welded tuff and obsidian and the distinctive occurrence of each units. Accordingly, the rhyolite lava flow outcrop in the top and foothills of Entoto ridge, predominantly in the western side. It also outcrops in the eastern part of the town from the Kokeb Tsibah School to the Benin Embassy with a thickness quite variable as it frequently forms dome structure. In this rock unit flow banding, folding, and jointing are common. The rhyolites are overlain by feldspar porphiritic trachyte and underlain by a sequence of tuffs and ignimbrites. Tuff and ignimbrites are welded and characterized by columnar jointing. The rhyolite made up of phenocrysts of plagioclase and altered rebeckite in a ground mass of glass with iron oxide. The thickness varies and reaches the maximum of 30m nearly Kotebe covering the rhyolitic lava flows.

Geochronologically, these rock units are dated 21.5MA by Morton (1974) and 22MA by Morton et al, 1979. Accordingly Zanettin et al, 1974 concluded from the general stratigraphy established by both rhyolite and trachyte of Entoto silicics belong to the Miocene Alaji rhyolite and basalt sequences.

C. **<u>Addis Ababa basalt</u>**: - These units which are mainly present in the central part of the town are underlain by the Entoto silicics and overlain by lower welded tuff of the Nazareth group. The maximum thickness exceeding the 130m was found at Kechene stream (Zanettin et al, 1974). Addis Ababa basalt units are the oldest rock post-dating of the Entoto silicics.

These units which are mainly present in the central part of the town are underlain by the Entoto silicics and overlain by lower welded tuff of the Nazareth group. The maximum thickness exceeding the 130m was found at Kechene stream. It is porphyritic in texture, composed of labradorite, bentonite, olivine and augite as phenocrysts. The groundmass is made of andesine, labradorite, olivoine, magnetite and pyroxene (Hailesillassie Girmay and Getaneh Assefa, 1989).

Morton, 1974: Vanier et al., 1985, identify the occurrence of these rocks in their distinctive localities of Addis Ababa. Oliovine porphyritic basalt outcrop in the central part of the town that includes Merkato, Teklehaymanot and Sidist Kilo, General Wingate School and French Embassy. The thickness of olivine porphyritic basalt varies from 1m or less in the foothills of Entoto, Lideta airfield and Filwoha to greater than 130m at Kechene stream.

The lower welded tuff overlies both types of basalt nearby the building college, the Kolfe police school, the Kokebe Tsibah School and Yeka-Mariam church (Hailesillassie Girmay and Getaneh Assefa, 1989). On the other hand, only in the George of the Kechene stream the olivine porphyritic basalt is overlain by the plagioclase porphyritic basalt, while elsewhere the relationship between them is very difficult to determine (Vernier et al., 1985). Age determinations for the Addis Ababa basalt give about 7MY and seem to have no time/composition equivalent (Morton, 1974).

D. **<u>Lower welded tuff</u>**: - Lower welded tuff outcrops as small discontinuous body in Filwoha, western parts of Addis Ababa and sululta (Assegid Getahun, 2007). It is glassy with abundant fiamme and has columnar joints. Generally it is overlain by the aphanitic basalt and underlain by the olivine and plagioclase porphyritic basalt. The age of this rock as dated by Morton et al, 1979 at Addis Ababa and sululta is 5.1 and

5.4MY respectively. This age overlap with the period of the activity of Wechecha Trachyte volcanoes and dated 4.6MY. Wechecha is located 15km west of Addis Ababa and probably the resources of the lower welded tuff at the both localities (Morton et al, 1979).

E. **Aphanitic basalt**: - The southern part of the Addis Ababa, specifically the areas of bole international airport and Lideta airfield are characterized by aphanitic basalt unit. It is underlain by a soil horizon that covers the plagioclase porphyritic basalt and overlain by soil horizon and tuff layers that lie below the young ignimbrite (Getaneh Assefa, 1985). The age of the basalt in Addis Ababa ranges from 3.4 to 3.6 MY (Morton et al, 1974).The rock body shows vertical curved columnar jointing together with sub-horizontal sheet jointing kaolin, lenses are present at the contact of this basalt with the younger ignimbrite. This is a sure evidence for the hydrothermal alterations along a NE-SW fracture system; which may affect both the basalt and the Entoto trachyte. Moreover the basalt is overlain by pumeacoues pyroclastic falls and the pyroclastic falls (Hailesillassie Girmay and Getaneh Assefa, 1989).

According to Getaneh Assefa (1985) the trachy-basalt outcrops are common around Repi and nearby General Winget School. It is underlain by the plagioclase and olivine porphyritic basalt and overlain by the younger ignimbrite from which it is separated by tuffs and agglomerates. Its relation with the rock of the group is not clear, but probably younger than the aphanitic basalt.

F. **Upper Welded tuff**: - This rock outcrops all over the southern part of the town including bole, Nefas silk and railway station; nevertheless it is also present in the central and northern parts of the town. It is gray colored, vertically and horizontally jointed and composed of sanidine, anorthoclase, rebeckite, quartz, pumice and unidentified volcanic fragments (Getaneh Assefa, 1985).

In addition to geological field surveying, geophysical investigation was applied by a team of Ethiopian geological survey geophysists. The gravity survey conducted by Goshu W/Yohannes, 1997 draws relevant conclusions from the interpretation of gravity data collected throughout the town. Accordingly, Bouguer anomaly low at Entoto with high elevation and high free air anomaly might be related to silicic center and/or isostatic flexural compensation. However, the southern low bouguer and low topography together with low free air anomaly is explained in terms of low density materials of silicic rocks and sediments, occurring in the topographyally depressed area. As the anomalies are sharp, focused and well-resolved, the lithological-morphological contact identified by the free air and topographic map of Addis Ababa can be located by zero contour with more certainty using second derivative map.

The high gradients revealed in all components of the gravity fields are the results of sharp density contrasts between different lithologies. Thus these gradients mark fault/contacts. Geological evidence suggests that the tectonic fabrics of Addis Ababa are determined by north-west, east-west and north-east trending faults, as the tectonic structural studies of Tsegaye Abebe et al, (1995) indicates. Close inspections of the gravity gradients in all maps show their consistency with the geology.

3.1.3 Seismicity

The East African Rift System (EARS) is one the geologic wonders of the world, a place where the earth's tectonic forces are presently acting to create new plates by splitting apart the old one. In simple terms, a rift can be thought of as a fracture in the earth's surface that widens over time, or more technically, as an elongate basin bounded by opposed steeply dipping normal faults. Form structural point of view it is one the world class example for the continental-continental diverging zone. Different scholars had conducted and also conducting their studies in this region (Kinde Samuel, 2002).

Addis Ababa is only 75-100km away from the western edge of the Main Ethiopian Rift Valley, which is a hotbed of tremors and active volcanoes. The presence of the Filwoha hot springs in the middle of Addis Ababa itself, for example, is nature's reminder that the city lies on fault lines that have been slowly building strains. It is the release of these strains accumulated over the years that cause the phenomenon of earthquake. According to a report published in 1999, a 6.5 magnitude earthquake, which seismologist say could happen in areas of close proximity to Addis Ababa, the country's major city, could cause as many as 4000-5000 deaths, 8000-10,000 injuries and a displacement of as many as 500,000 people and a total damage in excess of 12 Billion Birr (Kinde Samuel, 2002). Pierre Gouin, founder and long-time director of the Geophysical Observatory at the Addis Ababa University has extensively written about earthquake hazards in Ethiopia, particularly from the 1400's to 1977 in his known classic book: *Earthquake History of Ethiopia and the Horn of Africa*. In his book, Gouin describes the earthquakes of 1906 and 1961 that shook Addis Ababa and caused widespread panic.

According to a report by Ferguson (2013), the two plateaus (western Ethiopian plateau and south eastern Ethiopian plateau) are diverging from each other by 1-2cm per year. This extension of earth crust causes many faulting, fracturing, and displacing of the lithosphere. As a result this displacement, the earthquakes scaled to the crust movement can occur and cause several destructions. Due to its location right on one of the major tectonic plates in the world, i.e., the African and Arabian plates, earthquakes have been a fact of life in Ethiopia for a very long time. In the 20th century alone, a study done by Gouin, (1976) suggests that as many as 15,000 tremors, strong enough to be felt by humans, had occurred in Ethiopia proper and the Horn of Africa. A similar study by Fekadu Kebede (1996) the Ethiopian Geophysical Observatory at the Addis Ababa University published in 1996 indicated that there were a total of 16 recorded earthquakes of magnitude 6.5 and higher in some of Ethiopia's seismic active areas in the 20th century alone.

Addis Ababa is practically experienced by different earthquake waves in its history which has a magnitude of medium to high in Richter scale. The most significant earthquakes of the 20th century like the 1906 Langano earthquake, the 1961 Kara Kore earthquake, the 1983 Wondo Genet earthquake, the 1985 Langano earthquake, the 1989 Dobi graben earthquake in central Afar, and the 1993 Nazret earthquake were all felt in Addis Ababa, and the other major cities of Nazareth and Awassa.

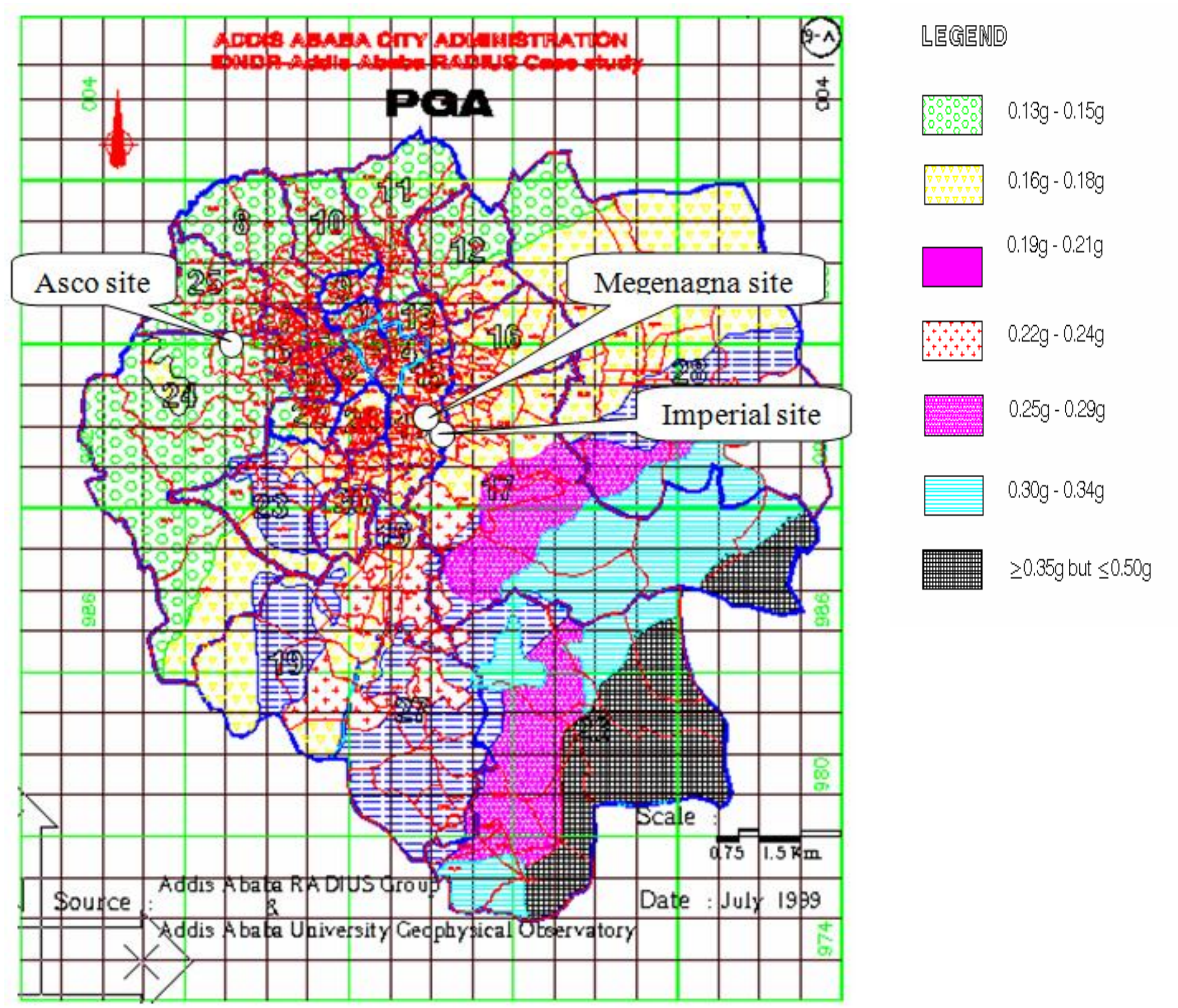


Figure 3.4 Seismic hazard map of Addis Ababa (RADIUS, 1999)

To make matters worse, the soil conditions in certain areas of Addis Ababa, particularly areas like Filwoha, Mesqel Square, Bole, Beqlo Bet, Nifas Silk, Lideta, Mekanisa etc., actually may aggravate the effect of earthquakes. These areas consist of layers of soft soil deposits (as much as 15m deep in the case of the Filwoha area) that further amplify the earthquake induced ground motion. These amplifications of earthquake forces will, inevitably, lead to foundation and structural failures. Buildings and structures in these areas are, therefore highly susceptible to more damage than their counter parts in areas like Entoto, Kolfe and Shola which have a thick layer of basalt rock nearer to the surface (Vernier et al., 1985).

3.1.4 Local Geology

3.1.4.1 Asco site

Geologically Asco site is a made ground where most of the top layer of the soils are damped for some reason. Topographically the site is characterized by gentle slope and the western extension of Entoto ridge plays a significant role in the variation of site topography. The following geotechnical layers were identified from excavations. The word block is used for excavation sites which were conducted to place the building.

<u>Layer 1</u>: coarse grained soil with gravels and cobbles

This is found at the top part of the site and varies in texture and thickness even within a single block site. Cobbles and coarse gravels are the main constitute of the layer. It is highly compressible, loose and easily flows during excavation. Its thickness is significantly variable and also sometimes its orientation is not even. The erratic distribution and uneven nature of this unit with its appearance near to surface reasonably leads to a conclusion of made ground. Plate 3.1 shows the excavation site for building site and shows layering of soils formations. In the left side picture shows the backfilled stream channel with upstream face.

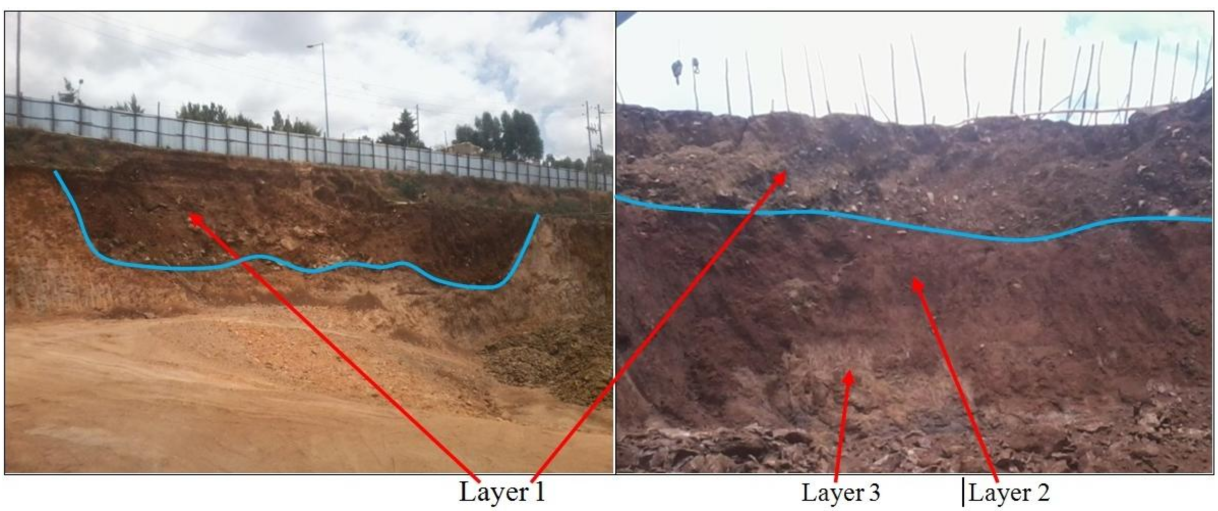


Plate 3.1- excavation outcrop showing different soil stratigraphy of Asco site (Block-9 and 6)

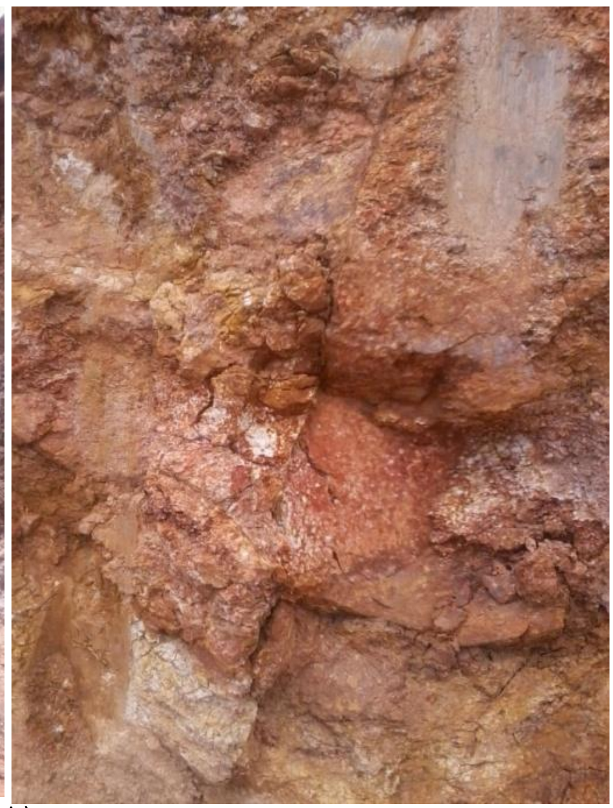
<u>Layer 2</u>: sandy Gravel soil

This layer is identified by its dark reddish color and relatively not complicated as the layer overlaying it. Texturally it is sandy silt and smoothly bedded as layering. It is loose to medium stiff and the slightly compressible. Its thickness ranges from 1.4m to 4.3m. In plate 3.1 the right side picture shows three different soil units. Layer 1 is coarse grained soil with gravels and cobbles, Layer 2 is sandy Gravel soil and Layer 3 is completely weathered rock material.

Layer 3: Completely weathered rock material

The geological formation of Entoto silicic rocks is completely weathered. The third layer is composed of this weathered material with coarse grained remnants of parent rock. Its color is smeared and variegated and its consistency is stiff. The depth of existence is variable from place to place and in most of the blocks it appears around 6m below natural ground level. Plate 3.2 shows the completely weathered rock material around excavation of Block 7.

a)



b)

Plate 3.2 a and b- completely weathered rock material

<u>Layer 4</u>: Soft Soil

In exception to the above soil profile at depth of 6.8m at block-6 a layer of clay soil was observed during excavation. It is uncommon clay soil with fiber strings in it and highly plastic soil. Visually this soil is different from common black cotton soil found around Addis Ababa. In addition to exceptionality of the soil practically the site was used for brick production factory. In this manner it is rational to conclude this clay soil is damped during the production of brick and due time damping of overlaying soil layers were conducted. Plate 3.3 indicates part of soft soil found underlying sandy gravel soil.

Sandy gravel soil Soft soil

Plate 3.3- soft soil underlying sandy garvel soil

3.1.4.2 Megenagna Site

The sub-surface geology of the proposed site is fairly correlated in all block sites. Apart from the concrete fill materials around demolished houses, the top most part of the building site is covered by soft to stiff dark grey and/or reddish brown Silty CLAY with a maximum thickness of 5.70m around block-3. This layer is underlain from top to bottom by greenish to light grey slightly to moderately weathered and widely to closely fractured tuff followed by stiff, reddish and/or brownish grey clayey SILT and then medium dense to very dense, brownish grey silty SAND with gravel. Underneath these soil layers, brownish grey, weak, moderately to highly weathered and widely to closely fractured Welded TUFF is encountered in all the blocks.

Based on visual description, in-situ and laboratory test results, the sub-surface geology is sub-divided into various geotechnical layers. Accordingly, the geotechnical investigations unravel the occurrence of four quasi homogenous geotechnical layers.

The lithology of the top part of the site is characterized by thin layer of lacustrine soil, pyroclastic deposit, and high to medium welding degree of tuff rock. As schematically indicated in the above figure the most top layer is clay soil (black cotton). It is characterized by dark in color highly plastic soil. The average thickness of this unit is about 1.2m overlaid by thin backfill boulder/cobble material. This unit has more or less wide area coverage in Addis Ababa area as it is mostly reported.

Underlying this unit there is a thin layer of red medium to fine grained silty soil. This unit is not common in all parts of the site. Block-4 and Block-5 are the typical location of this medium stiff soil. It vanishes soon as we shift to other blocks. The contact between the clay layer and this unit is not sharp as gradual change in color and texture took place. In both Blocks the thickness is similar (1.5m) and it pinches out and disappears at all. It is loosely compacted and easily compressible by field observation. A medium to high strengthen tuff rock underlies the red silt in Block-4 and Block-5 where as in the rest of blocks it is overlain by clay soil. Texturally medium grained and light grey colored with considerable amount of coarse glassy shinny mineral grains. It is varied in thickness from Block to block and a minimum of 3.5m is measured. In most of the cases it runs below the foundation level and it cannot be measured actually in the field. Structurally it is characterized by vertical joints and also linear horizontal to inclined jointing. Although it is weathered from both of its contacts the base is completely weathered and also underlain by very thin coarse grained easily friable soil. This thin unit has a thickness less than 30cm and identified by its variegated yellowish to light greenish color. From the exposure during excavation the lower most layers observed to be sandy clayey SILT. Light brownish in color and consist of coarse to fine grained particles with low plasticity behavior. The following descriptions are observed in the site during field work.


Plate 3.4- Clay layer at (block 2)

<u>Layer 1</u>: Soft to firm Silty CLAY

Apart from fill material and concrete floor, the top part of the building site is covered by soft to medium stiff dark grey Silty CLAY on the top most part and reddish brown, firm, Clayey SILT on the below the clayey soil with a maximum thickness of 5.70m around block-3. Plate 3.4 shows the CLAY soil which is 1.4m thick in this picture.

