Building Foundation Characterization Analysis: A Case Study in Ayat Area, Bole Sub-City, Addis Ababa, Central Ethiopia

Selamawit Tadesse

A Thesis Submitted to

School of Earth Sciences

Presented in Partial Fullfillment of the requirements for the Degree of Masters of Science (Engineering Geology)

ADDIS ABABA UNIVERSITY

Addis Ababa, Ethiopia

May, 2017

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SIGNATURE PAGE

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School of Graduate Studies

This is to certify that the thesis prepared by **Selamawit Tadesse**, entitled: *Building Foundation Characterization And Analysis: A Case Study In Ayat Area, Bole Sub-City, Addis Ababa, Central Ethiopia* and submitted in partial fulfillment of the requirements for the Degree of Master of Science (Engineering Geology) complies with the regulations of the University and meets the accepted standards with respect to originality and quality.

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ABSTRACT

Building foundation characterization and analysis: A case study in Ayat area, Bole sub-city, Addis Ababa, Central Ethiopia

Selamawit Tadesse Alene

Addis Ababa University, 2017

The present research study was conducted in North-Eastern part of Addis Ababa, Bole Sub City Bole Ayat area. The main objective of the present research work was to characterize the building foundation material and to assess its bearing capacity and settlement potential in the study area. To meet out the objectives of the present study, systematic methodology was followed. Literature review was undertaken to have general background on the subject matter. Besides previous works were reviewed, secondary data was collected from various sources, filed observation and laboratory tests were also conducted. Representative samples were also collected and analyzed. Secondary data on borehole logs, test reports and other documents on site investigations were procured from several companies and organizations. Also, test pit logging, sampling and testing were also undertaken to generate primary data. The secondary data was processed and all relevant parameter data/ information necessary for bearing capacity and settlement analysis was obtained. For building foundation characterization and analysis the study area was divided into 9 sites; Site 1 to Site 9, so that the data can be well managed and systematic analysis can be made. The bearing capacity was estimated through various analytical and semi empirical techniques, based on laboratory test results on soil properties and through in-situ properties by standard penetration test. The overall evaluation of bearing capacity of foundation soils at shallow depths in the study area reveals that ultimate bearing capacity in general increase with depth and with increasing footing dimensions. Through the limited analysis it is also found that the estimated total settlement is not within the limits of allowable settlement. Further, bearing capacity zonation maps are also forwarded through the present study. Finally, based on the results of the present study recommendations are also forwarded.

Key words: Building foundation, Bearing capacity, Settlement potential, Standard Penetration test,

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CHAPTER ONE INTRODUCTION

1.1 Background

Geo-hazards present a substantial danger to human life, property, infrastructure and environment. Bell (1999) defined problem soils as natural geo-hazards that are due to detrimental geotechnical properties of soil. According to Jones and Holtz (1973) and Bell (1999), global damage to infrastructure and associated remediation costs are often of far reaching economic consequences. In particular, with increasing number of the global population and related rapid urbanization and demand of new land for expansion of infrastructure, these soils pose significant problems for engineering structural foundations.

Soils with different origin, responses in different manner when are subjected to imposed loads of the structures. This response to load is basically controlled by the engineering properties of the soil. The foundation of a structure is that part which transmits the load of the superstructure to the ground. The static load implies to a pressure which is exerted to the foundation and which does not change with time (George, 1989).However, dynamic loads change with the time.

According to Leow (2005), foundation is an important part of every building, which interfaces the superstructures to the adjacent soil or rock below it. Without a proper design and construction of foundation, problems such as cracking or settlement of building may occur and even to the extent, the whole building may collapse within its design life. Therefore, proper foundation investigation and design is required for the safety and stability of the building structure.

The selection of foundation is the most important part of the design process and most difficult to define because the selection is governed by many factors such as soil condition, past site usage, adjacent construction, size of development and also the cost (Curtin et al., 1994).

1.2 Problem statement

In Ethiopia many residential building are under construction. Generally, due attention is not given for topography, site investigation, soil type for foundation, bearing capacity, strength of the soil and depth of the foundation with respect to the super structure.
The soil investigation is necessary when the loads from the building are large and the bearing capacity cannot be estimated based on type of soil condition at site. Due to lack of proper

design of foundation settlement of the building is always the main problem in building construction. The settlement of foundation causes cracks in building walls, beams, slabs etc. and building can even fail in case of large settlement.

In the present study area high raise buildings, mass housing project and different infrastructure are under construction by private companies and government agencies. In the recent years city administration of Addis Ababa is activating a massive housing program to solve housing problem. Many condominiums have been built in different phases of construction and in different parts of the city and houses have already been transferred to more than half a million people. In these projects generally, less attention is being paid due to their bulkiness. Also, the site contractors did not fulfill proper ground investigation and characterization in order to cut the cost. Due to such unfulfilled investigation, failure of buildings has occurred before their occupancy (Bowles, 1997). In the study area building failure, settlement problem in foundation and some distortions, cracking and tilting are observed in the building foundation. Further, maintenance and repair costs are generally greater than the original expense. Moreover, Addis Ababa is located in the western margin of the main Ethiopian rift valley where possible seismicity could also be a real challenge for the safe building design. Proper evaluation of the geological and geotechnical condition at the proposed building site is therefore highly important and this research is designed to address all such issue.

1.3 Objectives of the study

1.3.1 General objectives

The main objective of the present research work is to characterize the building foundation material and to assess its bearing capacity and settlement potential in the study area.

1.3.2 Specific objectives

- To characterize the building foundation conditions
- To estimate the general bearing capacity of the foundation soils in the study area and to zone it within various classes
- To assess settlement potential of the foundation material and to zone it within various classes
- To suggest suitable foundation type within various zones for the buildings
- To framed engineering recommendations to guide design and construction

1.4 Methodology

Various data collection method was exercised to accomplish the above objectives of the research. The two data collection methods that were followed are secondary data collection and primary data collection (the primary data are investigated with laboratory analysis). After collection of the data, systematic data processing and analysis was made.

1.4.1 Secondary data collection

Secondary data was collected with an intention to review different pre-existing geological, engineering geological reports and maps. The review was made to understand the investigation methods, analysis and final interpretations for various building foundation sites. Besides, borehole data, soil properties and classification data was also collected which was later utilized to estimate the bearing capacity and settlement potential of different foundation soils.

1.4.2 Primary data collection

Primary data is collected from test pitting. Logging and description of different geologic formation was done. From test pitting appropriate soil samples were collected.

Field data collections

During field data collection description and characterization of the soil was made by visual observation. Representative soil samples were collected from different soil units and laboratory tests were done to evaluate the index property of the representative samples.

Laboratory testing

Index properties are used to classify soils, to group soils into major strata, and to obtain estimates of structural properties. The following laboratory tests were conducted for the present study to evaluate the soil properties of the representative samples collected from the study area.

Based on the above data, comprehensive evaluation of the foundation condition of the site was done. The general methodology followed in the present study is presented in flow chart (Fig.1.1).

The index and engineering properties that were used for the present study are;

- Natural moisture content
- Atterberg limit
- Particle site distribution
- Free swell
- Unit weight
- Direct shear
- Consolidation test

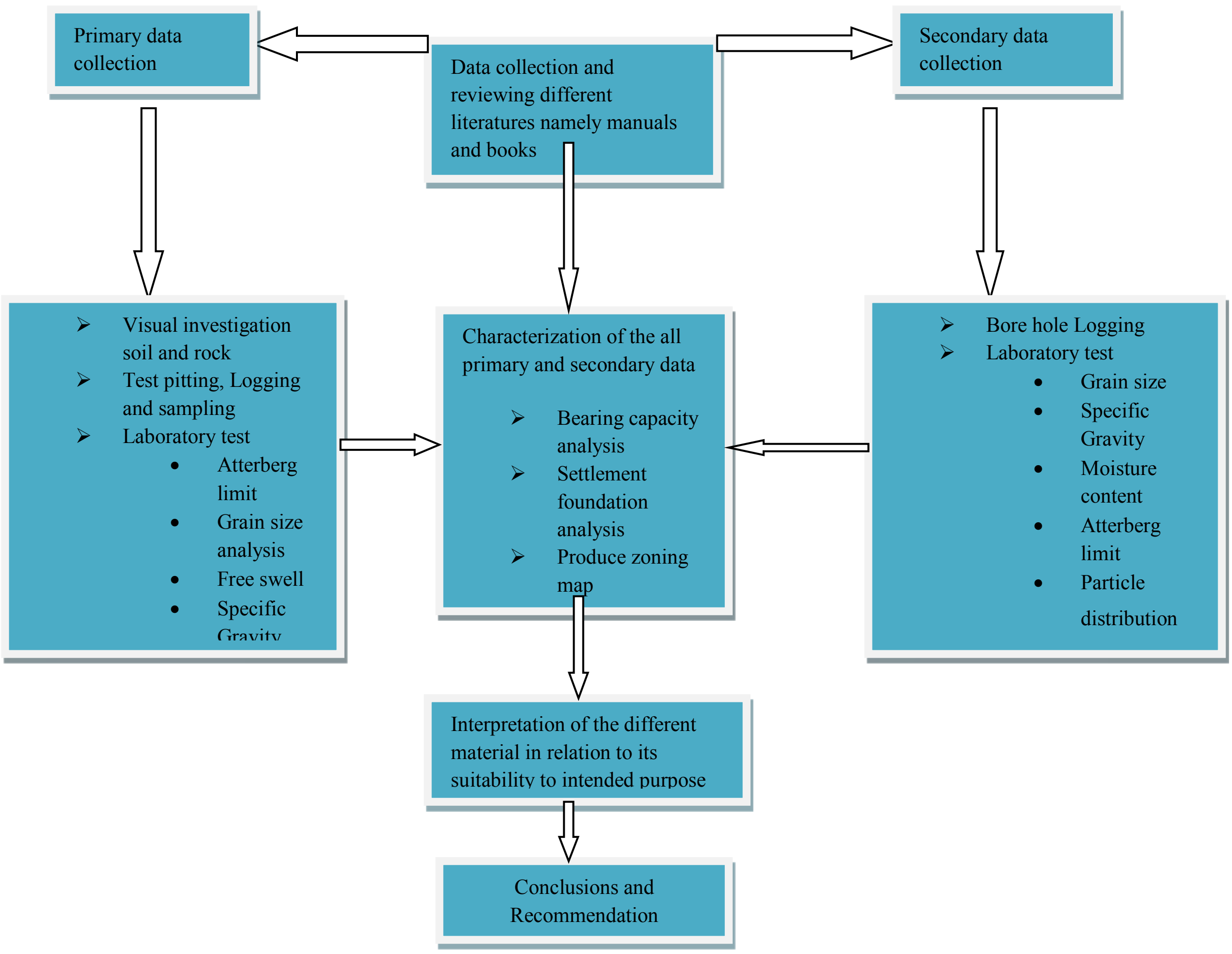


Fig 1.1 Flow chart showing general methodology followed for the present study

1.5 Scope of The Study

This study is proposed for the characterization of the building foundation in the study area. The results may be useful for the general characterization of the foundation soils in terms of bearing capacity and the settlement potential. Analysis of building foundation condition is useful for foundation condition estimation and its general suitability for building foundation.

1.6 Limitations To The Study

Availability of systematic secondary data was one of the limitation factor in the present study. Most of the geotechnical works for which secondary data is available are not conducted following standardized data collection procedures and also well-organized data archives were not maintained.

1.7 Chapter scheme

The present study is organized into six chapters.

Chapter one:-gives a general introduction to building foundation, statement of the problem, objectives of the study, methodology of the study, scope of the study, limitations of the study and organization of the study.

Chapter two:-over view of the study area, soils in the study area, buildings and associated problems in the study area and focus on regional and local geology of the study area.

Chapter three:-presents literature review related to the soils and its nature, foundation condition estimation, foundation of building and types of foundation, estimation of bearing capacity, soil consolidation and associated settlement potential.

Chapter four:-discusses about the methods and techniques. It discuss about the bearing capacity assessment; in-situ methods and bearing capacity estimation through equations and also discuss about the settlement potential estimation:- In-situ methods and Laboratory testing –consolidation.

Chapter five:-present investigation works and the data collected from field investigation and laboratory test results.

Chapter six:-presents the results obtained from the analysis, interpretation and discussion of site characterization, foundation characterization bearing capacity and settlement estimation of the area.

Chapter seven:-is about conclusion and recommendation of the study.

*****

CHAPTER TWO DESCRIPTION OF THE STUDY AREA

2.1 Preamble

This chapter deals with the description of geologic, topographic, climatic, Physiography and Drainage Pattern which has influenced the formation of foundation materials in the study area. This section also includes description of the seismic condition and soil type distribution in the study area and its surrounding.

2.2 Location and Accessibility

Bole Sub-city is one of the largest sub-cities located in eastern part of Addis Ababa. It shares boundary with Yeka Sub-city in the north, Kirkos and Nifas Silk Lafto Sub-cities in the west and Akaki Kaliti sub-city in the south. In Bole Sub-city the lowest elevation is 2,114 m in the southern periphery and the maximum elevation is 2,407 m above the sea level. The study area is located in the North-Eastern part of Addis Ababa specifically in Ayat area of Bole Sub City. The area is generally characterized by flat to sloping topographic feature. The study area is geographically bounded in between 990090.974m N- 997182.9m N latitude and 482961.594m E- 489307.021m E longitude of UTM Zone 37N (Fig.2.1). The total area covered by the study is about 45 km^2. The study area is accessible through Asphalt roads and gravel roads.

2.3 Climate

Addis Ababa has a humid subtropical highland climate (Koppen Cwb). Addis Ababa has a humid subtropical mild summer climate that is mild rainy summers and moderate seasonality. According to Ethiopian National Meteorological Agency (ENMA, 2014) the highest monthly average precipitation recorded was 296.5 mm in the month of August, (2011 -2015) with maximum and minimum temperatures of 26.5 °C and 8°C, respectively (2011 -2015). The main rainy season is from May to September. The hottest and driest months are usually April and May (Table 2.1).

2.4 Physiography and Drainage Pattern

The study area is located in the western margin of the Main Ethiopian Rift and represents a transition zone between the Ethiopian Plateau and the rift with poorly defined escarpment (Tamru Alemayehu et al., 2006). Generally, the elevation declines from northeast to south-west. It is bordered in left side by Furi Mountain, northern side by Entoto Mountain Ridge and Akaki River, right side by Wechecha Mountain and at lower part by Yerer Mountain. The elevation of the study area varies from 2230 m to 2440m. The physiography of the study area is a result of volcanism and erosion.

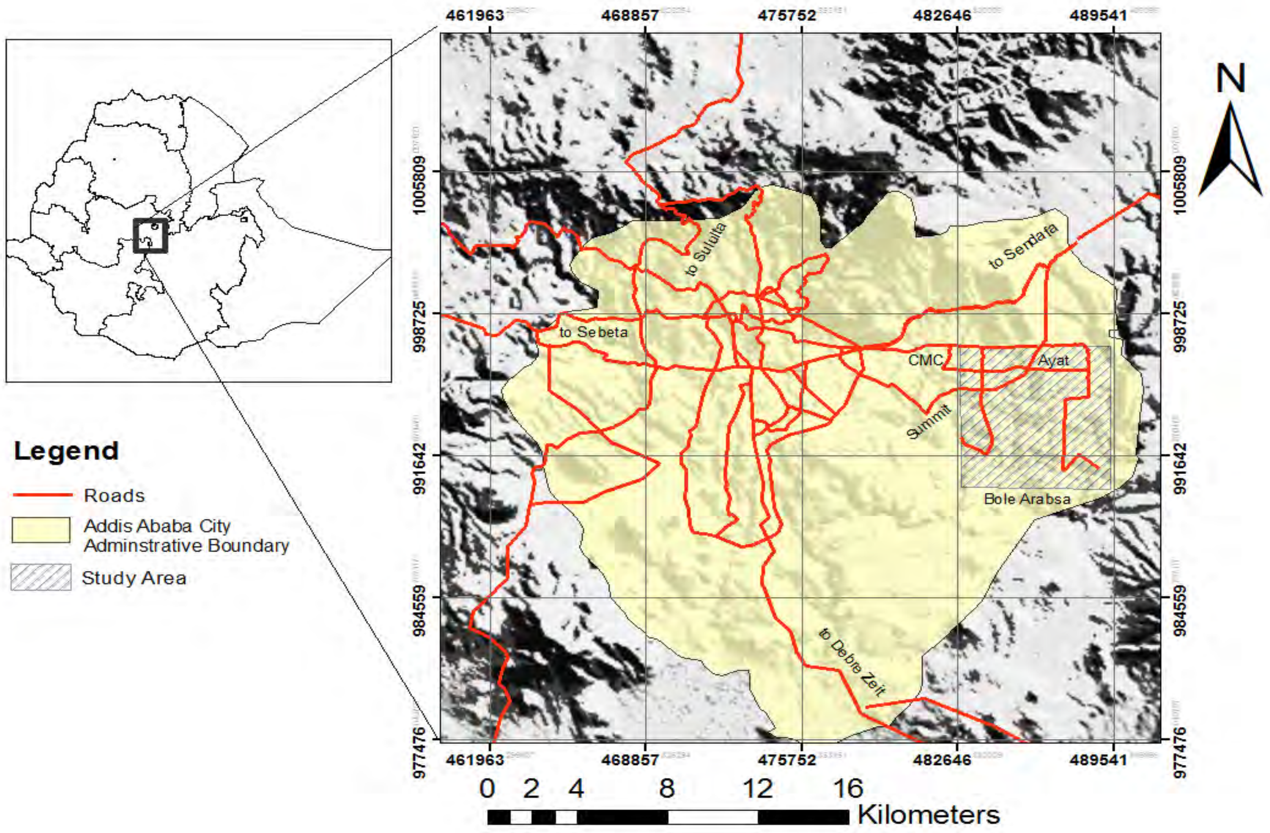


Fig. 2.1 Location map of study area

Table 2.1 The monthly precipitation, average maximum and average minimum temprature of the study area (Years 2011-2015)

Months	Average Max. (°C)					Average Min. (°C)					Precipitation				
	2011	2012	2013	2014	2015	2011	2012	2013	2014	2015	2011	2012	2013	2014	2015
Jan	23.7	24.5	24.5	24.3	23.8	9.3	8	9.3	9.5	8.3	3.4	-	-	-	-
Feb	25.4	25.5	26.3	25.2	26.3	9	8.5	10.4	11.7	10.3	13.6	-	-	41.7	-
Mar	24.7	26.7	26.3	25.8	26.4	11	10.7	13	12.1	11.9	27.9	34.5	63.5	29.7	21.3
Apr	-	24.7	26	26.4	-		12.1	13.2	13	-	-	75.1	114.4	33.7	-
May	24.9	26.5	25.1	25.9	-	12.8	12.7	12.8	13	-	86	58.5	78.5	62.1	-
Jun	23.7	24	23.3	25.1	-	12.3	12.3	12.1	12.8	-	148	72.8	101.4	41.8	-
Jul	21.8	24	21.3	22	-	12	12.1	11.9	12.5	-	183.1	228.8	157.6	179.7	-
Aug	20.7	20.7	20.8	21.5	-	12.3	11.9	12	11.8	-	296.5	281.6	270.2	253.6	-
Sep	21.5	22	22.8	21.7	-	11.9	11.7	11.4	12.2	-	141.3	176.9	126.7	95.1	-
Oct	24.2	23.9	23.2	23.2	-	9.3	9.9	10.5	10.5	-		1.2	45.3	34.8	-
Nov	23.5	24.4	23.9	23.7	-	10.5	9.5	9.8	9.1	-	11.9	-	3.2	-	-
Dec	22.8	-	23.4	22.9	-	7.3	-	7.9	8.4	-	-	-	-	-	-

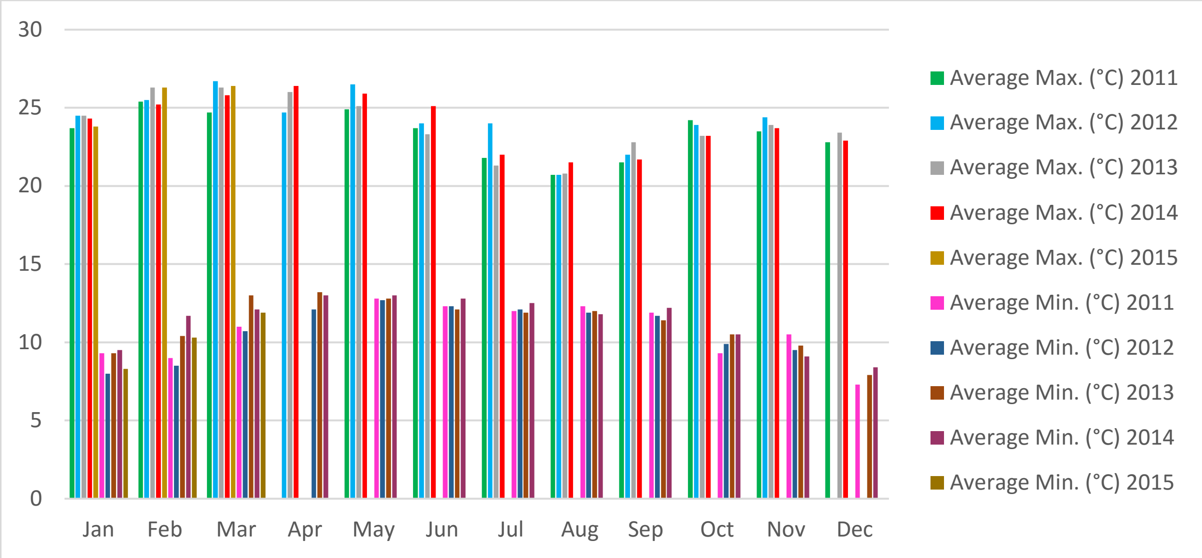


Fig. 2.2 Average temperature in Bole Sub-City for years 2011-2015

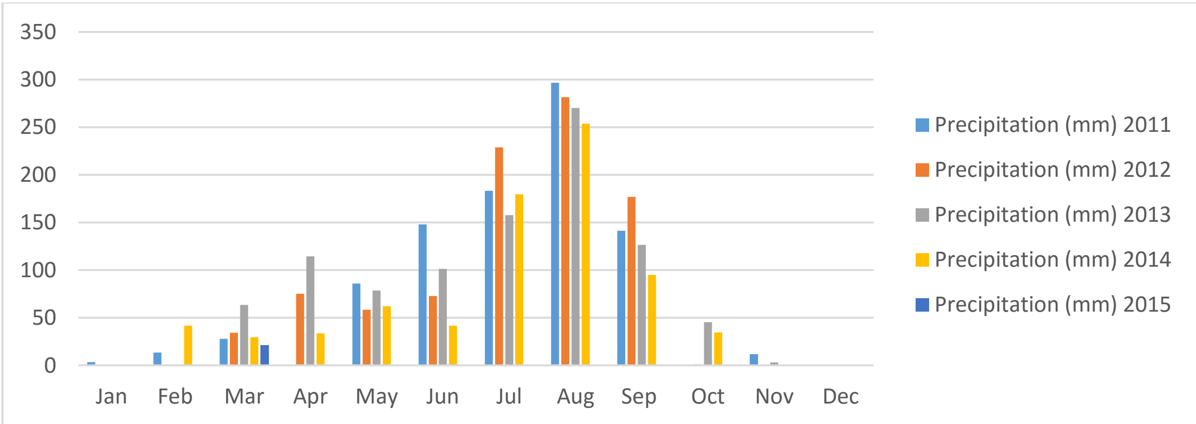


Fig. 2.3 Monthly average precipitation in Bole Sub-City for the years 2011-2015

Entoto mountain ranges are the major water source for the major permanent rivers which flow within and outside the zone. The Awash River basin covers much of the zone. The Akaki River basin is the main river system in the study area, the two rivers flow from north of Finfine city to the southwest and intersect at Abba Samuel dam lake and finally join the Awash River.

2.5 Regional geology

Addis Ababa is located in the western escarpment of the Main Ethiopian Rift system (MER).The geological history of the study area is an integral part of the evolution and development of the Ethiopian Plateau and the Rift system. The Miocene-Pleistocene volcanic succession in Addis Ababa area has been suggested by Haile Sellasie Girmay and Getaneh

Assefa (1989, as cited by Habtamu Solomon, 2010) from bottom to top are Alaji basalts, Entotosilicics, Addis Ababa basalts, Nazareth group, and Bofa basalts.