<u>Layer 2</u>: Reddish sandy SILT soil

Underlying clay soil reddish sandy silt soil exists with an average thickness of 2m. This soil unit is exceptionally found at block 4, 5 and 6. Plate 3.5 shows three soil formations clay, sandy silt and tuff unit

Plate 3.5- different soil and rock formations at (block 4)

<u>Layer 3</u>: Greenish to light grey, slightly to moderately weathered IGNIMBRITE

This layer is characterized by greenish to light grey, slightly moderately weathered, widely to closely fracture strong welded tuff/ignimbrite. It is found from a depth of 1.00m to 7.30m in block-1, 3.40m to 13.60m in block-2, 5.20m to 13.00m in block-3, 3.10m to 13.50m in block-4, 1.80m to 4.50m in block-5, 1.45m to 6.00m block-6. Plate 3.12 shows the different geologic units exposed during excavation at different Blocks in the site. The left side picture shows the tuff unit becoming less in thickness and finally diminishes in some part. The middle picture shows the inclined jointing in the tuff unit and in the right side picture stratigraphy of the site is clearly seen clay at the top followed by tuff unit and the tuff unit overlies sandy clayey silt soil.

Plate 3.6- vertical lithologic formations at (block 5, block- 3)

<u>Layer 4</u>: Soft to Stiff Clayey SILT

. This layer is characterized by soft to stiff, reddish to greyish brown Clayey SILT with extension from below foundation level around block-5. It is encountered from a depth of 6.30m in block-1, 8.70m in block-4, from 2.90m in block-5, 4.80m to 8.00m in block-6. Plate 3.13 shows the clayey SILT soil underlying welded tuff unit and in this picture the thickness of the visible part is 1.3m.

a

b

Plate 3.7- clayey SILT exposure underlying tuff unit

3.1.4.3 Imperial Site

The subsurface layer conditions were generally evaluated based on site observations and laboratory tests of soil borings drilled on the project sites. In general the subsurface vertical profile encountered at the site may be described by seven major geological soil stratums that can be seen in the geological log.

On a broad scale the geotechnical conditions in the study area are fairly consistent. Most of the area is covered by dark grey, stiff expansive clay (black cotton) soil up to a depth of around 2m and then followed by light grey, moderately strong, highly weathered and fractured tuff units. This layer extends up to a depth of 3.8m. Highly weathered and fragmented vesicular basalt is encountered in Block-2 at a depth of 18.70-20m. Geotechnical characteristic of each unit or layers were explained thoroughly from both field investigation and laboratory results as generalized for the site and described below.

Backfill

The area is previously used by construction materials supply enterprise where production of aggregate and concrete blocks were processed and manufactured. For the purpose of heavy machinery loads they have dumped different materials like red ash/scoria and selected materials. Partly around block four and block 8 the ground surface is covered by concrete material which was used by former institution. Thus, the depth of these back fill material reaches from a minimum of 20cm to a maximum of 50cm.

Layer 1: CLAY

Dark grey, stiff, highly plastic expansive CLAY covers the top natural surfaces of the project area. The thickness of this layer is around varies from 2m-3.6m, the maximum thickness of this unit is recorded at block 4 and block 8. It is observed and true that this layer has sticky nature as PI is very high and is not suitable for putting a foundation on it due to having varied properties. Plate 3.8 is the exposure of clay material during excavation and it covers the top part of the site.


Plate 3.8 clay unit found at Imperial site

Dark grey, stiff, highly plastic expansive CLAY covers the top natural surfaces of the project area. The thickness of this layer is around varies from 2m-3.6m, the maximum thickness of this unit is recorded at block 4 and block 8. It is observed and true that this layer has sticky nature as PI is very high and is not suitable for putting a foundation on it due to having varied properties. The picture is taken from top part of Block-8 after excavation was completed. As it loses its water due to exposure to new environment its surface is cracked and easily detached from its part. It was problematic even excavation as it falls and cannot have enough cohesion to stabilize the excavation wall.

Normally black cotton soil has high swelling value and potential affect light building structures through cracking of walls and floors, disconnected electric wires, pipe lines and mismatching of doors and windows. The other main features of this soil are problems related to settlement. The strength of the soil is very high when it gets dry while revealed very low bearing when it gets water (saturated). Thus, the building is subjected to settlement when the bearing capacity of the soil lowers due to the rising of ground water or recharging from subsurface water.

<u>Layer 2</u>: TUFF

The formation underlying the dark grey soil stratum is light grey, highly to completely weathered and highly fragmented tuff rock. At block 1 the minimum thickness is recorded as 0.4m. The maximum thickness is observed at block 4, 4m. The degree of fracturing is significantly differing from one block to block.

<u>Layer 3</u>: Sandy Silt and Sandy Clay Slit

This is a brownish to greenish grey moderately stiff clayey SILT soil is found underlying the above layer. The depth extends below the foundation depth and relatively it changes its color from block to block and seemingly to accommodate foundation. The field N-value that represents this layer ranges from 13-21. Plate 3.9 shows the geologic formation of imperial site. in this particular picture the top layer is clay unit followed by welded tuff underlien by sandy soil and finally sandy clayey silt soil is found.

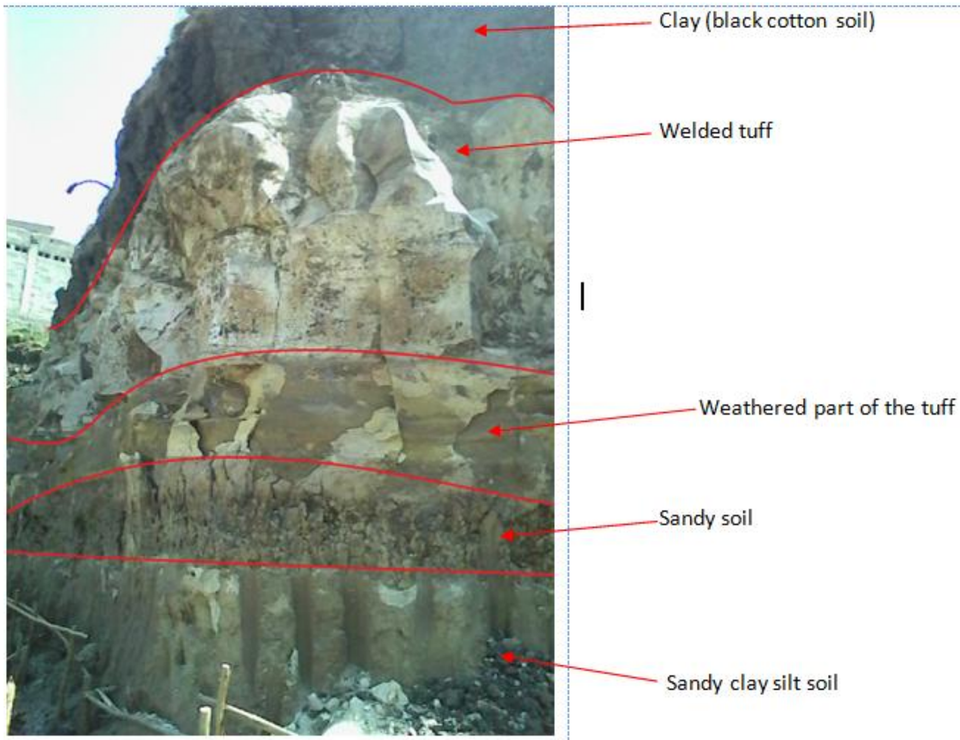


Plate 3.9- soil and rock formations at Imperial site

This site is stratigraphically similar to Megenagna site. As shown in the schematic picture of above, clay soil is the upper part of the layer. Clay (black cotton soil) covers the site with a minimum to maximum thickness of 1.2m and 2.9m respectively. Following the clay layer tuff layer is observed with average thickness of 1.5m. Underlying this unit there is a thin layer intermediate layer between the two tuff units. This unit is very light in density and in most of blocks it appears sandwiching between the upper tuff and lower tuff/Ignimbrite. Texturally this layer is defined as coarse glassy minerals and elongated porphyritc clast inclusions. The lower tuff is characterized by slightly strong than the upper tuff and around block-8 it is more related to ignimbrite than tuff. Plate 3.10 shows the excavation exposure of different layers in the site. as clearly seen from the picture the top dark colored part is clay soil followed by welded tuff unit and Ignimbrite unit at bottom. The two rock units are separated by coarse grained pyroclastic unit which has an average thickness of 0.5m.

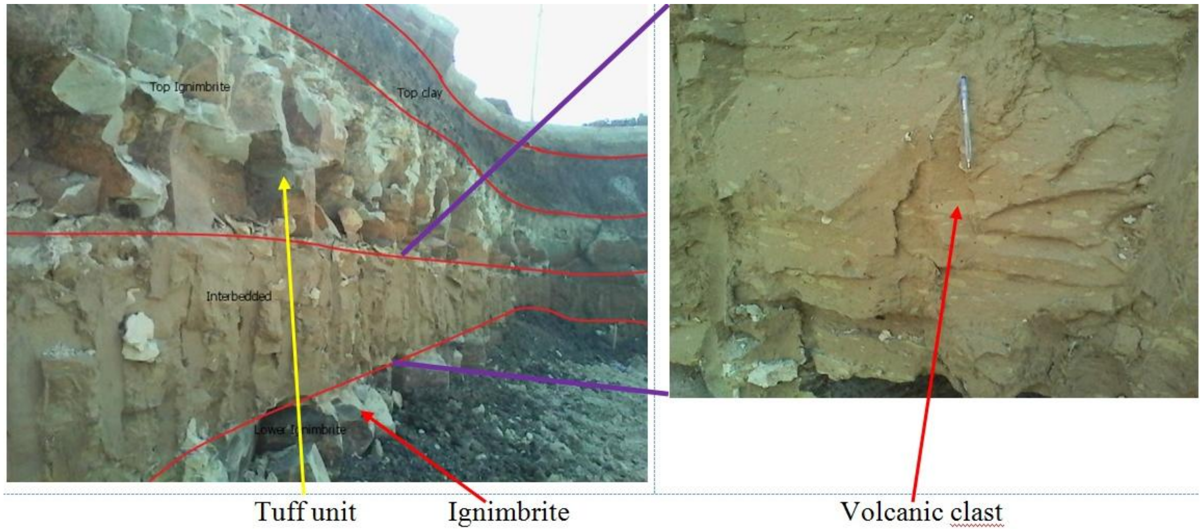


Plate 3.10- Tuff unit and Ignimbrite unit exposure with a sandwitched interbedded pyroclastic layer

CHAPTER FOUR METHODS AND TECHNIQUES

Various data collection methods were exercised to meet the objectives of the study. The two data collection methods are eventually employed for effective data collection from specified targets. The primary data are investigated with laboratory analysis which supports the theoretical descriptions.

4.1 Secondary Data Collection

Secondary data collection is intended to literature review of different pre-existing geological and engineering geological reports and maps has been collected with thorough evaluation of methodology followed and its final findings. Accordingly, regional and local hydro geologic setup, any available soil map and topographic maps has been used to characterize the study areas. Reports on geotechnical investigation of neighboring buildings and infrastructures were also utilized to estimate nature of subsurface condition of nearby sites.

4.2 Primary Data Collection

Primary data are collected from excavation and also borehole logs were used for subsurface sampling and evaluation. Logging and description of different geologic formation of each site emphasizing to soil and rock engineering properties has done. From the excavations appropriate disturbed and undisturbed soil samples were collected.

4.2.1 Field data collections

During field data collection the following were carried out

- Vertical and section logging of excavated block areas at different sites, visual description and characterization of soil and rock strata
- Representative samples were collected from of soil and rock units from representative locations at each sites
- Laboratory tests are involved to evaluate index properties of soil samples collected from site. Different index tests of soil and rock samples were conducted.

4.2.1.1 Asco Site

This site is composed of 13 blocks where space for 11 blocks was excavated and active (Fig. 4.1). Five representative samples were systematically collected from different soil profile as listed in the following table (Table 4.1).

Table 4.1 samples collected from Asco site (grain size analysis)

code	block No	depth (m)	Lab Analysis
ASB5-1	5	6.4	Grain size
ASB3-1	8	2.2	Grain size
ASB9-1	9	5.0	Grain size
ASB6-1	11	4.3	Grain size
ASB13-1	14	6.0	Grain size

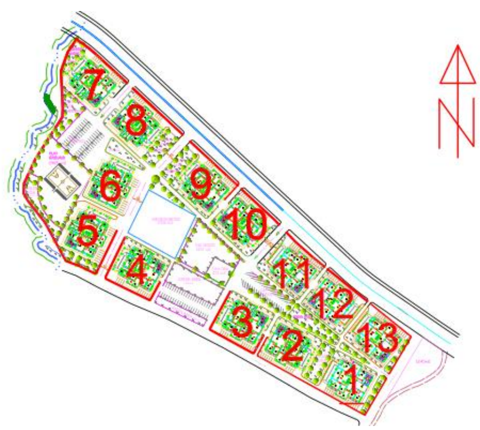


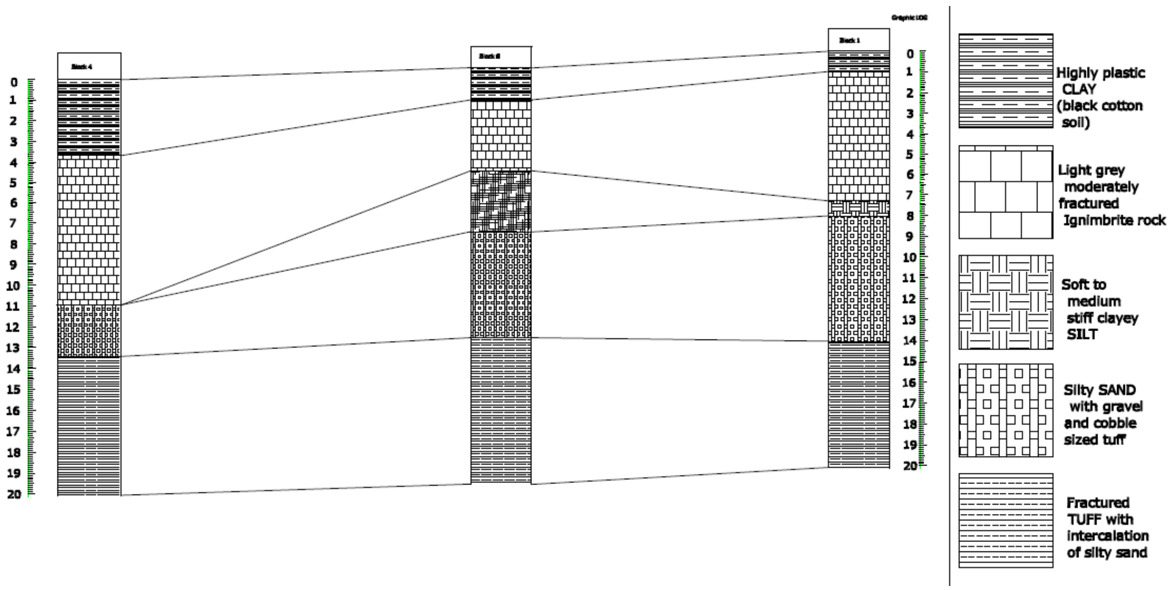
Fig. 4.1 Asco site layouts

4.2.1.2 Megenagna Site

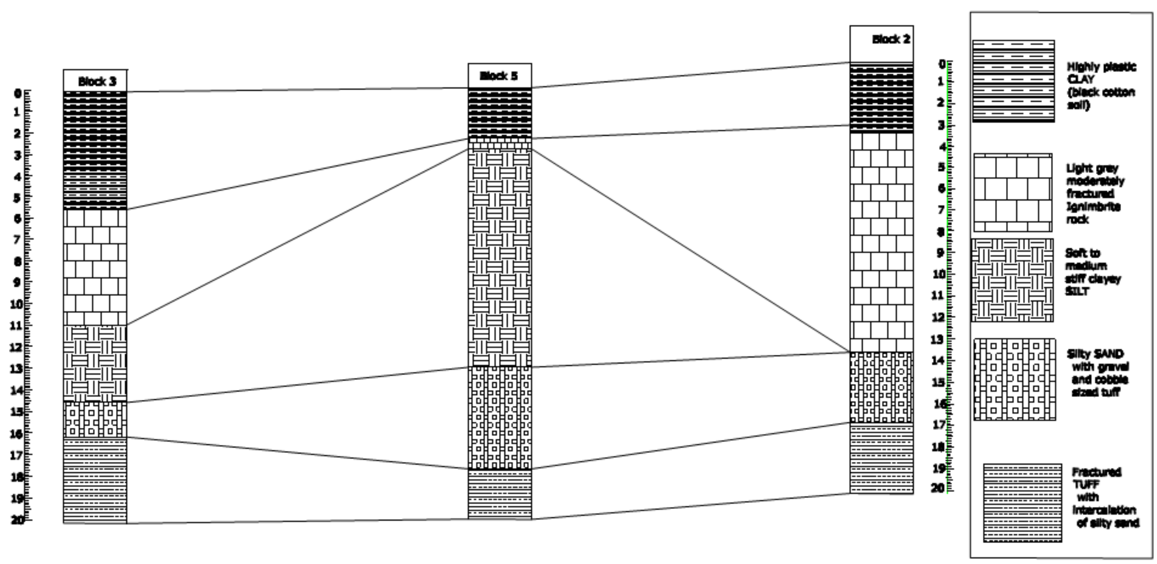
Here 6 blocks were designed and bulk excavation was conducted (Fig. 4.2). The site is more or less similar in geology with a variety of layer thicknesses from one block to the other block. Some of rock and soil layer pinched out in some blocks.

Fig. 4.2 Megenagna site layout (AB and CD are cross section lines)

Fig. 4.3 a and b shows the geologic profiles of Megenagna site. Different soil and rock layers are identified from the site and accordingly, the top soil layer is clay soil underlie by welded and moderately fractured tuff unit. Underlying the tuff unit soft to medium stiff silt occurred and pinches out in some part of the blocks which is underlie by silty sand soil. The bottom unit is highly fractured tuff with intercalation of silty sand soil.



a) AB line



b) CD line

Fig. 4.3 a and b, cross-sections of Megenagna site

12 samples were collected from representative soil profiles and rock strata. The type of samples and their laboratory analysis is summarized in the following table (Table 4.2).

Table 4.2 list of laboratory samples from Megenagna site

code	block No	depth (m)	type of laboratory conducted			
			grain size	consistency	UCS	consolidation
MSB6-1	6	6.0	✓	✓		
MSB2-1	2	4.3	✓	✓		
MSB5-1	5	5.5	✓	✓		
MSB6-2	6	5.0	✓	✓		
MSB1-1	1	6.7			✓	
MSB7-1	7	6.3			✓	
MSB1-2	1	6.9				✓
MSB5-2	5	7.0				✓
MRB2-1	2	6.3			✓	
MRB4-1	3	5.3			✓	
MRB1-1	1	3.3			✓	
MRB5-1	5	5.5			✓	

4.2.1.3 Imperial Site

Geologically imperial site is characterized by soil and rock layers with relatively uniform in occurrence. From the excavation data the site is characterized by ups and downs of elevation of strata's and some pinching out of layers. This site consists of 8 blocks as clearly seen in the picture below (Fig. 4.4). The following table lists out the samples collected from the site and corresponding with their respective type of laboratory analysis (Table 4.3). Fig. 4.5 a and b shows the cross sectional view of imperial site.

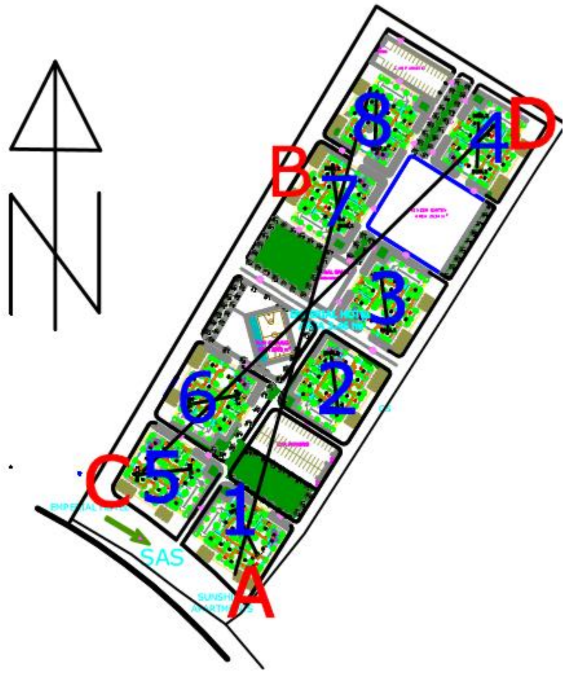
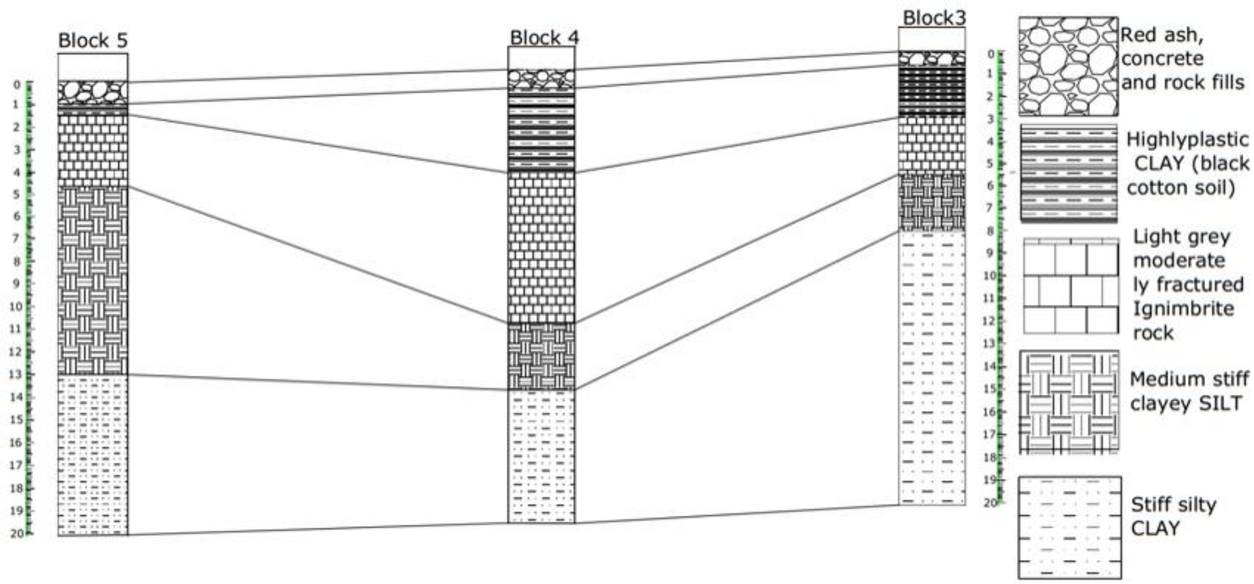


Fig. 4.4 Imperial site layouts (AB and CD are cross section lines)

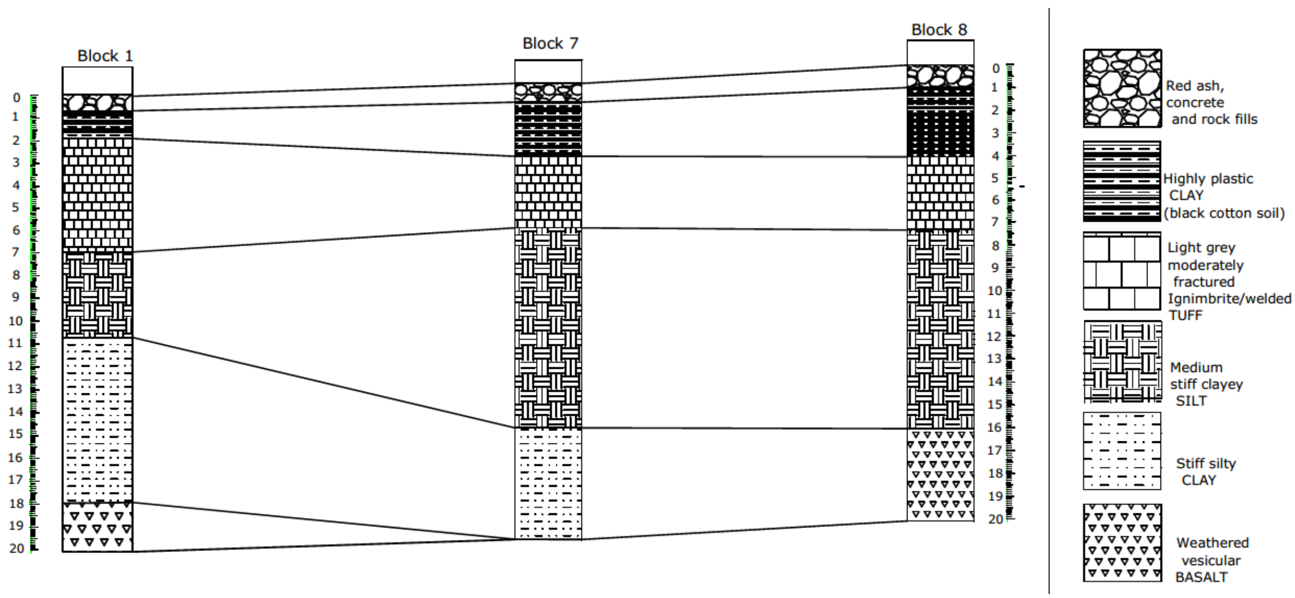
Table 4.3 list of samples collected from Imperial site

Code	Block No	Depth (m)	Type of Laboratory Test Conducted			
			Grain size	consistency	UCS	consolidation
ISB5-1	5	6.1	✓	✓		
ISB6-1	6	6.0	✓	✓		
ISB7-1	7	4.2	✓	✓		
ISB1-1	1	6.7	✓	✓		
ISB5-2	5	7.0			✓	
ISB2-1	2	6.0			✓	
ISB2-2	2	6.0				✓
ISB6-2	6	6.8				✓
IRB6-1	6	3.8			✓	
IRB4-1	4	6.0			✓	
IRB4-2	4	5.4			✓	
IRB8-1	8	6.8			✓	



a) CD line

Fig. 4.5 a and b shows the cross section of Imperial site the red ash and concrete materials are backfills and the top layer is clay soil overlying moderately fractured tuff unit. Medium stiff clay is the third layer from top to bottom and underlie by stiff silty clay.



b) AB line

Fig. 4.5 (a and b) cross section view of Imperial site

4.2.2 Laboratory Testing

Index properties are used to classify soils, to group soils in major strata, to obtain estimates of structural properties, and to correlate the results of structural properties tests on one portion of a stratum with other portions of the site or other similar deposits where only index test data are available. Either representative disturbed or undisturbed samples were utilized. Tests were assigned after review of boring data and visual identification of samples recovered. The following laboratory tests were conducted for this study to evaluate the soil and rock engineering property and of representative samples collected from sites.

- Particle distribution
- Consistency limit
- Unconfined compressive strength
- Consolidation test
- Natural Moisture content
- Free swell
- Unit Weight
- Direct shear

4.2.2.1 Particle Size

The usual procedure is to use a system of sieves having different mesh sizes, stacked on top of each other, with the coarsest mesh on top and the nest mesh at the bottom. After shaking the assembly of sieves, by hand or by shaking machine, each sieve will contain the particles larger than its mesh size, and smaller than the mesh size of all the sieves above it. In this way the grain size diagram can be determined. Special standardized sets of sieves are available as well as convenient shaking machines.

The size of the particles in a certain soil can be represented graphically in a grain size diagram. Such a diagram indicates the percentage of the particles smaller than a certain diameter, measured as a percentage of the weight. A steep slope of the curve in the diagram indicates a uniform soil; a shallow slope of the diagram indicates that the soil contains particles of strongly different sizes. For rather coarse particles, say larger than 0.005mm, the grain size distribution can be determined by sieving (Bowels, 1996).

Sieve analysis is normally conducted on soil samples where most particles will be retained on the No. 200 (0.075 mm) while the hydrometer test is conducted on soil samples where a majority of particles will pass the No. 200 sieve. In the sieve analysis, the soil sample is shaken through a stack of wire screens with standard size openings. The side dimension of a square hole thus becomes the definition of particle diameter. Particle size distribution curves can be used for soil classification, determination of hydraulic conductivity, identification of frost-susceptible soils and assessment of soil strength (Ranjan, 1993). Table 4.4 shows the percentage of fines passing each sieve sizes for samples collected from the sites.

Table 4.4 sieve analysis of soil samples from the study area

Site	Sample code	Percent of fines passed sieve size (mm)						
		12.5	6.3	4.75	2	0.6	0.425	0.075
Asco	ASB3-1	100	96.4	88.9	72	46.5	40.5	10.6
	ASB5-1	100	89.8	75.4	71	35.7	27.5	7.1
	ASB9-1	100	85.7	76.7	59	35.8	24.9	2.3
	ASB13-1	100	96.5	79.5	70.3	57.4	43.2	11
	ASB6-1	100	100	98.4	90.3	76.6	74.5	68.2
Megenagna	MSB6-1	100	94.9	92.4	84.1	72.1	69.1	52.4
	MSB2-1	100	98.4	97.4	94.4	90.7	89.9	87.4
	MSB5-1			100	99.6	97.4	96.4	92
	MSB6-2	100	94.9	92.4	84.1	72.1	69.1	56.7
Imperial	ISB5-1	100	100	100	100	100	99	92.81
	ISB6-1	100	100	100	100	99	96	89.7
	ISB7-1			94	92	91	90	87.79
	ISB5-2				99	97	95	87.39

4.2.2.2 Consistency Limit

The plastic limit (PL) is the moisture content of the soil at the boundary between the plastic and semi-solid states. The liquid limit (LL) is the moisture content of the soil at the boundary between the liquid and plastic states. At moisture contents greater than the liquid limit, the soil has little or no shear strength. The plasticity index (PI) is the range in moisture content between the liquid limit and the plastic limit, and represents the range of moisture contents over which the soil exhibits plastic deformation. The shrinkage limit (SL) is the moisture content below which an unloaded soil will not change in volume (George, 1989).

Plastic Limit

The plastic limit of a soil sample is ascertained by determining the minimum moisture content at which a sample of soil can be consistently rolled into threads 0.125 inches (3mm) in diameter and 10mm length without the material crumbling. Detailed parameters for this test can be found in BS 1377: part 2:1990.

Liquid Limit

The liquid limit test requires Casagrande liquid limit device and a specifically designed grooving tool. The liquid limit of a soil sample is determined by measuring the moisture content at which two halves of a soil mass will flow together over a distance of 0.5 inches (13 mm) along the bottom of a uniform groove separating the two halves, when a bowl containing the soil is dropped 0.4 inches (10 mm) at a rate of two impacts per second (Bell, 2007). Detailed parameters for this test can be found in BS 1377: part 2:1990. Table 4.5 shows the laboratory results of samples collected from Megenagna site and Imperial site and the percentage of liquid limit, plastic limit and plasticity index are presented in the table.

Albert M. Atterberg defined five different water contents describing soil consistency, know referred to as the Atterberg limits. Starting from a very wet state and then drying, the five water contents defined by Atterberg include the liquid limit, the plastic limit, the shrinkage limit, the sticky limit and the cohesion limit.

Table 4.5 shows the percentage of PI of samples

Site	Sample code	PL (%)	LL (%)	PI (%)
Megenagna	MSB6-1	49	77	28
	MSB2-1	68	34	34
	MSB5-1	38	74	36
	MSB6-2	49	78	27
Imperial	ISB5-1	29	80	51
	ISB6-1	34	68	34
	ISB7-1	47	58	11
	ISB7-2	45	82	37

4.2.2.3 Unit Weight

The unit weight of a soil is represented by the symbol Υ. Unit weight is commonly expressed in pounds per cubic foot or kilonewtons per cubic meter. Unit weights can be reported as wet unit weight, Υ_{wet} or dry unit weight, Υ_{dry}. Wet unit weight is calculated by dividing the total weight of a mass of soil containing water by its total volume. Dry unit weight is calculated by dividing the weight of dry soil by its total volume. Wet unit weight thus includes the weight of water as well as the soil particles while dry unit weight includes only the weight of the soil particles. Wet unit weight can be converted to dry unit weight by dividing wet unit weight by one plus the water content (Robert, 2010). Table 4.6 shows unit weight of samples from the sites.

Table 4.6 laboratory results of unit weight for samples

sites	Sample code	Unit weight (Kg/m3)
Megenagna	MSB2-1	18
	MSB5-2	17.5
Imperial	ISB1-2	16.8
	ISB5-3	17

4.2.2.4 Unconfined Compression Test

The unconfined compression test is a simple form of triaxial compression test where the confining pressure is zero. Axial force is the only external pressure imposed on the sample. Axial force begins at zero and increases until failure occurs in the sample. The soil sample must be capable of standing in the test apparatus under its own internal strength, so the unconfined compression test is limited to soils having some cohesion (Robert, 2010). A graph is produced (Fig. 4.6) showing axial load Vs unit strain for one sample.

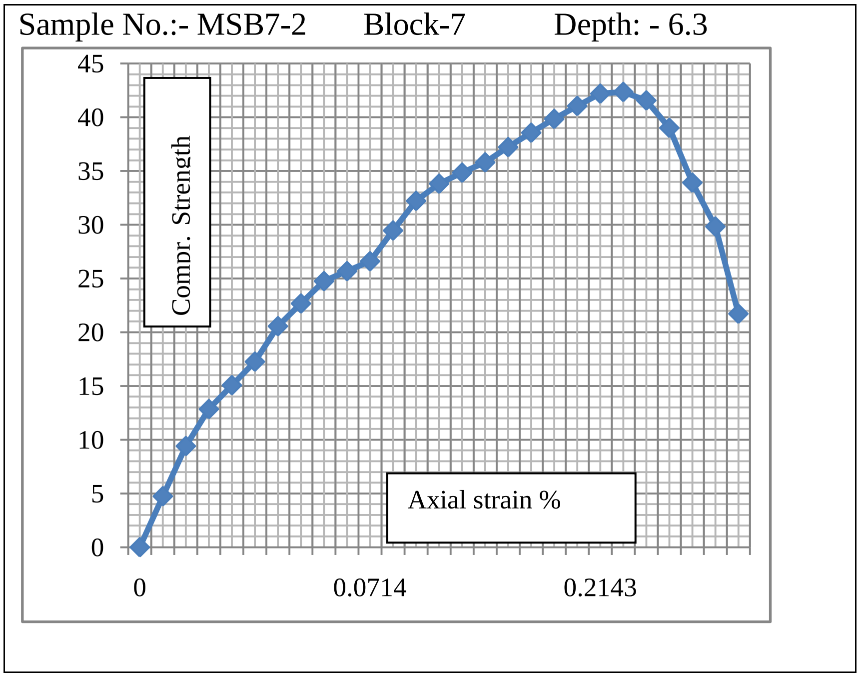


Fig. 4.6 USC of soil sample for one the sample from Megenagna site

4.2.2.5 Moisture Content

Soil moisture content is measured in accordance with BS 1377: part 2:1990. Moisture content is the defined as the ratio of mass of the water in a specimen to the mass of solids in the dry sample. The equation used to calculate moisture content is:

W (%) = Mw/Ms (100%)

The difference in weight between the wet and dry sample is the mass of water, Mw while the weight of the dry sample is the mass of the soil, Ms. Note that the equation defining water content differs from standard equations for determining the percentage of constituent materials. A specimen containing 25 grams of water and 25 grams of dry soil has a moisture content of 100%, but water comprises only 50% of the sample by weight (Ranjan, 1993).

The moisture content test requires only a scale and a means of drying the sample. The soil can be dried at a constant temperature of 110^{o} C using a conventional oven for 15-16 hours

(BS 1377: part 2: 1990), or by using a microwave oven requiring only a few minutes. Moisture content is an important soil property, which has been correlated with shear strength, hydraulic conductivity, compressibility and unit weight of the soil. Moisture content is important for interpretation of moisture-density relationships and forms the basis of Atterberg Limit testing (Ranjan, 1993).

4.2.2.6 Free Swell

There are certain types of soils that can swell, particularly clay in the montmorillonite (a very soft phyllosilicate group of minerals that typically form in microscopic crystals, forming a clay) family. Swelling occurs when the moisture is allowed to increase causing the clay soil to increase in volume. There are a number of reasons for this to occur: the elastic rebound of the soil grains, the attraction of the clay mineral for water, the electrical repulsion of the clay particles and their adsorbed cations from one another, or the expansion of the air trapped in the soil voids. In the montmorillonite family, adsorption and repulsion predominate and this can cause swelling (George, 2009). Table 4.7 shows the laboratory results of NMC (natural moisture content) and FS (free swell) of samples collected from the study area.

Table 4.7 shows the percentage of NMC and FS

Sites	Sample code	NMC %	free swell %
Megenagna	MSB3-2	26	50
	MSB2-2	37	160
	MSB4-1	115	23
	MSB6-1	30	40
Imperial	ISB5-2	27	155
	ISB8-2	40	25
	ISB1-1	36	45

4.2.2.7 Consolidation Test

The consolidation properties determined from the consolidation test are used to estimate the magnitude and the rate of both primary and secondary consolidation settlement of a structure. This test is performed to determine the magnitude and rate of volume decrease that a laterally confined soil specimen undergoes when subjected to different vertical pressures. From the measured data, the consolidation curve (pressure-void ratio relationship) can be plotted. This data is useful in determining the compression index, the recompression index and the preconsolidation pressure (or maximum past pressure) of the soil. In addition, the data obtained can also be used to determine the coefficient of consolidation and the coefficient of secondary compression of the soil (Robert, 2010).

The amount of settlement induced by the placement of load bearing elements on the ground surface or the construction of earthen embankments will affect the performance of the structure. The amount of settlement is a function of the increase in pore water pressure caused by the loading and the reduction of this pressure over time. The reduction in pore pressure and the rate of the reduction are a function of the permeability of the in-situ soil. All soils undergo elastic compression, primary and secondary consolidation (Bell, 2007). Table 4.8 to 4.10 shows the laboratory result of consolidation tests for respective sites.

Asco site

Table 4.8 Results of laboratory consolidation test for Asco site

ASB6-2				Depth 6.6-6.9m		
pressure (KN/m2)	change in height (mm)	void ratio (e)	t50 (min)	mean specimen height (mm)	vol. of compressibility ,Mv (m2/MN)	coefficient of consolidation, Cv(m2/yr)
50	0.154	1.4175	18.2	19.93	0.1876	0.571
100	0.453	1.3930	12.7	19.65	0.1507	0.697
200	0.534	1.3576	26.9	19.53	0.0881	0.494
400	0.965	1.3243	29.5	19.10	0.1099	0.3543
800	1.985	1.1983	26.7	18.7	0.1375	0.3254
1600	3.724	1.0360	17	17.12	0.1069	0.5463
800	3.069	1.0628				
200	2.670	1.0853				
50	2.591	1.1975				

Imperial site

Table 4.9 a and b) Results of laboratory consolidation test for Imperial site

			ISB2-2	Depth 5.8-6		
pressure KN/m2	change in height (mm)	void ratio eo	t50 (min)	mean specimen height (mm)	volume of compressibility mv, (m2/MN)	coefficient of consolidation, cv (m2/yr)
200	0.956	0.7549	40	19.52	0.251	0.248
400	1.59	0.6965	80	18.73	0.1722	0.114
800	2.686	0.5955	65	17.86	0.1583	0.128
1600	4.264	0.4501	70	16.53	0.12535	0.1014
800	4.05	0.4698				
200	3.49	0.5214				

ISB6-2			Depth 7-7.2			
pressure KN/m2	change in height (mm)	void ratio eo	t50 (min)	mean specimen height (mm)	volume of compressibility mv, (m2/MN)	coefficient of consolidation, cv (m2/yr)
100	0.16	1.9099	30	19.92	0.0806	0.344
200	0.402	1.8744	25	19.72	0.1235	0.404
400	0.984	1.789	50	19.31	0.153	0.194
800	1.846	1.6626	10	18.59	0.11871	0.898
1600	2.932	1.5033	80	17.61	0.07953	0.1008
800	2.868	1.5127				
200	2.206	1.6098				
100	1.734	1.679				

Megenagna site

Table 4.10 a and b) Results of laboratory consolidation test

MSB6-2				Depth 7-7.2		
pressure (KN/m2)	change in height	void ratio (e)	t50 (min)	mean specimen height (mm)	vol. of compressibility, Mv (m2/MN)	coefficient of consolidation, Cv(m2/yr)
50	0.128	1.4272	15	19.94	0.1288	0.689
100	0.206	1.4177	21.3	19.83	0.0788	0.48
200	0.408	1.393	28.7	19.69	0.1031	0.351
400	0.78	1.3476	40	19.41	0.0968	0.2448
800	0.378	1.2745	30.5	18.92	0.0803	0.3052
1600	2.322	1.1592	31.5	18.09	0.0667	0.27
800	2.088	1.1878				
200	1.682	1.2374				
50	1.036	1.3163				

MSB5-2				Depth 6.8-7m		
pressure (KN/m2)	change in height (mm)	void ratio (e)	t50 (min)	mean specimen height (mm)	vol. of compressibility ,Mv (m2/MN)	coefficient of consolidation, Cv(m2/yr)
50	0.176	1.4015	18.7	19.91	0.1776	0.551
100	0.334	1.3824	13.8	19.75	0.1607	0.735
200	0.498	1.3625	27.5	19.58	0.0841	0.363
400	0.91	1.3126	30	19.3	0.1079	0.3227
800	1.914	1.191	26.7	18.59	0.1388	0.3365
1600	3.376	1.0139	15	17.27	0.1099	0.5168

800	3.124	1.0444				
200	2.882	1.0737				
50	2.534	1.1159				

4.2.2.8 Unconfined compressive Strength for Rocks

Megenagna site and Imperial site are characterized by rock layers which vary in thickness from site to site. Mostly these rock units are founded above and at foundation level and will be excavated to get the needed space for basements. Block-4 and Block-8 of Imperial site, Block-2, Block-3, and Block-4 of Megenagna site are characterized by thick welded tuff/ignimbrite rock. Samples are collected to evaluate the strength of these rocks and the results are tabularized in the table below (Table 4.11).

Table 4.11 Results of laboratory UCS test of rock samples

Sites	Block No.	depth	compressive strength(Kg/cm2)	compressive strength (Mpa)
Imperial	4	5.7	173.65	17.37
	4	6.7	218.1	21.81
	8	5.2	107.51	10.75
	8	6.5	105.52	10.55
Megenagna	1	5.4	29.67	2.97
	2	3	13.51	1.35
	4	4.4	10.86	1.086
	3	3.34	11.12	1.11

CHAPTER FIVE ANALYSIS AND DISCUSSION

5.1 Site characterization

Grain size distribution of the soil samples collected from the sites were analyzed and results are presented in Table 4.4 in Chapter 4. Accordingly, Asco site is characterized by coarse grained soils with little amount of fines. Gravels and sands are the main soil composition of this site. Geologically the layer underlying fill material in the site is completely weathered rock material. The most top layer is characterized by gravel and rock fragment s which is fill material. Fig. 5.1 shows percent of fine passed and gradation curve for soils sample collected from Asco site.

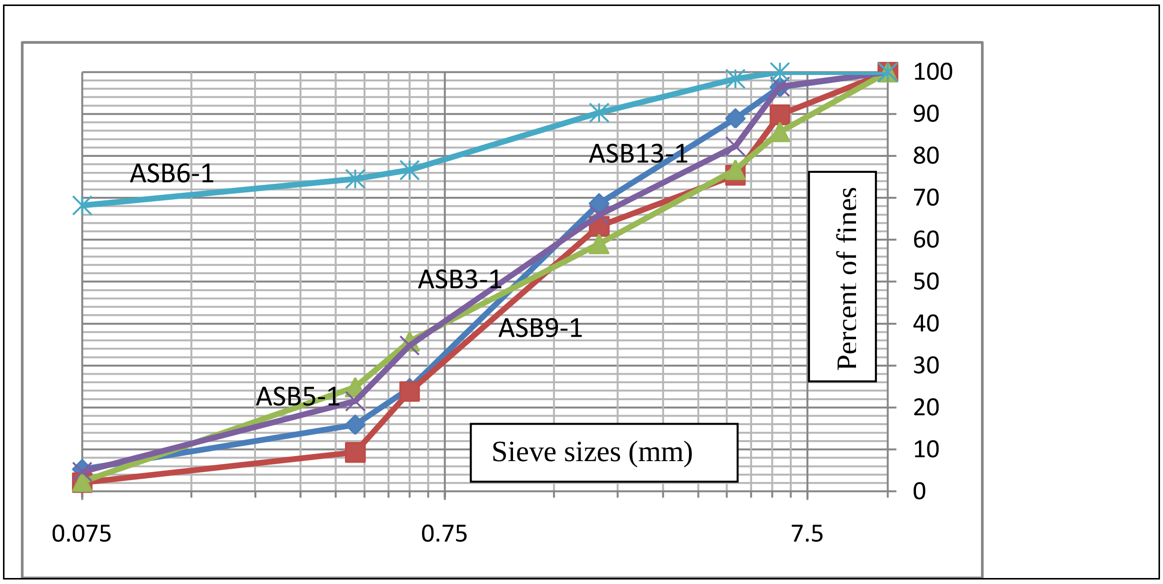


Fig. 5.1 grain size distribution curve for Asco site

The particle gradation curve shows the soil layers are well graded and in some portion of the graphs it seems gap graded. Coefficient of uniformity and coefficient of gradation is given as:

$$C_u = {D_{60}}/{D_{10}} \ and \ C_c = {D_{30}}^2/{D_{10} * D_{60}}$$

Where,

C_u= uniformity Coefficient C_c= gradation coefficient

D_{60}, D_{10} and D_{30} are sieve sizes at respective percent of fines

In Table 5.1 the uniformity coefficient varies from 1.323 to 2.486 and the gradation coefficient varies from 1.612 to 5.991. According to literatures well graded granular soil can have coefficient of gradation ranging from 1 to 3. Hence the soils are well graded granular soil.