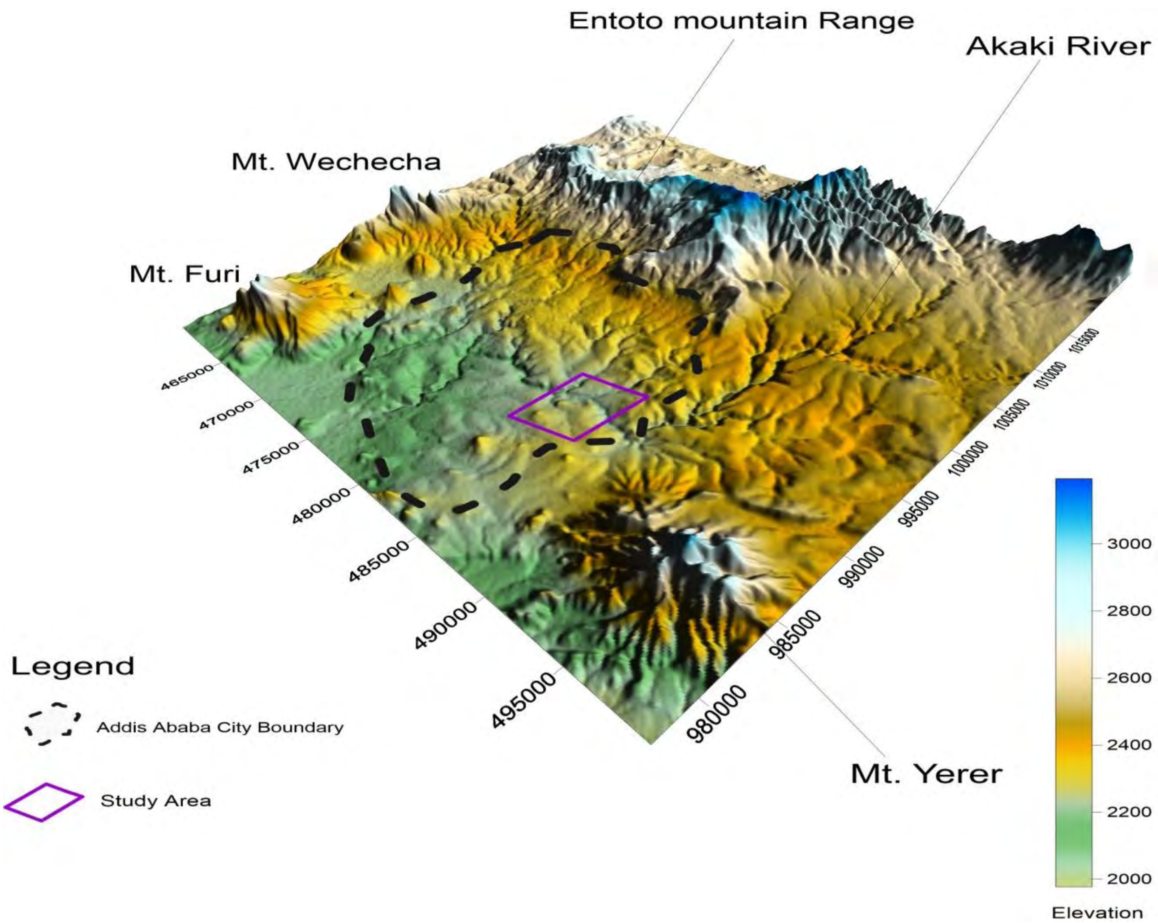


Fig. 2.4 Physiographic map of Addis Ababa

2.5.1 Alaji Basalt

Alaji Basalt group are volcanic rocks (Alaji rhyolite and Basalt) and parts of the escarpment were out poured from the end of Oligocene until middle Miocene. This unit is composed of basalts, which show variation in texture from highly pophyricto aphyric. Within this unit, there is an intercalation of gray and glassy welded tuff. The outcrop of Alaji basalt extends from the crest of Entoto(ridge bordering the northern parts of Addis Ababa) towards the north (Haile Sellasie Girmay and Getaneh Assefa,1989). The unit is underlay in by tuffs and ignimbrites. Its stratigraphic relationship with the Entotosilicics is difficult to determine as they occur in a fault contact. Mohr (1967, as cited in Tamru Alemayehu et al., 2006) proved that the Entoto trachyte overlies the Alaji basalt.

2.5.2 Entoto Silicics

These early Miocene aged silicic volcanic could represent localized terminal episodes to massive Oligocene fissure-basalt activity in the Addis Ababa region (Morton et al.,1979). The thickness of the flow becomes maximum on the top of Entoto ridge and thin both towards the plateau and the plain east of Addis Ababa. According to Zanettin and Justin (1974,as cited in Tamru Alemayehu et al.,2006) these lavas make up a thick pile of flows accumulated along east west fissures (east west fault running from Kassam River to Ambo) and up lifted north wards. This unit is uncomfortably overlain by Addis Ababa basalt on the foothills of Entoto and underlain by Alaji basalt. The Entoto silicics composed of rhyolite and trachyte with minor amount of welded tuff and obsidian (Haile Sellasie Girmay and Getaneh Assefa, 1989). It also outcrops in the eastern part of the town from the Kokebe Tsebah School to the British Embassy. The thickness is 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 porphyritic trachyte and underlain by a sequence of tuffs and ignimbrites. Tuffs and ignimbrites are welded and characterized by columnar jointing (Assegid Getahun, 2007).

2.5.3 Addis Ababa Basalt

According to Haile Sellasie Girmay and Getaneh Assefa (1989), Addis Ababa basalt is the oldest visible rock post-dating the Entoto silicic. It is mainly found in the central part of the town and underlain by the Entoto silicics and overlain by Lower welded Tuff of the Nazareth group. The maximum thickness exceeding130m is found at Ketchen stream. It is porphyritic in texture, composed of labradorite, bytownite, olivine and augite as phenocrysts. The ground mass is made of andesine, labradorite, olivine, agnetite and pyroxene (Haile Sellasie Girmay and Getaneh Assefa, 1989).

Olivine porphyritic basalts outcrop in the central part of the town that includes Mercato, Teklehaymanote and Sidist Kilo. The distribution of plagioclase porphyritic basalt is almost the same as that of the olivine porphyritic basalt, but only little more north wards. It outcrops in an area, which includes Sidist Kilo, General Winget School and French Embassy. The thickness of the former varies from 1m or less in the foothills of Entoto, Lideta Airfield and Filwoha to greater than 130m at Ketchane stream (Morton, 1974; Varnieretal., 1985 as cited in Tamru Alemayehu et al., 2006). The Lower Welded Tuff overlies both types of basalt near by the Building College, the Kolfe Police School, the Kokobe Tseba School and Yeka

Mariam Church. On the other hand, only in the gorge of the Ketchane stream, the olivine pophyric basalt is overlain by the plagioclase porphyritic basalt, while elsewhere the relationship between them is very difficult to determine (Varnier et al., 1985 as cited inTamru Alemayehu et al., 2006). Addis Ababa basalt yield ages clustering around 7my and seems to have no time/composition equivalent (Morton et al., 1974as cited in Tamru Alemayehu et al., 2006).

2.5.4 Nazaret Group

The units identified in this group are denoted as Lower Welded Tuff, Aphanitic basalt and Upper Welded Tuff. The group is underlain by Addis Ababa basalt and overlain by Bofa basalts. The rocks outcrop mainly south of Filwoha fault and extend towards Nazaret (Tamru Alemayehu et al., 2006; Mohammed, 2007).

Lower Welded Tuff

It is the rocky outcrops as small discontinuous body in Filwoha, western parts of Addis Ababa and Sululta. Generally, it is overlain by the aphanitic basalt and underlain by the olivine and plagioclase prophyritic basalt. The age of this rock unit, as dated by Morton et al.(1979, as cited in Tamru Alemayehu et al., 2006), at Addis Ababa and Sululta is 5.1 and 5.4 million years, respectively. This age overlap with the period of the activity of Wachecha trachyte volcanoes, dated 4.6 million years. Wachecha is located15km west of Addis Ababa and probably the sources of the Lower welded tuff at both localities (Morton et al., 1979 as cited in Tamru Alemayehu et al., 2006; Kabite, 2011).

Aphanitic Basalt

This basalt covers the southern part of the town, especially the areas of Bole International Airport and Lideta Airfield. The rock body shows vertical curved columnar jointing together with sub horizontal sheet jointing. Kaolin and lenses are present at the contact of this basalt with the younger ignimbrite. This makes an evidence for the hydrothermal alterations along a NE-SW fracture system, which may affects both the basalt and the Entoto trachyte. Moreover the basalt is overlain by pumeacoues pyroclastic falls and the pyroclastic falls. 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. The crystals of plagioclase show marked flow ignimbrites. Trachy-basalt outcrops around Repi and nearby General Wingate

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 rocks of the group is not clear, but probably younger than the aphanitic basalt (Getaneh Assefa et al.,1985 as cited in Tamru Alemayehu et al.,2006; Kabite, 2011).

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 sandine, an orthoclase, rebecite, quartz, pumice and unidentified volcanic fragments (Getaneh Assefa et al., 1989). The welded tuff is underlain by aphanitic basalts and overlain by young olivine basalts. An age determinations made on a sample collected nearby Haile Gebreselassie road resulted 3.2million years, that overlap with the activity of Yerer trachytic volcano‘s (Morton et. al., 1979 as cited in Tamru Alemayehu et al., 2006).

Young Trachyitic Flow

This rock is predominating in the south west part of the town, from Dama hotel towards Furi and Repi along the hills and foothills of Hana Mariama and Tulu Iyou. It is porphyritic with phenocrysts of plagioclase (albite-oligoclase) sandine, biotite within a ground mass of microlities of feldspar.

Moreover, it is underlain by the tuff that covers the young ignimbrite and overlaying by alternating flows of plagioclase porphyritic basalt and rhyolite especially in the Repihill. Its relation with the young olivine prophyrytic basalt is not clear as they outcrop in different parts of the areas; however, in a small outcrop nearby Aba Samuel Lake south of the project area, the trachyte underlies the olivine porphyritic basalt (Mohammed, 2007; Tamru Alemayehu et al., 2006).

2.5.5 Young Olivine Porphyritic (Bofa) Basalt

They outcrop south ward from Akaki River where they appear in the form of boulders reaching a thickness of 10meter. They are restricted and dominant in the south east part of the town i.e. Debre Zeit Road. They contain phencorysts of plagioclase, olivine that is partially and completely altered to idingisite and augite within a ground mass composed of plagioclase, magnetite, pyroxene and olivine. This basalt is underlain by the tuffs, which cover the welded tuff. The age of this basalt is 2.8 MY (Mohammed, 2007;Tamru Alemayehu et al., 2003).

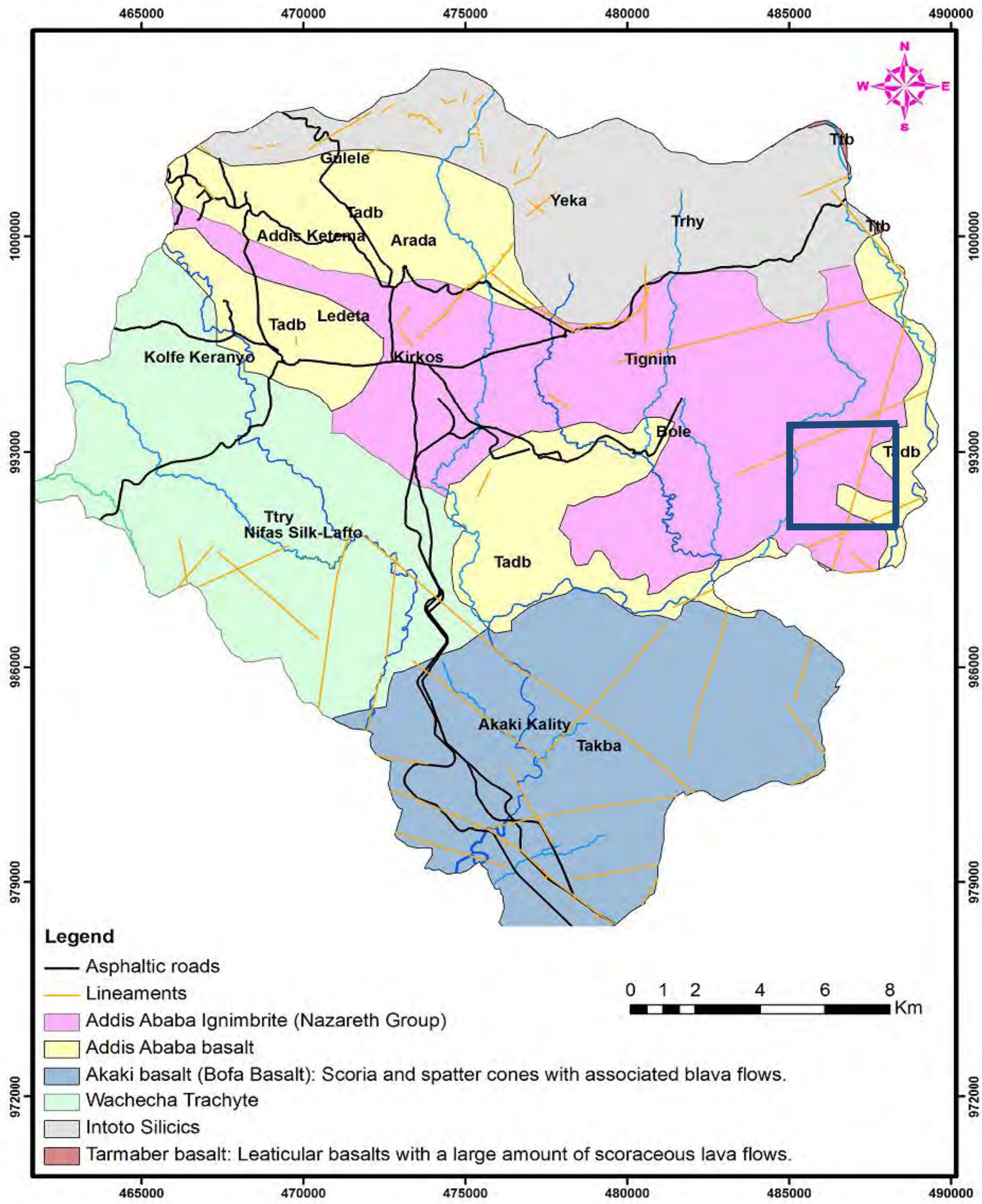


Fig. 2.5 Geological Map of Addis Ababa (after WWDSE. 2008)

2.6 Seismicity

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).

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 20thcentury 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.

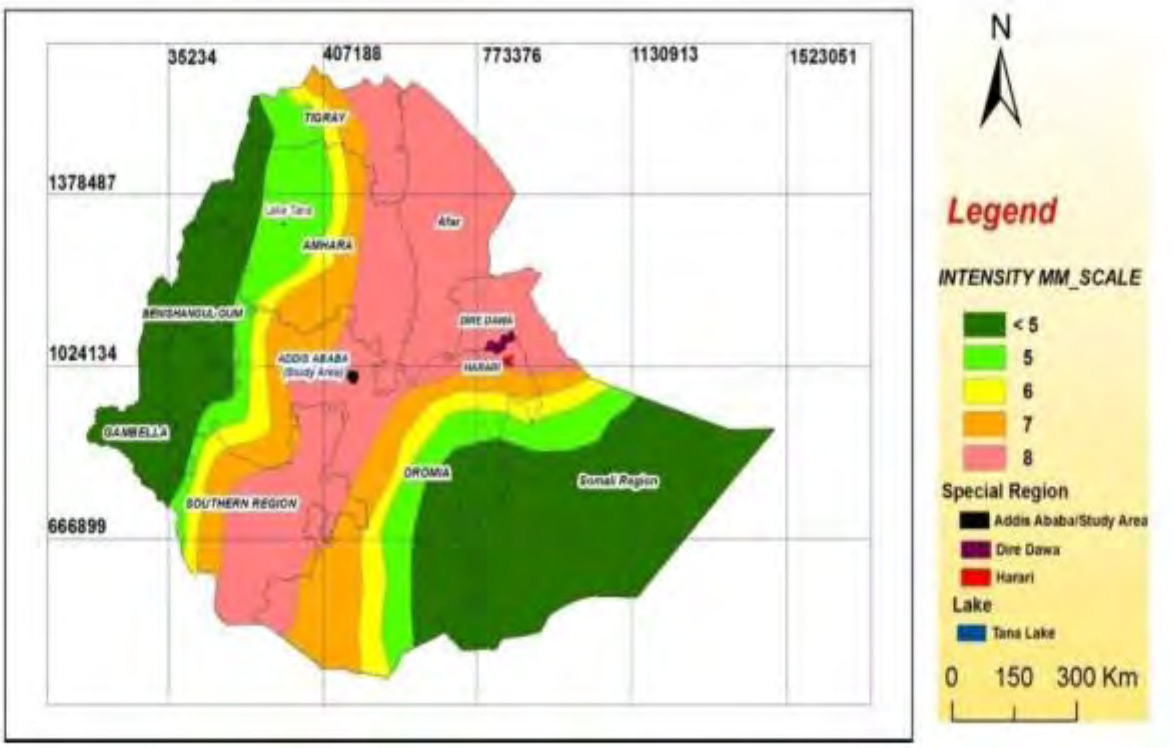


Fig. 2.6 Seismic risk map of Ethiopia 100 years return period, 0.99 probability (After Laike Mariam Asfaw, 1986)

A similar study by Fekadu Kebede (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.

According to seismic risk map of Ethiopia (Fig.2.6) 100 year return period, 0.99 probability by Laike Mariam Asfaw (1986), the country is divided into zones of approximately equal seismic risks based on the known distribution of past earthquakes. According to Johnson and Degraff (1988), these seismic intensity zones are related to the ground acceleration (Table-2.2).

Table 2.2 Seismic Intensity Zone related to Ground Acceleration

Intensity (MM)	< 5	5	6	7	8
Ground Acceleration (g).	0.01	0.02	0.05	0.1	0.2

(Source: Johnson and Degraff (1988)

The study area falls in the intensity scale 8, thus the estimated ground acceleration as per Johnson and Degraff (1888) will be 0.2g. The intensity scale 8 indicates that the present study area lies in the high seismic risk zone. Therefore, for safe design of small engineering structures in Addis Ababa, a ground acceleration of 0.2g should be considered. However, site specific seismic investigations should be conducted for big engineering structures.

2.7 Local Geology and Topography

Topographic relief has an important role in soil formation. For a deep residual soil to develop, the rate at which weathering advances into the earth’s crust must exceed the rate of removal of the products of weathering by erosion. Topography controls the rate of weathering by partly determining the amount of available water and the rate at which it moves through the zone of weathering. In addition to this, it also controls the effective age of the profile by controlling the rate of erosion of weathered material from the surface. Thus, deeper residual profiles will generally be found in valleys and gentle slopes rather than on high ground or steep slopes (Blight, 1997 as cited in Hana Tibebu, 2008).

The topography of the study areas favors the development of heterogeneous soil profiles by the decomposition of rocks on which it lies. Thus, residual soils are commonly seen in most parts of the city with varying thickness. On the other hand, due to intensive erosional activities there is poor soil development (shallow soil profile) or patchy occurrences on most parts of the slope. The dominant type of soil in the southern parts of the city, where surface water is poorly drained, is expansive clay soil.

The site is also characterized as slightly flat with flat ground topography. The boreholes were drilled to a maximum depth of 15 m-30 m in order to establish the subsurface geology and groundwater condition of the site. These bore hole were drilled by different governmental and non-governmental organization companies.

Data from field geology and bore hole logging suggests that the study area is mostly dominated by slightly to moderately weathered IGINIMBRITE ROCK with joints closely to medium spaced. Tuffs and ignimbrites are welded and characterized by columnar jointing.
In some areas dark highly weathered and decomposed to moderately weathered rocks were found occasionally with vesicular BASALT.

2.7.1 Ignimbrite rock

It is exposed forming a relatively slightly flat to small ridge where its measured thickness ranges from 2 - 6m. The observed ignimbrite rock is light grey in color and slightly to moderately weathered, closely jointed/ fractured, medium grained and weak to medium strong (Plate 2.1).This formation covers smaller area and is overlain by expansive clay

Plate 2.1 Highly weathered ignimbrite exposed in the study area

2.7.2 Volcanic tuff

Stratigraphically this unit is found overlaying basaltic lava. It covers a flat topography and vertically jointed. It has a white to light gray color appearance in its freshly broken surface. On its weathered face, it is gray in color. It has moderately thin horizontal beds where the thickness ranges from 60 to 70 cm. The joints have a general SE-NW orientation (Plate 2.2). The joints affecting this unit are spaced 0.40 to 1.10 m and the aperture ranges from 5 to 8 cm and are filled with clayey soil.

Plate 2.2 Volcanic tuff

2.7.3 Basalt unit

It is exposed forming a relatively flat to small ridge where its measured thickness ranges from 3 - 5m. The observed basalt rock has variety of color on its weathered discolored surface: light yellowish, dark gray, and reddish brown and part of it is changed into soil whereas its fresh surface exhibits dark gray color. This basaltic lava is moderately to highly weathered and has weak strength. Vertical joints of N-S, SE and SW trend (Plate 2.3) affect this lava. These joints are filled with clay and are widely spaced to very widely spaced where the spacing range from 1.5 to 3m. The joint surfaces are undulating.

Plate 2.3 Basalt lava

Structures

The structures mainly present in exposed rocks in the study area are joints. The general orientation of these joint sets is presented in Table 2.3 and Fig. 2.7.

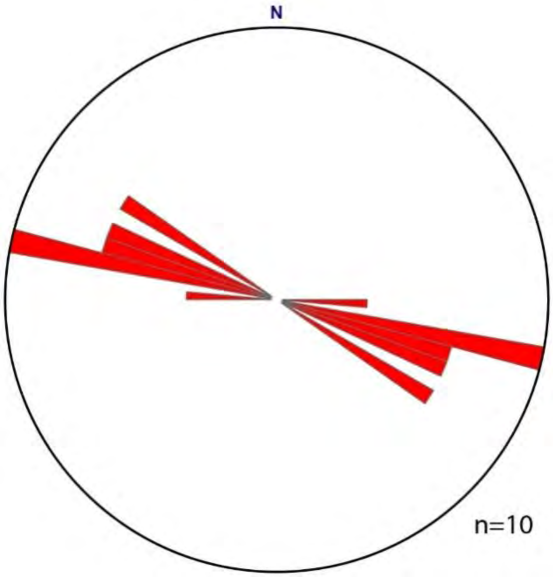
Table 2.3 Structural field measurement of joints

No.	strike	dip	strike	Dip
Joints affecting Basalt lava			Joints affecting Volcanic Tuff	
1	145	90NE	300	65NE
2	N-S	75NE	110	90SW
3	N-S	85NE	100	80NE
4	145	80NE	100	70SW
5	240	85NW	285	90SW
6	210	80NW	E-W	85NS
7	N-S	90NW	285	90SW
8	190	90NE	100	90NE
9	160	70SW	300	65NE
10	150	85SW	290	90SW

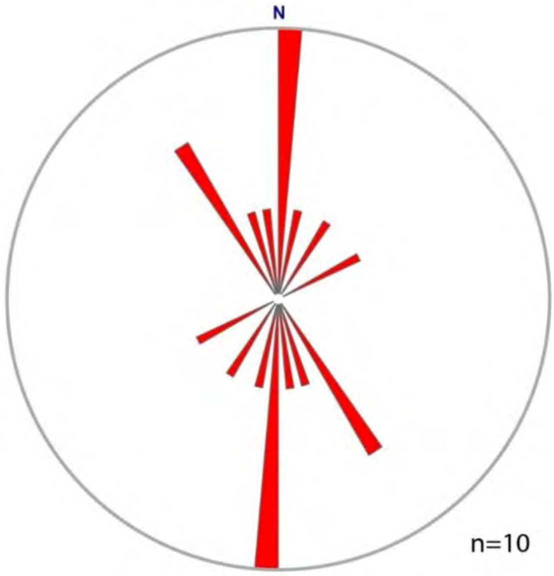
2.8 Soil description

2.8.1 Black cotton soil

Based on the visual description Black Cotton soil cover the top most part of the study area. This layer is identified as soft dark, firm to stiff soil layer. The soil has a thickness of about 0.2 – 3m. These soils are highly plastic, fine grained and expansive in nature.



(a)



(b)

Fig. 2.7 Rose diagram of joints measured in the field joints affecting tuff (a) basalt (b)

2.8.2 Light to dark grey soil

The soil from this layer can be characterized as light to dark grey, stiff to very stiff and highly plastic silty CLAY. The thickness of this soil layer varies from 2 to 10.7 m in the study area.

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CHAPTER THREE LITERATURE REVIEW

3.0 Preamble

During the present study a thorough literature review was undertaken to have a general background on the subject matter and to develop a conceptual framework within which general methodology for the present study was evolved. Emphasis was given to understand the building foundation conditions and its assessment techniques.

Various literature sources were reviewed which include maps and reports, books, journal articles and materials available on internet. Besides, previous studies and investigation reports were assessed and included as a part of literature review.

3.1 Soil and its nature

Most of the building foundations are laid down on soils. Thus, the soils form the foundation for the safety and stability of the building structures. Different soils behave in different manner when subjected to the loads imposed by the structures. This behavioral variation to imposed loads is on account of genetic origin of soils, its mineral composition and its index and engineering properties. Therefore, before knowing the soil performance as engineering foundation it is mandatory to know what type of soil it is and what type of properties it possess (Arora, 2004).

The term soil is defined by different subject of matter for different purpose. According to Webster's dictionary soil means the upper layer of the earth in which plant grow, consisting of a mixture of organic remains, clay and rock particles (Soil taxonomy, 1999). In terms of agronomy soil means the upper layer of the earth that may be excavate, specifically, the loose surface material of the earth in which plants grow (Soil taxonomy, 1999). In field of agronomy the main concern is to use soil for raising crops. In the field of geology Earth's crust is assumed to consist unconsolidated sediments, called mantle or regolith, overlying rocks, due to that the term soil is used for the upper layer of mantle which can support plants(Arora, 2004). In geotechnical engineering the upper layer part of the earth crust is called top soil. The top soil contains a large quantity of organic matter and is not suitable for construction material or foundation structures (Arora, 2004). The top soil is removed from the earth surface before construction.

The term 'soil' in soil engineering is defined as an unconsolidated material, composed of solid particles, produced by the disintegration of rocks (Arora, 2004).The void space between the particles may contain air, water or both. The solid particles may contain organic matter. The soil particles can be separated by such mechanical means as distributed in water (Arora, 2004).