Table 5.1 shows the uniformity coefficient and gradation coefficient for Asco site

Sample code	D_{10}	D_{30}	D_{60}	C_u	C_c
ASB3-1	0.08	0.35	0.87	2.486	1.760
ASB5-1	0.079	0.375	0.93	2.480	1.914
ASB9-1	0.32	0.67	0.87	1.299	1.612
ASB13-1	0.082	0.65	0.86	1.323	5.991

The SPT N-value of the Asco site varies from low to high value according to (Bernand, 1990). According to AASHTO, 1988 in Bowels, 1996, the SPT N blow counts fairly correspond to the relative density for granular soils of the soil.

Table 5.2 correlation of average SPT blow count and Relative density, packing state, Friction angle, phi for Asco site

Block No	Average SPT N value	Relative density	State of packing	Friction angle, phi
1	20	Medium dense	Compact	30-35
2	24	Medium dense	Compact	30-35
3	22	Medium dense	Compact	30-35
4	20	Medium dense	Compact	30-35
5	21	Medium dense	Compact	30-35
6	13	Medium dense	Compact	30-35
7	22	Medium dense	Compact	30-35
8	34	Dense	Dense	35-40
9	32	Dense	Dense	35-40
10	29	Dense	Compact	30-35
11	17	Medium dense	Compact	30-35
13	19	Medium dense	Compact	30-35
14	20	Medium dense	Compact	30-35

From Table 5.2 the relative density and state of compactness of the soil fall under a category of medium dense and compact respectively. This is because the average SPT value considered is conducted for the whole length not specific to a given soil layer. The friction angle of the soil also exhibits a fair result averaging to 32.5^0.

According to (Tokimatsu and Yoshimi, 1993) the presence of fines reduces the SPT N-values. The correlation of SPT blow counts and depth exhibits significant reduction where soil layer changes from coarse grain to fine grained soil. Fig. 5.2 shows the SPT blow count against depth, produced by NOVOSPT software and represent average SPT N-value at block-6 and block-8 respectively. In case of block-6 the soil stratification is changed from coarse grained to fine grain and also the lowest average SPT-N-value is recorded at this block at a depth of 5.5m. At, Block-6 at a depth of 5m a dark brown soft soil were exposed during excavation and the graph indicates this change by turning its alignment. Again at a depth of 9m the site geology is characterized by completely weathered rock material and the graph exhibits this change by turning to right side. Actually the point of turn on the graph is at around 7.5m this is because the lower part of the soft soil is consolidated and potentially resistant to SPT blow similar to underlying soil. Block-8 is also characterized by high average SPT N-value from the site the graph also exhibit keep increasing in values. For the rest of blocks average SPT blow count and depth correlation is attached under appendix.

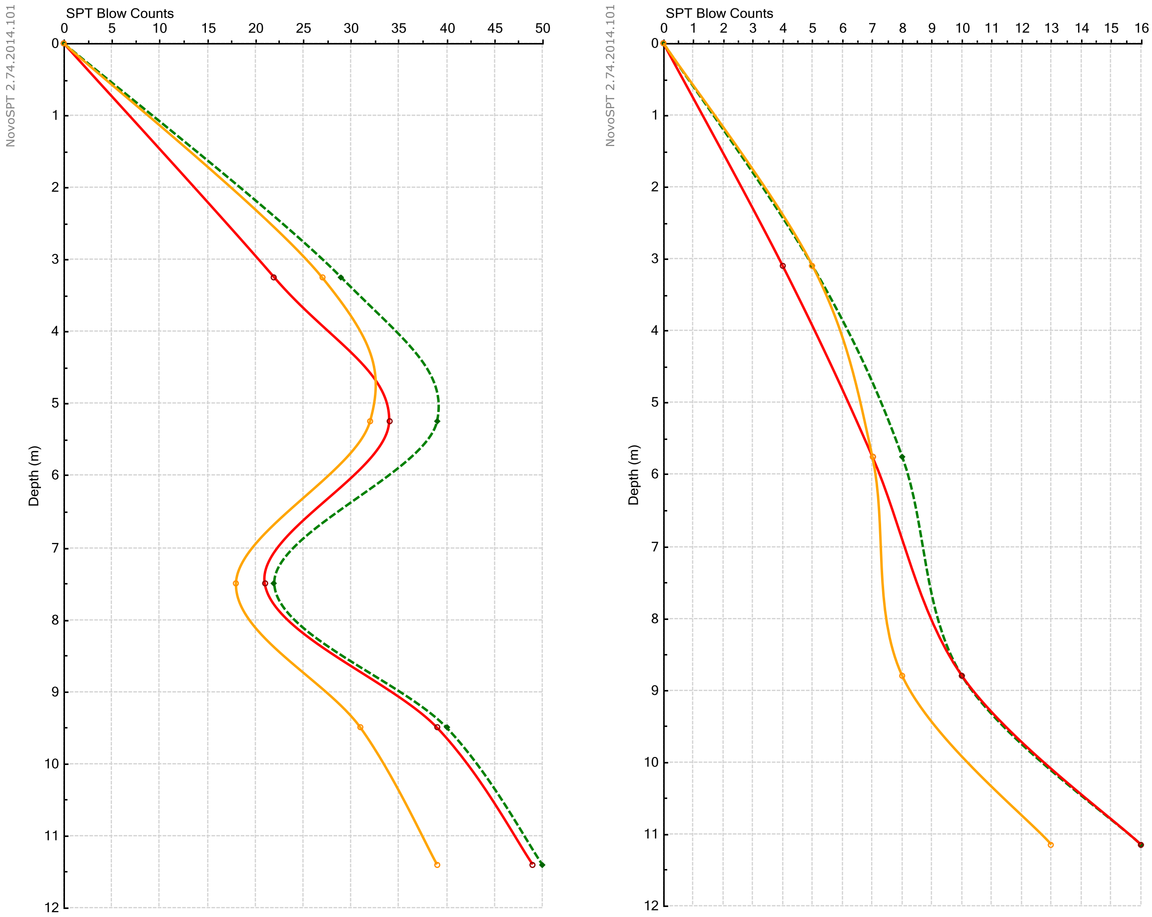


Fig. 5.2 Asco Site SPT Vs Depth produced by SPT NOVO Software green-SPT Blow count, Yellow-SPT $N_{(60)}$ and Red-SPT $N_{1(60)}$ (right left side Fig. is for block-6 and the right side is for block-8)

Megenagna and Imperial sites are generally characterized by fine grained soil. Clay is the most top layer of both sites and followed by thick welded tuff/ignimbrite overlies clayey SILT soil. Table 4.4 in Chapter 4 presents the percentage fines of these two sites. In the table the percent of fines passed 0.075mm sieve is higher which indicates the layer below welded tuff/ignimbrite is fine grained in texture. Similarly, the imperial site is also characterized by fine grained clayey SILT material. The SPT blow count against depth in both Megenagna and Imperial site (Fig. 5.3 and 5.4) also exhibits the change is soil texture where low SPT indicates the fine grained soils. The turning of the graph to right side also indicates at respective depth there is an occurrence of more coarse grained soil. Similar graph are attached under Appendix for the three sites.

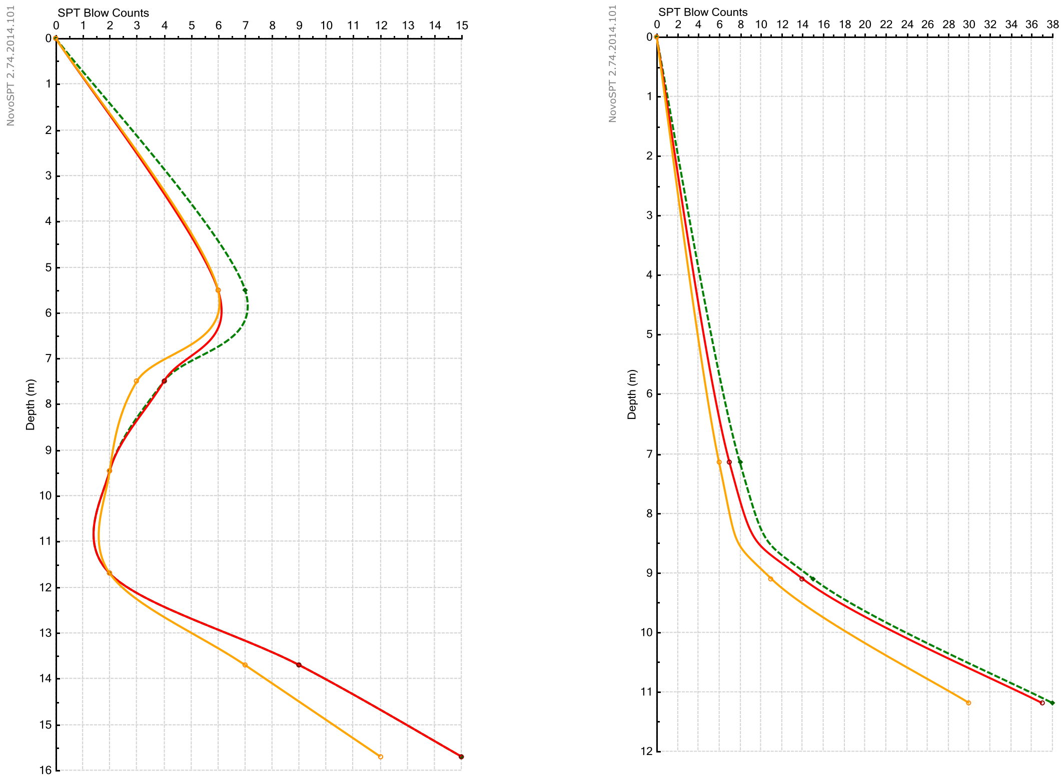


Fig. 5.3 Megenagna site SPT Vs Depth produced by SPT NOVO Software green-SPT Blow count, Yellow-SPT $N_{(60)}$ and Red-SPT $N_{1(60)}$ (right left side Fig. is for block-5 and the right side is for block-2)

Consistency behavior of soils in Megenagna, Imperial and Asco sites are presented in Table 4.5 in chapter 4. The plasticity index of the representative samples ranges from 19% to 51%. The plasticity state of the soils is presented in Table 5.3 below.

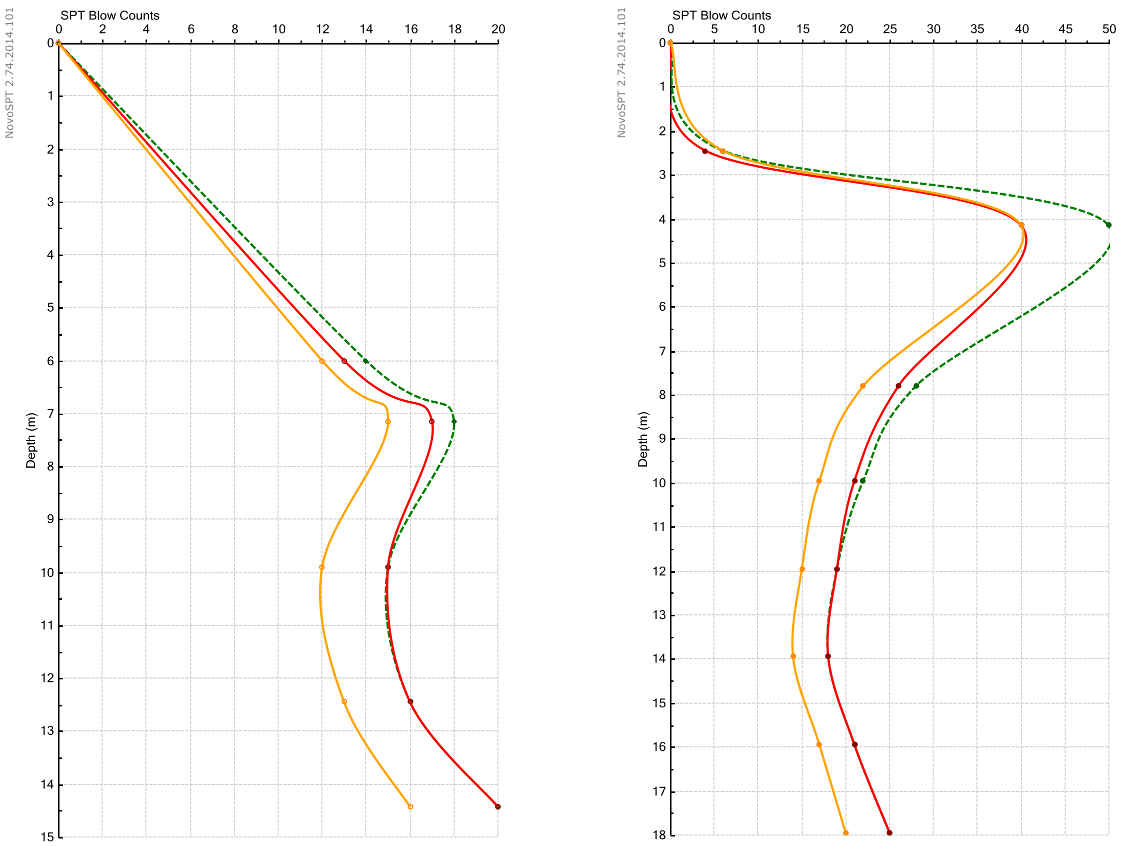


Fig. 5.4 Megenagna site SPT Vs Depth produced by SPT NOVO Software green-SPT Blow count, Yellow-SPT $N_{(60)}$ and Red-SPT $N_{1(60)}$ (right left side Fig. is for block-5 and the right side is for block-2)

Table 5.3 laboratory results of PI and state

Site	Sample Code	PI (%)	State
Megenagna	MSB6-1	19	Medium Plastic
	MSB2-1	34	High Plastic
	MSB5-1	36	High Plastic
	MSB6-2	28	High Plastic
Imperial	ISB5-1	51	Very High Plastic
	ISB6-1	34	High Plastic
	ISB7-1	11	Medium Plastic
	ISB7-2	37	High Plastic
Asco	ASB5-2	46	Very High Plastic

Plasticity chart is developed from plasticity index and liquid limit showing most of the samples fall below A-line (Fig. 5.5) indicating the soil is highly plastic clayey silt soil. According to Bowels, (1996) the major problems of these soils regarding engineering structures are often associated with both bearing-capacity considerations and consolidation settlements. Silts with a large PI may be called plastic silts. These silts exhibit nearly the same characteristics as those of soft clays

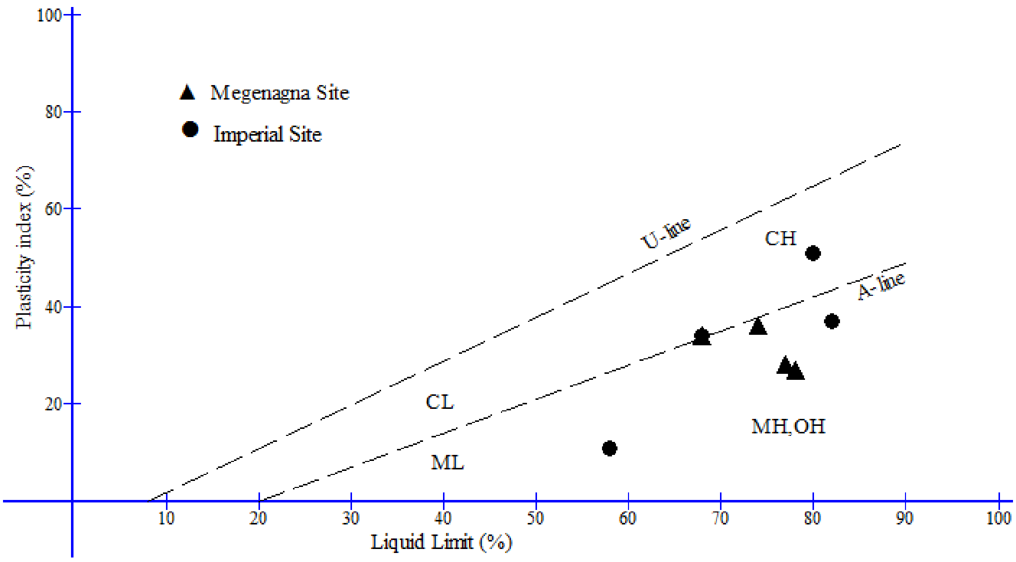


Fig. 5.5 Plasticity chart

The plasticity chart in Fig. 5.5 shows most of the samples fall under A-line. This indicates that the samples can be categorized as high plastic silty soil. One sample from taken from Imperial block-5 falls above A-line indicating the soil is high plastic CLAY soil.

AASHTO, (1988) also correlates the average SPT blow counts with consistency of the soil. Table 5.4 exhibits the correlation of and most of the samples fall under a category of very stiff consistency except at block-3 and 5 of Megenagna site where medium consistency is correlated. This indicates in both sites the soil is potentially consistent.

Table 5.4 Correlation of SPT blow count and consistency of soil at Megenagna and Imperila sites (AASHTO, 1988)

Sites	Block No	Average SPT N value	Limits	Consistency
Meegenagan	2	24	16-30	Very stiff
	3	34	31-60	Hard
	5	6	5-8	Medium stiff
	6	20	16-30	Very stiff
Imperial	1	22	16-30	Very stiff
	2	17	16-30	Very stiff
	3	16	16-30	Very stiff
	4	14	9-15	Stiff
	5	16	16-30	Very stiff
	6	22	16-30	Very stiff
	7	21	16-30	Very stiff
	8	19	16-30	Very stiff

Table 5.5 soil classification at Megenagna and Imperial site

Sites	Sample code	Depth (m)	unit weight	NMC %	Specific gravity	Wet sieve (mm)			Atterberg Limit			USC Classificatio
						2	0.425	0.075	LL	PL	PI	
Megenagna	MSB5	5.5	-	46.6	-	99.2	97.4	92.9	73	38	35	MH
	MSB6	6.7	10.7	52.9	2.54	85.2	68.4	60.6	65	44	21	MH
	MSB3	6.5	-	-	-	99.9	98.5	76.6	35	21	14	CL
Imperial	ISB5	6.5	-	-	-	99	96	87.39	80	29	51	MH
	ISB6	7	-	-	-	100	92	53.85	52	25	27	MH
	ISB1	7				95	77	37.9	36	19	17	CL

Table 5.5 exhibit the soil classification at Megenagna and Imperial site, where most of the soil sample falls under a category of high plastic SILT soil according to Unified soil classification system.

According to Bowels, (1996) most deposits contain some clay particles (with the resulting plasticity/cohesion) or quantities of fine to medium sand. In passing, note that as little as 5 percent clay can give a silt "cohesion"; 10 to 25 percent clay particles may result in the deposit being a "clay." In both these types of soil it is necessary to make a best estimate of the allowable bearing capacity to control a shear failure with a suitable factor of safety and to estimate the probable consolidation settlements.

Megenagna and Imperial sites are also characterized by welded tuff unit. These rock layers in some of the cases are capable of found at foundation level (7m). Where the foundation level is supposed to lie on these rock layers, Tuff unit will be the bearing layer and hence, its strength should be evaluated. The thickness of this unit is very instantly variable and it will be a factor for the tuff unit to sufficiently support the super structure. As identified in the table below the rock strength falls to very weak for Megenagna site and to weak for Imperial site. With the exception of a few porous limestone and volcanic rocks and some shales, the strength of bedrock in situ will be greater than the compressive strength of the foundation concrete (Bowels, 1996). Accordingly, the rock angle of internal friction is seldom less than 40° (often 45° to 55°) and rock cohesion ranges from about 3.5 to 17.5MPa. The upper limit on allowable bearing capacity is, as previously stated, taken as /c' of the base concrete or not more than the allowable bearing pressure of metal piles. The angle of internal friction of rock is pressure-dependent, similar to soil. Also, inspection of rock parameters from a number of sources indicates that, similar to sand, we could estimate angle of internal friction to be 45° for most rock except limestone or shale where values between 38° and 45° should be used and in most cases estimate undrained cohesion near to

5MPa as a conservative value. The unconfined compressive strength for rock samples from Megenagna and Imperial site is used to evaluate the general strength of the rock.

Table 5.6 rock compressive strength for megenagna and Imperial site According to (Sabatini et al., 2002)

Sites	Block No.	depth	compressive strength (Mpa)	Limits	Description	Grade
Imperial	4	5.7	17.37	5-25	Weak	R2
	4	6.7	21.81	5-25	Weak	R2
	8	5.2	10.75	5-25	Weak	R2
	8	6.5	10.55	5-25	Weak	R2
Megenagna	1	5.4	2.97	1-5	Very weak	R1
	2	3	1.35	1-5	Very weak	R1
	4	4.4	1.086	1-5	Very weak	R1
	3	3.34	1.11	1-5	Very weak	R1

According to this correlation (Table 5.6) samples from Megenagna site exhibit very weak in strength and samples from imperial site exhibit weak rock strength. This indicates that the tuff unit at Imperial site is stronger than that of Megenagna site.

Unconfined compressive strength is also conducted for soil samples from Megenagna and Imperial site (Table 5.7). NVADACDM, (1992) correlates the consistency of soil with its unconfined compressive strength. The laboratory test of compressive strength of soil samples from the two sites were correlated to it consistency and the soil samples and falls most of them are very stiff to stiff. This indicates the soils are fair in consistency from engineering point of view.

Table 5.7 SPT Vs compressive strength (after, NAVAC DM, 1992)

Site	Block No.	UCS (KPa) Results	Depth	Limits	SPT N-value	Consistency
Megenagna	5	9.6	7.0m	>4	>22	Very stiff
	6	22.7	8.0m	>4	>22	Very stiif
Imperial	6	24.6	7.0m	>4	>22	Very stiff
	1	19.6	7.0m	>4	>22	Very stiff
	5	3.7	8.0m	<4	<22	Stiff

Burmister, (1951a) classified based on its plasticity index (Table 5.8). Accordingly soil samples from Megenagna and Imperial site found to be silty clay to clay. The clay soils are According to Bowels, (1996) the major problems are often associated with the very soft to soft, deposits from both bearing-capacity8 considerations and consolidation settlements.

Table 5.8 soil identification (After Burmister, 1951a in ASTM, annual book of standards, part 19)

Site	Block No.	PI %	Depth (m)	Limits	Degree of overall plasticity	Identification (Burmister system)
Megenagna	6	49	6.5	>40	Very high	Clay
	2	68	7.0	>40	Very high	Clay
	5	38	6.0	20-40	High	Silty clay
	6	49	4.5	>40	Very high	Clay
Imperial	5	29	6.5	20-40	High	Silty clay
	6	34	6.0	20-40	High	Silty clay
	7	47	7.2	>40	Very high	Clay
	4	45	5.0	>40	Very high	Clay

5.2 Foundation Characterization

5.2.1 Bearing Capacity Analysis

Foundation analysis refers to the determination of the bearing layer and depth, allowable bearing pressure and type of foundation that could be adopted safely and economically. Factors such as the load to be transmitted to the foundation and the subsurface condition of the soil have been considered in selecting the foundation type.

The bearing capacity of the soils depends on the depth, size, shape, and strength of geologic material. Bearing capacity estimation is conducted using various methods. In this study the SPT N values are used to estimate bearing capacity of the soil. The allowable bearing capacity is calculated from SPT blow counts for both isolated and mat foundation for different depth.

5.2.1.1 Isolated Footing

Adjusting the N-values, a design N-values are chosen from consecutive depths where the test is performed. The design N-values are taken as the average of adjusted N-values which are found in between ½ B above and 2B below the proposed footing depths where B is the width of the foundation.

The footing depth is considered at depths of 6.0m below the ground level at the locations of drilled boreholes and the foundation width is selected based on Bowels, (1996) equation (equation below) for isolated foundation and ranges from 20.0 to 60.0m for a mat foundation. The bearing capacity for the soil layer is calculated from the SPT N-values using Meyerhof's equation as follows (Bowles, 1996):-

$$q_{all} = \frac{N'}{F_2} * \left(1 + {}^{F_3}\!/_{B}\right)^2 * K_d \qquad\qquad B > F_4$$

$$q_{all} = allowable\ bearing\ pressure\ for\ settlement\ limited\ to\ 25mm$$
$$K_d = 1 + 0.33(D/B) \leq 1.33$$
$$F_2 = 0.08$$
$$F_3 = 0.3$$
$$F_4 = 1.2$$
$$B = width\ of\ foundation$$
$$D = Depth\ of\ foundation$$

The following allowable bearing pressures are calculated for different foundation widths at the respective depths for settlement limited to 25mm. Foundation width is a significant parameter since a large foundation width will affect the soil to a greater depth and strains integrated over a greater depth will produce a larger settlement. Allowable bearing capacity is calculated for Asco, Imperial and Megenagna sites presented in Table 5.9.

Asco Site

Table 5.9 estimation of allowable bearing capacity for Asco site using SPT N-values, Bowels, (1996) equation for Asco site

Block No.	Avg. depth (m)	Avg. N55	Avg. BC (all) (KPa)	Block No.	Avg. depth (m)	Avg. N55	Avg. BC (all) (KPa)
1	3	18	342	4	3	14	281
	5	12	221		5	14	261
	7	12	214		7	11	196
	9	14.5	252.5		9	14	247
	11	15	259.5		11	10.5	183
	13	12.5	215.5		13	9.5	164
	15	11	189		15	19	329
	17	10.5	179.5		17	13	224
	19	10	170.5		19	14	240
2	3.5	8.5	169	5	5	11	209
	5.5	23	426		7	11	200
	9.5	20.5	367		9	12	212
	11.5	14	244		11	13.5	236
	11.5	17	295		13	10	174
	13.5	15	260		15	14	242
	15.5	8	137		17	17	293
	17.5	10	172		19	17	292
3	3	9.5	191	6	3.35	5	99
	5	10.5	196		5.5	5	92
	7.5	16	288		7.8	6.5	116
	9	17	297		10.4	5.5	96
	12.6	14.5	251		13.1	14	243

	14.8	12	208		15.9	14	242
	18	12	206				
Block No.	Avg. depth (m)	Avg. N55	Avg. BC (all) (KPa)	Block No.	Avg. depth (m)	Avg. N55	Avg. BC (all) (KPa)
7	3	7.5	150	10	5	12.5	291
	5.25	22	410		7	17	545
	8.45	9	161		9	18.5	222
	10.75	26	457		11	19.5	297
	14.9	26	451		13	16	321
8	3	25.5	501		15	9.5	336
	5	21	390		17	15	224
	7.6	26.5	476		19	39	189
	9	26.5	470	11	3	12.5	251
	11	29.5	517		5	13.5	158
	13	27	469		7	11	225
	15	25.5	441		9	8.5	236
	16.6	24	414		11	17.5	191
9	3	15.5	151		13.1	9	209
	5.5	28.5	429		15	7	225
	8.1	22	276		17	6	155
	9.1	22	501		19.2	9	120
	11	23	509	12	3	8	161
	13	20	418		5.5	16	296
	15	22	398		7.45	14.5	260
	17	10	344		11.2	10.5	184
	19	40	377		12.8	12.5	218
13	3	7.5	150				181
	5.35	17	315				171
	7.5	25	449				
	12.9	4.5	78				
	15.75	8.5	147				
	18.1	12.5	215				

The allowable bearing capacity calculated in the above table (Table 5.9) is for respective depth with a given adjusted SPT N-value for Asco site. As soil layers very depth wise its bearing capacity also varies where in more consistent soil it increases and for less consistent soil it decreases. According to bowels, (1996) in normal soil stratigraphy where soil consistency increases with depth bearing capacity increases. Moreover, these results are displayed to understand the soil behavior with depth on graphic view. Allowing all the results of all blocks on a single graph mess-up and difficult to understand. Hence, depending on the pattern of graph the entire site are displayed in different graphs under a category of two parts. In the first graph (Fig. 5.6) allowable bearing capacity increases around a depth of 8m and decreases around a depth of 13m. The increase in bearing capacity is due to the soil profile change where fill material is underlying by in situ completely weathered rock material.

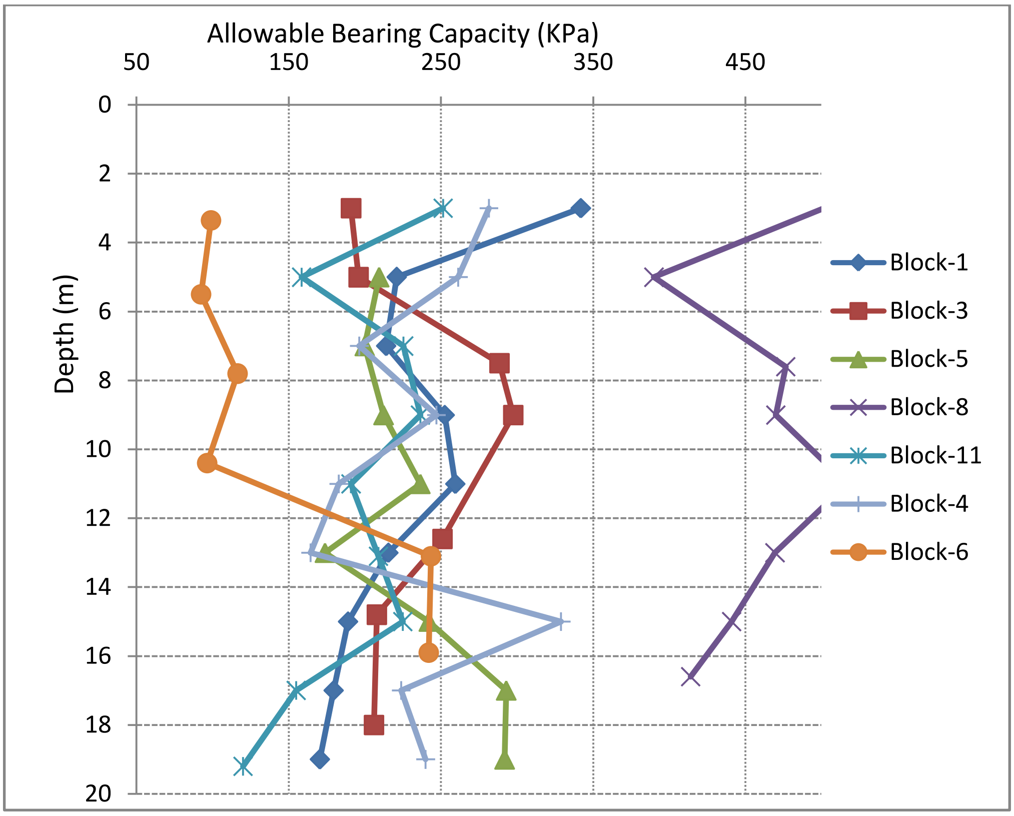


Fig. 5.6 Allowable bearing capacity and depth correlation Part 1(block-1, 3, 4, 5, 6, 8, 11, 4)

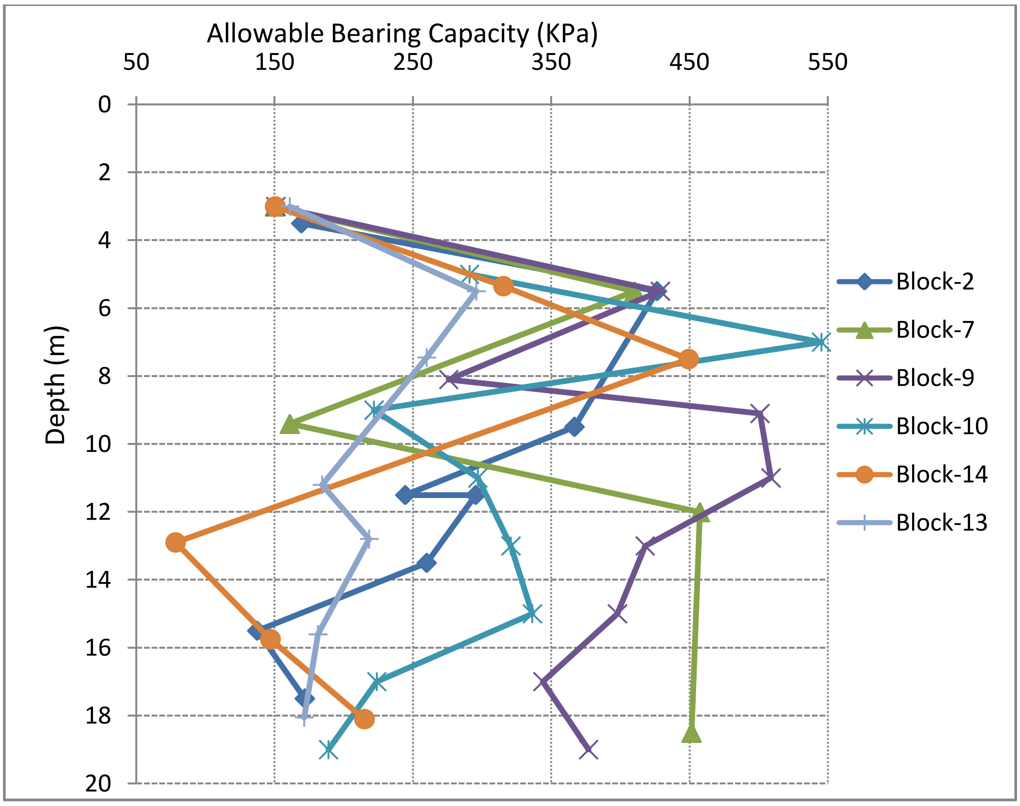


Fig. 5.7 Allowable bearing capacity and depth correlation part 2 (block 2, 7, 6, 10, 13, 14)

In the second graph (Fig. 5.7) allowable bearing capacity elevates up to 6-7m and decreases beyond this depth. Again the change soil consistency and grain size is responsible for such sharp variation in soil bearing capacity. To understand overlying of geology and allowable bearing capacity it is viable to analyze on the site overview (Fig. 5.8).

Fig. 5.8 Overlie of allowable bearing capacity and soil profile for Asco site

Imperial Site

Similarly the allowable bearing capacity for imperial site is correlated with its soil profile. Table 5.10 exhibit the allowable bearing capacity with depth for Megenagna site. The results with its respective depth are displayed on a single graph to understand its pattern. Accordingly similar to Asco site the pattern of the graph indicates two categories and it is easily readable and understandable to view these patterns in different graphs. As the graph pattern indicated the geology of the site is variable with considerable soil consistency and grain size (Fig. 5.9 and 5.10). The right side of the site the graph shows a decrease in allowable bearing capacity to a depth of 10m as general indication. On the other side the left side of the site is characterized by increase in allowable bearing capacity up to average of 10m and decreases beyond it.

Table 5.10 estimation of allowable bearing capacity for Imperial site using SPT N-values, Bowels, (1996) equation

	Avg. depth (m)	Avg. N55	Avg. BC (all) (KPa)		Avg. depth (m)	Avg. N55	Avg. BC (all) (KPa)
BLOCK 1	4.5	11	208	BLOCK 2	2.45	14	293
	6.45	13	237		6.55	14	255
	8.475	13	232		8.45	12	214
	10.45	13	229		10.45	12	211
	12.475	14	244		12.45	12	209
	14.45	14	243		14.45	14	243
BLOCK 3	2.45	13	272		16.45	14	241
	8.05	13	233		18.45	14	240
	9.85	11	194	BLOCK 4	2.45	16	335
	11.85	15	262		4.45	13	246
	13.95	14	243		6.3	13	237
	16.95	14	241		15.45	11	190
BLOCK 5	6	15	275	BLOCK 6	4.2	13	248
	7.15	14	253		5.95	13	238
	9.9	16	282		8.45	16	285
	12.45	16	279		10.45	16	281
	14.425	13	225		12.425	11	192
BLOCK 7	2.525	13	271	BLOCK 8	2.45	13	272
	6.45	14	255		4.15	14	268
	8.45	14	250		7.8	14	251
	10.45	14	246		9.95	11	194
	12.45	14	244		11.95	12	210
	14.45	11	191		13.95	13	226
	16.45	13	224		15.95	15	259
					17.95	14	241

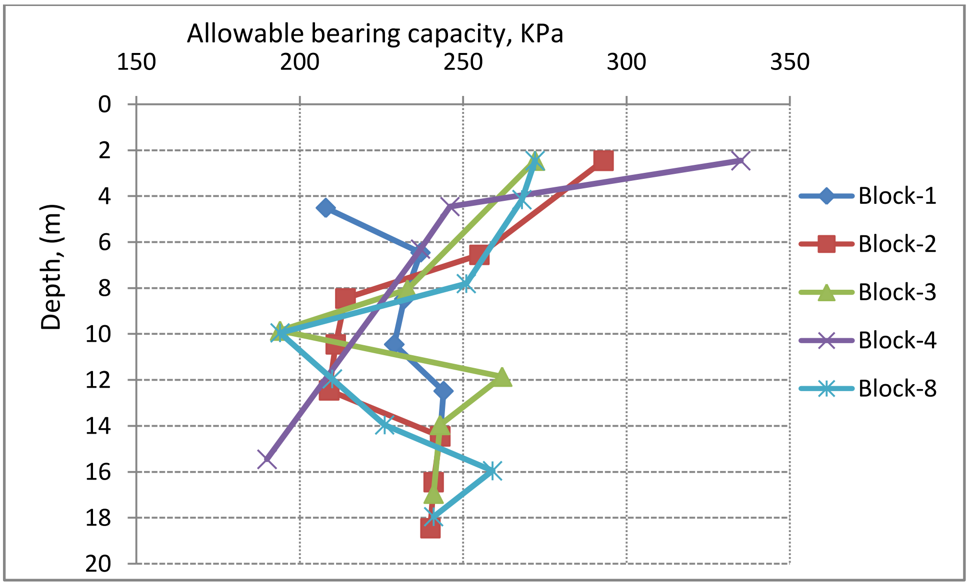


Fig. 5.9 Allowable bearing capacity and depth correlation for Imperial site part-1 (block 1, 2, 3, 4, 8)

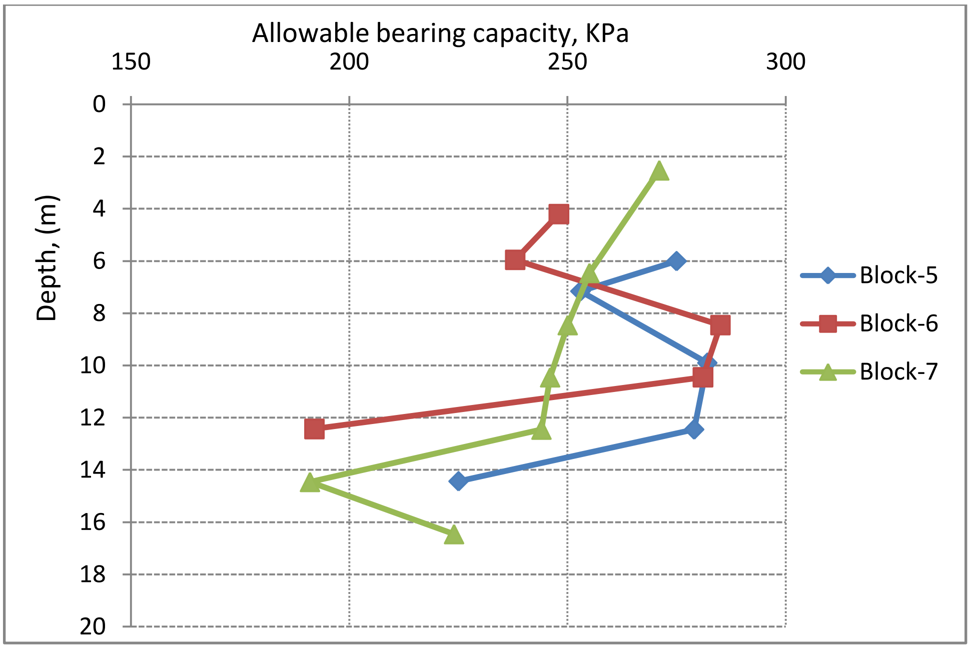


Fig. 5.10 Allowable bearing capacity and depth correlation for Imperial site part 2 (block 5, 6, 7)

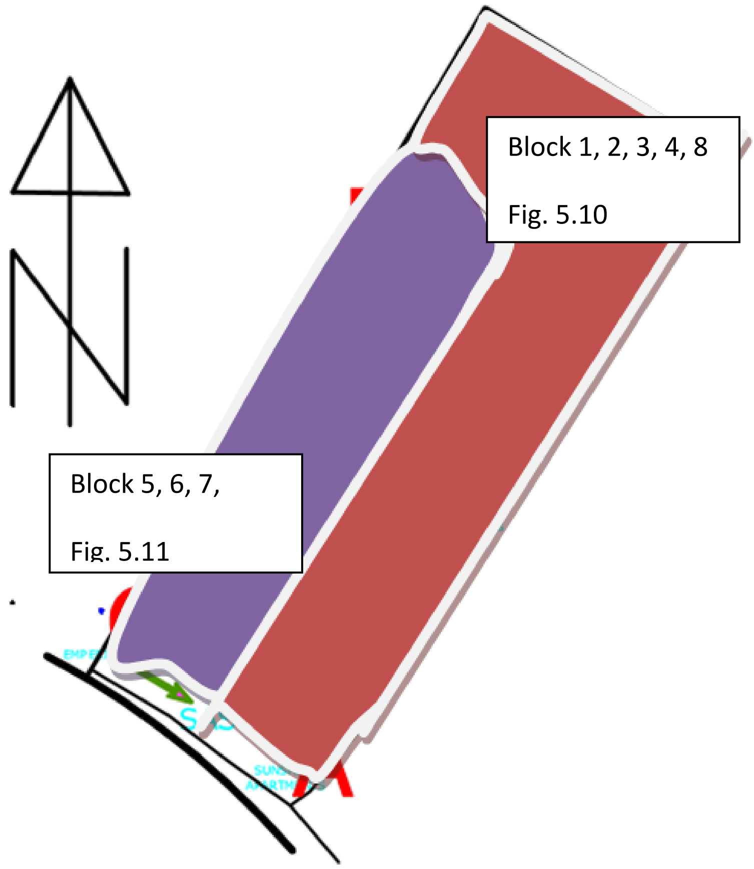


Fig. 5.11 Overlie of allowable bearing capacity and soil profile for Imperial site

Megenagna Site

In case of Megenagna site in block-2 the allowable bearing capacity decreases where as in the rest of blocks it increases in depth wise (Table 5.11). This indicates the soil is more consistent as depth is concerned. Similar to the other two sites the graph is plotted to visualize the relation between allowable bearing capacity and depth. The graph (Fig. 5.12) exhibits the increment of allowable bearing capacity depth wise for block 1, 5, and 6 where as for block 2 it decreases. Fig. 5.13 shows the bearing capacity and soil layer based on Fig. 5.12

Table 5.11 estimation of allowable bearing capacity for Imperial site using SPT N-values, Bowels, (1996) equation

Block No.	Avg. depth (m)	Avg. SPT N55	Avg. BC (all) (KPa)
Block-1	8.2	11	197
	9.7	15	265
	12.1	19	332
Block-2	9.685	20	353
	15.16	18	311
Block-5	5.5	6	111
	7.5	3	126
	9.45	9	159
	11.7	11	192
	13.7	12	208
	15.7	13	224
Block-6	7.15	7	126
	9.1	12	213
	11.19	22	386