Soil is a three-phase material consisting of solid particles, water, and air. Its mechanical behavior is largely dependent on the size of its solid particles and voids. The solid particles are formed from physical and chemical weathering of rocks. Therefore, it is important to have some understanding of the nature of rocks and their formation (Helwany, 2007).

In general, soil is a natural body comprised of solids (mineral and organic matter), liquid and gases that occurs on the land surface and is characterized by one or both of the following, horizons or layers that are distinguishable from the initial material as a result additions losses, transfers and transformations of energy and matter or the ability to support rooted plants in a natural environment (Soil taxonomy, 1999).

Soils are produced from rocks through the process of weathering and natural erosion; there are two type of weathering physical and chemical weathering to help break down rock(parent rock) (Arora, 2004).Physical weathering is cause for disintegration of rock fragment. Physical weathering can occur due to temperature change, pressure, and erosion by water, ice and wind. Chemical weathering is changes the composition of rocks. Chemical weathering decomposed from the serious chemical process such as acidification, dissolution and oxidation (Helwany, 2007).

Further, Helwany (2007) describe the particle size and the distribution of various particle sizes of a soil depend on the weathering agent and the transportation agent. Soils are categorized as gravel, sand, silt or clay depending on the predominant particle size involved. Gravels are small pieces of rocks. Sands are small particles of quartz and feldspar. Silts are microscopic soil fractions consisting of very fine quartz grains. Clays are flake-shaped microscopic particles of mica, clay minerals, and other minerals. The average size (diameter) of solid particles ranges from 4.75 to 76.2 mm for gravels and from 0.075 to 4.75mm for sands. Soils with an average particle size of less than 0.075mm are either silt or clay or a combination of the two. Soils can be divided into two major categories: Cohesionless and Cohesive.

Arora (2004) describe Cohesionless soils, such as gravelly, sandy, and silty soils, have particles that do not adhere together even with the presence of water. Arora (2004) describe Cohesive soils are characterized by their very small flake like particles, which can attract water form plastic matter by adhering to each other. Ranjan (1993) describe 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. 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.

Johnson (1996) describes the Soil deformation may occur by change in stress, water content, soil mass, or temperature. 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/time delayed consolidation is the reduction in volume associated with a reduction in water content, and it occurs in all soils. Consolidation occurs quickly in coarse-grained soils such as sands and gravels, and it is usually not distinguishable from elastic deformation (Johnson, 1996; Robert, 1990; Ranjan, 1993).

According to Robert (1990) and Ranjan (1993) 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 position (Robert, 1990).

Ranjan (1993) describe the time required for the settlement to occur vis-a 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 give us an idea of how much settlement will undergo after it is constructed and whether such a settlement 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) and Robert (1990), describe the compression settlement of each soil stratum is computed from the consolidation test data and from the soil stresses before and after construction.

3. 2 Foundation condition estimation

According to Bowles (1997), the foundation is the part of an engineered system that transmits to and into the underlying soil or rock the loads supported by the foundation structure. The same author state that the term superstructure is commonly used to describe the engineered part of the system bringing load to the foundation, or substructure.

According to Das (2011) and Arora (2004), in general there are two types of foundations for buildings and bridges; Shallow foundations and Deep foundations.

According to Das (2011), Spread footings, wall footings, and mat foundations are type of shallow foundations. The same author state that pile and drilled shaft foundations are type of deep foundations.


Plate 3.1 Spread (Isolated) Footing Plate 3.2 Mat Foundation

George (1989) describes 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 soil.

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 foundation 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 soil and rocks, and then designing a suitable foundation structure (Bell, 2007; Robert, 2010).

According to Bell (2007), a satisfactory foundation must meet three requirements:

(i) It must be placed at an adequate depth to prevent frost damage, undermining by scour, or damage from future construction nearby.
(ii) It must be safe against breaking into the ground
(iii) It must not settle enough to disfigure or damage the structure.

3.3 Foundation Selection and Classification

According to Arora (2004), a foundation is required for distributing the loads of the superstructure on a large area, and it should be designed such that; (i) The soil below dose not fail in shear and (ii) The settlement is within the safe limit.

Foundation may be broadly classified into two categories(Arora, 2004;Day, 2006): (i) shallow foundation which transmits the loads to the strata at shallow depth, and (ii) Deep foundation which transmits the loads to considerable depth below the ground surface.

Spread footings are used for distributing concentrated column loads over a large area so that the bearing pressure is less than or equal to allowable soil pressure (Arora, 2004).

Foundation engineering deals with the selection of the type of foundation, such as using a shallow or deep foundation system and another important aspect of foundation engineering involves the development of design parameters, such as the bearing capacity or estimated settlement of the foundation (Day, 2006).

According to Bowles (1997), foundations may be classified based on where the load is carried by the ground, producing: (i) Shallow foundations - termed bases, footings, spread footings, or mats. The depth is generally $D/B \leq 1$ but may be somewhat more and (ii) Deep foundations- piles, drilled piers, or drilled caissons. $L_p/B > 4$ + with a pile.

Bowles, (1997) also stated that supporting capacity of the soil, from either strength or deformation considerations, is seldom over 1000 kPa but more often on the order of 200 to 250 kPa. This means the foundation is interfacing two materials with a strength ratio on the order of several hundred and as a consequence the loads must be "spread" to the soil in a manner such that its limiting strength is not exceeded and resulting deformations are tolerable but shallow foundations accomplish this by spreading the loads laterally, hence the term spread footing and where a spread footing (or simply footing) supports a single column, a mat is a special footing used to support several randomly spaced columns or to support several rows of parallel columns and may underlie a portion of or the entire building.

According to Bowles (1997), deep foundations are analogous to spread footings but distribute the load vertically rather than horizontally and a major consideration for both spread footings (and mats) and piles are the distribution of stresses in the stress influence zone beneath the foundation. Below a critical depth of about 5B the soil has a negligible increase in stress (about 0.02qo) from the footing load. This influence depth depends on B, because these B values are in a possible range beneath a large building, so any poor soil below a depth of 2 m would have a considerable influence on the design of the wider footings (Bowles, 1997).

Table3.1 Foundation types and typical usage (Bowles, 1997)

Foundation type	Use	Applicable soil conditions
Shallow foundations(generally D/B≤ 1		
Spread footings,	Individual columns, walls	Any conditions where bearing capacity is adequate for applied load. May use on a single stratum; firm layer over soft layer or soft layer over firm layer. Check settlements from any source.
Combined footings	Two to four columns on footing and/or space is limited.	Same as for spread footings above.
Mat foundation	Several rows of parallel columns; heavy column loads; use to reduce differential settlements.	Soil bearing capacity is generally less than for spread footings, and over half the plan area would be covered by spread footings. Check settlements from any source.
Deep foundations(generally Lp/B≥ 4+)		
Floating foundation	In groups of 2* supporting a cap that interfaces with columns	Surface and near surface soils have low bearing capacity and competent soil is at great depth. Sufficient skin resistance can be developed by soil to pile perimeter to carry anticipated loads.
Bearing pile	Same as for floating pile	Surface and near surface soils not relied on for skin resistance; competent soil for point load is at a practical depth (8 - 20m).
Drilled piers or caissons	Same as for piles; use fewer; for large column loads	Same as for piles. May be floating or point bearing (or combination). Depends on depth to competent bearing stratum.

According to Murthy (1990), two important factors that are to be considered in stability of a structure are: (i) the foundation must be stable against shear failure of the supporting soil and (ii) the foundation must not settle beyond a tolerable limit to avoid damage to the structure.

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

According to Day (2006), the selection of a particular type of foundation is often based on a number of factors, such as:

(i) Adequate depth: the foundation must have an adequate depth to prevent frost damage. For such foundations as bridge piers, the depth of the foundation must be sufficient to prevent undermining by scour.
(ii) Bearing capacity failure: the foundation must be safe against a bearing capacity failure.
(iii) Settlement: the foundation must not settle to such an extent that it damages the structure.
(iv) Quality: the foundation must be of adequate quality so that it is not subjected to deterioration, such as from sulfate attack.
(v) Adequate strength: the foundation must be designed with sufficient strength that it does not fracture or break apart under the applied superstructure loads. The foundation must also be properly constructed in conformance with the design specifications.
(vi) Adverse soil changes: the foundation must be able to resist long-term adverse soil changes. An example is expansive soil, which could expand or shrink causing movement of the foundation and damage to the structure.
(vii) Seismic forces: the foundation must be able to support the structure during an earthquake without excessive settlement or lateral movement. Based on an analysis of all of the factors listed above, a specific type of foundation (i.e., shallow versus deep) would be recommended by the geotechnical engineer.

The following sections discuss various types of shallow and deep foundations. A shallow foundation is often selected when the structural load will not cause excessive settlement of the underlying soil layers and in general, shallow foundations are more economical to construct than deep foundations (Day, 2006).

Based on economic considerations, mat foundations are often constructed for the following reasons (NAVFAC DM-7.2, 1982):

(i) Large individual footings. A mat foundation is often constructed when the sum of individual footing areas exceeds about one-half of the total foundation area.

(ii) Cavities or compressible lenses. A mat foundation can be used when the subsurface exploration indicates that there will be unequal settlement caused by small cavities or compressible lenses below the foundation. A mat foundation would tend to span over the small cavities or weak lenses and create a more uniform settlement condition.

(iii) Shallow settlements. A mat foundation can be recommended when shallow settlements predominate and the mat foundation would minimize differential settlements.

(iv) Unequal distribution of loads. For some structures, there can be a large difference in building loads acting on different areas of the foundation. Conventional spread footings could be subjected to excessive differential settlement, but a mat foundation would tend to distribute the unequal building loads and reduce the differential settlements.

(v) Hydrostatic uplift. When the foundation will be subjected to hydrostatic uplift due to a high groundwater table, a mat foundation could be used to resist the uplift forces.

3.4 Selection of Foundation Depth

The type of foundation, whether shallow or deep and the depth of undercutting and embankment depends on the depths to acceptable bearing strata as well as on the type of structure to be supported (USACE, 1992).

(i) Dense sands and gravels and firm to stiff clays with low potential for volume change provide the best bearing strata for foundations.

(ii) Standard penetration resistance values from the SPT and cone resistance from the CPT should be determined at a number of different lateral locations within the construction site. These tests should be performed to depths of about twice the minimum width of the proposed foundation.

(iv) Minimum depth requirements should be determined by such factors as depth of frost action, potential scour and erosion, settlement limitations, and bearing capacity.

3.5 Foundation Settlement Analysis

Suitability of soil for building foundations depend on the physical and engineering geological characteristics of soil and the performance of engineering works will depend on the correct assessment of engineering properties to determine suitability and to predict performance of soil for its intended use (Arora, 1997). From building foundation point of view safe performance of the foundation depends on bearing capacity of the foundation material and its settlement potential. Settlement of a foundation is vertical and downward movement due to a volume decrease of the soil on which it is built. Poor shear strength directly correlates to poor bearing capacity which ultimately reflects the unsafe building foundation.

According to Arora (1997), when a soil mass is subjected to a compressive force its volume decreases and the property of the soil due to which a decrease in volume occurs under compressive force is known as the compressibility of the soil. The compression of soils can occur due to one or more of the causes; (i) Compression of solid particles and water in the void, (ii) Compression and expulsion of air in the void or (iii) Expulsion of water in the void.

A study of consolidation characteristic is extremely useful for forecasting the magnitude and time of the settlement of the foundation (Arora, 1997). According to Coduto (2001), the vertical downward load is usually the greater load acting on foundations and the resulting vertical downward movement is usually the largest and most important movement we call this vertical downward movement settlement. Sometimes settlement also occurs as a result of other causes unrelated to the presence of the foundation such as consolidation due to the placement of a fill. Although foundations with zero settlement would be ideal, this is not an attainable goal. Stress and strain always go together, so the imposition of loads from the foundation always causes some settlement in the underlying soils.

Foundation settlements must be estimated with great care for buildings, bridges, towers, power plants, and similar high-cost structures. For structures such as fills, earth dams, levees, braced sheeting and retaining walls a greater margin of error in the settlements can usually be tolerated (Arora, 1997). In the vertical direction the settlement will be defined asΔH. The principal components of ΔH are particle rolling and sliding, which produce a change in the void ratio, and grain crushing, which alters the material slightly. Only a very small fraction of ΔH is due to elastic deformation of the soil grains.

- **Immediate,** or those that take place as the load is applied or within a time period of about 7 days.

Immediate settlement analyses are used for all fine-grained soils including silts and clays with a degree of saturation S ≤90 percent and for all coarse grained soils with a large coefficient of permeability [say, above 10^{-3} m/s].

The immediate settlement of the corner of a rectangular base of dimensions B' X L' on the surface of an elastic half-space can be computed from an equation from the Theory of Elasticity [e.g., Timoshenko and Goodier (1951)] as follows:

$$\Delta H = qoB' \frac{1-\mu^2}{Es} \left(Ii + \frac{1-2\mu}{1-\mu} I2\right) If \ldots\ldots \text{ eq. } 3.1$$

$$\Delta H = qoB' \frac{1-\mu^2}{Es} mIsIf \qquad \ldots\ldots \text{ eq. } 3.2$$

Where q_o = intensity of contact pressure in units of Es

B'= least lateral dimension of contributing base area in units of ΔH

I_i = influence factors, which depend on L'/B' thickness of stratum H, Poisson's ratioμ, and base embedment depth D

m = number of corners contributing to settlement ΔH. At the footing center m = 4; at a side m= 2, and at a corner m = 1. Not all the rectangles have to have the same L'/B' ratio, but for any footing, use a constant depth H.

I_f= influence factor find using figure Bowles, 1997.

- **Consolidation,** or those that are time-dependent and take months to years to develop.

The Leaning Tower of Pisa in Italy has been undergoing consolidation settlement for over 700 years. The lean is caused by the consolidation settlement being greater on one side. This, however, is an extreme case with the principal settlements for most projects occurring in 3 to 10 years.

Consolidation settlement analyses are used for all saturated, or nearly saturated, fine grained soils where the consolidation. For these soils we want estimates of both settlement ΔH and how long a time it will take for most of the settlement to occur (Bowles, 1997).

- Settlements may be estimated from the SPT N- value in granular soils.
- The settlement estimate is based on the size and type of foundation.

According Arora (2004), the allowable maximum settlement depends upon the type of soil, the type of foundation and the structural framing system. The maximum settlement ranging from 20mm to 300mm is generally permitted for various structures. Theoretically, no damage is done to superstructure if the soil settles uniformly. However, settlements exceeding 150mm may cause trouble in utilities such as water pipe lines, sewers, telephone lines and also in access from streets.

IS: 1904 (1966), permits a maximum settlement of 40mm for isolated foundation on sand and 65mm for those on clay.

As per Ethiopian building code (EBCS) allowable settlement for isolated foundation on sand is 25mm whereas for clay soils it is 50mm. The allowable settlement is higher for clays because progressive settlements on clayey soils permit better strain adjustments in the structural members. The maximum permissible settlement for raft foundations on sand is 40mm to 65mm and that on clay 65 to 100mm. The permissible settlements for rafts are more than those for isolated foundation because the raft bridges over soft patches of the soil and the differential settlements are reduced (Arora, 2004).

3.6 Bearing capacity

Bearing capacity is the ability of soil or rock to-safely carries the load placed on the soil from any engineered structure without undergoing a shear failure. Applying a bearing pressure, which is safe with respect to failure, does not ensure that settlement of the foundation will be within acceptable limits. Therefore, settlement analysis should also be performing. The general accepted method of bearing capacity analysis is to assume that the soil below the foundation along a critical plane of failure (slip path) is on the verge of failure and to calculate the bearing pressure applied by the foundation required to cause this failure condition. This is the ultimate bearing capacity qu (Bowles., 1996).Experimental investigations have indicated that when a footing fails due to insufficient bearing capacity, distinct failure patterns are developed, depending upon type of failure mechanism. Failure is accompanied by appearance of failure surfaces and by building of sheared mass of soil (Vesic,1963).

The physical and engineering properties of soils of principal interest for the analysis and design of foundation elements primarily include the following:

- Strength parameters :- angle of internal friction, (Φ), soil cohesion (C)
- Stress-strain modulus (or modulus of elasticity), shear modulus, and Poisson's ration, angle of internal friction, (Φ), soil cohesion (C)
- Compressibility indexes for amount and rate of settlement
- Gravimetric-volumetric data (Unit weight, specific gravity, void ratio, or porosity, water content, plastic limit, liquid limit).

Some or all of the above laboratory tests are routinely performed as part of the foundation design process (Bowles, 1997). While performing the above tests in the laboratory, there must be great care and supervision, for there exist all or some of the below mentioned most common problems for all laboratory tests and it is impossible to eliminate all but can be minimize. These are the following;

- Recovery of good quality samples. It is not possible to recover samples with zero disturbances, but if the disturbance is a minimum - a relative term – the sample quality may be adequate for the project.
- Necessity of extrapolating the results from the laboratory tests on a few small samples, which may involve a small volume to the site, which involves several thousands of cubic meters.
- Laboratory equipment limitations
- Ability and motivation of the laboratory personnel.

BEARING CAPACITY EQUATIONS

In general Bearing Capacity estimation equations was developed by different parameters those are;

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

$$q_{ult} = cN_cS_c + qN_q + 0.5\lambda BN_ɣS_ɣ$$

$$N_q = \frac{a2}{acos2(45+ \quad /2)}$$

$$A = e^{(0.75Л- Φ/2)tanΦ}$$

$$N_c = (N_q - 1)cotΦ$$

$$N_ɣ = \frac{tan}{2}\left(\frac{Kpy}{cos2} - 1\right)$$

Where, C = cohesion

q_{ult} = ultimate bearing capacity pressure

B= foundation width, ft

Nc, Nγ, Nq = dimensionless bearing capacity factors for cohesion c, soil weight in the failure wedge, and surcharge q terms

$S_{C,}\ S\gamma$ = shape factor

γ = unit weight of soil

Φ = angle of shear resistance

K_p = coefficient of passive earth pressure.

q = Surcharge at the ground level

Meyerhof (1963) also derived an equation for bearing capacity with additional parameters which was not included in Terzagh estimation. Shape and depth factors are primarily incorporated and bearing capacity constant, N_γ was modified.

$$q_{ult} = cN_cS_cd_c + qN_qS_qd_q + 0.5\lambda B' N_\gamma S_\gamma d_\gamma$$

$$q_{ult} = cN_cS_ci_c + qN_qS_qi_q + 0.5\lambda B' N_\gamma S_\gamma i_\gamma$$

$$N_q = e^{Л\tan\Phi}(45 + \Phi/2)$$

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

$$N_\gamma = (N_q - 1)\tan(1.4\Phi)$$

Where, C = cohesion

q_{ult} = ultimate bearing capacity pressure

B= foundation width, ft

Nc, Nγ, Nq = dimensionless bearing capacity factors for cohesion c, soil weight in the failure wedge, and surcharge q terms

γ = unit weight of soil

Φ = angle of shear resistance

q = Surcharge at the ground level

Sc, Sq and Sγ = shape factors

dc, dq and dγ = depth factors

ic, iq and iγ = inclination factor

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

$$Q_{ult} = cN_cS_cd_ci_cg_cb_c + qN_qS_qd_qi_qg_qb_q + 0.5\lambda B' N_\gamma S_\gamma d_\gamma i_\gamma g_\gamma b_\gamma$$

$$N_q = e^{Л\tan\Phi}\tan^2(45 + \Phi/2)$$

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

$$N_\gamma = 1.5(N_q - 1)\tan\Phi$$

Where,C = cohesion

q_{ult} = ultimate bearing capacity pressure

B= foundation width, ft

Nc, Nγ, Nq = dimensionless bearing capacity factors for cohesion c, soil weight in the failure wedge, and surcharge q terms

γ = unit weight of soil

Φ = angle of shear resistance

q = Surcharge at the ground level

Sc, Sq and Sγ = shape factors

dc, dq and dγ = depth factors and ic, iq and iγ = inclination factor.

The above estimation was commonly used for cohesive soils where 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, (1997) derived the following estimation method.

$$q_{all} = \frac{N}{F2} * (1 + {F3}/{B}) * K_d \qquad B > F4$$

q_{all} = allowable bearing pressure for settlement limited to 25mm, F = factor of safety

B = width of foundation

D = depth of foundation

Bowels, (1997) also recommend estimating bearing capacity for mat foundation using SPT blow counts.

$$Q_{ult} = cN_cS_cd_c + qN_qS_qd_q + 0.5\lambda BN\gamma S\gamma d\gamma$$

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

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 important to the bearing capacity of sand; β, Ɣ, Φ. 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 B., 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 the ground conditions and the importance of the structure.

3.7 Estimation of Ultimate Bearing Capacity (qu) of soil by using Standard Penetration Test (SPT) N-value

According to Bowles (1997), the ultimate bearing capacity of footings on soils can be determined with the help of standard penetration resistance value, N. Value of N are determined at a number of selected locations of boreholes, at vertical intervals of 750 mm or at change of strata, whichever occurs earlier. The method has been standardized as ASTN D 1586 since 1958 with periodic revisions to date. The SPT is widely used to obtain the bearing capacity of soils directly. One of the earliest published relationships was that of Terzaghi and Peck (1967). This has been widely used, but an accumulation of field observations has shown these curves to be overly conservative.

According to Bowels (1997), correlations between SPT N value and soil or weak rock properties are wholly empirical. It is therefore important that user of the SPT and the data it produces has a good appreciation of those factors controlling the test, which are: (i) Variations in the test apparatus, (ii) The disturbance created by boring the hole; and (iii) The soil into which it is driven

In general, the above approaches have both merits and demerits according to the stress type, shape of foundation and ground condition they are used.

3.8 Previous works

There are no much studies and significant previous practical experience of building foundation characterization and analysis in Ethiopian. Some related studies to the present work, however, were conducted in the past for academic purposes. The work of Lamesgin Mesele (2014), Merga Negesa(2014), Weynshet Tadesse (2015) and Hanna Tibebu (2008) are a related work that is referred for this research work.

Lamesgin Mesele (2014) conducted characterization and assessment of the bearing capacity of most common foundation materials in Addis Ababa city. He has undertaken a comparative assessment of the bearing capacity of foundation material in Addis Ababa, through the development of a geotechnical data base from historical borehole logs and laboratory test results data. In addition in his research work, he has prepared a bedrock topography and soil thickness map of Addis Ababa. He made assessment of bearing capacity of foundation materials in Addis Ababa through different computational methods and made comparisons of results on bearing capacities within each soil type and among different soil types. He concluded that the N_c and N_q terms are the same for all authors except slight variation in Terzaghi. However, the N_γ factor has the widest range of values there by resulting in considerable variations in the bearing capacity, especially for greater angles of shearing resistance. These conclusions can be drawn from this work;

- Among the bearing capacity factors, the N_γ term brings very big differences in bearing capacity values, especially for larger footings (mats).
- For footings founded on red clays in Addis Ababa, Terzaghi, EBCS and SPT methods gave reasonable bearing capacity values. On the other hand, for the mat foundations, Terzaghi (1943), Meyehof (1951), Hansen (1970) and EBCS-7 (1995) methods gave close results. For expansive soils, the computed bearing capacity values for shallow foundations are found to be nearly identical. However, the SPT method gives relatively high bearing capacity values. For a saturated tuff, the resulted bearing capacity values for all analytical methods are almost similar. Unlike the analytical methods, the SPT method resulted very high bearing capacity values for a saturated tuff. With regard to paleosols, roughly similar range of bearing capacity value been obtained using Terzaghi (1943) and EBCS-7 (1995) methods. On the other hand, relatively higher values have been obtained using Meyerhof, Hansen and Vesic approaches. For larger footings (mats) founded on paleosols, the Vesic method gave high bearing capacity values, whereas the rest of the analytical methods gave roughly similar values. Unlike the analytical methods, bearing capacity values obtained using SPT method is found to be high for both footings and mats.

Merga Negesa (2014) conducted characterization and foundation analysis for selected sites under Addis Ababa housing project. He has undertaken a characterization and foundation analysis of three sites namely Asco site, Megenagna site and Imperial site which are under preparation for apartment buildings. In his research work 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 his work the following conclusions are drawn;

- 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.
- 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.

3.9 Evolution of Methodology for the present study

Based on systematic literature review, general background knowledge about characterization of building foundation and analysis techniques was developed. With a thorough conceptual framework a comprehensive methodology was developed.

Historical geotechnical investigation reports have been extensively reviewed from both governmental and non-governmental organizations so as to understand the most common foundation material types and the foundation analysis practiced in Addis Ababa. Geotechnical data from these investigations and ongoing projects, and subsequent interpretations, provided the necessary geotechnical framework for the present study. During the present study in order to achieve the objective, field observation and selected laboratory tests were conducted. Representative soil samples were taken from test pit and soil index properties were evaluated to understand the basic soil types and to have some comparison with the soil types procured as secondary data. Based on the primary data and secondary data characterization of the building foundation was made for bearing capacity and settlement potential. Finally, based on the analysis of the results, interpretations were made and accordingly conclusions and recommendations are forwarded.

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CHAPTER FOUR METHODS AND TECHNIQUES

4.0 Preamble

In order to achieve the objective of the study various data collection methods was exercised. The two data collection sources which were followed are secondary data and primary data collection (the primary data are investigated with laboratory analysis).

The general methodology followed in the present research involved:

- Review of literatures, manuals and specifications
- Field work
- Secondary data collected
- Primary data collected
- Laboratory tests on collected samples
- Bearing capacity assessment
- Settlement potential estimation
- Data analysis and Interpretation of results

4.1 Literature review

Literature review basically involved the retrieval of data on physiography and climatic conditions, soil types, the regional and local geology, ground water condition and geo-hazards of the study area. Besides, literature on geotechnical investigation techniques as applied to foundation studies in the present study area was collected from various sources and later it was thoroughly reviewed. For this, different textbooks, journals, various official documents and reports on geotechnical investigation for building site characterization were reviewed.

Other literatures that deal about soil formation, soil classification, bearing capacity of foundation classes, foundation type selection, settlement problems and foundation, and recommendations for construction were also reviewed. For these purpose different maps, previously done foundation investigations, building design manuals, reference books and journals from different sources have been used for better understanding of aspects related to the current research topic.

4.2 Field work

A systematic field work was carried out to assess information on the land use and land cover, physiographic and geological setup of the study area. Besides, necessary data was collected for the geotechnical characterization of the soil/rocks in the study area. The field work has been aided by land use and land cover map, topographical map, geological map to get various inputs for the description on the land use and land cover, the topography, local geology and soils conditions. The soil/rock types of the study area have been identified visually.

The identification was made based on their color, and size and distribution of their constituents. Test pits have been dug for determination of depth of soil, description of soil profile, soil sampling and assessment of groundwater conditions.

4.3 Secondary data collection

Secondary data was collected from governmental and non-governmental organizations to review different pre-existing geological, engineering geological reports, maps, foundation types and foundation analysis.

The review was made to understand the investigation methods, analysis and final interpretations for various building foundation sites. Besides, borehole data, soil properties and classification data was also collected which was later utilized to estimate the bearing capacity and settlement potential of different foundation soils. In this regard, the secondary data was procurement and processed from; bore hole logging, in-situ test and laboratory test reports on different soils. Besides, processed secondary data on bearing capacity and settlement potential of foundation soils in the study area was also reviewed and utilized to meet out the objectives of the present study.

The characterization of the soil was made through visual observation for soil texture, color and through laboratory testing on representative soil samples from test pits. Laboratory test include; grain size distribution, Atterberg limit, soil classification, specific gravity, moisture content etc.

For bearing capacity estimation the basic soil properties were used these include cohesion (C), shearing resistance (Ø) and unit and weight of soil (γ). These properties were adopted mainly through the test reports from secondary sources. Besides in-situ SPT data was also used from as secondary data.

For settlement potential estimation consolidation test was conducted and secondary data was also utilized. The data used for the estimation of bearing capacity and settlement analysis was obtained from borehole log and laboratory tests reports for more than 300 projects with 0 to 30 m depth information from the present study.

4.4 Primary data collection

Primary data for the present study was collected through representative test pits. Further, logging of test pits was done to describe soil profile through physical examination based on color, soil texture, grain size distribution etc. Further, from test pits appropriate soil samples were also collected for laboratory testing. In total 7 test pits were made in the study area. The selection of representative test pits was made after careful study of the soil map of the study area, reconnaissance field study and review of secondary data. The depth of these test pits varies from 0 to 3 m. For each of these test pits GPS coordinates were recorded. Systematic logging of test pits was made. Within each test pits depth of each soil unit was measured, soil color, texture, degree of saturation was also recorded. In total 7 soil samples, disturbed and undisturbed, were collected from the test pits. Later, these samples were tested in the laboratory to define the index and engineering properties of the soils in the study area.

During the field investigation following systematic activities were carried out;

(i) Visual description and identification of soil based on ASTM D 2488
(ii) Vertical section logging of test pits at different sites, visual description and characterization of soil and rock strata
(iii) Representative soil samples were collected at different depth from different soil horizons.
(iv) Later, laboratory tests were conducted on representative soil samples to evaluate index properties of soil for which different tests of soil samples were conducted.

4.5 Laboratory Testing

Index properties are used to classify soils, to group soils into major strata, and to obtain estimates of structural properties. The following laboratory tests were conducted for the present study to evaluate the soil properties of the representative samples collected from the study area. In total, 14 disturbed and 3 undisturbed soil samples were tested to define the index properties of the soils in the study area. Also, engineering properties were determined for 7 soil samples.

A – Test pit at Name the place B – Teat Test pit at Name the place

Plate 4.1 Test pits made during present study

The index properties tests that were carried out include; Natural moisture content, Atterberg limit, Particle size distribution, Free swell and Unit weight. Test for engineering properties included; Direct shear, unconfined compressive strength and Consolidation test. These test were carried out at Addis Ababa University, school of earth science, engineering geology laboratory, transport construction design share company laboratory and the test reports are presented as chapter 5. Thus, based on the test results comprehensive evaluation of the foundation condition in the present study area was made.

4.5.1 Natural Moisture Content

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

W (%) = Mw/Ms (100%)   …..eq. 4.1

The difference in weight between the wet and dry sample is the mass of water, Mw is the weight of the dry sample and Ms is the mass of the soil. Note that the eq. 4.1 defining water content differs from standard equations for determining the percentage of constituent materials. A specimen containing 25 g of water and 25 g of dry soil has a moisture content of 100%, but water comprises only 50% of the sample by weight (Ranjan, 1993).

The soil was dried at a constant temperature of 110 ^{0}C using a conventional oven for about 15 hours (BS 1377: part 2: 1990). 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).

4.5.2 Atterberg Limit

The consistency of a fine-grained soil is the physical state in which it exists; it relates largely to the water content. Consistency denotes the degree of firmness of the soil that is indicated by tests in the field as soft, firm, stiff or hard (Abramson et al., 1996). As the water content of a fine-grained soil is increased gradually from 0%, it goes through different consistencies, namely brittle solid, semi-solid, plastic and liquid states as shown in Fig. 4.1. Atterberg limits are the borderline water contents between two such states (Mills and Cameron, 2002).

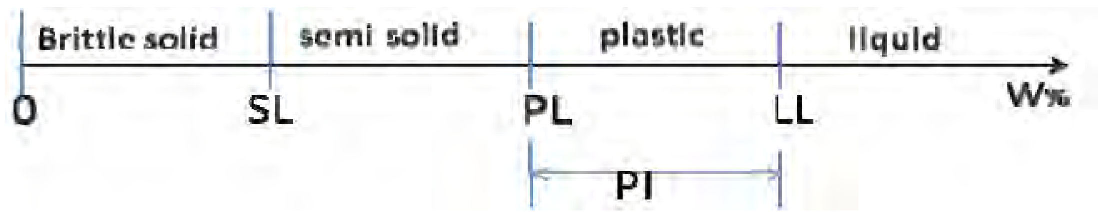

Fig. 4.1 Consistency limits (Redraw this figure)

Plastic Limit (PL): plastic limit is the lowest water content at which the soil changes from a fluid to plastic state. According to description of plasticity of fine soils in terms of range of plasticity index given by IAEG(1983),the clay, sandy clay/silty and clayey/silty sand soils are moderately to extremely plastic and the silty soils are moderately to highly plastic type.

Liquid Limit (LL): liquid limit is the lowest water content at which the fine-grained soil behaves like a viscous mud and flowing under its own weight. It is the transition water content between plastic and liquid states. Generally, soils having high values of liquid and plastic limits are considered as poor foundation materials.

Plasticity Index (PI): plasticity Index is a measure of the range of water content over which soil is in a plastic state. Soils with high PI tend to be clay, those with a lower PI tend to be silt, and those with a PI of 0 tend to have little or no silt or clays (Holtz, 1981).

PI =LL –PL

Table 4.1:- Soil expansivity predicted by plasticity index (IS 1498)

Degree of expansion	Plasticity index (%)
Low	< 12
Medium	12 – 23
High	23 – 32
Very high	>32

Plastic index is important in classifying the fine-grained soils. The more plastic a soil means the more compressible, higher shrinkage-swell potential and the lower is its permeability will be (Abramsonetal.,1996).

The larger the plasticity index, the greater will be the engineering problems associated with using the soil as an engineering material, such as foundation support for residential building and road sub-grades (Bowles,1992).

In the present study Liquid limit and plastic limits were determined for soil samples collected from seven test pits within the study area. These limits were determined employing AASHTO T-89/90 classification methods used for identified fine grain soil. In liquid limit Casagrand liquid limit apparatus was used to determine the amount of clay content and in plastic limit simple rolling procedure to remove moisture content of soil was utilized.

4.5.3 Grain Size and Grain Size Distribution

According to Das (1994), soils are usually comprised of more than one particle size and the principal particle sizes of soils are; gravel, sand, silt and clay therefore all soils are made up of one or all of these distinct components in combination and each component has a definite grain size range characteristic reaction in the soil mass.

The grain size distribution of a coarse-grained soil is generally determined through sieve analyses, where the soil sample is passed through a stack of sieves and the percentages passing different sizes of sieves are noted. The grain size distribution of the fines are determined through hydrometer analysis, where the fines are mixed with distilled water to make 1000ml of suspension and a hydrometer is used to measure the density of the soil-water suspension at different times and hydrometer analysis is effective for soil fractions down to about 0.5μm (Das, 1994).

In the present study the particle size distribution analyses of the disturbed soil samples have been carried out by sieve analysis for materials coarser than 0.075mm and by hydrometer test for fine material (AASHTO, 2008). Mostly soils containing both coarse and fine grains therefore it is necessary to do sieve and hydrometer analysis to obtain the complete grain size distributions data (Das, 2011).

4.5.4 Free swell

In the present study free swell index soil test analysis of disturbed soil sample from seven test pits in the study area was determined by IS: 2720 (part 40)- 1985. The test was used to determined expansivity of soil in the study area. The result of the test are present later in chapter 5.

Table 4.3 Soil expasivity predicted by free swell index (IS 1498)

Degree of expansion	Free swell index (%)
Low	< 50
Medium	50 – 100
High	100 – 200
Very high	> 200

4.5.5 Unit Weight

In the present study unit weight of the soil was determined by ASTM D 2937-00 – Standard Test for Density of Soil in Place by the Drive Cylinder Method. This test was performed to determine the in-place density of undisturbed soil obtained by pushing or drilling a thin-walled cylinder.

The bulk density is the ratio of mass of moist soil to the volume of the soil sample, and the dry density is the ratio of the mass of the dry soil to the volume the soil sample.

4.5.6 Direct shear test

In the present study direct shear soil test was carried out according to AASTO T-236. The test is performed by deforming a specimen at a controlled rate on a single shear plane determine by configuration of the apparatus. This test is used to determined shear strength parameters, angle of internal friction and cohesion of material. These shear strength parameters of soil were used to compute the bearing capacity of the foundation soils.

4.5.7 Consolidation

In the present study consolidation test was conducted according to ASTM D 2423 standard. This test is useful 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 pre-consolidation pressure (or maximum past pressure) of the soil.

4.6 Estimation of bearing capacity of soil

For the present study ultimate bearing capacity (Qult) and allowable bearing capacity (Qall) were computed by utilizing MS Excel computation sheet developed by Raghuvanshi (2017). This computational sheet facilitate to compute bearing capacity proposed by Terzaghi (1943), Hensen (1957, 1970), Vesic (1973), Meyerhof (1963) and Ethiopian Building Standard code (EBSC-7). The computational sheet facilitates to do analysis for variable footing dimensions at different depths.

The basic input data required for computation of bearing capacity are foundation soil properties; cohesion (C), shearing resistance (Ø), unit weight of soil (γ), footing dimensions, depth of footing, depth to ground water table and mode of application of load. The bearing capacity factors, depth, shape and inclination factors are computed automatically. In order to compute allowable bearing capacity (Qall) a factor of safety of 3 was considered for the present study.

4.7 Settlement potential estimation

Settlement estimation is done for this specific site 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 primary consolidation settlement. Therefore, the calculation of consolidation settlement is presented in chapter 5. Consolidation test was conducted on undisturbed samples collected from the study area.

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CHAPTER FIVE DATA COLLECTION AND ANALYSIS

5. Preamble

This chapter deals with the collection of primary and secondary data for foundation analysis. Besides, it also presents characterization of soils for building foundation in the study area. For soil characterization and foundation analysis both secondary and primary data was utilized. Borehole logs, soil laboratory tests and SPT data were used as secondary data. Besides, representative primary data was collected through test pits and systematic laboratory tests were also conducted.

5.1 Data collection

In order to characterize the soils from the study area both secondary and primary data was utilized. The soil properties that were used from the secondary and the primary sources and the total number of test results or samples used are presented in Table 5.1. The summary of laboratory test results is presented in Annexure 1. Fig. 5.1 shows the location of boreholes that were used for the present study.

5.1.1 Secondary data

In order to achieve the objectives of the present study it was necessary to source a large quantity of geotechnical records mainly through the boreholes present in the study area. In this regard geotechnical data from various sources was procured. These include Saba Engineering PLC, Building construction design and share company PLC, Adiss Geosystem PLC, Arcon Building PLC and Addis Ababa saving house development enterprise. The data used for the estimation of bearing capacity and settlement analysis was obtained from borehole logs and laboratory tests reports for more than 300 projects.

5.1.2 Primary data

For the present study 7 test pits were made at different locations in the study area (Fig. 5.1). These test pits were made to collect representative samples for the determination of soil properties and to have general comparison of test results with that of the data obtained from the secondary sources.

Table 5.1 Summary of laboratory test result used in the present study

S.No	Type Of Laboratory Test	Secondary data	Primary data	Total	Test Designation
1	Grain Size analysis	539	7	546	AASHTO T88
2	Atterberg Limit (LL and PL)	527	7	534	AASHTO T89 & 90
3	Free Swell	231	7	238	AASHTO T256
4	Specific gravity	379	7	386	AASHTO T100
5	Natural moisture content	110	7	117	AASHTO T265
6	Unconfined Compressive Strength(UCS)	71	3	74	ASTM D2938
7	Bulk Unit weight	82	3	85	ASTM D2937
8	Consolidation	41	3	44	
9	Standard penetration test	1150	-	1150	ASTM D 1586 –99 & BS 5930: 1981

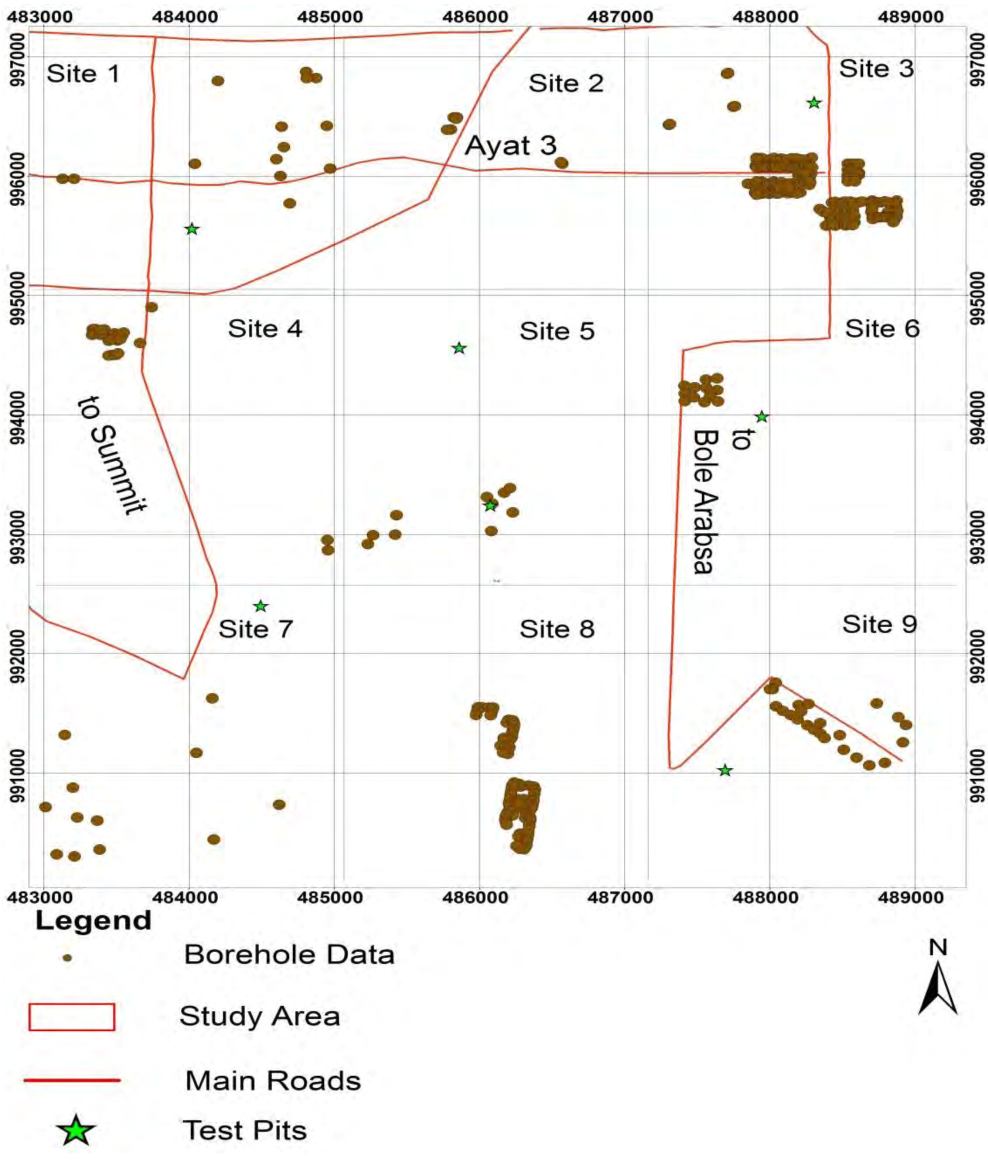


Fig.5.1 Location of boreholes and test pits that were used for the present study

Each of these test pits was systematically logged and description of various soils based on visual observations was made. Besides, representative disturbed and undisturbed samples were also collected for the determination of various properties through laboratory testing. Table 5.2 presents the location of test pits that were used during the present study.

Table 5.2 Location of test pits used to generate primary data during the present study

Test Pit No	GPS location (UTM)			Name of Place	Depth (m)	Description
	Easting	Northing	Elevation (m)			
1	487694	991024	2297	Close to CMC square	3m	Light grey to dark (fine grain)
2	486076	993243	2301	Ayat Condominium	3m	Dark to black color
3	484020	995563	2367	Ayat 5	3m	Light gray complexly weathered, Decompose, tuff
4	485861	994564	2307	Close to quarry site	3m	Light dark to light brown
5	484493	992400	2306	Close to summit	3m	Light gray complexly weathered, Decompose, tuff
6	488306	996620	2216	Around Moha Soft drink factory	3m	Light grey to dark (fine grain)
7	487946	993990	2306	Close to bole Arabsa	3m	Dark gray, medium stiff, high plastic silty clay

The soil samples collected from the test pits were tested in the laboratory of Construction design Share company PLC, Addis Ababa for Grain size distribution, Atterberg limits, Free swell, specific gravity and natural moisture content. To determine these properties 7 soil samples were utilized (Table 5.3). Besides, unconfined compressive Strength (UCS), bulk unit weight and consolidation tests were also conducted on 3 representative samples (Table 5.4).

Table 5.3 Summary of Laboratory test results (primary data) for index properties and classification

Test Pit No	Sample No.	depth	Atterberg Limit (AASHTO T89 & 90)		Soil Classification (AASHTO M-145)	Wet Sieve Analysis (AASHTO T-88), Sieve Size (mm) Vs % passing						Free Swell %	NMC, % AASHTO T-265
			LL %	PI %		19	12.5	4.75	2	0.425	0.075		
1	S1	1.5m	73	25	A-7-5(18)			100	98	89	79	120	43
2	S2	1.5m	55	17	A-7-5(14)			100	99	96	84	120	41
3	S3	1.5m	68	27	A-7-5(19)			100	96	86	80	130	38
4	S4	1.5m	62	19	A-7-5(15)			100	94	82	72	130	40
5	S5	1.5m	52	16	A-7-5(16)			100	97	91	82	110	44
6	S6	1.5m	59	21	A-7-5(14)		100	84	81	74	67	110	45
7	S7	1.5m	95	48	A-7-5(20)				100	89	86	180	39

A perusal of Table 5.3 clearly shows that all the soils fall into A-7-5 soil group of AASHTO M-145. The liquid limit (LL) varies from 52 to 95 % and the plasticity index varies from 16 to 48 %. Further, the free swell of soils fall in the range 110 to 180 % and natural moisture content (NMC) varies from 39 to 45 %.

Further, results (Table 5.4) show that UCS for soil samples varies from 36.4 to 50 kPa. The bulk density for soil samples fall in a range of 15 to 18 kN/m^3. The consolidation test result shows that the compression index (C_C) and volume change (m_v) varies from 0.193to 0.249 and 0.079 to 0.231, respectively.

Table 5.4 Summary of Laboratory test results on UCS, Bulk unit weight and consolidation

Test Pit No	Sample No.	Depth of sample (m)	UCS (kPa)	Bulk Unit Weight (kN/m^3)	Consolidation	
					Cc	Mv
1	S1	1.5	50	18	0.206	0.113
2	S3	1.5	36.4	15	0.249	0.231
7	S9	1.5	43.4	16	0.193	0.079

5.2 Characterization of foundation material

For the purpose of characterization and foundation analysis the study area was divided into 9 sites; Site 1 to Site 9, as shown in Fig. 5.1. This was mainly done so that the data can be well managed and systematic analysis can be made. The foundation analysis results are also presented site wise. The location details of these Sites and distribution of boreholes and test pits (made during present study) is presented in Table 5.5.

In the present study area two prominent type of soils and two different types of rock units are present. The soil types are; light to dark grey silty clay/ clayey silt soil and soft dark, silty clay (Black Cotton) soil. Among the rocks; weak tuff layer and weak to medium strong ignimbrite rocks are present. Following section presents a detailed description on these soils and rock types.

5.2.1 Light to dark grey soil

From the field observations through test pits, laboratory tests and geotechnical bore hole log reports light to dark grey soils are mainly dominated in Site 1, 4 and 7 (Fig. 5.1). This soil was encountered underlying by pyroclastic ash and tuff deposits and reaches to the maximum drilled depth. The thickness of this soil layer varies from 2 to 10.7 m in the study area as observed from the logs of various boreholes. The soil from this layer can be characterize as light to dark grey, stiff to very stiff and highly plastic silty CLAY/clayey SILT soil.

Table 5.5 Location details of Sites and distribution of boreholes and test pits in the study area

Site No	GPS Location (UTM)			Name of important Place	Boreholes			Test Pit
	Easting	Northing	Elevation (m)		Total No of Borehole Present	Boreholes used for the present study	Depth of Boreholes (m)	
Site 1	483000-485000	997000-995000	2330	CMC square	15	11	0 – 15m	1
Site 2	485000-487000	995000-997000	2363	Around Ayat real state	11	11	0 – 15m	-
Site 3	487000-489307	995000-997000	2400	Ayat condominium	98	16	0 – 30m	1
Site 4	483000-485000	992600-995000	2300	Summit Square	26	10	0 – 15m	-
Site 5	485000-487000	992600-995000	2370	Bole Ayat 3	10	4	0 – 15m	2
Site 6	487000-489307	992600-995000	2340	Bole Ayat 5	23	8	0 – 30m	1
Site 7	483000-485000	990090-992573	2250	Moha soft drink factory	12	4	0 – 15m	1
Site 8	485000-487000	992573-990090	2270	Close to beshale river	69	12	0 – 15m	-
Site 9	487000-489307	992573-990090	2290	Close bole arabsa condominium	24	7	0 – 15m	1

The summary of laboratory test results on the representative soil samples, both from secondary and primary data on light to dark grey soils a represented in Table 5.6. A perusal of results (Table 5.6) clearly shows that 99% of the material passed through No 200 sieve. Further, the liquid limit and plasticity index for these soils varies from of 87 to 113% and 47 to 80%, respectively.

The free swell varies from 90 to 180 %, whereas, natural moisture content varies from 30 to 45%. In general, the UCS for these soils varies from 85.8kPa to 200.1kPa. The coefficient of compression (Cc) for these soils varies from 0.193-0.256.

The shear strength parameters C and φ varies from 11 – 30 kPa and 11 – 21^{o}, respectively. The results in general show that these soils are very high plastic and are fine grained expansive in nature.

As per Unified Soil Classification System (USCS) these soils may be placed under MH and CH types of soils (Fig. 5.2).Further, in situ Standard Penetration Test (SPT) results from the secondary data on this soil indicate that the N value varies from 6 to 25 blows per 300 mm of the test section. Based on the N value the soils falling within this group are represented as medium stiff to very stiff soils.