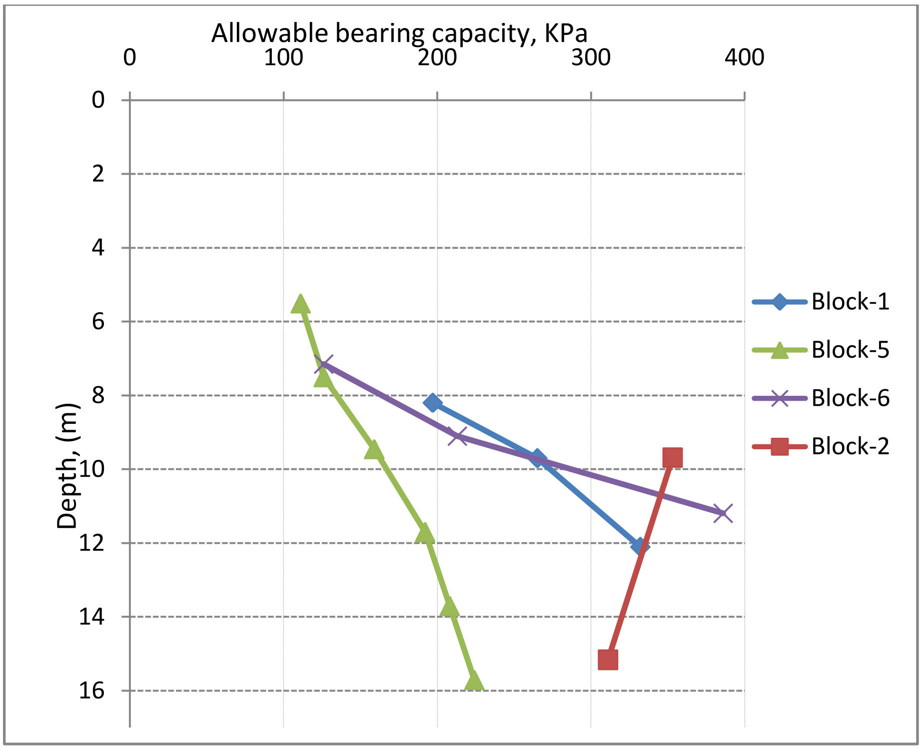


Fig. 5.12 Allowable bearing capacity and depth correlation for block 5, 6, 7

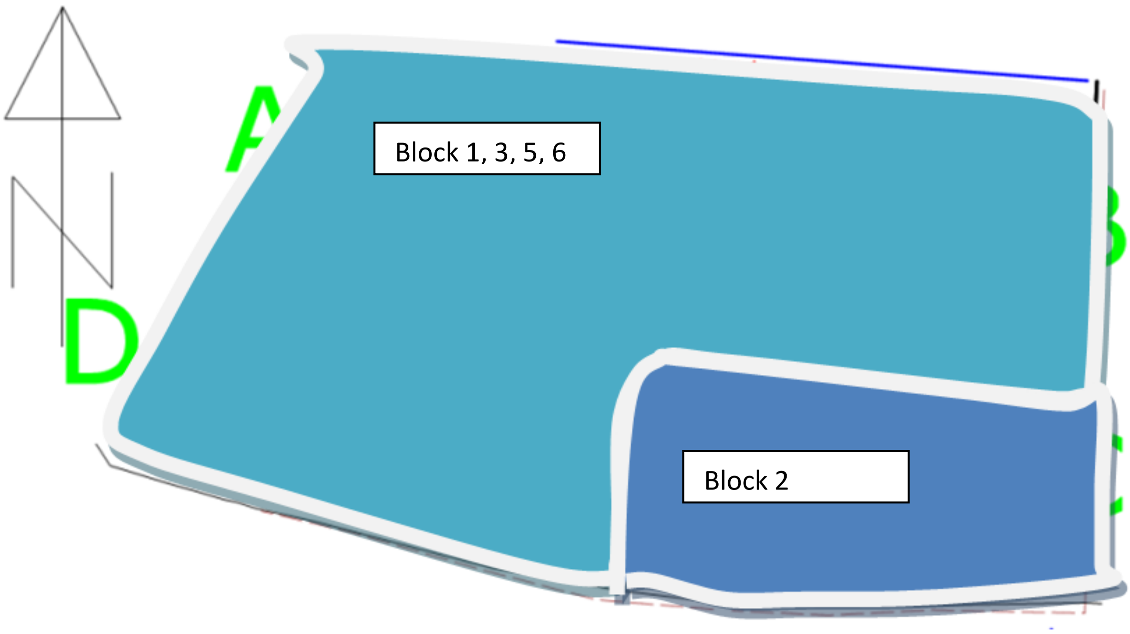


Fig. 5.14 Overlie of allowable bearing capacity and soil profile

5.2.1.2 Mat Foundation

According Bowels, (1996) mat foundation may be used where the base soil has a low bearing capacity and/or the column loads are so large that more than 50 percent of the area is covered by conventional spread footings. It is common to use mat foundations for deep basements both to spread the column loads to a more uniform pressure distribution and to provide the floor slab for the basement.

Mat foundation is commonly used where the base soil has a low bearing capacity and/or the column loads are so large that more than 50 percent of the area is covered by conventional spread footings. It is common to use mat foundations for deep basements both to spread the column loads to a more uniform pressure distribution and to provide the floor slab for the basement (Bowels, 1996). The mat foundation also used to bridge over horizontal variation of the soil layer on the ground.

The bearing capacity for the soil layer is calculated from the SPT N- values using Meyerhof's equation as follows (Bowles, 1996):

$$Q_{all} = \frac{(N_{55} * S * K_d)}{0.08 * 25}$$

$$K_d = 1 + 0.33(D/B) \qquad < 1.33$$

$$Where, S = allowable\ Settlement\ (mm)$$

$$N_{55} = adjusted\ SPT\ blow\ count$$

$$B = width\ of\ foundation$$

Asco site

Table 5.12 estimation of allowable bearing capacity for mat foundation Asco site using SPT N-values, Bowels, (1996) equation foundation width of 20m, 30m, 40m, 50m.

Block No.	Width mat (m)					
	20	30	40	50	60	
1	323.24	313.53	308.68	305.76	303.82	Q_{all} (KPa)
2	439.28	426.09	419.49	415.53	412.29	
3	337.94	327.8	322.72	319.68	317.65	
4	333.9	323.88	318.86	315.85	313.85	
5	386.48	374.88	369.07	365.59	363.27	
6	250.06	242.55	238.79	236.54	235.04	
7	250.44	242.92	239.16	236.9	235.4	
8	728.73	706.85	695.91	689.35	684.97	
9	509.74	494.44	486.78	482.19	479.13	

10	379.8	368.4	362.69	359.27	356.99	
11	335.2	325.13	320.1	317.08	315.07	
13	275.07	266.81	262.68	260.21	258.55	
14	364.12	353.19	347.72	344.44	342.26	

5.3 Settlement Analysis

Settlement is another criterion for evaluating the performance of structures because excessive settlements will result in poor performance of the structure. Different codes set the limiting settlement for the type of the structure and foundations. In case of EBSC-7, 1995 the allowable settlement 50mm and 75mm for sandy and clay soil respectively. These settlements depend on a number of parameters basically soil type.

Settlement estimation is done for this specific project by considering the dominant clayey SILTY 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 undisturbed samples collected from the sites (Table 4.8-4.10).

It is essential to design a foundation system which ensures differential and total settlements do not exceed allowable values. Settlements are usually classified as immediate settlement, (settlement which can take place as the load is applied) and consolidation settlement, (settlement which is time dependent and taking months to years to develop). The principal settlements for most projects occur in 1 to 5 years (Frederick L., 1979). Both types of settlement analyses are in the form of these two equations:

$$\epsilon = \frac{\Delta H}{H} = \frac{\Delta e}{1 + e_o}$$

Where, ε is vertical strain, H is layer thickness, ΔH is the compression of the layer and e_o and Δe are initial void ratio and change in void ratio respectively.

The amount of consolidation settlement is computed using the compression index C_c obtained from the empirical equation. More commonly C_c is used in computing ΔH as follows:

$$\Delta H = \frac{C_c H}{1 + e_o}\left[\log(P_o' + {\Delta P}/{P_o'}\right]$$

Where,

C_c = compression index from consolidation test

e_o = average in-situ void ratio in the stratum for which C_c applies (e_o=wG_s)

H = thickness of stratum

P'_o = effective overburden pressure at mid-height of H

ΔP = average increase in pressure from foundation in layer H in same unit as P'_o

The detail calculation for settlement and the data used for calculating are attached in appendix- of this report.

Asco site

Table 5.13 (a, b, c, d, e, f, g and h) Asco site settlement estimation

a)

the Spread Footing = 2 x 2m	Footing/ Mat dimensions		Settlement (mm)	Depth (m)	Contact pressure q_o(KPa)
	width/2	length/2			
6.5	1	1	7.48	0.8	220.12
7.3	1	1	3.36	0.8	98.89
8.1	1	1	1.20	0.8	35.29
8.9	1	1	0.58	0.8	17.01
9.7	1	1	0.34	0.8	9.86
			12.96		

b)

For Isolated Foundation with 2.0m width									
Layer Description	Depth	Layer thickness (m)	e_o	P'_o	Average increasse in pressure kPa) Δp	C_c	DH (mm)	Sum (mm)	75% of Total (mm)
Clayey SILT/Sity CLAY	6.5	0.0	1.160	114.40	220.12	0.107	0.00		
	7.3	0.8	1.160	128.48	98.89	0.107	9.82	9.82	7.37
	8.1	0.8	1.160	142.56	35.29	0.107	3.81	13.63	10.22
	8.9	0.8	1.160	156.64	17.01	0.107	1.77	15.41	11.55
	9.7	0.8	1.160	170.72	9.86	0.107	0.97	16.37	12.28
	10.5	0.8	1.160	184.80	6.40	0.107	0.59	16.96	12.72
						Immediate (mm) =12.96			
						Consolidation (mm)=12.72			
						Secondary (mm)=0.00			
						Total settlement (mm)=25.68			

c)

For the Spread Footing=4.0x4.0m	Footing/ Mat dimensions		Settlement (mm)	Depth (m)	Contact pressure q_o(kPa)
	width/2	length/2			
6.5	2	2	13.56	1.6	192.35
8.1	2	2	6.09	1.6	86.41
9.7	2	2	5.56	1.6	30.84
11.3	2	2	2.68	1.6	14.87
			27.89		

d)

For Isolated Foundation with 4.0m width									
Layer Description	Depth	Layer thickness (m)	e_o	P'_o	Average increase in pressure kPa) Δp	C_c	DH (mm)	Sum (mm)	75% of Total (mm)
Clayey SILT	6.5	0.0	1.160	114.40	192.35	0.107	0.00		
	8.1	1.6	1.160	142.56	86.41	0.107	16.31	16.31	12.23
	9.7	1.6	1.160	170.72	30.84	0.107	5.72	22.03	16.52
	11.3	1.6	1.160	198.88	14.87	0.107	2.48	24.51	18.38
						Immediate (mm) =27.89			
						Consolidation (mm)=18.38			
						Secondary (mm)=0.00			
						Total settlement (mm)=46.27			

e)

For the Spread Footing=6.0x6.0m	Footing/ Mat dimensions		Settlement (mm)	Depth (m)	Contact pressure q_o(kPa)
	width/2	length/2			
6	3	3	5.70	0.6	299.85
6.6	3	3	5.48	0.6	287.98
7.2	3	3	4.56	0.6	239.80
7.8	3	3	3.46	0.6	181.84
8.4	3	3	2.56	0.6	134.71
9	3	3	1.92	0.6	100.78
9.6	3	3	1.46	0.6	77.00
10.2	3	3	1.14	0.6	60.19
			26.29		

f)

For 6.0m width foundation									
Layer Description	Depth	Layer thickness (m)	e_o	P'_o	Average increasse in pressure kPa) Δp	C_c	DH (mm)	Sum (mm)	75% of Total (mm)
Clayey SILT	6.0	0.0	1.160	105.60	299.85	0.107	0.00		
	6.6	0.6	1.160	116.16	287.98	0.107	16.09	16.09	12.07
	7.2	0.6	1.160	126.72	239.80	0.107	13.71	29.80	22.35
	7.8	0.6	1.160	137.28	181.84	0.107	10.89	40.69	30.52
	8.4	0.6	1.160	147.84	134.71	0.107	8.36	49.05	36.79
	9.0	0.6	1.160	158.40	100.78	0.107	6.36	55.41	41.56
	9.6	0.6	1.160	168.96	77.00	0.107	4.85	60.26	45.19
	10.2	0.6	1.160	179.52	60.19	0.107	3.73	63.99	47.99
						Immediate (mm) =26.29			
						Consolidation (mm)=47.99			
						Secondary (mm)=0.00			
						Total settlement (mm)=74.28			

g)

For mat foundation =50x50m	Footing/ Mat dimensions		Settlement (mm)	Depth (m)	Contact pressure q_o(kPa)
	width/2	length/2			
6	25	25	34.49	5	258.55
11	25	25	33.12	5	248.31
16	25	25	27.58	5	206.77
21	25	25	20.92	5	156.80
26	25	25	15.49	5	116.15
31	25	25	11.59	5	86.90
36	25	25	8.86	5	66.39
41	25	25	6.92	5	51.90
			165.60		

h)

For Mat Foundation with 50.0m width									
Layer Description	Depth	Layer thickness (m)	e_o	P'$_o$	Average increasse in pressure kPa) Δp	C_c	DH (mm)	Sum (mm)	75% of Total (mm)
Clayey SILT	6.0	0.0	1.160	105.60	258.55	0.107	0.00		
	11.0	5.0	1.160	193.60	248.31	0.107	88.78	88.78	66.58
	16.0	5.0	1.160	281.60	206.77	0.107	59.23	148.00	111.00
						Immediate (mm) =165.60			
						Consolidation (mm)=111.00			
						Secondary (mm)=0.00			
						Total settlement (mm)=276.60			

As indicated in previous sections geologically Asco site is characterized by coarse grained soil where immediate settlement is high. However, block-6 is characterized by soft soil and this is also indicated in the bearing capacity of the site where the minimum allowable bearing capacity is recorded. During estimation of settlement for this site the laboratory consolidation of this unit is used for the entire site where it is assumed to be the worst case in the site. Under this circumstance, the results are summarized for the three sites in Table 5.16.

Megenagna site

The settlement of this site is calculated based on the results of consolidation test from this site and the detail settlement estimation for immediate and consolidation is done for different foundation width as presented in the following table (Table 5.14).

Table 5.14 (a, b, c, d, e, f, g and h) Megenagna site settlement estimation

a)

For the Spread Footing = 2 mx 2m	Footing/ Mat dimensions		Settlement (mm)	Depth (m)	Contact pressure qo(kPa)
	width/2	length/2			
6.5	1	1	7.82	0.8	230.00
7.3	1	1	3.51	0.8	103.33
8.1	1	1	1.25	0.8	36.87
8.9	1	1	0.60	0.8	17.78
9.7	1	1	0.35	0.8	10.30
			13.54		

b)

For Isolated Foundation with 2.0m width									
Layer Description	Depth	Layer thickness (m)	e_o	P'_o	Average increase in pressure kPa) Δp	C_c	DH (mm)	Sum (mm)	75% of Total (mm)
Clayey SILT	6.5	0.0	1.402	114.40	230.00	0.189	0.00		
	7.3	0.8	1.402	128.48	103.33	0.189	16.14	16.14	12.10
	8.1	0.8	1.402	142.56	36.87	0.189	6.29	22.43	16.82
	8.9	0.8	1.402	156.64	17.78	0.189	2.94	25.37	19.02
	9.7	0.8	1.402	170.72	10.30	0.189	1.60	26.97	20.23
	10.5	0.8	1.402	184.80	6.69	0.189	0.97	27.94	20.96
							Immediate (mm) =13.54		
							Consolidation (mm)=20.96		
							Secondary (mm)=0.00		
							Total settlement (mm)=34.49		

c)

For the Spread Footing=4.0x4.0m	Footing dimensions		Settlement (mm)	Depth (m)	Contact pressure qo(kPa)
	width/2	length/2			
6.5	2	2	12.69	1.6	180.00
8.1	2	2	5.70	1.6	80.86
9.7	2	2	5.20	1.6	28.86
11.3	2	2	2.51	1.6	13.91
			26.10		

d)

For Isolated Foundation with 4.0m width									
Layer Description	Depth	Layer thickness (m)	e_o	P'_o	Average increase in pressure kPa) Δp	C_c	DH (mm)	Sum (mm)	75% of Total (mm)
Clayey SILT	6.5	0.0	1.402	114.40	180.00	0.189	0.00		
	8.1	1.6	1.402	142.56	80.86	0.189	24.57	24.57	18.43
	9.7	1.6	1.402	170.72	28.86	0.189	8.54	33.11	24.83
	11.3	1.6	1.402	198.88	13.91	0.189	3.70	36.81	27.61
							Immediate (mm) =26.10		
							Consolidation (mm)=27.61		
							Secondary (mm)=0.00		
							Total settlement (mm)=53.71		

e)

For the Foundation=6x6m	Footing dimensions		Settlement (mm)	Depth (m)	Contact pressure qo(kPa)
	width/2	length/2			
6	3	3	5.71	0.6	300.00
6.6	3	3	5.48	0.6	288.12
7.2	3	3	4.56	0.6	239.92
7.8	3	3	3.46	0.6	181.93
8.4	3	3	2.56	0.6	134.77
9	3	3	1.92	0.6	100.83
9.6	3	3	1.47	0.6	77.04
10.2	3	3	1.15	0.6	60.22
			26.30		

f)

For foundation with 6.0m width									
Layer Description	Depth	Layer thickness (m)	e_o	P'_o	Average increase in pressure kPa) Δp	C_c	DH (mm)	Sum (mm)	75% of Total (mm)
Silty CLAY	6.0	0.0	1.402	105.60	300.00	0.189	0.00		
	6.6	0.6	1.402	116.16	288.12	0.189	25.58	25.58	19.18
	7.2	0.6	1.402	126.72	239.92	0.189	21.79	47.36	35.52
	7.8	0.6	1.402	137.28	181.93	0.189	17.31	64.67	48.50
	8.4	0.6	1.402	147.84	134.77	0.189	13.29	77.96	58.47
	9.0	0.6	1.402	158.40	100.83	0.189	10.10	88.06	66.04
	9.6	0.6	1.402	168.96	77.04	0.189	7.70	95.76	71.82
	10.2	0.6	1.402	179.52	60.22	0.189	5.93	101.69	76.27
							Immediate (mm) =26.30		
							Consolidation (mm)=76.27		
							Secondary (mm)=0.00		
							Total settlement (mm)=102.57		

g)

For the Mat Foundation =50x50m	Footing/ Mat dimensions		Settlement (mm)	Depth (m)	Contact pressure qo(Kpa)
	Width/2	Length/2			
6	25	25	33.35	5	250.00
11	25	25	32.03	5	240.10
16	25	25	26.67	5	199.93
21	25	25	20.22	5	151.61
26	25	25	14.98	5	112.31
31	25	25	11.21	5	84.03
36	25	25	8.56	5	64.20
41	25	25	6.69	5	50.18
46	25	25	6.38	5	40.08
			160.09		

h)

For Mat Foundation with 50.0m width									
Layer Description	Depth	Layer thickness (m)	e_o	P'_o	Average increasse in pressure kPa) Δp	C_c	DH (mm)	Sum (mm)	75% of Total (mm)
Silty CLAY	6.0	0.0	1.402	105.60	250.00	0.189	0.00		
	11.0	5.0	1.402	193.60	240.10	0.189	137.84	137.84	103.38
	16.0	5.0	1.402	281.60	199.93	0.189	91.68	229.52	172.14

Immediate (mm) =160.09
Consolidation (mm)=172.14
Secondary (mm)=0.00
Total settlement (mm)=332.23

Imperial site

This site is also similar to Megenagna site and the settlement is estimated both immediate and consolidation settlements are estimated based on the laboratory consolidation test result for samples from the site. The following table (Table 5.15) presents the detail settlement estimation of Imperial site by considering different foundation width.