Table -5.6 Summary of index and engineering properties of light to dark grey soils

Index and Engineering proper Index and Engineering property		Range	Mean	Coefficient of variation (CV)%
Grain Size Distribution	Gravel	0.1 -29.4	14.75	21
	Sand	0.3 - 9.9	5.1	28
	Silt	6 – 97.8	51.9	34
	Clay	1 – 82	41.5	42
Liquid limit (LL)		87 – 113	100	57
Plastic Index (PI)		47 – 80	64	59
Soil Classification (USCS)		MH and CH	-	-
Free swell		90 – 180	135	64
Unit Weight (KN/m^3)		15 – 21	18	55.6
Moisture content		30 – 45	38	60.8
Void ratio (e)		0.0473 -0.0845	0.0659	67.6
UCS (KPa)		74 – 200	137	69.9
Swelling pressure (KPa)		105 – 130	118	64
Coefficient of compression (Cc)		0.193-0.256	0.2245	10
Cohesion(C) (KPa)		11 – 30	21	53
Angle of internal friction (Ø)		11 – 21	16	54.6
SPT- N		4 – 25	15	8

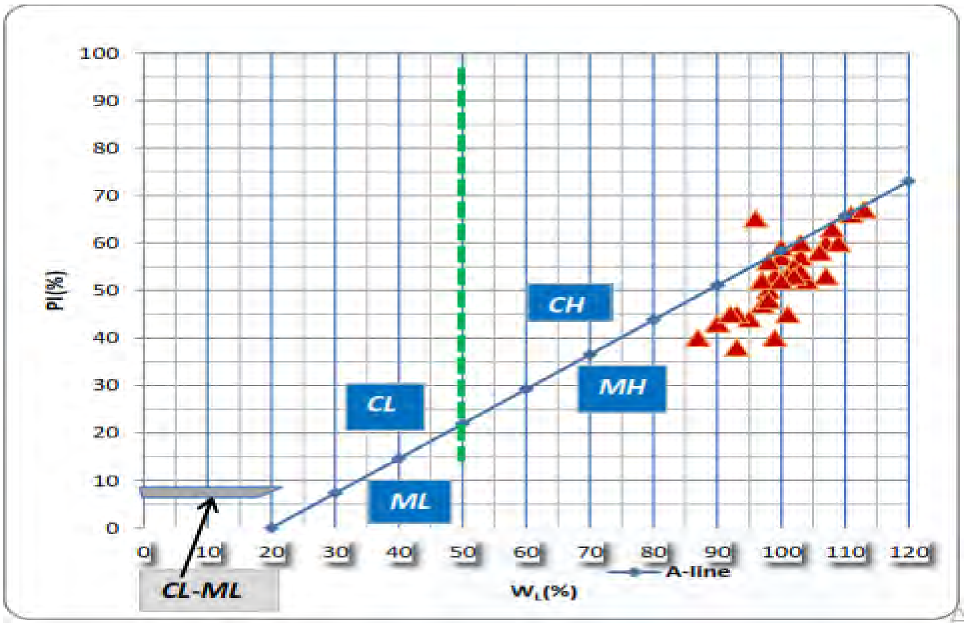


Fig 5.2 Casagrande's Plasticity Chart for soils present at Site 1, 4 and 7

5.2.2 Soft dark, clay (Black Cotton soil)

From the field observations through test pits, laboratory tests and geotechnical bore hole log reports soft dark clay (black cotton) soils are mainly dominated in Site 2, 3,5,6,7,8 and 9 (Fig. 5.1). This soil mostly covers the top most part in the study area and is composed of loose, reddish brown scoracious material. The thickness of this soil layer varies from 0.3 to 3m in the study area, as observed from the logs of various boreholes and within the test pits made during the present study. The soil from this layer can be characterize as black, soft consistency, medium stiff, high plastic silty CLAY (Black cotton) soils.

The summary of laboratory test results on the representative soil samples, both from secondary and primary data on soft dark clay (black cotton) soils are presented in Table 5.7. A perusal of results (Table 5.7) clearly shows that 98% of the material passed through No 200 sieve. The liquid limit and plasticity index for these soils varies from of 37 to 89 % and 7 to 47 %, respectively.

The free swell varies from 90 to 190 % which shows high to very high expansion potential. The natural moisture content varies from 26 to 61%. In general, the UCS for these soils varies from 34 – 207.8 kPa. The coefficient of compression (Cc) for these soils varies from 0.3488 to 0.677. The shear strength parameters C and φ varies from 10 to 80 kPa and 9 – 21^{o}, respectively. The results in general show that these soils are very high plastic and are fine grained expansive in nature.

As per Unified Soil Classification System (USCS) these soils may be placed mainly under ML, MH and CL types of soils (Fig. 5.3).Further, in situ Standard Penetration Test (SPT) results from the secondary data on this soil indicate that the N value varies from 4 to 17 blows per 300 mm of the test section. Based on the N value the soils falling within this group are represented as soft to stiff consistency with soils with very loose to medium dense.

Table -5.7 Summary of index and engineering properties of soft dark, clay (black cotton)

Index and Engineering properties		Range	Mean	Coefficient of variation (Cv)%
Grain Size Distribution	Gravel	-	-	-
	Sand	1.9 – 48.7	25	37
	Silt	14.6 – 85.3	50	49
	Clay	1.2 – 84.1	42	56
Liquid limit (LL)		37 – 89	63	55
Plastic Index (PI)		7 – 47	27	59
Soil Classification (USCS)		MH and ML	-	-
Unit Weight (KN/m^3)		16 – 22	19	53
Free swell		90 – 190	140	62
Moisture content		26 – 61	44	57
Void ratio (e)		0.7270 –1.4002	1.0636	63.8
UCS (KPa)		34 – 207.8	121	66.38
Swelling pressure (KPa)		100 – 140	120	66
Coefficient of compression (Cc)		0.3488-0.677	0.5129	7
Cohesion(C) (KPa)		10 – 80	45	49
Angle of internal friction (Ø)		9 – 21	15	52
SPT- N		4 – 17	11	7.6

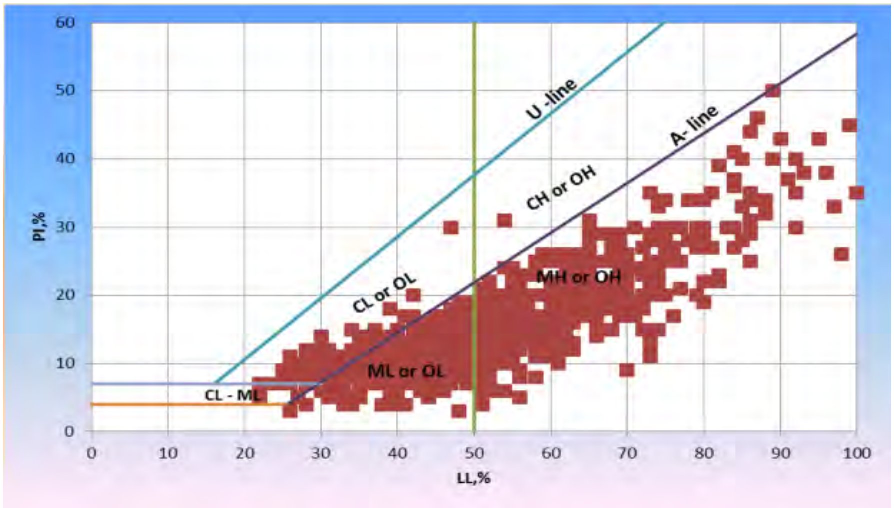


Fig. 5.3 Plasticity chart (USCS, AASTO) showing soils from Site 8 and 9

5.2.3 Weak TUFF layer

This rock type is encountered below the plastic silty SAND/sandy SILT and continues to the maximum drilled depth in some of the boreholes. The depth of weak tuff layer from surface in general varies from 0.5 to 6m, as observed in borehole logs from the study area. The rock in general is very weak, dark grey, moderately to highly weathered, fractured, friable welded or medium dense to very dense tuff. The rock quality designation (RQD) values as observed from various borehole log reports shows that it varies from 0 to 60% and in majority of cases it is less than 25 %. Thus, according to Deere et al (1967) classification the rock mass may be categorized as very poor to poor quality rock.

5.2.4 Weak to medium strong IGNIMBRITE rock

The ignimbrite rock layer is found sandwiched between the soil layers and as per the borehole logs data it is present at a depth ranging from about 5 to 8.5 m from the normal ground surface. The thickness of ignimbrite rock layer generally varies from 0.95 to 2.7 m, however as per some borehole records it continues to the maximum drilled depth (8.5m). The rock in general is light grey in color¸ slightly to moderately weathered, closely jointed/fractured, medium grained and weak to medium strong. The rock quality designation (RQD) values as observed from various borehole log reports shows that it varies from 0 to 80 % and in majority of cases it is less than 50%. Thus, according to Deere et al (1967) classification the rock mass may be categorized as very poor to poor quality rock. Further, the uniaxial compressive strength, tested through rock core show that the UCS values vary from10.61 to 25.73 MPa. Thus, according to Hoek and Brown (1977) classification these rocks may be characterized as ‘weak rocks.

5.2.4 Highly to completely weathered BASALT layer

This basalt rock layer is starting from at minimum and maximum depths of 8.6m and 13.5m. It is represented grayish white to reddish brown, fine grained, massive, jointed, and highly to completely weathered BASALT, weak to very weak. In general, the rock quality designation (RQD) value as from observed from various borehole log reports show that varies from 0% - 51% and the majority of cases it is less than 50%. Thus, according to Deere et al (1967) classification the rock mass may be categorized as very poor to poor quality rock. Further, the uniaxial compressive strength, tested through rock core show that the UCS values vary from 8.43 to 19.71 MPa. Thus, according to Hoek and Brown (1977) classification these rocks may be characterized as 'weak rocks.

5.3 Bearing capacity (q_u) Assessment

Before any foundation analysis is made, the fundamental material parameters should be known. The basic soil properties that are required for the bearing capacity estimation are shear strength parameters (cohesion and angle of shearing resistance) and the unit weight of the soil. Other factors such as the depth and width of foundation should also be known (Arora, 1997; Ranjan and Rao, 2002).Thus, for the computation of bearing capacity the above mentioned soil properties were obtained for each site from the secondary and primary data sources and are presented in Table 5.8. The bearing capacity of foundation soils varies with depth and footing dimensions therefore, in the present study bearing capacity was computed by considering different depths and variable footing dimensions. For the present study the bearing capacity was only compute for square footings. Also, it was assumed that only vertical static loads will be acting on the foundation and ground surface is horizontal. It is also assumed that ground water table is deep and it may not affect the foundation soils. Table 5.9 gives details for depths and footing dimensions for various sites for which bearing capacity was computed.

For the present study ultimate bearing capacity (Qult) and allowable bearing capacity (Qall) were computed by utilizing MS Excel computation sheet developed by Raghuvanshi (2017).This computational sheet facilitate to compute bearing capacity proposed by Terzaghi (1943), Hensen (1957, 1970), Vesic (1973), Meyerhof (1963) and Ethiopian Building Standard code (EBSC-7).The computational sheet facilitates to do analysis for variable footing dimensions at different depths. The basic input data required for computation of bearing capacity are foundation soil properties; cohesion (C), shearing resistance (Ø), unit weight of soil (γ), footing dimensions, depth of footing, depth to ground water table and mode of application of load. The bearing capacity factors, depth, shape and inclination factors are computed automatically. In order to compute allowable bearing capacity (Qall) a factor of safety of 3 was considered for the present study.

The following section presents bearing capacity estimation for each site except for site 7 and site 8. In site 7 and 8 the important soil property (cohesion (C), shearing resistance (Ø), unit weight of soil (γ)) were not available due to which bearing capacity estimation was not carried out. Bearing capacity estimation was made for individual borehole for sites 1 to 6 and 9. The results presented for ultimate bearing capacity (Qult) and allowable bearing capacity (Qall) are given in range within which values of Qult and Qall fall within each site for different depths and variable footing dimensions. The results on Qult and Qall are also presented graphically for comparison and better understanding.

Table 5.8 Input soil parameters used for different sites for bearing capacity computations

Sites	**No of Boreholes**	**Depth (m)**	**Soil Properties used in bearing capacity analysis**		
			Unit weight of soil (γ) (kN/m3)	**Cohesion (C) (kN/m2)**	**Angle of shearing resistance (ϕ) (deg.)**
Site 1	**11** (BH 1 to 11)	**0.5**	15.9 to 20	21 to 28	14.68 to 20.78
		1			
		1.5			
		2			
		2.5			
Site 2	**11** (BH 1 to 11)	**0.5**	16 to 19	10 to 29	16 to 21
		1			
		1.5			
		2			
		2.5			
Site 3	**15** (BH 1 to 15)	**0.5**	16.3 to 22.3	23.33 to 69	6 to 21
		1			
		1.5			
		2			
		2.5			
Site 4	**10** (BH 1 to 10)	**0.5**	17 to 21.73	11 to 33	11.3 to 19
		1			
		1.5			
		2			
		2.5			
Site 5	**4** (BH 1 to 4)	**0.5**	17.8 to 19	16 to 17	17 to 19
		1			
		1.5			
		2			
		2.5			
Site 6	**8** (BH 1 to 8)	**0.5**	16 to 18.79	21.3 to 38	17 to 21.3
		1			
		1.5			
		2			
		2.5			
Site 9	7 (BH 1 to 7)	**0.5**	17.4 to 22.7	22 to 32	14.8 to 20.8
		1			
		1.5			
		2			
		2.5			

5.3.1 Site 1

Site 1 is located in the northern part of the study area and falls within UTM coordinates 483000E; 995000N to 485000E; 997000N. The important places which falls within Site 1 are; CMC Square, CMC Condominium and Safari. For the bearing capacity analysis soil properties data from 11 boreholes (BH 1 to 11) was considered.

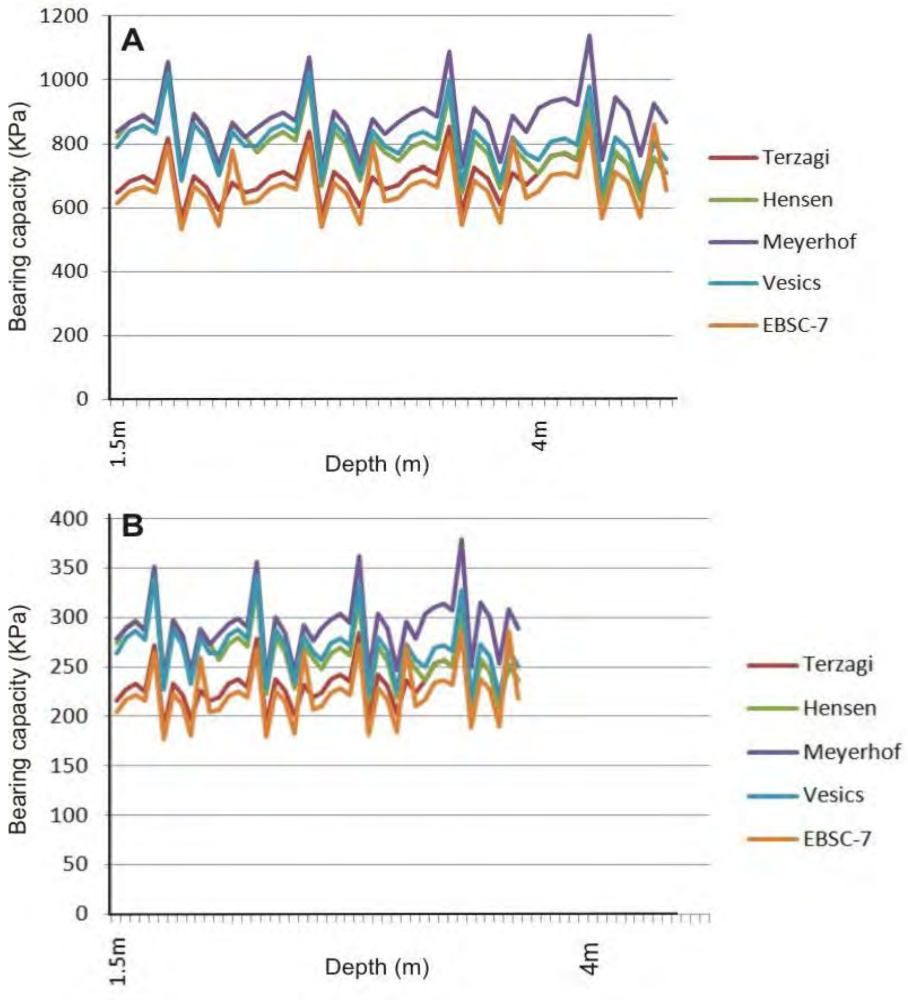
Table 5.9 Depths and footing dimensions for various sites for which bearing capacity was computed

<table>
<tr><th>Sites</th><th>Depth</th><th colspan="6">Footing dimensions for which bearing capacity was computed (m)</th></tr>
<tr><td rowspan="5">Site 1</td><td>0.5</td><td rowspan="5">0.5 x 0.5</td><td rowspan="5">1 x 1</td><td rowspan="5">1.5 x 1.5</td><td rowspan="5">2 x 2</td><td rowspan="5">2.5 x 2.5</td><td rowspan="5">4 x 4</td></tr>
<tr><td>1</td></tr><tr><td>1.5</td></tr><tr><td>2</td></tr><tr><td>2.5</td></tr>
<tr><td rowspan="5">Site 2</td><td>0.5</td><td rowspan="5">0.5 x 0.5</td><td rowspan="5">1 x 1</td><td rowspan="5">1.5 x 1.5</td><td rowspan="5">2 x 2</td><td rowspan="5">2.5 x 2.5</td><td rowspan="5">4 x 4</td></tr>
<tr><td>1</td></tr><tr><td>1.5</td></tr><tr><td>2</td></tr><tr><td>2.5</td></tr>
<tr><td rowspan="5">Site 3</td><td>0.5</td><td rowspan="5">0.5 x 0.5</td><td rowspan="5">1 x 1</td><td rowspan="5">1.5 x 1.5</td><td rowspan="5">2 x 2</td><td rowspan="5">2.5 x 2.5</td><td rowspan="5">4 x 4</td></tr>
<tr><td>1</td></tr><tr><td>1.5</td></tr><tr><td>2</td></tr><tr><td>2.5</td></tr>
<tr><td rowspan="5">Site 4</td><td>0.5</td><td rowspan="5">0.5 x 0.5</td><td rowspan="5">1 x 1</td><td rowspan="5">1.5 x 1.5</td><td rowspan="5">2 x 2</td><td rowspan="5">2.5 x 2.5</td><td rowspan="5">4 x 4</td></tr>
<tr><td>1</td></tr><tr><td>1.5</td></tr><tr><td>2</td></tr><tr><td>2.5</td></tr>
<tr><td rowspan="5">Site 5</td><td>0.5</td><td rowspan="5">0.5 x 0.5</td><td rowspan="5">1 x 1</td><td rowspan="5">1.5 x 1.5</td><td rowspan="5">2 x 2</td><td rowspan="5">2.5 x 2.5</td><td rowspan="5">4 x 4</td></tr>
<tr><td>1</td></tr><tr><td>1.5</td></tr><tr><td>2</td></tr><tr><td>2.5</td></tr>
<tr><td rowspan="5">Site 6</td><td>0.5</td><td rowspan="5">0.5 x 0.5</td><td rowspan="5">1 x 1</td><td rowspan="5">1.5 x 1.5</td><td rowspan="5">2 x 2</td><td rowspan="5">2.5 x 2.5</td><td rowspan="5">4 x 4</td></tr>
<tr><td>1</td></tr><tr><td>1.5</td></tr><tr><td>2</td></tr><tr><td>2.5</td></tr>
<tr><td rowspan="5">Site 9</td><td>0.5</td><td rowspan="5">0.5 x 0.5</td><td rowspan="5">1 x 1</td><td rowspan="5">1.5 x 1.5</td><td rowspan="5">2 x 2</td><td rowspan="5">2.5 x 2.5</td><td rowspan="5">4 x 4</td></tr>
<tr><td>1</td></tr><tr><td>1.5</td></tr><tr><td>2</td></tr><tr><td>2.5</td></tr>
</table>

As can be seen from Fig. 5.1 the boreholes are well distributed in Site 1. For all boreholes cohesion (C), shearing resistance (Ø), unit and weight of soil (γ) fall within a range of 21 to 28 KN/m^2, 14.68 to 20.78^o and 15.9 to 20 kN/m^3, respectively. The bearing capacity was computed for variable square footing dimension (0.5, 1, 1.5, 2, 2.5 and 4 m) at different depth (0.5, 1, 1.5, 2 and 2.5 m). The summary of results for ultimate bearing capacity (Qult) and allowable bearing capacity (Qall) for Site 1 is presented in annex 1 and 2, respectively. The results on bearing capacity for Site 1 computed for representative footing dimension 1.5 x 1.5m are also presented graphically in Fig. 5.4.The average Qult and Qall values for Site 1 by Terzaghi (1943), Hensen (1957, 1970), Vesic (1973), Meyerhof (1963) and EBSC-7 at different depths and various footing dimensions are presented in Table 5.10.

Table 5.10 Average Qult and Qall values for Site 1

Depth (m)	Footing dimension (m x m)					
	0.5 x 0.5	**1 x 1**	**1.5 x 1.5**	**2 x 2**	**2.5 x 2.5**	**4 x 4**
	Average range of Allowable bearing capacity (Qult) (KPa)					
0.5	587 - 886	597 - 908	605 – 909	605 - 916	602 – 912	612 - 928
1	607–914	627–917	630 – 923	630 – 933	627 - 931	633- 941
1.5	687–936	633 –942	638–948	635 - 946	631 - 944	638 - 963
2	643 - 956	651 - 973	657 – 981	680 - 987	680 – 996	698- 1008
2.5	685 - 976	711 - 973	713 – 993	696 – 1004	701 - 1025	720 – 1026
	Average range of Allowable bearing capacity (Qall) (KPa)					
0.5	125 - 235	130 – 240	140 - 250	140 -250	140 - 245	145- 260
1	135 - 255	165 – 275	180 – 290	180 - 295	180 - 280	180 - 300
1.5	150 - 270	180 – 290	210- 320	210 - 320	210 - 315	210 – 330
2	150 - 280	190 – 300	210 – 310	250- 360	245 – 350	280 - 400
2.5	160 – 290	230 – 340	280 – 380	280 - 390	290 – 400	290 - 420



A – Ultimate bearing capacity B – Allowable bearing capacity

Fig. 5.4 Bearing capacity estimations for Site 1, computed for footing dimension1.5 x 1.5 m

5.3.2 Site 2

Site 2 is located in the northern part of the study area and falls within UTM coordinates 485000 to 487000 Easting and 995000 to 997000Northing. The important places which fall within Site 2 are Woreda 16 Ayat area, around Ayat real estate, Woreda 10 around Gabriel church and Woreda 10 former Kebele 04. For the bearing capacity analysis soil properties data from 11 boreholes (BH 1 to 11) was considered. For all boreholes cohesion (C), shearing resistance (Ø), unit and weight of soil (γ) fall within a range of 10 to 29 KN/m^2, 16 to 21^o and 16 to 19 KN/m^3, respectively.

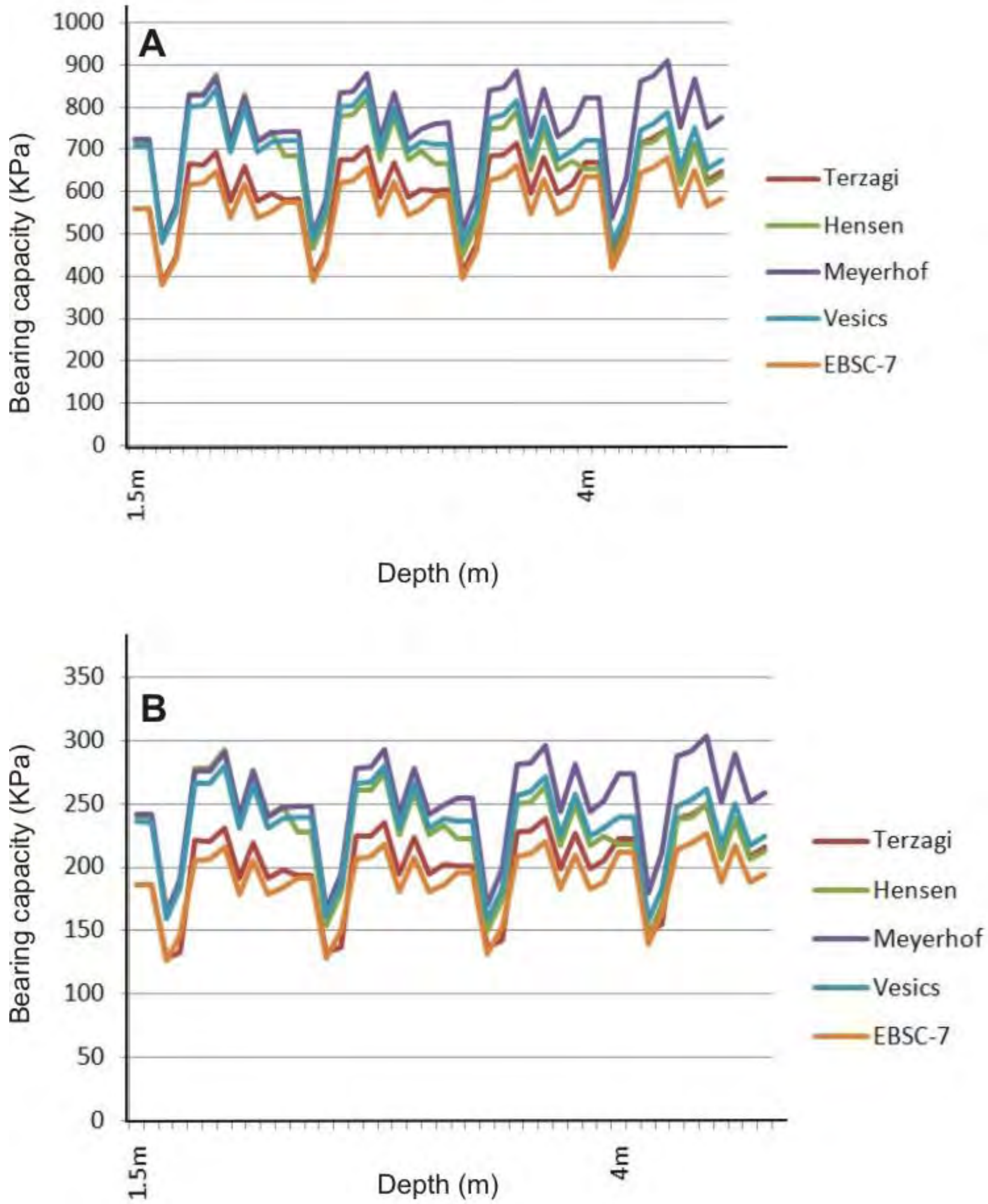
The bearing capacity was computed for variable square footing dimension (0.5, 1, 1.5, 2, 2.5 and 4 m) at different depth (0.5, 1, 1.5, 2 and 2.5 m). The summary of results for ultimate bearing capacity (Qult) and allowable bearing capacity (Qall) for Site 2 is presented in Annex 3 and 4, respectively. The results on bearing capacity for Site 2 computed for representative footing dimension 1.5 x 1.5 m are also presented graphically in Fig. 5.5. The average Qult and Qall values for Site 2 by Terzaghi (1943), Hensen (1957, 1970), Vesic (1973), Meyerhof (1963) and EBSC-7 at different depths and various footing dimensions are presented in Table 5.11.