Table 5.15 (a, b, c, d, e, f, g and h) Imperial site settlement estimation

a)

For the Spread Footing = 2m x 2m	Footing/ Mat dimensions		Settlement (mm)	Depth (m)	Contact pressure q_o(kPa)
	width/2	length/2			
6.5	1	1	7.82	0.8	230.00
7.3	1	1	3.51	0.8	103.33
8.1	1	1	1.25	0.8	36.87
8.9	1	1	0.60	0.8	17.78
9.7	1	1	0.35	0.8	10.30
			13.54		

b)

For Isolated Foundation with 2.0m width									
Layer Description	Depth	Layer thickness (m)	e_o	P'_o	Average increase in pressure kPa) Δp	C_c	DH (mm)	Sum (mm)	75% of Total (mm)
Clayey SILT	6.5	0.0	1.160	114.40	230.00	0.107	0.00		
	7.3	0.8	1.160	128.48	103.33	0.107	10.16	10.16	7.62
	8.1	0.8	1.160	142.56	36.87	0.107	3.96	14.12	10.59
	8.9	0.8	1.160	156.64	17.78	0.107	1.85	15.97	11.97
	9.7	0.8	1.160	170.72	10.30	0.107	1.01	16.97	12.73
	10.5	0.8	1.160	184.80	6.69	0.107	0.61	17.59	13.19

Immediate (mm) =13.54
Consolidation (mm)=13.19
Secondary (mm)=0.00
Total settlement (mm)=26.73

c)

For the Spread Footing = 4m x 4m	Footing/ Mat dimensions		Settlement (mm)	Depth (m)	Contact pressure q_o(kPa)
	width/2	length/2			
6.5	2	2	12.69	1.6	180.00
8.1	2	2	5.70	1.6	80.86
9.7	2	2	5.20	1.6	28.86
11.3	2	2	2.51	1.6	13.91
			26.10		

d)

For Foundation with 4.0m width									
Layer Description	Depth	Layer thickness (m)	e_o	P'_o	Average increasse in pressure kPa) Δp	C_c	DH (mm)	Sum (mm)	75% of Total (mm)
Clayey SILT	6.5	0.0	1.160	114.40	180.00	0.107	0.00		
	8.1	1.6	1.160	142.56	80.86	0.107	15.47	15.47	11.60
	9.7	1.6	1.160	170.72	28.86	0.107	5.38	20.84	15.63
	11.3	1.6	1.160	198.88	13.91	0.107	2.33	23.17	17.38
							Immediate (mm) =26.10		
							Consolidation (mm)=17.38		
							Secondary (mm)=0.00		
							Total settlement (mm)=43.48		

e)

For the Spread Footing = 6m x 6m	Footing/ Mat dimensions		Settlement (mm)	Depth (m)	Contact pressure q_o(kPa)
	width/2	length/2			
6	3	3	5.71	0.6	300.00
6.6	3	3	5.48	0.6	288.12
7.2	3	3	4.56	0.6	239.92
7.8	3	3	3.46	0.6	181.93
8.4	3	3	2.56	0.6	134.77
9	3	3	1.92	0.6	100.83
9.6	3	3	1.47	0.6	77.04
10.2	3	3	1.15	0.6	60.22
			26.30		

f)

For Foundation with 6.0m width									
Layer Description	Depth	Layer thickness (m)	e_o	P'_o	Average increase in pressure (kPa) Δp	C_c	ΔH (mm)	Sum (mm)	75% of Total (mm)
Clayey SILT	6.0	0.0	1.160	105.60	300.00	0.107	0.00		
	6.6	0.6	1.160	116.16	288.12	0.107	16.10	16.10	12.07
	7.2	0.6	1.160	126.72	239.92	0.107	13.71	29.81	22.36
	7.8	0.6	1.160	137.28	181.93	0.107	10.89	40.70	30.53
	8.4	0.6	1.160	147.84	134.77	0.107	8.36	49.07	36.80
	9.0	0.6	1.160	158.40	100.83	0.107	6.36	55.43	41.57
	9.6	0.6	1.160	168.96	77.04	0.107	4.85	60.28	45.21
	10.2	0.6	1.160	179.52	60.22	0.107	3.73	64.01	48.01
							Immediate (mm) =26.30		
							Consolidation (mm)=48.01		
							Secondery (mm)=0.00		
							Total settlement (mm)=74.3		

g)	For Mat Foundation = 50m x 50m	Footing/ Mat dimensions		Settlement (mm)	Depth (m)	Contact pressure q_o(kPa)
		width/2	length/2			
	6	25	25	33.35	5	250.00
	11	25	25	32.03	5	240.10
	16	25	25	26.67	5	199.93
	21	25	25	20.22	5	151.61
	26	25	25	14.98	5	112.31
	31	25	25	11.21	5	84.03
	36	25	25	8.56	5	64.20
	41	25	25	6.69	5	50.18
				160.09		

h)

For Mat Foundation with 50.0m width									
Layer Description	Depth	Layer thickness (m)	e_o	P'_o	Average increase in pressure (kPa) Δp	C_c	DH (mm)	Sum (mm)	75% of Total (mm)
Silty CLAY	6.0	0.0	1.160	105.60	250.00	0.107	0.00		
	11.0	5.0	1.160	193.60	240.10	0.107	86.76	86.76	65.07
	16.0	5.0	1.160	281.60	199.93	0.107	57.71	144.47	108.35
							Immediate (mm) =160.09		
							Consolidation(mm)=108.35		
							Secondary (mm)=0.00		
							Total settlement (mm)=268.44		

Table 5.16 summary of settlement estimation for Asco, Megenagna and Imperial site

Sites	Foundation Size (B) (m)	Settlement (mm)		Total settlement (mm)
		Immediate	Consolidation	
Asco site	2m	12.96	12.72	25.68
	4m	27.89	18.38	46.27
	6m	26.29	47.99	74.28
	50m	165.60	111.00	276.60
Megenagna	2m	13.54	20.96	34.49
	4m	26.10	27.61	53.71
	6m	26.30	76.27	102.57
	50m	160.09	172.14	332.23
Imperial	2m	13.54	13.19	26.73
	4m	26.10	17.38	43.48
	6m	26.3	48.01	74.3
	50m	160.09	108.35	268.44

According to EBSC-7, 1995 (Fig. 5.14) settlement can be analyzed from SPT N-value and a standard is given similar to the following graph. SPT N-values are plotted on this graph to correlate settlement for each SPT N-value. Thus, one con search for his SPT N-value on this graph and correlate settlement potential for his SPT N-value.

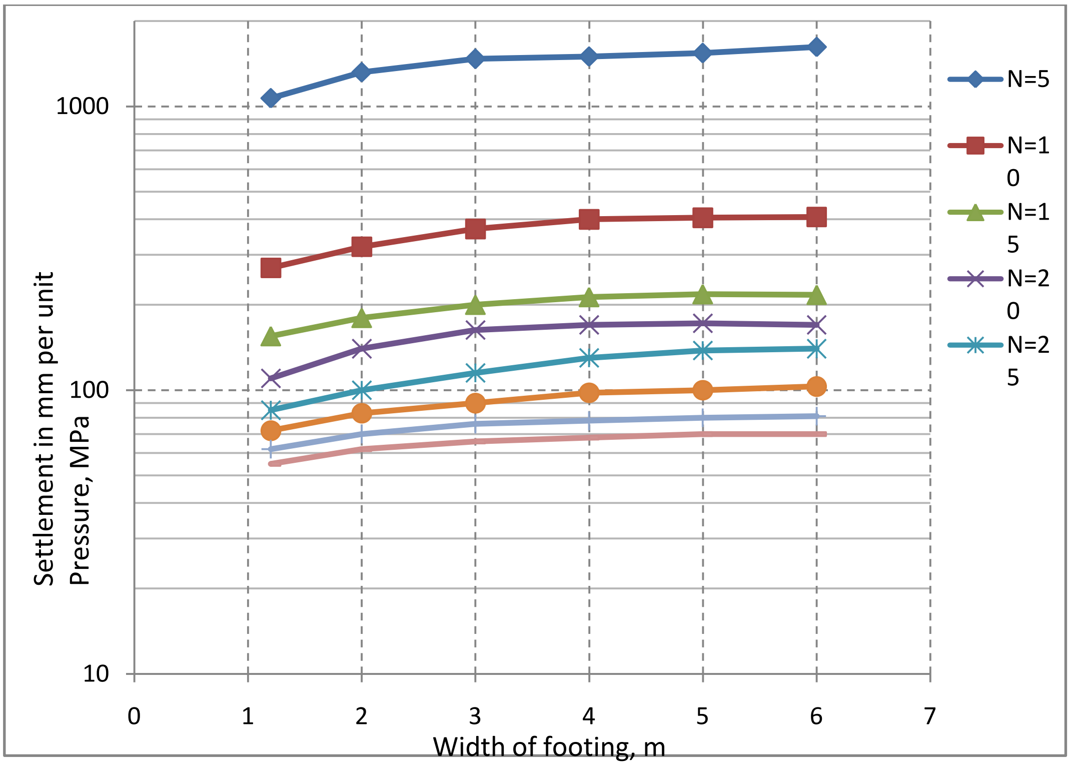


Fig. 5.15 settlement from SPT N-value (EBSC-7, 1995)

According to EBSC-7, 1995 the following table (Table 5.17) produced, after averaging SPT N-value for each blocks at different sites. Even though, the data digitizing the graph and reading corresponding settlement value is done with some $\pm$ the value in the table are potentially close to what will be calculated from the SPT N value and settlement correlation. Increasing foundation width increases the settlement value as also seen from the original graph. The value in the table is settlement in mm per unit pressure (MPa).

Table 5.17 rough settlement correlation and SPT N-value (EBSC-7, 1995)

Sites	Block No.	SPT N-value	Settlement (mm) under different footing/MPa			
			3	4	5	6
Asco	1	13	270	285	290	300
	2	15	200	210	217	220
	3	15	200	210	217	220
	4	13	270	285	290	300
	5	12	275	288	294	310
	6	10	390	398	400	400
	7	20	170	185	188	190
	8	27	125	145	148	150
	9	22	168	175	178	184
	10	16	198	205	208	212
	11	10	390	398	400	400
	12	12	275	288	294	310
	13	12	275	288	294	310
Imperial	1	13	270	285	290	300
	2	13	270	285	290	300
	3	13	270	285	290	300
	4	12	275	288	294	310
	5	14	271	286	290	297
	6	14	271	286	290	297
	7	13	270	285	290	300
	8	13	270	285	290	300
Megenagna	1	15	200	210	217	220
	2	19	175	187	192	196
	5	9	393	399	405	409
	6	13	270	285	290	300

CHAPTER SIX CONCLUSION AND RECOMMENDATION

6.1 Conclusion

The massive construction of buildings is part of city construction, renewal and expansion. It is indicative of population growth and need of housing. As overall booming of construction sectors in indicates economic way of a given country. The population growth of Addis Ababa city amplifies the need of housing. Massive condominium housing projects have been conducted since the beginning of housing program. Geotechnical works in touch the construction industry with soil and rock material. Geotechnical site characterization and foundation estimation is one of the most valuable tasks to be done beyond the uncertainty of the ground. Under mass housing projects the due to large site areal coverage and number of data analysis in most cases poor out puts are produced and hence the structures will fail before its actual performance.

Addis Ababa housing projects constructs several condominiums and apartments in the city. One the major failure was recorded at Jemo condominium site and a block is tilted 10^0-15^0. The present study has the objective of characterization and foundation analysis of three sites namely Asco site, Megenagna site and Imperial site which are under preparation for apartment buildings.

In order to achieve the objectives of the present study literature review of previous works, filed observation and laboratory tests were conducted. Representative samples were collected from each sites and soil index property evaluation was conducted. Additionally, strength property is also done. Totally, 30 samples were collected from these three sites.
From the study the following findings are deduced:

- From engineering geology point of view Asco site is characterized by coarse grained well graded to gap graded soil materials. From the test it is found to be gravely sand according to UCS classification. Samples from the rest of the sites are found to be highly plastic silt (MH) soil.
- Bearing capacity analysis based on Bowels, 1996 SPT method employed and if foundation level has to be stick to around 7m below natural ground level the sites run over range of the following allowable bearing capacity.

  Asco site ranges from 116KPa-476KPa

Imperial site ranges from 214KPa-285KPa

Megenagna site ranges from 126KPa-300KPa

- Correlation of bearing capacity and depth shows a significant overlapping with geology of the sites indicating change in soil profile.
- Settlement estimation was conducted based on consolidation results of laboratory tests and EBSC-7, (1995) method. In both cases the settlement increases with width of foundation and the value is also beyond allowable limits.
- The unconfined compressive strength of rock samples collected from Megenagna and Imperial site shows the Imperial site samples are found to be stronger than Megenagna site.

6.2 Recommendations

Basedonthepresentstudyresultsfollowingrecommendationsareforwarded:

Isolated and mat foundations were considered to support the proposed structures depending on the super structure loads anticipated. Settlements were not within the allowable limit. Therefore, the following methods of construction and design are recommended.

- to improve bearing capacity for site exhibit low bearing capacity it is recommended to remove the soil and replace with selected under optimum compaction to a depth of 2.5B footing foundation and 1.5m for a mat foundation
- at block-8 the excavation indicates backfilled old stream channel and additional investigation should be done to know the depth of stream bed and its property
- eventhough some of the blocks can be supported by isolated footing, to avoid differential settlement and also for the buildings are designed to have two basements, it is recommended to use mat foundations to support the proposed structures. For Mat foundation, the recommended maximum settlement value for clays is 75-125mm (Bowles, 1996)
- it is also recommended isolated footing where rock strata is thicker than 6m, can accommodate the load sufficiently (Block-3 and Block-4 of Megenagna site and Block-4 of imperial site)

References

Afework Sisay, (2004). Assessment of Damage of Buildings Constructed on Expansive Soil Areas of Addis Ababa. Unpublished Thesis, Addis Ababa, Ethiopia.

Assiged Getahun, 2007. Geology of Addis Ababa City. Unpub. Report, EIGS, Addis Ababa, Ethiopia

Asrat Worku, (2011). Recent Developments in the Definition of Design Earthquake Ground Motions Calling For a Revision of the Current Ethiopian Seismic Code - EBCS 8: 199, *Zede Journal of Ethiopian Engineers and Architects.* **28**:25pp.

ASTM, (1977). Annual book of standards, NY, USA.

Belayneh Desta, (2009). Assessment of expansive soils and Foundation problems around mekanisa-lebu area, Addis Ababa. Unpub. MSc. Thesis, Addis Ababa University

BCS 1377:1990 part 2. British Code of Stanadards. London
Bell, F.G (2007). Engineering Geology, 2nd ed., Butterworth-Heinemann, Burlington

Blyth, F.G and Freitas, M.H. (1984). Geology for Engineers, 7th ed., Butterworth-Heinemann, Burlington

Bowels, J.E (1996). Foundation Analysis and Design, 5th ed., Mcgraw-Hill companies, Singapore, 220pp

David F., (2013). Soil Mechanics and Civil Engineering. Wiley and Sons, Ny, USA.

Ethiopian Building Standard Code (EBSC-7,1995). Foundation. Ministry of Urban and Development and Construction, Addis Ababa, Ethiopia.

Fekadu Kebede, (1996). Seismic Hazard Assessment for the Horn of Africa, *Zede Journal of Ethiopian Engineers and Architects*, Addis Ababa, Ethiopia, 1996.

Frederick L., (1979). Foundation Engineering. Mcgraw-Hill companies, Berlin, Germany.

Haile Sellasie Girmay and Getaneh Assefa (1989), Addis Ababa-Nazareth Volcanics: A Miocene-Pleistocene Volcanic Succession in Ethiopian Rift, *Ethiopian Journal of Science*, Addis Ababa, Ethiopia.

Holtz,W.G.andGibbs,H.J.,1956,Engineering Properties of Expansive Clays, Transactions, American Society of Civil Engineers, vol.121, pp. 641-677.

IDNDR RADIUS Project, Addis Ababa Case Study, Final Report. Prepared by Addis Ababa RADIUS group, et al, September 1999.

George David P. (2009) Engineering Geology- principles Practices, Springer, Verlag Berlin Heidelberg.

Grim R., (1952). Clay and its Mineralogical formation. Wiley and Sons, NY

Gopal Ranjan, (1993). A.S.R.R.A.O Engineering Geology, Wiley Eastern limited, Rorkee

Goshu W/Yohannes (1997). Report on Gravity Field of Addis Ababa. Part 1. Unpub. Report, EIGS, Addis Ababa, Ethiopia.

Johnson H., (1996). Soil Engineering and Foundation Investigation. Elsilver, NY.
Kazmin V., (1972). Geology of Ethiopia. Unpub. Report EIGS, Addis Ababa, Ethiopia

Kebede Tsehayu and Tadesse H/Mariam, (1990). A Report on Engineering Geological Map of Addis Ababa. Unpublished Report, Addis Ababa, Ethiopia

Laike Mariam Asfaw (1982). Site Amplification at the Western Escarpment of the East African Rift System Near Addis Ababa, *Bulletin of the Seismological Society of America*, Vol. **72**, No. 1, , 327-329pp.

Mengesha Tefera,Tewodros Chernet and Workneh Haro,(1996), Geological Map of Ethiopia (1:2000000), 2nd Edition and Explanatory notes. Bulletin N0.3. GSE.

Mohr, P.A. (1963). The Ethiopian Rift System, *Geoph. Observ*, Vol. **9**, Addis Ababa, Ethiopia

Mohr P., (1963). The Ethiopian Rift System. Unpub. Report EIGS, Addis Ababa, Ethiopia.

Morton W.H, (1974). Geological Map of Addis Ababa. Unpub. Report, Addis Ababa University, Addis Ababa, Ethiopia

Morton, W.H. and Rex (1979). Riftward Yanging of Volcanic Rocks of the Addis Ababa Region, Rift Valley Ethiopian, *Nature*, **280**: 284-288

Price D.G. (2009). Engineering Geology, Springer-Verlag Berlin Heidelberg

Pierre, G. (1981). Earthquake History of Ethiopia and the Horn of Africa, Bulletin of the Seismological Society of America, Ottawa, Canada, Vol. **71**: 573-574pp

Robert Stott, (2010). Foundation Manual. US corps, USA.

Samuel Kinde (1998). Proposed Considerations for Revision of EBCS-8:1995 for Conservative Seismic Zoning and Stringent Requirements for Torsionally Irregular Buildings

Sabatini P.J, Bachus, R.C., Mayne, J.A., Schneider P.W., and Zettler T.E., (2002). Evaluation of soil and rock properties. Technical report. Atlanta, Georgia.
Tamiru Alemayehu , (2003). Ground Water Vulnerability Mapping of the Addis Ababa Water Supply Aquifers, Addis Ababa, Ethiopia

Tilahun Mammo. (2005). Site-specific ground motion simulation and seismic response analysis at the proposed bridge sites within the city of Addis Ababa, *Engineering Geology*, Vol. **79**, 127-150 pp.

Tokimatsu K. and Yomoshi Y., (1993). Criteria of Soil classififcation with SPT and fines content, *Geot. Jap*. Vol. **3**, 255pp

Tsegaye Abebe, Mazzarini, F., Innocenti, F. and Manetti, P., (1995). The Yerer Tullu Wellel Extensional Structure (central Ethiopia) Evidence from Remote Sensing, Petrological and Geocronological Data. Unpub. Report EIGS, Addis Ababa, Ethiopia

US corps of Engineers (1986). Soil Mechanics Manual. NAVFAC DM 7.02, Naval Facilities Command, USA

Vernier A., and Tesfaye Cherinet, Haile Selassie Girmay, (1985). Hydrogeology of Addis Ababa Area. Unpub. Report, Addis Ababa, Ethiopia

Woltham Tony, (1996). Fundamentals of Engineering Geology, 2nd ed., Taylor and Francis Group, New York

Yohannes Abebe, (2009). “Ethiopia and its GDP in recent”. Unpub. Technical Report

Zanettin B. and Justin E. (1974). The Volcanics of Western Afar and Ethiopian Rift Margins. Padova, Italy

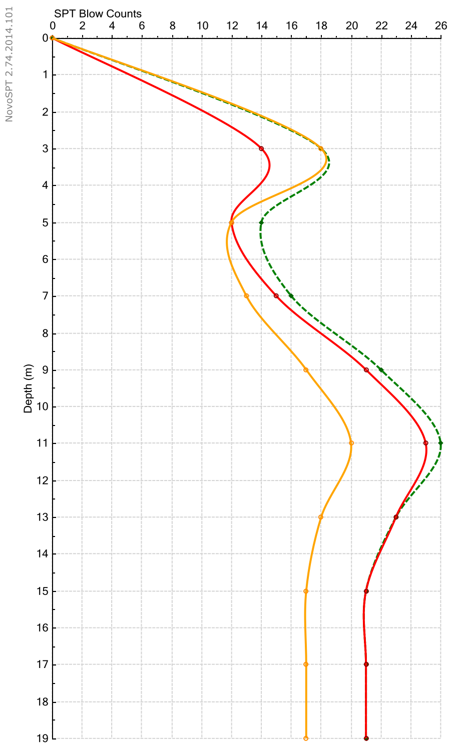
Zanettin, B., (1997). Volcanic Succession, Tectonics and Magmatology in cetral Ethiopia

(www.http://aashde.gov.et) accessed on 23-sept-2013

(www.cga.gov.et) accessed on 4-dec-2013

Asco SPT Vs depth correlation using NOVO SPT

green line-SPT N values redline -SPT N_{60} yellow-SPT $N_{1\ (60)} = N_{60}*C_n$



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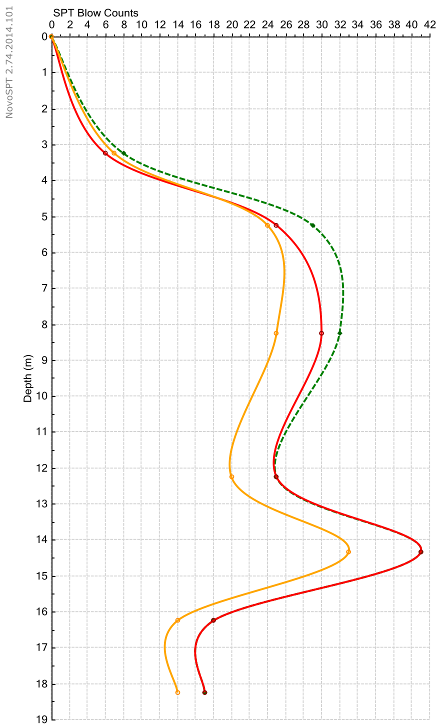

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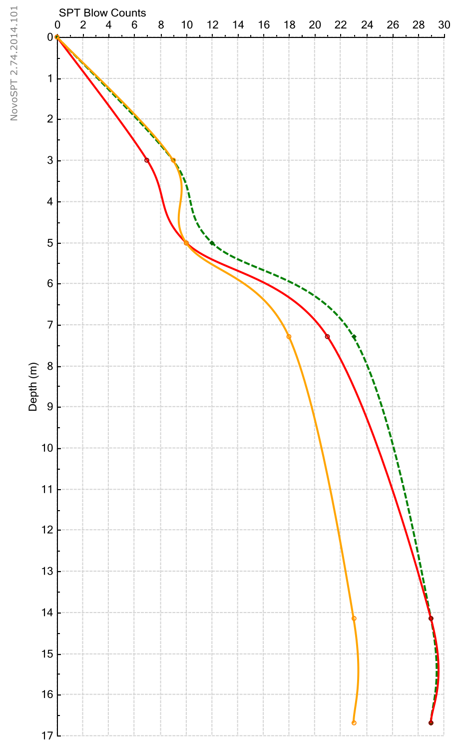

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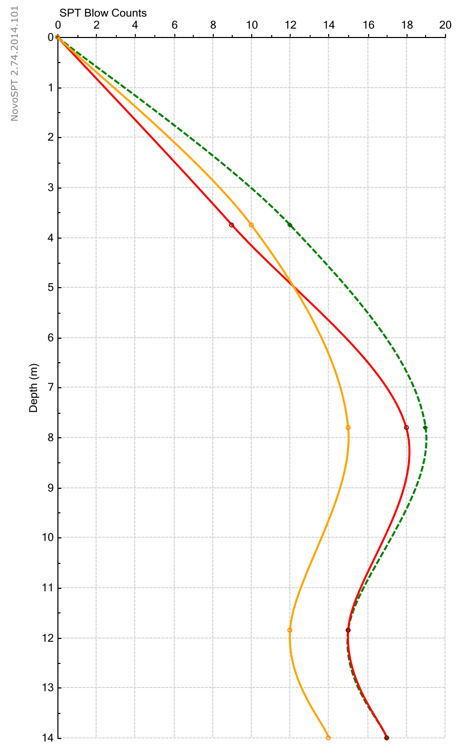

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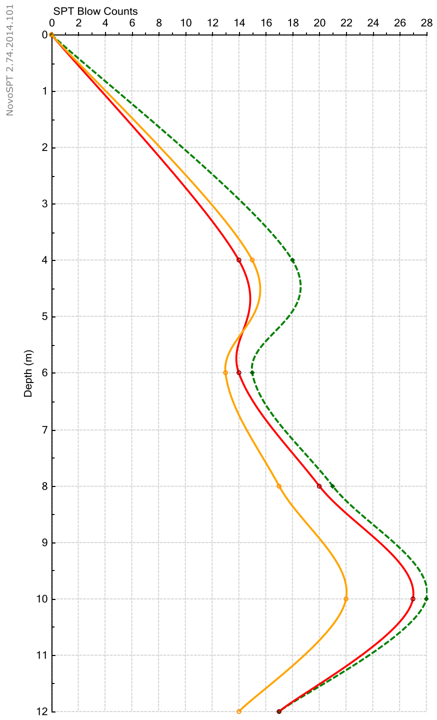

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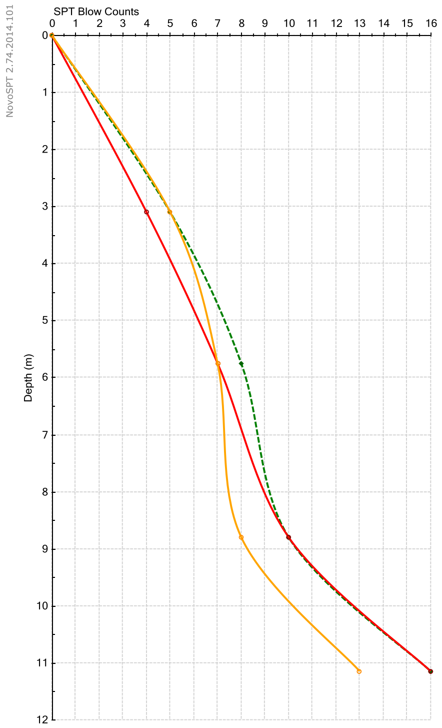

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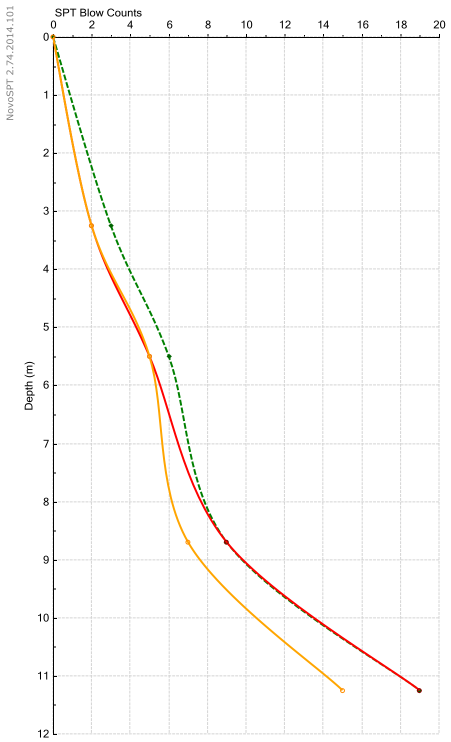

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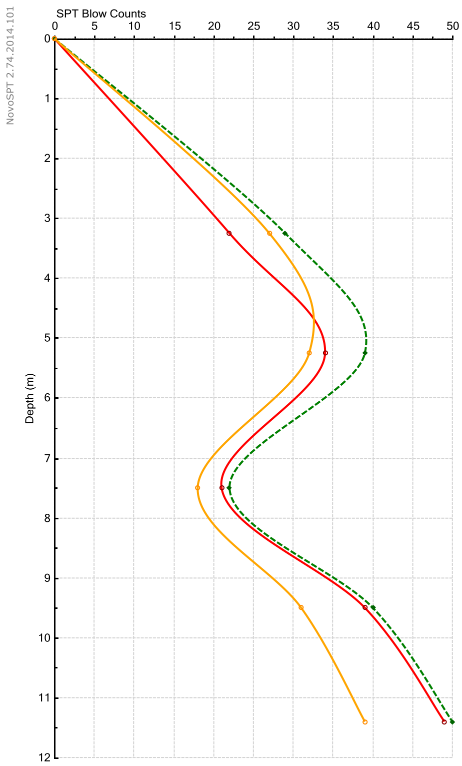

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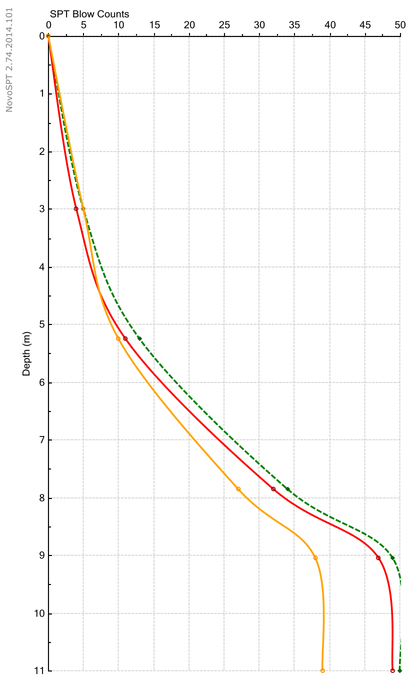

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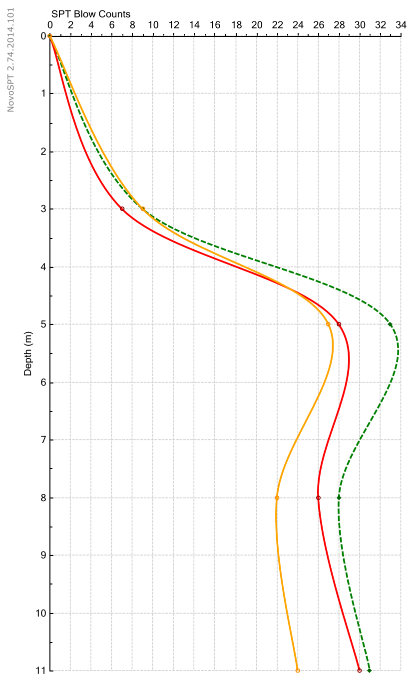

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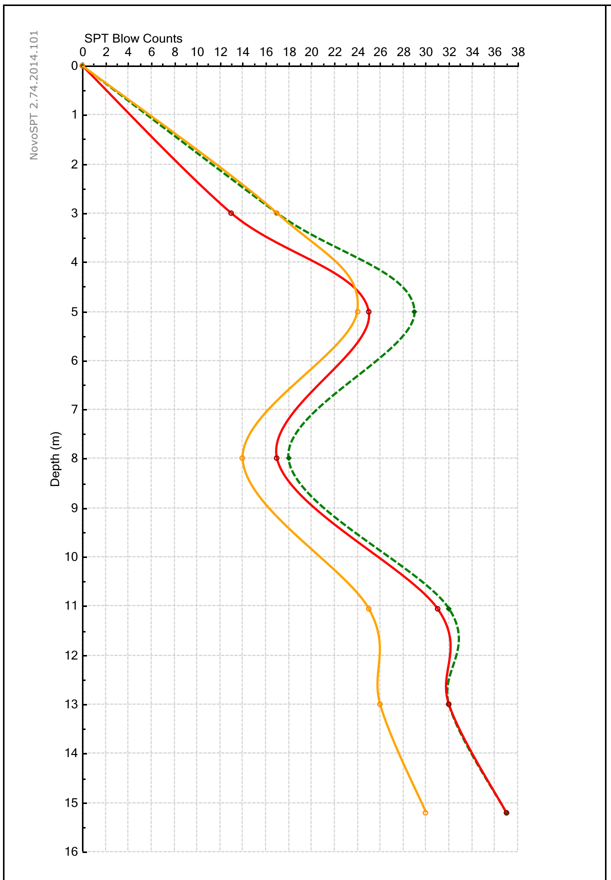

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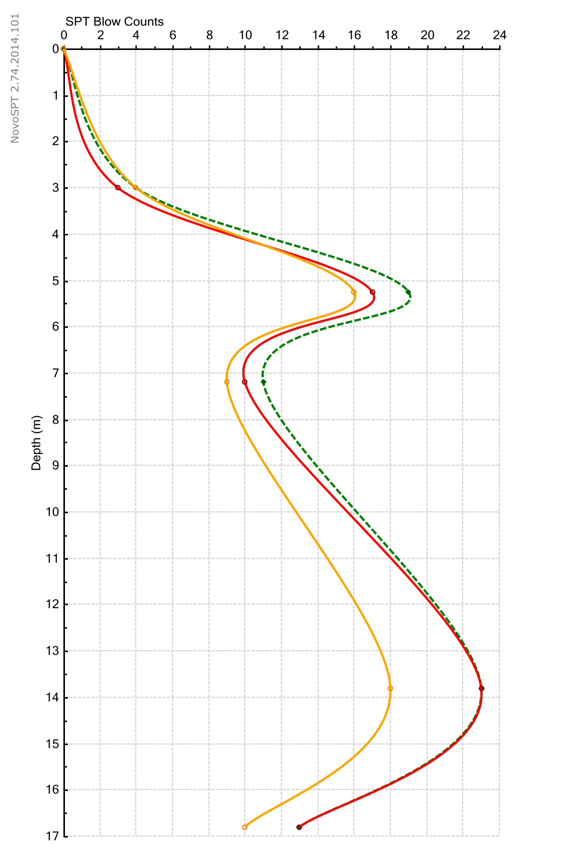

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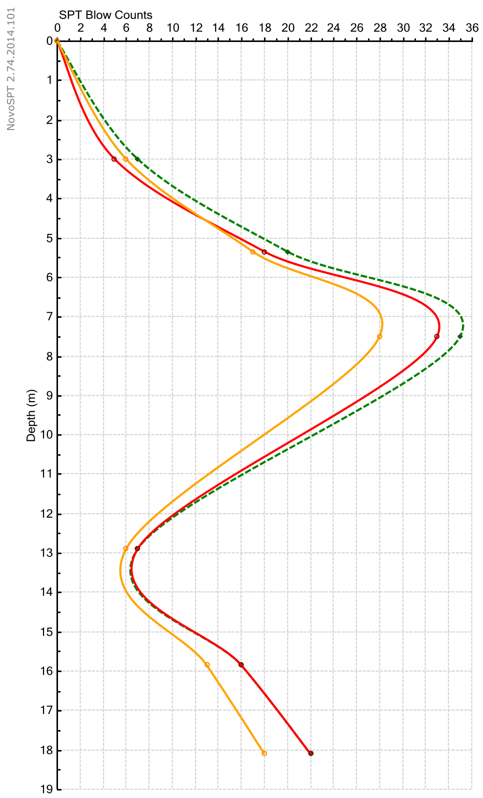
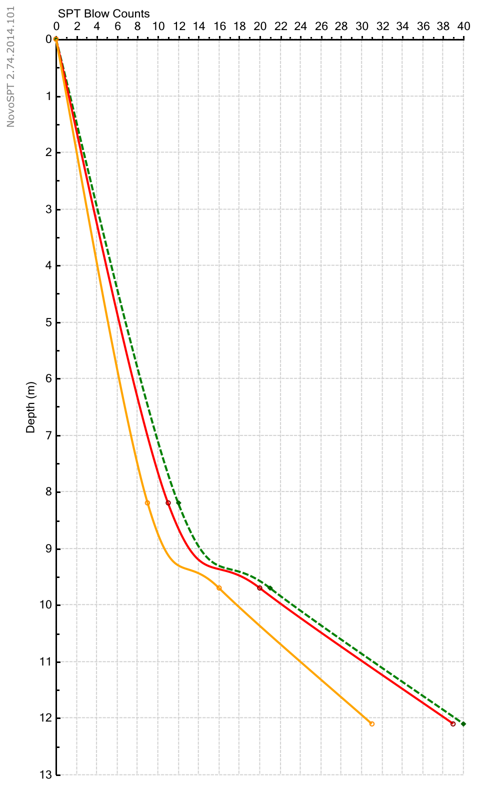


NovoSPT 2.74.2014.101
SPT Blow Counts
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36
Depth (m)
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19

Block-14

Megenagna site SPT Vs depth using NOVO SPT



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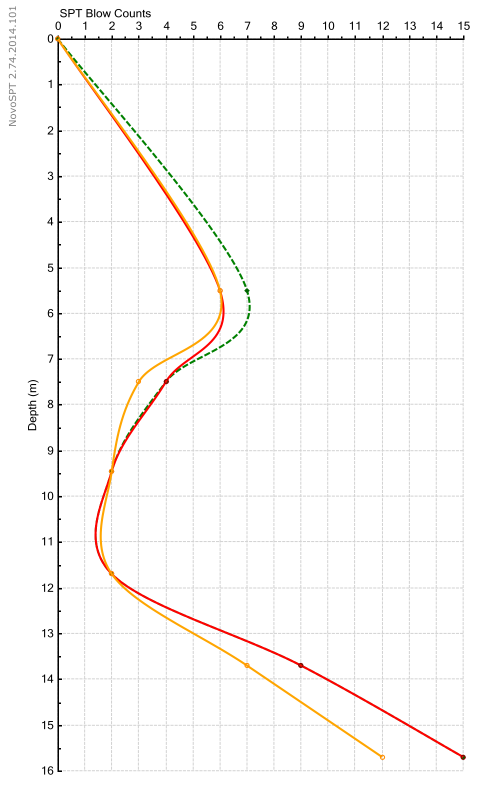

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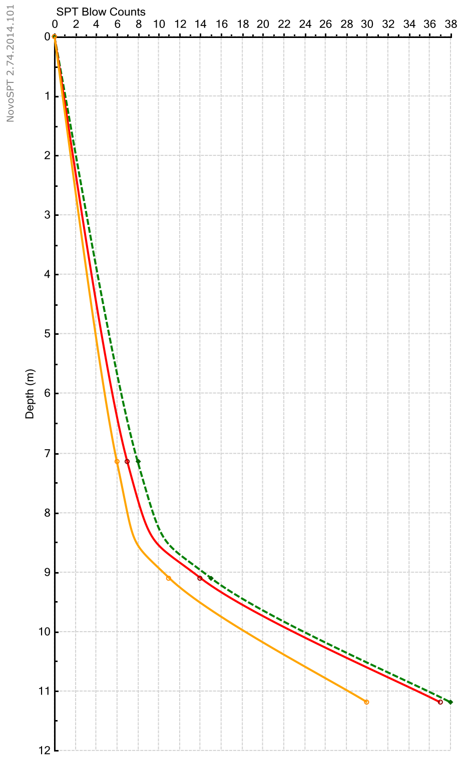

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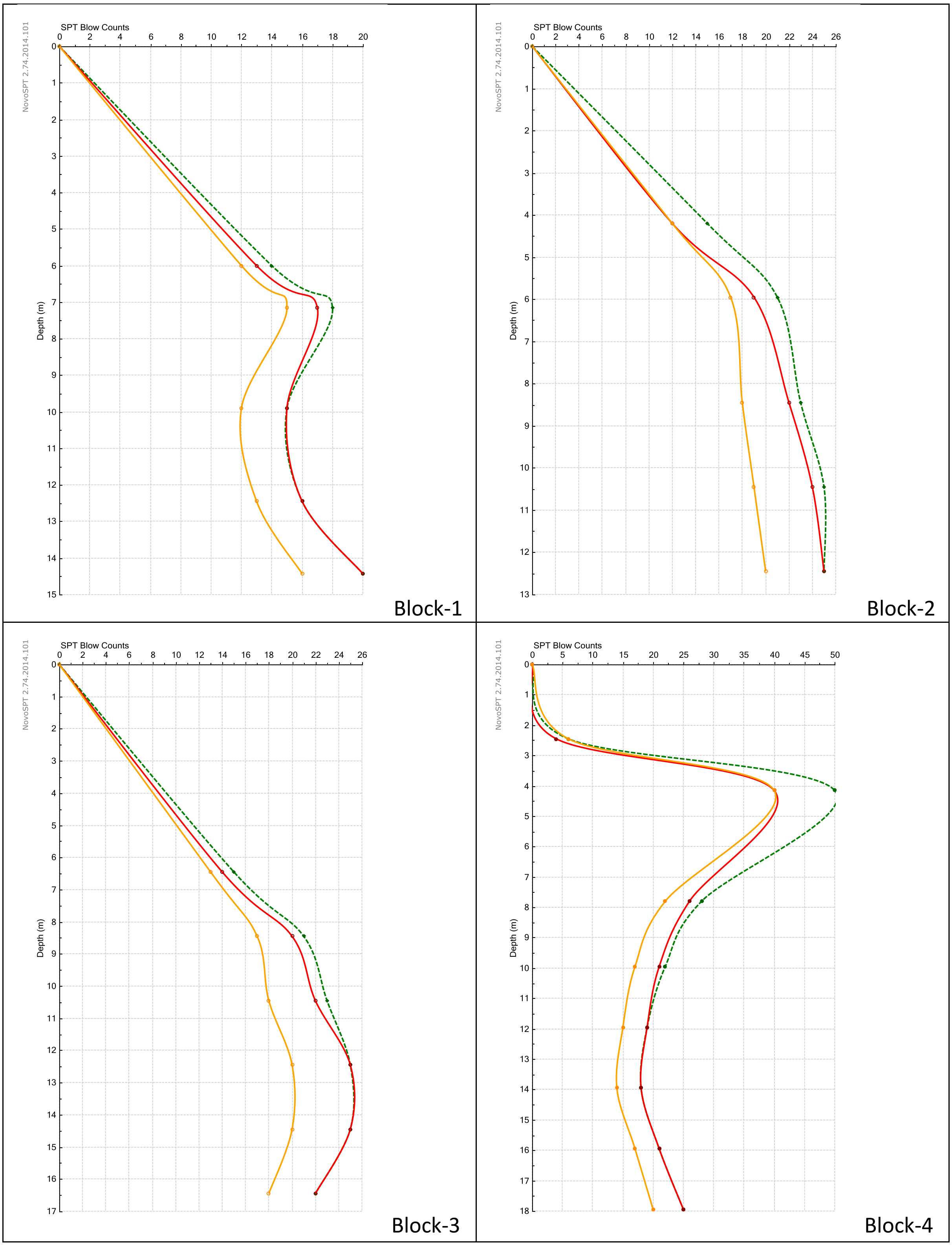
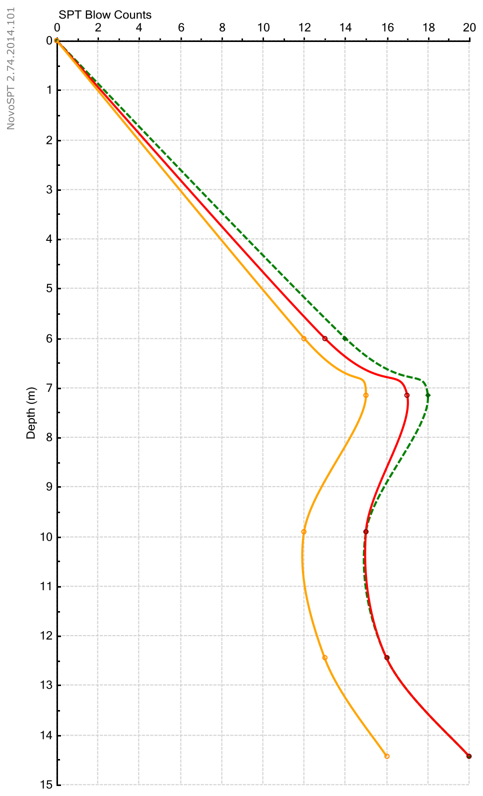
IMPERIAL SPT Vs DEPTH





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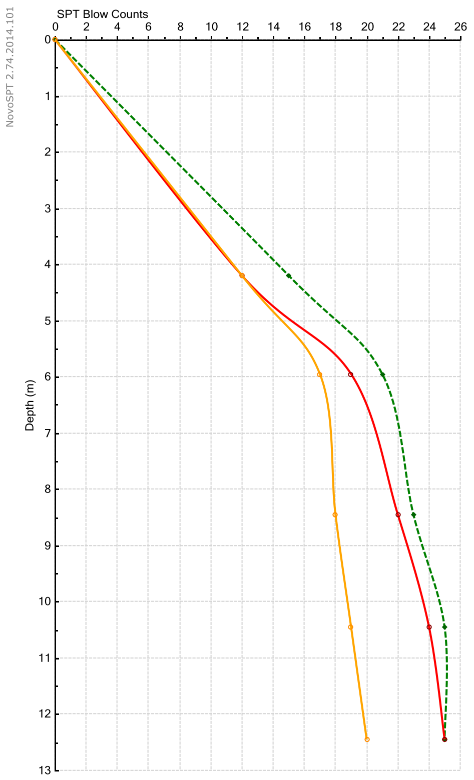

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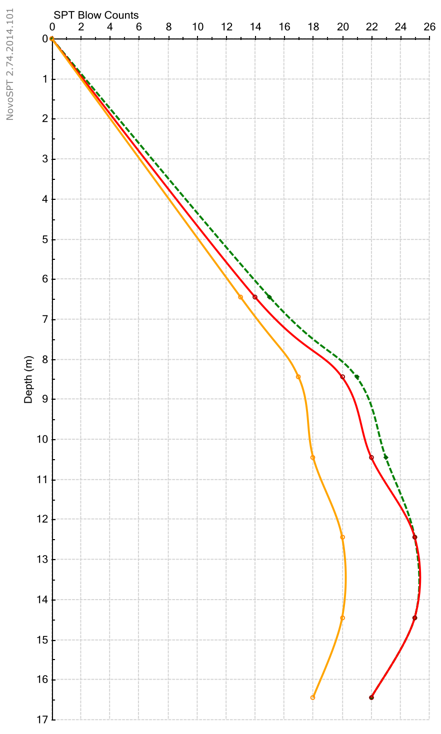

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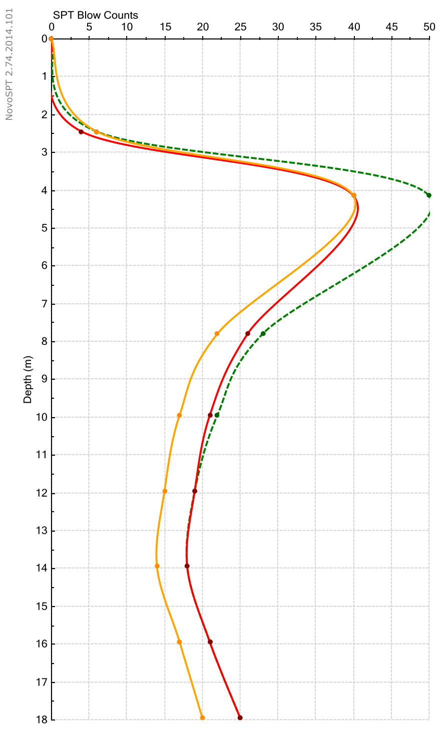

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