Table 5.11 Average Qult and Qall values for Site 2

Depth (m)	Footing dimension (m x m)					
	0.5 x 0.5	**1 x 1**	**1.5 x 1.5**	**2 x 2**	**2.5 x 2.5**	**4 x 4**
	Average range of Allowable bearing capacity (Qult) (KPa)					
0.5	405 - 605	405- 730	410- 750	410- 756	410- 770	410– 790
1	411- 613	425- 763	420- 781	430- 786	420– 780	440– 800
1.5	421– 623	435– 783	443– 791	444– 796	440– 800	460– 810
2	451– 653	465– 790	470– 800	470– 770	460– 800	470– 820
2.5	490- 693	500- 800	490- 810	500– 820	480– 810	490– 840
	Average range of Allowable bearing capacity (Qult) (KPa)					
0.5	110 - 200	110 - 205	100 - 200	100 – 210	100 – 220	110 – 240
1	110 - 210	110 - 230	115 - 245	115 – 250	120 – 250	145 – 260
1.5	120 - 220	140 - 240	140 - 270	140 – 260	140 – 265	150 – 270
2	150 - 250	170 - 270	170 - 280	180 – 280	190 – 320	205 – 340
2.5	190 - 320	200 - 340	210 - 340	220 – 340	230 – 360	250 – 370

5.3.3 Site 3

Site 3 is located in the northern part of the study area and falls within UTM coordinates 487000 to 489307.002 Easting and 995000 to 997000 Northing. The important places which fall within Site 3 are; Ayat Condominium, Woreda 10 around Ayat Chefe and EADG hospital. For the bearing capacity analysis soil properties data from 15 boreholes (BH 1 to 15) was considered.



A – Ultimate bearing capacity B – Allowable bearing capacity

Fig. 5.5 Bearing capacity estimations for Site 2, computed for footing dimension 1.5 x 1.5 mm

In this site data for maximum number of boreholes (total 98 boreholes) was available, a good concentration of boreholes can be seen in Site 3 (Fig.5.1). For all boreholes cohesion (C), shearing resistance (Ø), unit and weight of soil (γ) fall within a range of 23.33 to 69 kN/m^2, 6 to 21^o and 16.3 to 21.3 KN/m^3, respectively. The bearing capacity was computed for variable square footing dimension (0.5, 1, 1.5, 2, 2.5 and 4 m) at different depth (0.5, 1, 1.5, 2 and 2.5 m).

The summary of results for ultimate bearing capacity (Qult) and allowable bearing capacity (Qall) for Site 3 is presented in annex 5 and 6, respectively. The results on bearing capacity for Site 3, computed for representative footing dimension 1.5 x 1.5m, are also presented graphically in Fig. 5.6.The average Qult and Qall values for Site 3 by Terzaghi (1943), Hensen (1957, 1970), Vesic (1973), Meyerhof (1963) and EBSC-7 at different depths and various footing dimensions are presented in Table 5.12.

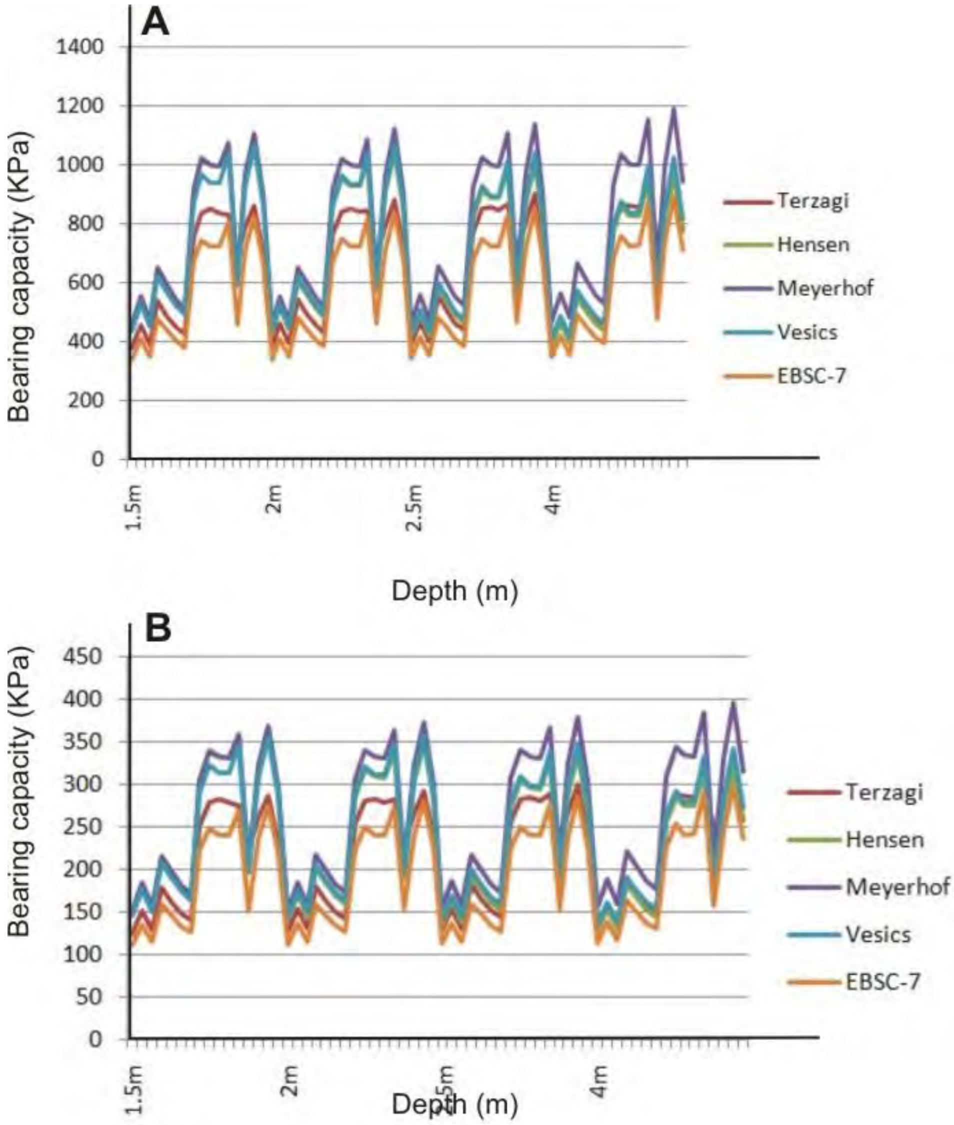
Table 5.12 Average Qult and Qall values for Site 3

Depth (m)	Footing dimension (m x m)					
	0.5 x 0.5	**1 x 1**	**1.5 x 1.5**	**2 x 2**	**2.5 x 2.5**	**4 x 4**
	Average range of Allowable bearing capacity (Qult) (kPa)					
0.5	530- 930	530– 940	530- 960	535– 965	540- 970	540– 970
1	550- 950	560- 970	570- 990	570- 1000	575– 1010	560- 1000
1.5	570– 970	580– 990	590– 1000	600– 1020	610– 1040	610- 1010
2	590– 1000	600- 1020	610– 1060	610- 1050	620– 1070	615- 1050
2.5	600- 1020	610- 1050	630–1100	620- 1070	650– 1100	630- 1070
	Average range of Allowable bearing capacity (Qult) (kPa)					
0.5	100 - 255	100 - 270	100 - 280	100 – 285	100 – 290	100 – 310
1	100 – 265	100 - 300	100 - 310	110 – 320	110 – 320	135 – 345
1.5	110 - 330	110 – 320	130 - 340	130 – 335	130 – 330	130 – 340
2	120 - 340	150 – 360	170 - 370	170 – 380	180 – 390	200 – 410
2.5	180 – 380	200 – 410	210 - 410	210 – 420	220 – 430	230 – 440

5.3.4 Site 4

Site 4 is located in the western part of the study area and falls within UTM coordinates 483000 to 485000 Easting and 992600 to 995000 Northing. The important places which fall within Site 4 are Summit square and summit condominium. For the bearing capacity analysis soil properties data from 10 boreholes (BH 1 to 10) was considered. In this site boreholes (total number 26) distribution is not even and mostly concentrated at one location (Fig.5.1).For all boreholes cohesion (C), shearing resistance (Ø), unit and weight of soil (γ) fall within a range of 11 to 33 KN/m^2, 11.3 to 19^o and 17 to 21.3 kN/m^3, respectively.

The bearing capacity was computed for variable square footing dimension (0.5, 1, 1.5, 2, 2.5 and 4 m) at different depth (0.5, 1, 1.5, 2 and 2.5 m). The summary of results for ultimate bearing capacity (Qult) and allowable bearing capacity (Qall) for Site 4 is presented in annex 7 and 8, respectively. The results on bearing capacity for Site 4, computed for representative footing dimension 1.5 x 1.5 m, are also presented graphically in Fig. 5.7. The average Qult and Qall values for Site 4 by Terzaghi (1943), Hensen (1957, 1970), Vesic (1973), Meyerhof (1963) and EBSC-7 at different depths and various footing dimensions are presented in Table 5.13.



A – Ultimate bearing capacity B – Allowable bearing capacity

Fig. 5.6 Bearing capacity estimations for Site 3, computed for footing dimension 1.5 x 1.5 m

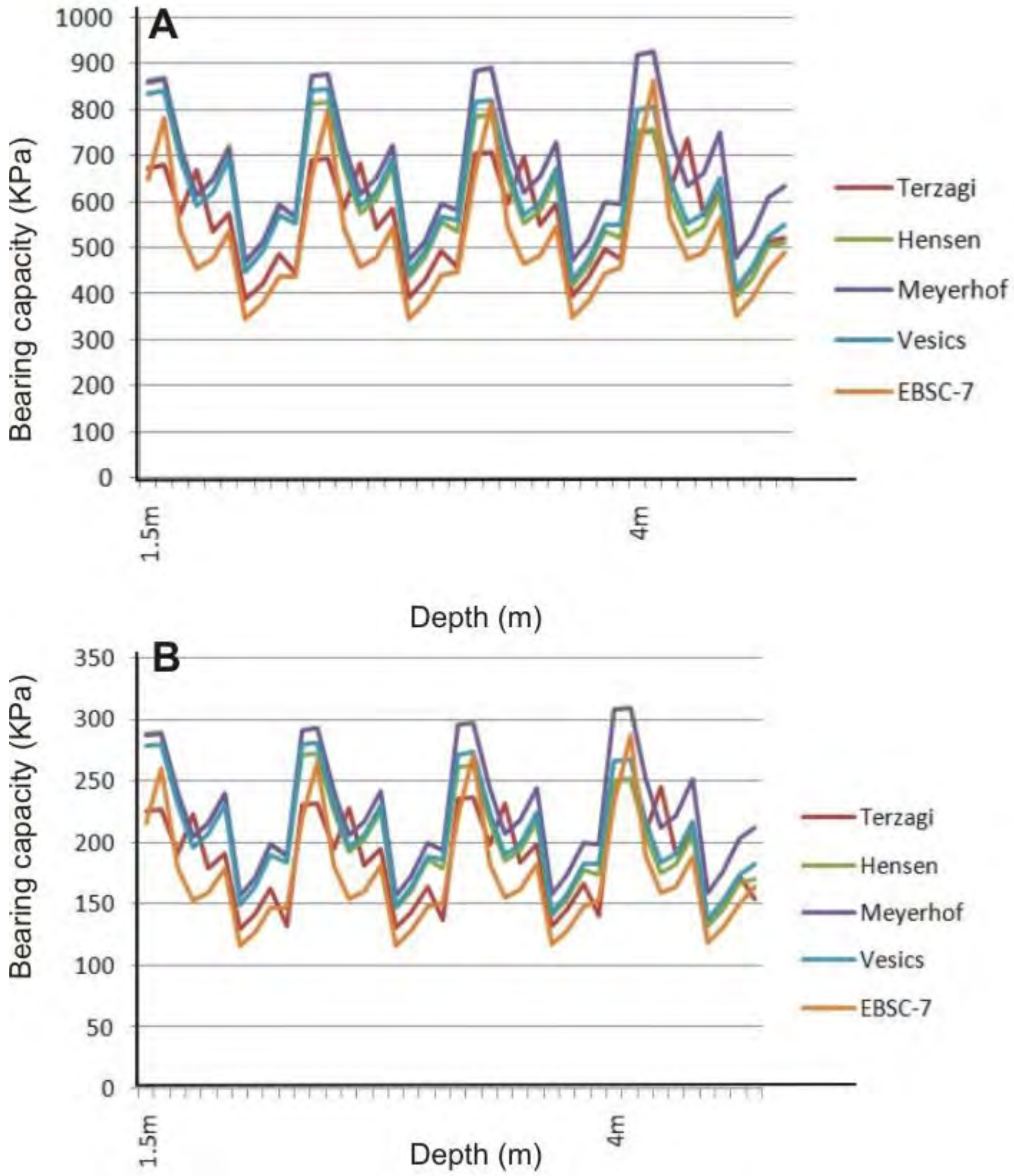
5.3.5 Site 5

Site 5 is located exactly in the central part of the study area and falls within UTM coordinates 485000 to 487000 Easting and 992600 to 995000 Northing. The important place which falls within Site 5 is Ayat 3.

For the bearing capacity analysis soil properties data from 4 boreholes (BH 1 to 4) was considered. In this site boreholes (total number 10) distribution is scattered and mostly concentrated at one location (Fig.5.1). For all boreholes cohesion (C), shearing resistance (Ø), unit and weight of soil (γ) fall within a range of 16 to 17 kN/m^2, 17 to 29^o and 17.8 to 19 kN/m^3, respectively.

Table 5.13 Average Qult and Qall values for Site 4

Depth (m)	Footing dimension (m x m)					
	0.5 x 0.5	**1 x 1**	**1.5 x 1.5**	**2 x 2**	**2.5 x 2.5**	**4 x 4**
	Average range of Allowable bearing capacity (Qult) (kPa)					
0.5	390 - 745	390- 755	400 - 760	405 - 770	410 - 790	405 - 770
1	400–765	405–775	410 - 780	415- 790	420 - 800	415 - 790
1.5	410 - 785	415 - 795	420 - 800	425 - 810	430 - 820	435 - 830
2	430 - 795	435 - 805	440 - 810	445 - 820	450 - 840	445 - 850
2.5	450 - 800	455 - 815	460 - 820	465 - 840	470 - 860	465 - 840
	Average range of Allowable bearing capacity (Qult) (kPa)					
0.5	100 - 200	100 - 200	100 – 200	100 - 210	100 – 220	110 – 240
1	100 - 220	100 - 230	110 – 240	120 - 250	120 – 260	145 – 280
1.5	110 - 250	120 - 260	140- 270	130 - 270	130 – 270	130 – 280
2	130 - 270	140 - 290	160 – 300	180 - 310	190 – 320	210 – 340
2.5	170 - 290	200 - 310	210 – 330	210 - 340	220 – 350	240 – 370



A – Ultimate bearing capacity B – Allowable bearing capacity

Fig. 5.7 Bearing capacity estimations for Site 4, computed for footing dimension 1.5 x 1.5 m

The bearing capacity was computed for variable square footing dimension (0.5, 1, 1.5, 2, 2.5 and 4 m) at different depth (0.5, 1, 1.5, 2 and 2.5 m). The summary of results for ultimate bearing capacity (Qult) and allowable bearing capacity (Qall) for Site 5 is presented in Annex 9 and 10, respectively. The results on bearing capacity for Site 5, computed for representative footing dimension 1.5 x 1.5m, are also presented graphically in Fig. 5.8. The average Qult and Qall values for Site 5 by Terzaghi (1943), Hensen (1957, 1970), Vesic (1973), Meyerhof (1963) and EBSC-7 at different depths and various footing dimensions are presented in Table 14.

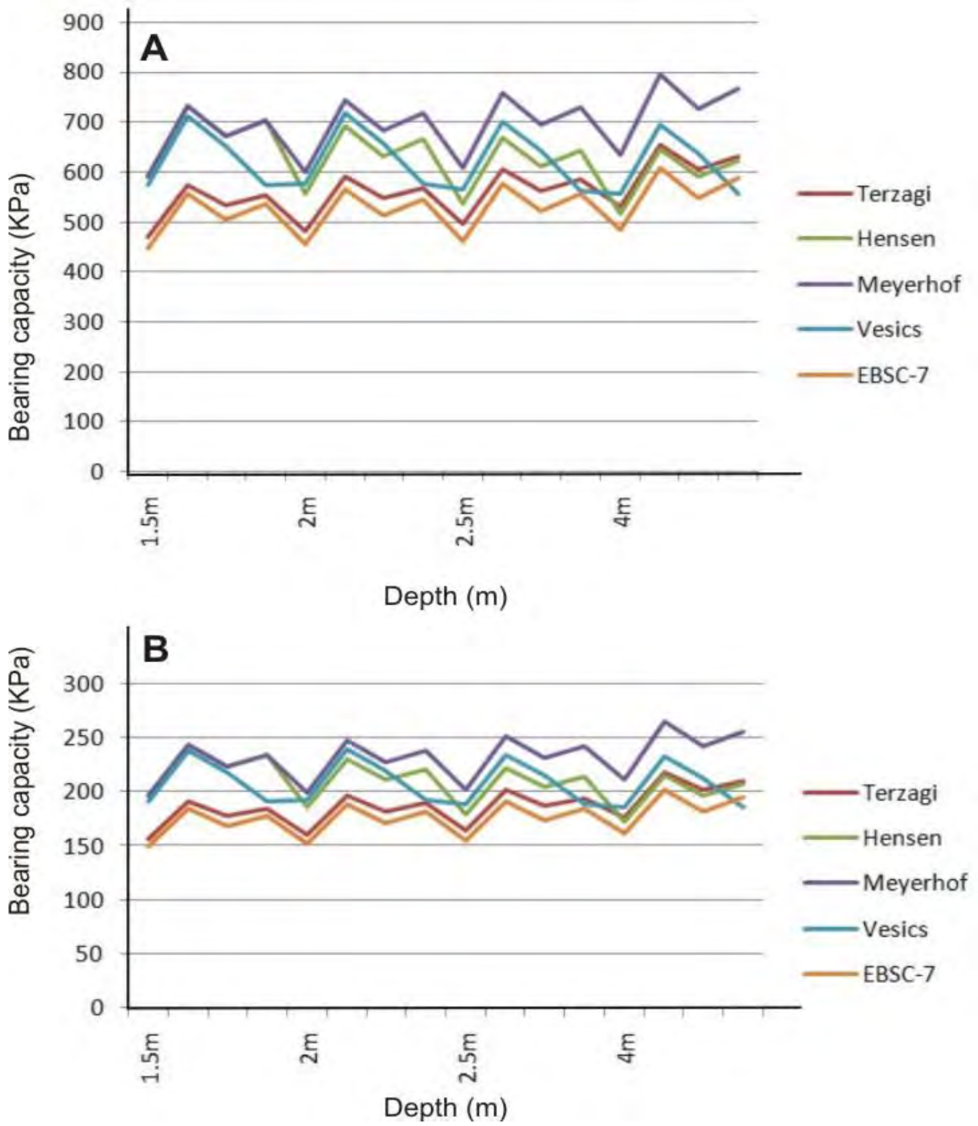
5.3.6 Site 6

Site 6 is located in the eastern part of the study area and falls within UTM coordinates 487000 to 489307.021 Easting and 992600 to 995000 Northing. The important place which fall within Site 6 is Bole Ayat 5.For the bearing capacity analysis soil properties data from 8 boreholes (BH 1 to 8) was considered. In this site boreholes (total number 23) distribution is only concentrated at one location (Fig.5.1). For all boreholes cohesion (C), shearing resistance (Ø), unit and weight of soil (γ) fall within a range of 21.3 to 38 kN/m^2, 17 to 21.3^{o}and 16 to 18.79 kN/m^3, respectively. The bearing capacity was computed for variable square footing dimension (0.5, 1, 1.5, 2, 2.5 and 4 m) at different depth (0.5, 1, 1.5, 2 and 2.5 m).

Table 5.14 Average Qult and Qall values for Site 5

Depth (m)	Footing dimension (m x m)					
	0.5 x 0.5	**1 x 1**	**1.5 x 1.5**	**2 x 2**	**2.5 x 2.5**	**4 x 4**
	Average range of Allowable bearing capacity (Qult) (kPa)					
0.5	470 - 600	475 - 605	480–610	490– 620	505– 630	505– 635
1	490–620	495- 630	500 - 640	510– 650	515– 660	520– 665
1.5	510 - 640	515 - 650	520 - 660	530- 670	535– 680	540– 680
2	530 - 660	535 - 670	540 - 690	530– 680	555– 710	560– 710
2.5	550 - 680	555 - 690	560 - 700	540- 690	575- 730	580–735
	Average range of Allowable bearing capacity (Qult) (kPa)					
0.5	115- 180	110 - 185	110 – 170	115 – 215	120 – 220	140 – 220
1	120 - 190	130 - 205	140 – 210	150 - 210	155 – 210	180 – 230
1.5	130 - 220	150 - 220	175- 225	175- 225	175- 230	180- 235
2	160 - 230	180 - 250	200 – 260	210 - 270	220 – 275	230 – 300
2.5	180 -260	220 - 270	230 – 290	240 - 290	260 - 305	275 – 325

The summary of results for ultimate bearing capacity (Qult) and allowable bearing capacity (Qall) for Site 6 is presented in annex 11 and 12, respectively. The results on bearing capacity for Site 6, computed for representative footing dimension 1,5 x 1.5 m, are also presented graphically in Fig. 5.9. The average Qult and Qall values for Site 6 by Terzaghi (1943), Hensen (1957, 1970), Vesic (1973), Meyerhof (1963) and EBSC-7 at different depths and various footing dimensions are presented in Table 15.



A – Ultimate bearing capacity B – Allowable bearing capacity

Fig. 5.8 Bearing capacity estimations for Site 5, computed for footing dimension 1.5 x 1.5 m

5.3.7 Site 9

Site 9 is located in the southeastern part of the study area and falls within UTM coordinates 487000 to 489307.021 Easting and 992573.949 to 990090.974 Northing. The important places which fall within Site 9 are; areas close to Bole Arabsa condominium and areas close to Legetafo River. For the bearing capacity analysis soil properties data from 7 boreholes (BH 1 to 7) was considered. In this site boreholes (total number 24) distribution is only concentrated at one location (Fig.5.1). For all boreholes cohesion (C), shearing resistance (Ø), unit and weight of soil (γ) fall within a range of 22 to 32 kN/m^2, 14.8 to 20.8^o and 17.4 to 22.7 kN/m^3, respectively. The bearing capacity was computed for variable square footing dimension (0.5, 1, 1.5, 2, 2.5 and 4 m) at different depth (0.5, 1, 1.5, 2 and 2.5 m). The summary of results for ultimate bearing capacity (Qult) and allowable bearing capacity (Qall) for Site 9 is presented in annex 13 and 14, respectively. The results on bearing capacity for Site 9, computed for representative footing dimension 1.5 x 1.5 m, are also presented graphically in Fig. 5.10. The average Qult and Qall values for Site 9 by Terzaghi (1943), Hensen (1957, 1970), Vesic (1973), Meyerhof (1963) and EBSC-7 at different depths and various footing dimensions are presented in Table 5.16.

A detailed discussion on the results of Ultimate bearing capacity and Allowable bearing capacity (qa) obtained by various equations at different locations for variable depth and different footing dimensions is made in Chapter 6. Besides, a systematic interpretation is also presented in Chapter 6 in the light of objectives of the present study.

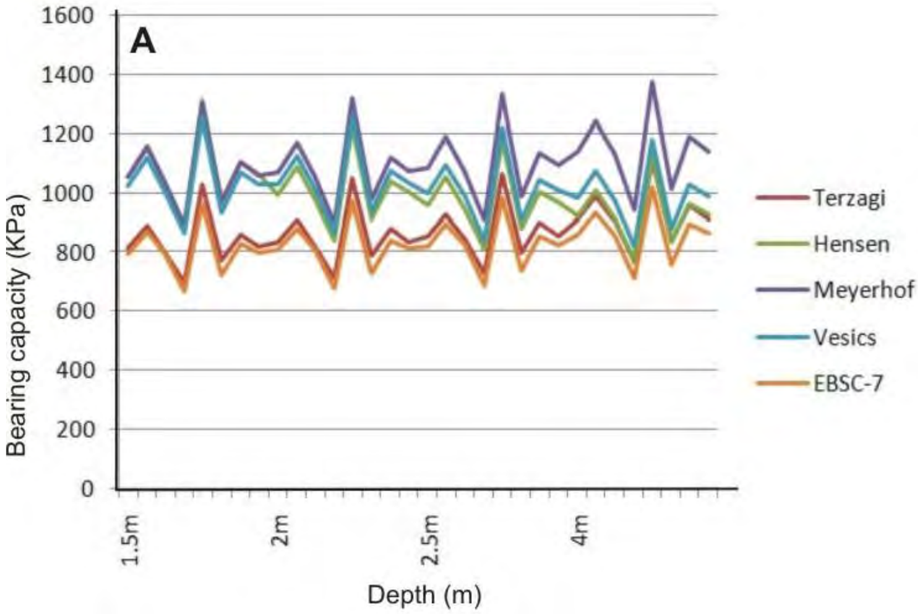
Table 5.15 Average Qult and Qall values for Site 6

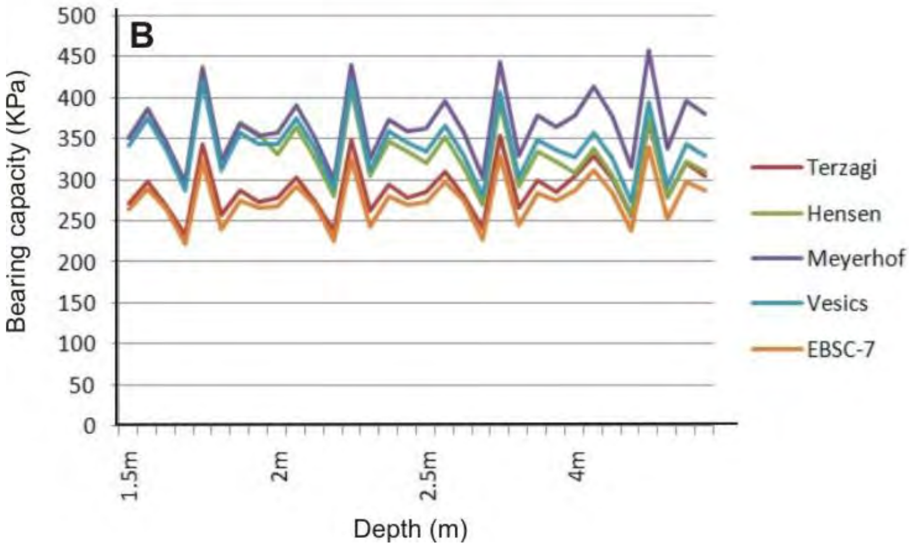
Depth (m)	Footing dimension (m x m)					
	0.5 x 0.5	**1 x 1**	**1.5 x 1.5**	**2 x 2**	**2.5 x 2.5**	**4 x 4**
	Average range of Allowable bearing capacity (Qult) (kPa)					
0.5	690–1110	695- 1110	700 - 1140	705 - 1170	705 - 1190	700 - 1170
1	705–1130	710–1140	715 - 1160	715 - 1190	720- 1200	740 - 1190
1.5	715–1150	720 - 1160	730 - 1180	735 - 1200	740- 1220	760 - 1210
2	735– 1170	740- 1190	750- 1205	760 - 1230	770 - 1245	775 - 1220
2.5	755–1190	760 - 1210	770 - 1230	775 - 1245	785- 1265	790 - 1250
	Average range of Allowable bearing capacity (Qult) (kPa)					
0.5	170 – 320	180 - 320	190 - 325	205 - 330	220 – 340	230 – 360
1	190 – 350	225 - 345	230 - 350	240 - 370	245 – 375	270 – 400
1.5	210 – 360	230 - 370	265- 395	265- 390	260 – 385	265- 390
2	230 – 370	260 - 430	280 - 430	305 - 440	310 – 440	320 – 460
2.5	260 – 390	290 - 415	310 - 420	320 - 430	330 – 470	365 – 495

5.4 Estimation of q_u by using Standard Penetration Test (SPT)

In the present study an attempt was also made to estimate the bearing capacity by utilizing insitu SPT data. The SPT data was procured as secondary data from site 8 in12 number of locations. Fig. 5.11 shows the location for which SPT data was utilized for the present study. Further, Table 5.31 presents the details of SPT data that was utilized in the present study.

As mention in Chapter-3, the SPT is widely used to obtain the bearing capacity of the soils directly. Meyerhof (1956, 1974, as cited in Bowels 1996) published equations for the computation of the allowable bearing capacity for 25-mm settlement. Bowels (1996) modified the Meyerhof's equations for an approximate 50 % increase in the allowable bearing capacity.





A – Ultimate bearing capacity B – Allowable bearing capacity

Fig. 5.9Bearing capacity estimations for Site 6, computed for footing dimension 1.5 x 1.5 m

$qa = \frac{N}{F1} KdB \leq F4$ ……eq.5.1

$qa = \frac{N}{F1} (\frac{B+F3}{F1})^2 \, Kd$ B > F4 …… eq. 5.2

Where qa = allowable bearing pressure for 25mm (1- inch) settlement

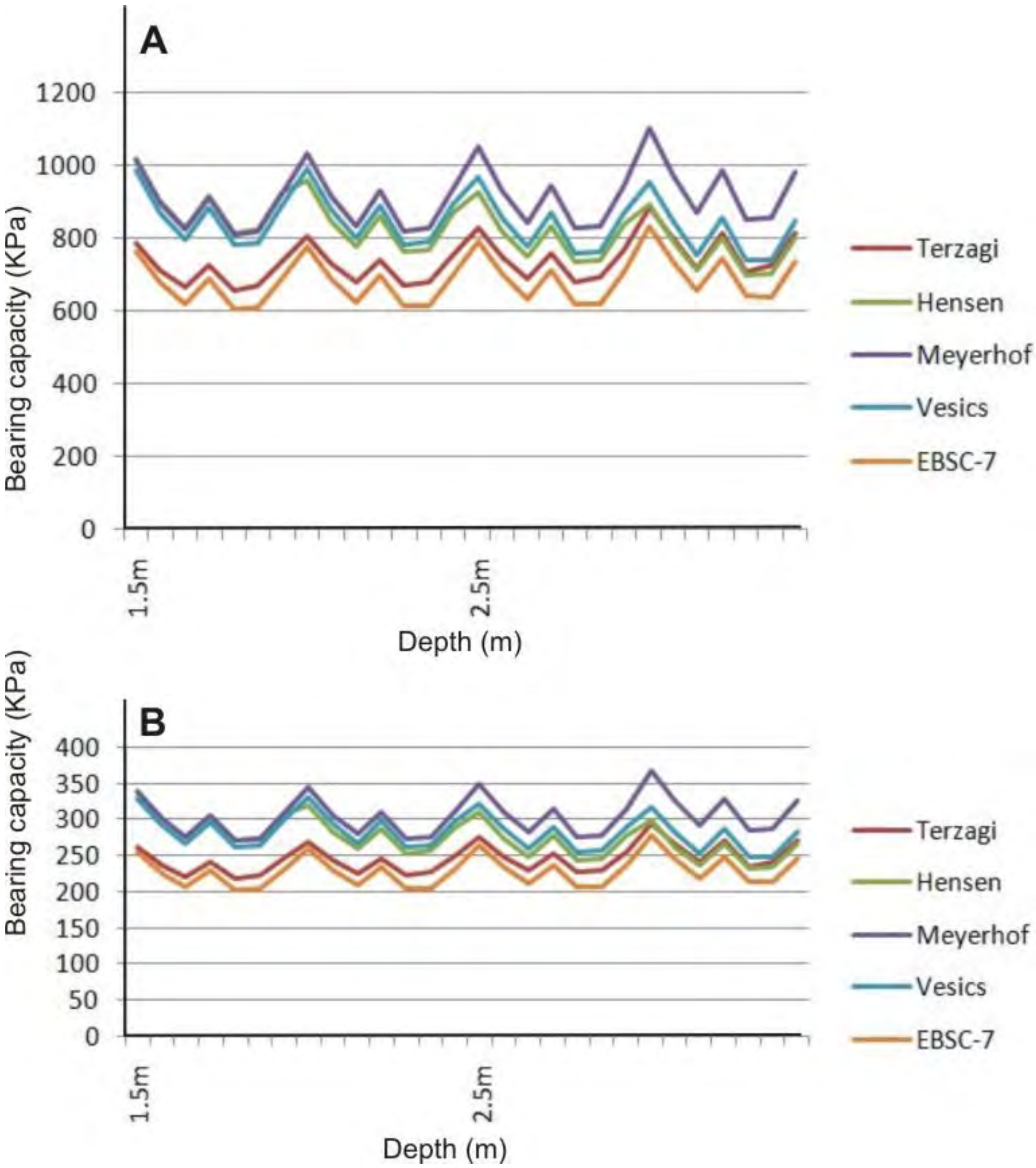
$Kd = 1 + 0.33 \frac{D}{B} \leq 1.33$ …… eq. 5.3

factors are as follows:

	N_{55}	N_{70}
F1	0.05	0.04
F2	0.08	0.06
F3	0.3	0.3
F4	1.2	1.2

Table 5.16 Average Qult and Qall values for Site 9

Depth (m)	Footing dimension (m x m)					
	0.5 x 0.5	**1 x 1**	**1.5 x 1.5**	**2 x 2**	**2.5 x 2.5**	**4 x 4**
	Average range of Allowable bearing capacity (Qult) (kPa)					
0.5	675 – 850	680- 860	695- 870	700– 880	700– 890	700–890
1	695– 875	700– 885	705– 890	710– 900	715– 900	720 – 895
1.5	715– 900	720– 910	725– 920	730– 940	735– 950	740 – 915
2	635– 915	740– 930	745– 940	750– 960	755– 970	750 – 935
2.5	655- 930	760– 950	765– 960	770- 980	775 - 990	760 – 975
	Average range of Allowable bearing capacity (Qult) (kPa)					
0.5	160 - 250	160 – 250	170 - 245	190 – 300	190 – 300	210 – 290
1	180 - 290	200 – 290	210 - 290	215 - 290	220 – 290	245 – 310
1.5	220 - 300	220 – 300	240- 305	240- 305	240- 305	240- 315
2	230 - 310	240 – 320	250 - 330	280 – 350	290 – 355	300 – 380
2.5	250 - 330	280 – 350	305 - 375	310 - 380	320 – 390	340 – 405



A – Ultimate bearing capacity B – Allowable bearing capacity

Fig. 5.10 Bearing capacity estimations for Site 9, computed for footing dimension 1.5 x 1.5 m


Fig. 5.11 Location in Site 8 for which SPT data was utilized in the present study

The depths at which the SPT N- values are obtained, the SPT N-values and the adjusted N-values (i.e. N'70) are given below and they are considered for determining the design N-values. The N values are converted to N77 standard energy ratio value according to Bowles (Bowles, 1988).

N'70= CN x N x n_1x n_2x n_3x n_4…… eq. 5.3

Where;N'70= adjusted N

CN = adjustment for overburden pressure $(p''o/p'o)^{1/2}$

p'_o= overburden pressure

p''_o= reference overburden pressure (95.76 kPa or 1.0 kg/cm^2)

n_1 = E_r/E_rb(where $E_r$is average energy ratio that depends on the drill system and $E_{rb}$is the standard energy ratio).

$E_r$is taken as 50 and E_{rb}as 70.

n_2= Rod length correction

Rod length > 10 m = 1,

Rod length 6-10 m = 0.95,

Rod length 4-6 m = 0.85,

Rod length 0-4 m = 0.75

n_3= sampler correction (1.00 in our case)

n_4= borehole diameter correction (1.00 in our case)

After adjusting the N-values based on the above formula and taking corresponding average values, a design N-values were chosen from consecutive depths where the test was performed. The design N-values are taken as the average of N-values which are found in between ½ B above and 2B below the proposed footing depths where B is the width of the footing.

The SPT - N corrected values used for the computation of allowable bearing capacity (qa) in the present study. The raw N values were corrected by following the procedure as mentioned above. Since there are number of correction factors and variable depths therefore it is not possible to present this data in tabular format.

The computed Allowable bearing capacity (qa) at different locations for variable depth and different footing dimensions is presented in Table 5.17.

A detailed discussion on Allowable bearing capacity (qa) obtained from SPT data at different locations for variable depth and different footing dimensions is made in Chapter 6. Besides, a systematic interpretation is also presented in Chapter 6 in the light of objectives of the present study.

5.5 Settlement Analysis

Settlement is another criterion for evaluating the performance of a building. Excessive settlements will result in poor performance of the building structure. Different building codes set the limiting settlement for the type of the structure and foundations. The proposed foundation types shall also meet this criterion (Bowels, 1984).

For saturated fine grained soils, the major part of the settlement is contributed by the consolidation settlement (Arora, 1997). Hence, the calculation of consolidation settlement for the soils in the study area is presented below.

Consolidation settlement computed using the following formula according to Bowels (1984):

$\Delta H = [CcH/(1 + eo)]\ [\log (p'o + \Delta p/ p'o)]$…… eq.5.4

Where, Cc is the compression index from e vs log p plot, $e_o$is the in-situ void ratio in the stratum, His the thickness of stratum, p'o= effective overburden pressure at mid-height of H andΔp = average increase in pressure from foundation in layer H in same unit as p'o.
For the present study settlement analysis was made for 3 sites. The details for these sites are presented in Table 5.18.

Table 5.17 Allowable bearing capacity (qa) as computed from SPT data in the present study

Site (as per Fig. 5.1)	SPT Location	Depth	Footing dimension (m)					
			1.5	2	2.5	3	3.5	4
			Allowable bearing capacity (qa) (kPa)					
Site 8	**1**	**2**	462	424	382	356	338	325
		2.5	469	431	409	378	357	341
		3	507	466	442	426	401	382
		3.5	506	465	441	425	414	393
		4	511	470	445	430	419	410
Site 8	**2**	**2**	550	505	456	424	403	387
		2.5	562	516	490	453	428	409
		3	602	553	525	506	475	453
		3.5	617	566	537	518	505	480
		4	637	585	555	536	522	511
Site 8	**3**	**2**	258	237	214	199	189	181
		2.5	271	249	236	219	206	198
		3	281	258	244	236	222	211
		3.5	289	265	252	243	236	225
		4	292	268	255	245	239	234
Site 8	**4**	**2**	485	446	402	374	355	341
		2.5	517	475	450	416	393	376
		3	528	485	460	443	417	397
		3.5	565	519	492	475	463	439
		4	579	532	505	487	474	465
Site 8	**5**	**2**	470	431	389	362	344	330
		2.5	470	431	409	378	357	342
		3	510	468	444	428	403	384
		3.5	523	480	455	439	428	406
		4	558	512	486	469	457	448
Site 8	**6**	**2**	500	459	414	385	366	352
		2.5	517	475	450	417	393	376
		3	508	467	443	427	401	383
		3.5	522	480	455	439	428	406
		4	505	464	440	424	413	405
Site 8	**7**	**2**	482	443	399	371	352	339
		2.5	482	443	420	388	366	351
		3	540	496	471	454	427	407
		3.5	536	492	467	450	438	417
		4	538	495	469	452	441	432
Site 8	**8**	**2**	391	359	323	301	286	275
		2.5	391	359	340	315	297	284
		3	431	396	375	362	340	324
		3.5	434	398	378	364	355	337
		4	427	392	372	359	350	343
Site 8	**9**	**2**	413	379	342	318	302	290
		2.5	421	387	367	339	320	306
		3	431	396	375	362	340	324
		3.5	440	404	383	370	360	342
		4	438	402	382	368	359	352
Site 8	**10**	**2**	442	406	366	341	323	311
		2.5	425	390	370	342	323	309
		3	431	396	375	362	340	324
		3.5	422	387	367	354	345	328
		4	410	377	357	345	336	329
Site 8	**11**	**2**	369	339	306	285	270	260
		2.5	383	351	333	308	291	279
		3	387	356	337	325	306	291
		3.5	412	378	359	346	337	320
		4	396	364	345	333	324	318
Site 8	**12**	**2**	430	395	356	331	314	302
		2.5	450	413	392	362	342	327
		3	451	414	393	379	356	340
		3.5	480	440	418	403	393	373
		4	484	445	422	407	396	389

Table 5.18 Details of sites for which settlement analysis was made

Site No.	Location from where sample was collected (UTM)			Depth (m)	Data source	Remarks
	Easting	Northing	Elevation (m)			
Site 1	484692.61	995772.87	2330	2 to 4m	Secondary	North part of the study area
Site 3	487932.31	996230.94	2399	2 to 4 m	Secondary	North part of the study area around Ayat condominium
Site 5	486089.11	993255.79	2312	2 to 4 m	Secondary	Central part of the study area close to quarry site

5.5.1 Site 1

This site is located in the northern part of the study area, the location details are given in Table 5.18. The input data used for settlement analysis at Site 1 is presented in Table 5.19. The results are presented in Table 5.20.

For Allowable bearing pressure cohesion (C), shearing resistance (Ø), unit and weight of soil (γ) fall within a range of 21 to 28 KN/m^2, 14.68 to 20.78^o and 15.9 to 20 kN/m^3, respectively.

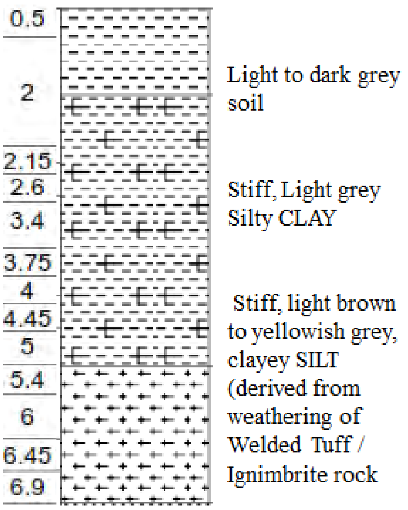


Table 5.19 Input data used for the settlement analysis at Site 1

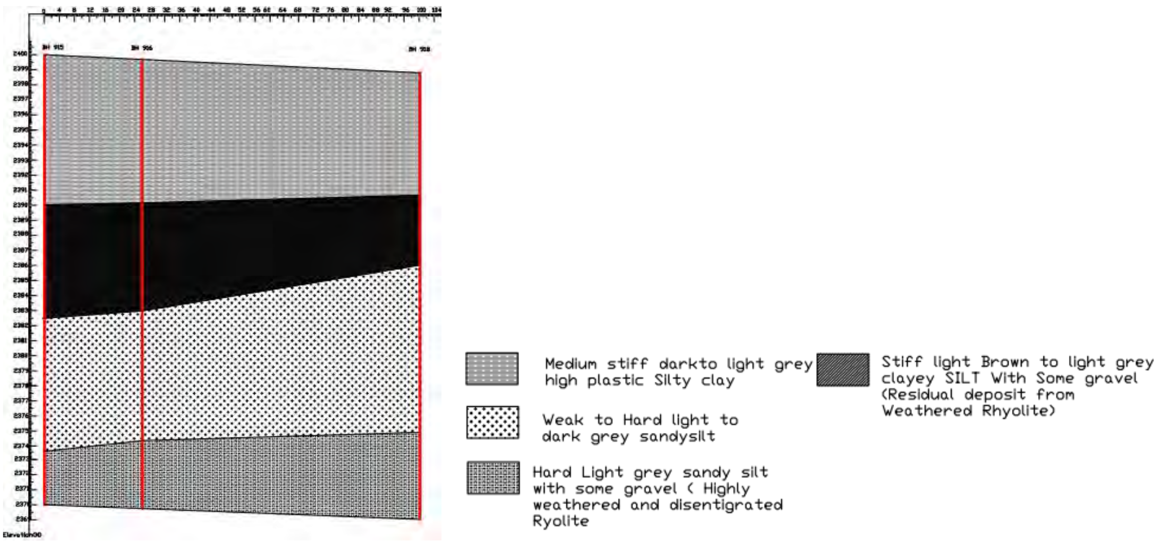
Site No.	Footing depth (m)	Footing width (m x m)	Allowable bearing pressure (KPa)	Thickness of stratum (H) (m)	Effective overburden pressure (Po) (KPa)	Average increase in pressure (ΔP)(Kpa)	Insitu void ratio (e_o)	Change in void ratio (Δe)	Compression index (Cc)
1	2	1.5	410	5	127.5	74.71	1.2	0.0395	0.197
	2	2.0	388	5	127.5	98.41	1.2	0.0489	0.197
	2	2.5	375	5	127.5	121.20	1.2	0.0572	0.197
	2	4.0	358	5	127.5	158.70	1.2	0.0692	0.197
2	2.5	1.5	302	5	102	55.75	1.49	0.0473	0.25
	2.5	2.0	286	5	102	73.57	1.49	0.059	0.25
	2.5	2.5	256	5	102	114.23	1.49	0.0816	0.25
	2.5	4.0	245	5	102	120.21	1.49	0.0845	0.25
3	3.5	1.5	387	5	102	75.29	1.173	0.0521	0.22
	3.5	2.0	367	5	102	99.87	1.173	0.0643	0.22
	3.5	2.5	354	5	102	123.08	1.173	0.0746	0.22
	3.5	4.0	327	5	102	155.51	1.173	0.0873	0.22
4	4	1.5	785	5	136	158.97	1.2	0.0662	0.22
	4	2.0	745	5	136	182.47	1.2	0.0803	0.22
	4	2.5	718	5	136	208.31	1.2	0.0917	0.22
	4	4.0	686	5	136	245.4	1.2	0.108	0.22

Table 5.20 Results for the settlement analysis at Site 1

Site No.	ΔH (mm)	Footing Dimension (m x m)			
		1.5	**2.0**	**2.5**	**4.0**
1	**Settlement**	89.68	111.23	129.92	157.23
2	**Settlement**	95.06	118.39	163.81	169.76
3	**Settlement**	119.87	148.03	171.63	208.61
4	**Settlement**	150.54	182.47	208.31	245.40

5.5.2 Site 3

This site is located in the northern part of the study area, around Ayat, the location details are given in Table 5.18. The input data used for settlement analysis at Site 3 is presented in Table 5.21. The results are presented in Table 5.22.



For Allowable bearing pressure cohesion (C), shearing resistance (Ø), unit and weight of soil (γ) fall within a range of 23.33 to 69 kN/m^2, 6 to 21^o and 16.3 to 21.3 KN/m^3, respectively. In the footing depth 2m – 3.5m.

Table 5.21 Input data used for the settlement analysis at Site 3

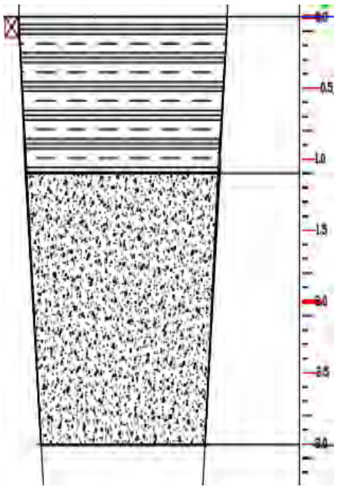
Site No.	Footing depth (m)	Footing width (m x m)	Allowable bearing pressure (kPa)	Thickness of stratum (H) (m)	Effective overburden pressure (Po) (KPa)	Average increase in pressure (ΔP)(Kpa)	Insitu void ratio (e_o)	Change in void ratio (Δe)	Compression index (Cc)
1	2	1.5	287	5	93.50	133.86	1.0794	0.0662	0.1326
	2	2.0	264	5	93.50	145.65	1.0794	0.0803	0.1326
	2	2.5	230	5	93.50	138.93	1.0794	0.0917	0.1326
	2	4.0	190	5	93.50	141	1.0794	0.108	0.1326
2	2.5	1.5	289	2.5	148.75	437.62	1.1437	0.0662	0.3701
	2.5	2.0	367	2.5	148.75	515.58	1.1437	0.0803	0.3701
	2.5	2.5	330	2.5	148.75	478.59	1.1437	0.0917	0.3701
	2.5	4.0	287	2.5	148.75	435.72	1.1437	0.108	0.3701
3	3.5	1.5	297	5	72	145.8	0.988	-	0.236
	3.5	2.0	371	5	85.64	139	0.988	-	0.236
	3.5	2.5	382	5	103.45	143	0.988	-	0.236
	3.5	4.0	297	5	122	126	0.988	-	0.236

Table 5.22 Results for the settlement analysis at Site 3

Site No.		Footing Dimension (m xm)			
	ΔH (mm)	**1.5**	**2.0**	**2.5**	**4.0**
1	**Settlement**	41.81	52.61	46.9	57.6
2	**Settlement**	76.62	99.68	89.19	75.97
3	**Settlement**	110.9	115.2	119.9	124

5.5.3 Site 5

This site is located in the centraln part of the study area, around quarry site, the location details are given in Table 5.18. The input data used for settlement analysis at Site 5 is presented in Table 5.23. The results are presented in Table 5.24.



0.00 – 1.2m
Dark gray, medium stiff, high plastic, silty CLAY.

1.2 – 4.00m
Light gray, medium stiff, high plastic, clayey SILT.

For Allowable bearing pressure cohesion (C), shearing resistance (Ø), unit and weight of soil (γ) fall within a range of 16 to 17 kN/m^2, 17 to 29^o and 17.8 to 19 kN/m^3, respectively. In the footing depth 2m – 4m.

Table 5.23 Input data used for the settlement analysis at Site 5

Site No.	Footing depth (m)	Footing width (m x m)	Allowable bearing pressure (kPa)	Thickness of stratum (H) (m)	Effective overburden pressure (Po) (KPa)	Average increase in pressure (ΔP)(kpa)	Insitu void ratio (e_o)	Change in void ratio (Δe)	Compression index (Cc)
1	2	1.5	370	5	127.5	74.71	1.2	0.0395	0.129
	2	2.0	290	5	127.5	98.41	1.2	0.0489	0.129
	2	2.5	245	5	127.5	121.20	1.2	0.0572	0.129
	2	4.0	220	5	127.5	158.70	1.2	0.0692	0.129
2	2.5	1.5	466	5	127.5	87.59	1.14	0.0293	0.129
	2.5	2.0	443	5	127.5	116.28	1.14	0.0363	0.129
	2.5	2.5	427	5	127.5	142.93	1.14	0.0421	0.129
	2.5	4.0	393	5	127.5	179.05	1.14	0.0491	0.129
3	3.5	1.5	436	5	102	147.53	0.77	0.0532	0.129
	3.5	2.0	426	5	102	189.09	0.77	0.0624	0.129
	3.5	2.5	379	5	102	198.32	0.77	0.0643	0.129
	3.5	4.0	361	5	102	232.22	0.77	0.0679	0.129
4	4	1.5	387	5	102	128.33	0.77	0.0485	0.129
	4	2.0	382	5	102	166.39	0.77	0.0576	0.129
	4	2.5	378	5	102	197.70	0.77	0.0641	0.129
	4	4.0	372	5	102	222.35	0.77	0.0688	0.129

Table 5.24 Results for the settlement analysis at Site 5

Site No.		Footing Dimension (m xm)			
	ΔH (mm)	1.5	2.0	2.5	4.0
1	Settlement	74.57	74.82	74.33	74.65
2	Settlement	68.45	84.84	98.42	114.83
3	Settlement	105.25	123.38	127.05	139.63
4	Settlement	95.83	113.83	126.81	136.11

The results showed that settlement at Site 1, Site 3 and Site 5 varies in a range from 89 to 245 mm, 41 to 124 mm and 74 to 136 mm, respectively. A detailed discussion on settlement results is made in Chapter 6.

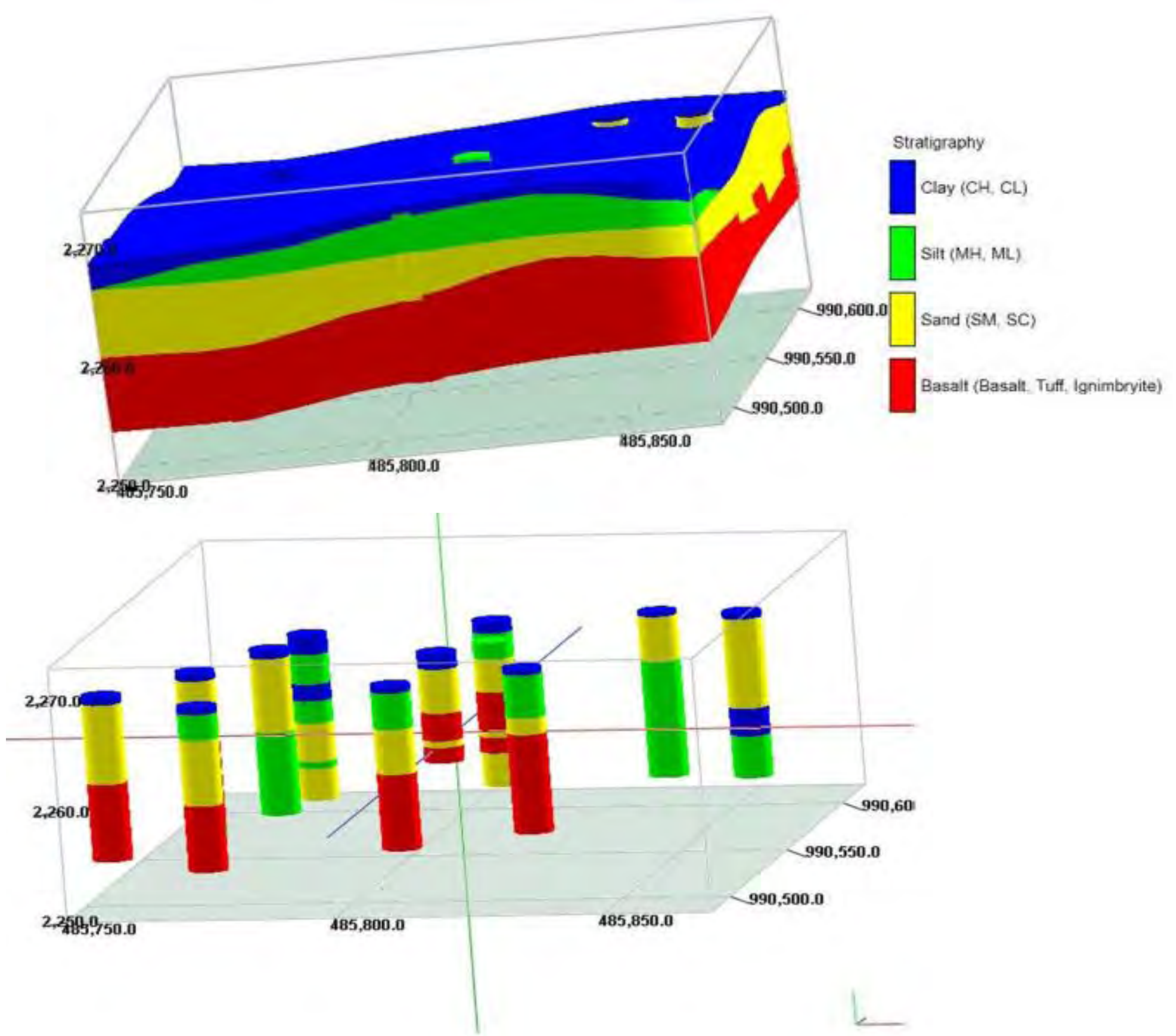


Fig 5.12 3D stratigraphic model of the study area and stratigraphic model of borehole of the study area.

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CHAPTER SIX RESULTS, INTERPRETATION AND DISCUSSION

6. Preamble

The present chapter deals with a detailed discussion on results. Besides, systematic interpretations are also made with respect to the general objective of the present study.

6.1 Site characterization

6.1.1 Light to dark grey soil

The most fundamental index properties of soils for foundation analysis are moisture content, Atterberg limits, free swell and unit weight. The results on index properties on representative soil samples from the study area are already presented in Table 5.6 in Chapter 5. The moisture content of light to dark grey soils ranges from 30 to 45%. Moisture content can provide valuable information on possible foundation problems. For example, if a clay layer located below a proposed shallow foundation has a water content of 100%, then it is likely that this clay will be highly compressible. Likewise if the same clay layer below the shallow foundation has a water content of 5%, then it is likely that the clay layer is dry and desiccated and could subject the shallow foundation to expansive soil uplift (Day, 2006).

The unit weight is another basic index property for determining the vertical overburden pressure (vertical stress distribution) for any foundation analysis. Besides, it is an important input parameter for the bearing capacity estimation for the foundations. From Table 5.6, unit weight of light to dark clay soils ranges from 15 to 21 kN/m^3. Further, the liquid limit (LL) and plasticity index (PI) for these soils varies from of 87 to 113% and 47 to 80%, respectively. Also, the free swell (FS) for these soils varies from 90 to 180 %, According to Bowels (1996), soils having PI value of >35% and LL value of >70% show very high potential for volume change. According to USACE (1992), soils having PI values of > 35% and LL values of > 60% show high degree of expansion. Soils having free swell values greater than 100% are considered potential problems (Holtz and Gibbs, 1956 as cited in Murthy, 2009). Thus, based on LL, PI and FS values light to dark soils in the study area are problematic. The results in general show that these soils are very high plastic and are fine grained expansive in nature. As per Unified Soil Classification System (USCS) these soils may be placed under MH and CH types of soils (Fig. 5.2).

One of the most important properties of soils for estimating the settlement is the void ratio. The void ratio also indicates the softness or stiffness of a soil. Void ratio of light to dark grey soil ranges from 0.0473 -0.0845. According to Bowles (1996), void ratio less than or equal to 0.35 indicates the most densest state while void ratio greater than or equal to 1 implies that the soil is loose or soft. This implies that light to dark grey soil in the study area are most probably in the densest state and also another important engineering property of expansive soils is the swelling pressure. Perusal of Table-5.1 also demonstrated that swelling pressure of light to dark grey soils in study area is in the range of 105kPa to 130 kPa. Hence, structures built on such soils should have an imposing pressure greater than the swelling pressure. If the imposed bearing pressure on the foundation by the super structure is less than the swelling pressure, the structure is likely to be lifted up at least locally which would lead to cracks in the structure (Murthy,2009). The shear strength parameters C and φ varies from 11 – 30 kPa and 11 – 21^{o}, respectively. These shear strength parameter are important input for bearing capacity estimation of the soils. Further, in situ Standard Penetration Test (SPT) results indicate that the N value for these soils varies from 6 to 25 blows which represents that light to dark grey soils in the study area are medium stiff to very stiff soils.

5.1.2 Soft dark, clay (Black Cotton soil)

The soil from this layer can be characterize as black, soft consistency, medium stiff, high plastic silty CLAY (Black cotton) soils. As per the results, these soils may be placed mainly under ML, MH and CL types of soils (USCS Classification system) (Fig. 5.3). The natural moisture content varies from 26 to 61%. Further, the liquid limit and plasticity index for these soils varies from of 37 to 89 % and 7 to 47 %, respectively. The free swell varies from 90 to 190 %. As already stated, according to Bowels (1996), soils having PI value of > 35% and LL value of > 70% show very high potential for volume change. According to USACE (1992), soils having PI values of >35% and LL values of >60% show high degree of expansion. Therefore, these soils possess high to very high expansion potential. Void ratio of lsoft dark clay soil ranges from 0.7270 –1.4002. According to Bowles (1996) this range of void ratio suggests that the soft dark clays in the study area range from medium dense to loose and soft soils.

Further, in situ Standard Penetration Test (SPT) results indicate that the N value varies from 4 to 17 blows which suggest that these soils are soft to stiff in its consistency and are very loose to medium dense with respect to its relative density.

6.1.3 In situ foundation soil characteristics

Standard penetration test (SPT) is important to understand in-situ strength of soils and to show a relationship with other soil properties.

During the present study SPT data was procured from 12 locations in Site 8 of the study area. This was the only complete SPT data which was available in the entire study area. The SPT data results from all 12 locations shows that SPT-N value in general ranges from 4 to 25. Based on these values the foundation soils in the study area can be characterized as medium stiff to hard with relative density of loose to very dense.

Based on the standard correlation of SPT – N values, the foundation soils in the study area have variable consistency: 10% of the test results showed medium stiff soils, 49% of the test results showed stiff soils, 6% of the test results showed very stiff soils, and 35% of the test results showed hard soils (Table 6.1).

Correlation of SPT-N value with relative density (Table 6.1) shows that: 23% of the soils are loose, 48% of the soils are medium dense and 29% of the soils are dense. In general the majority of the soils in the study area have stiff consistency and medium dense relative density. It implies that in general the foundation soils in the present study area are suitable for foundation and may not generally pose problems of stability. However, foundation conditions may vary for individual site and detailed foundation analysis needs to be made for each case separately.

6.2 Foundation characterization

6.2.1 Bearing capacity estimation

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 to be considered in selecting the foundation type (Arora, 1997).

Table 6.1 SPT N-value correlation with consistency and relative density

Consistency		**Test results from present study (%)**	**Compactness / Relative density**		**Test results from present study (%)**
N’ = 4 – 8	Medium stiff	10%	N’ = 4 - 10	Loose	23%
N’ = 8 – 15	Stiff	49%	N’ = 10-30	Medium dense	48%
N’ = 15 – 30	Very stiff	6%	N’ = 30-50	Dense	29%
N’ > 30	Hard	35%	N’ > 50	Very dense	0.13%

Before any foundation analysis is made, the fundamental material parameters should be known. The basic material properties are shear strength properties (cohesion and angle of shearing resistance) and the unit weight. Other factors such as the depth and width of foundation, and the level of ground water should also be determined. Based on these parameters ultimate bearing capacity and allowable bearing capacity of the study area were analyzed with different approaches with the help of computational sheets for bearing capacity analysis develop in Microsoft (MS) excel. The program for computation of bearing capacity for shallow foundation is written by Raghuvanshi (2016).

Taking this into consideration and the minimum permissible width of foundations by EBCS-7 (1995), which is greater than or equal to 1m, bearing capacity computation has been done for different depth intervals between 1m and 7.5m from the natural ground. Accordingly, ultimate bearing capacity of the foundation soils of the study area at different footing depths has been computed by using different approaches. For this analysis, the shape of the footing was assumed to be square. Further, in the present computation, the load was assumed to be vertical and its line of action was considered along the center of the foundation.

Also, in this study the SPT-N values were also used to estimate the bearing capacity of the foundation soils. The allowable bearing capacity was calculated from SPT blow counts for both isolated and mat foundation for different depths. Bearing capacity was estimated based on soil properties and SPT-N values and the results are already presented in Chapter 5.

In order to determine the foundation characteristics during the present study, data information from various sources was utilized. The required input data for foundation analysis was obtained from; borehole profile along with standard penetration test result, test reports on soil properties, laboratory test result (primary data), Ethiopian Building Code Standard EBCS-7 (1995) and experience and engineering judgment. In the following section Site wise discussion on results on foundation characterization is presented.

Site 1

Site 1 is located in the northern part of the study area and covers locations such as CMC square, CMC condominium and Safari. The foundation soils that are present at shallow depth are light to dark grey soils. The soils are characterized as light grey to light brown, stiff to very stiff, highly plastic silty clay/clayey silt soil. The ultimate bearing capacity and allowable bearing capacity for soils at Site 1 in the study area was computed by Terzaghi

(1943), Hensen (1957, 1970), Vesic (1973), Meyerhof (1963) and EBSC-7 methods at different depths by considering various footing dimensions (Table 5.11 and 5.12). The results shows that the average allowable bearing capacity for foundation soils varies from 125 to 420 kPa for a depth range of 0.5 to 2.5 m and a footing dimension from 0.5 x 0.5 m to 4 x 4m.

Site 2

Site 1 is located in the northern part of the study area and covers locations such as 16 Ayat, Ayat real estate, Woreda 10, Gabriel church and Woreda 10 former Kebele 04. The foundation soils at shallow depth are characterized by soft dark clay (black cotton clay). The soil is loose, reddish brown and has a thickness ranging from 1.3 to 2m. Below these soils light to dark grey soils are present. The ultimate bearing capacity and allowable bearing capacity for soils at Site 2 in the study area was computed by Terzaghi (1943), Hensen (1957, 1970), Vesic (1973), Meyerhof (1963) and EBSC-7 methods at different depths by considering various footing dimensions (Table 5.13 and 5.14). The results shows that the average allowable bearing capacity for foundation soils varies from 110 to 370 kPa for a depth range of 0.5 to 2.5 m and a footing dimension from 0.5 x 0.5 m to 4 x 4m.

Site 3

This site is located in the northern part of the study area and covers important places such as; Ayat condominium, Woreda 10 around Ayat chefe and EADG hospital. The foundation soils present at shallow depth are soft dark clay (black cotton clay). This soil layer is characterized as black, medium stiff, highly plastic silty clay (black cotton soil) extending up to 2.00 m below the ground level. Below these soils light to dark grey soils are present. The results on ultimate bearing capacity and allowable bearing capacity for soils at Site 3 in the study area are presented in Table 5.16 and 5.17. The results shows that the average allowable bearing capacity for foundation soils varies from 100 to 440 kPa for a depth range of 0.5 to 2.5 m and a footing dimension from 0.5 x 0.5 m to 4 x 4m.

Site 4

Site 4 is located in the western part of the study area and covers places such as Summit square and summit condominium. The foundation soil at shallow depth is characterized as light grey to light brown, stiff to very stiff, highly plastic silty CLAY/clayey SILT soils. The results on ultimate bearing capacity and allowable bearing capacity for soils at Site 4 in the study area are presented in Table 5.19 and 5.20. The results shows that the average allowable bearing capacity for foundation soils varies from 100 to 370 kPa for a depth range of 0.5 to 2.5 m and a footing dimension from 0.5 x 0.5 m to 4 x 4m.

Site 5

Site 5 is located exactly in the central part of the study area and covers Ayat 3. The foundation soils at shallow depth are characterized by light to dark grey soil and soft dark clay soil. The dark grey expansive soil extend upto 3 m whereas soft to medium stiff silty clay have a depth upto 0.8 m. The results on ultimate bearing capacity and allowable bearing capacity for soils at Site 5 in the study area are presented in Table 5.22 and 5.23. The results shows that the average allowable bearing capacity for foundation soils varies from 115 to 325 kPa for a depth range of 0.5 to 2.5 m and a footing dimension from 0.5 x 0.5 m to 4 x 4m.

Site 6

This site is located in the eastern part of the study area and cover Bole Ayat 5 area. The foundation soil is characterized as black, soft, highly plastic silty CLAY (Black Cotton soil) extending to a depth that varies from 0.50 to 1.80 m. The results on ultimate bearing capacity and allowable bearing capacity for soils at Site 6 in the study area are presented in Table 5.25 and 5.26. The results shows that the average allowable bearing capacity for foundation soils varies from 170 to 495 kPa for a depth range of 0.5 to 2.5 m and a footing dimension from 0.5 x 0.5 m to 4 x 4m.

Site 9

Site 9 is located in the south-eastern part of the study area and the important places which fall within this site are; Bole Arabsa condominium and areas close to Legetafo River. The foundation soils at shallow depths are characterized as black, silty CLAY (Black cotton soil) which varies in thickness from 0.3 to 2.5m. The results on ultimate bearing capacity and allowable bearing capacity for soils at Site 9 in the study area are presented in Table 5.28 and 5.29. The results shows that the average allowable bearing capacity for foundation soils varies from 160 to 405 kPa for a depth range of 0.5 to 2.5 m and a footing dimension from 0.5 x 0.5 m to 4 x 4m.

Overall Evaluation of bearing capacity of foundation soils

The overall evaluation of bearing capacity of foundation soils at shallow depths in the study area as computed by Terzaghi (1943), Hensen (1957, 1970), Vesic (1973), Meyerhof (1963) and EBSC-7 methods reveals that ultimate bearing capacity in general increase with depth and with increasing footing dimensions (Table 6.2). The results also showed that Terzaghi, EBCS and SPT methods provide bearing capacity values which are comparable within close range. Similarly, Meyerhof, Hansen and Vesic methods also shows comparable bearing capacity values within close range, although these values are higher than the former ones.

Table 6.2 presents average range of ultimate bearing capacity (Qult) as computed by different methods at different depth and variable footing dimensions. Similarly, Table 6.3 presents average range of allowable bearing capacity (Qall) as computed by different methods at different depth and variable footing dimensions.

6.2.2 Bearing capacity of foundation soils based on SPT-N

Allowable bearing pressures for the selected foundation layers shall be discussed based on correlation of the relative compaction of the in-situ ground as indicated from in situ test (SPTN-values). Isolated footings are the simplest to construct and economical type of foundations. The allowable bearing capacity of these types of footings can be determined by using different methods; in-situ tests (SPT N-Values), laboratory tests and visual identification.

In the present study, calculation of the bearing capacity was made with an assumption that the depth of the foundation varies from 2.5 to 5 m and the width of the footing ranges from 2 to 5 m. Allowable bearing capacity (qall) from SPT-N value was computed for 12 locations in the present study and the results are presented in Table 5.31. The result shows that the Allowable bearing capacity (qall) in Site 8 (from where SPT data was made available) varies in a range of 237 to 946 kPa. This range of Allowable bearing capacity (qall) is quite suitable for bearing pressures imposed by G+4 to G+7 buildings, as the bearing pressure for G+4 and G+7 buildings is about 156 kPa to 244 kPa, respectively (Addis Ababa Housing Construction Project Office).

Table 6.2 Average range of Ultimate bearing capacity (Qult) (kPa)

Depth (m)	Footing dimension (m x m)					
	0.5 x 0.5	1 x 1	1.5 x 1.5	2 x 2	2.5 x 2.5	4 x 4
Site 1 -						
0.5	587 - 886	597 - 908	605 – 909	605 – 916	602 – 912	612 - 928
1	607–914	627–917	630 – 923	630 – 933	627 - 931	633- 941
1.5	687–936	633 –942	638–948	635 – 946	631 - 944	638 - 963
2	643 - 956	651 - 973	657 – 981	680 – 987	680 – 996	698- 1008
2.5	685 - 976	711 - 973	713 – 993	696 – 1004	701 - 1025	720 – 1026
Site 2						
0.5	405 - 605	405- 730	410- 750	410- 756	410- 770	410– 790
1	411- 613	425- 763	420- 781	430- 786	420– 780	440– 800
1.5	421– 623	435– 783	443– 791	444– 796	440– 800	460– 810
2	451– 653	465– 790	470– 800	470– 770	460– 800	470– 820
2.5	490- 693	500- 800	490- 810	500– 820	480– 810	490– 840
Site 3						
0.5	530- 930	530– 940	530- 960	535– 965	540- 970	540– 970
1	550- 950	560- 970	570- 990	570- 1000	575– 1010	560- 1000
1.5	570– 970	580– 990	590– 1000	600– 1020	610– 1040	610- 1010
2	590– 1000	600- 1020	610– 1060	610- 1050	620– 1070	615- 1050
2.5	600- 1020	610- 1050	630–1100	620- 1070	650– 1100	630- 1070
Site 4						
0.5	390 - 745	390- 755	400 - 760	405 – 770	410 - 790	405 - 770
1	400–765	405–775	410 - 780	415- 790	420 - 800	415 - 790
1.5	410 - 785	415 - 795	420 - 800	425 – 810	430 - 820	435 - 830
2	430 - 795	435 - 805	440 - 810	445 – 820	450 - 840	445 - 850
2.5	450 - 800	455 - 815	460 - 820	465 – 840	470 - 860	465 - 840
Site 5						
0.5	470 - 600	475 - 605	480–610	490– 620	505– 630	505– 635
1	490–620	495- 630	500 - 640	510– 650	515– 660	520– 665
1.5	510 - 640	515 - 650	520 - 660	530- 670	535– 680	540– 680
2	530 - 660	535 - 670	540 - 690	530– 680	555– 710	560– 710
2.5	550 - 680	555 - 690	560 - 700	540- 690	575- 730	580–735
Site 6						
0.5	690–1110	695- 1110	700 - 1140	705 – 1170	705 - 1190	700 - 1170
1	705–1130	710–1140	715 - 1160	715 – 1190	720- 1200	740 - 1190
1.5	715–1150	720 - 1160	730 - 1180	735 – 1200	740- 1220	760 - 1210
2	735– 1170	740- 1190	750- 1205	760 – 1230	770 - 1245	775 - 1220
2.5	755–1190	760 - 1210	770 - 1230	775 – 1245	785- 1265	790 - 1250
Site 9						
0.5	675 – 850	680- 860	695- 870	700– 880	700– 890	700 - 890
1	695– 875	700– 885	705– 890	710– 900	715– 900	720 - 895
1.5	715– 900	720– 910	725– 920	730– 940	735– 950	740 – 915
2	635– 915	740– 930	745– 940	750– 960	755– 970	750 – 935
2.5	655- 930	760– 950	765– 960	770- 980	775 - 990	760 - 975

6.3 Bearing capacity zonation in the study area

In the present study an attempt was made to prepare bearing capacity zonation by considering a representative 1.5 m footing depth and by considering various footing dimensions.

Table 6.3 Average range of Allowable bearing capacity (Qall) (kPa)

Depth (m)	Footing dimension (m x m)					
	0.5 x 0.5	1 x 1	1.5 x 1.5	2 x 2	2.5 x 2.5	4 x 4
Site 1						
0.5	125 - 235	130 – 240	140 - 250	140 -250	140 - 245	145- 260
1	135 - 255	165 – 275	180 – 290	180 – 295	180 - 280	180 - 300
1.5	150 - 270	180 – 290	210- 320	210 - 320	210 - 315	210 – 330
2	150 - 280	190 – 300	210 – 310	250- 360	245 – 350	280 - 400
2.5	160 – 290	230 – 340	280 – 380	280 – 390	290 – 400	290 - 420
Site 2						
0.5	110 - 200	110 - 205	100 - 200	100 – 210	100 – 220	110 – 240
1	110 - 210	110 - 230	115 - 245	115 – 250	120 – 250	145 – 260
1.5	120 - 220	140 - 240	140 - 270	140 – 260	140 – 265	150 – 270
2	150 - 250	170 - 270	170 - 280	180 – 280	190 – 320	205 – 340
2.5	190 - 320	200 - 340	210 - 340	220 – 340	230 – 360	250 – 370
Site 3						
0.5	100 - 255	100 - 270	100 - 280	100 – 285	100 – 290	100 – 310
1	100 – 265	100 - 300	100 - 310	110 – 320	110 – 320	135 – 345
1.5	110 - 330	110 – 320	130 - 340	130 – 335	130 – 330	130 – 340
2	120 - 340	150 – 360	170 - 370	170 – 380	180 – 390	200 – 410
2.5	180 - 380	200 – 410	210 - 410	210 – 420	220 – 430	230 – 440
Site 4						
0.5	100 - 200	100 - 200	100 – 200	100 - 210	100 – 220	110 – 240
1	100 - 220	100 - 230	110 – 240	120 - 250	120 – 260	145 – 280
1.5	110 - 250	120 - 260	140- 270	130 - 270	130 – 270	130 – 280
2	130 - 270	140 - 290	160 – 300	180 - 310	190 – 320	210 – 340
2.5	170 - 290	200 - 310	210 – 330	210 - 340	220 – 350	240 – 370
Site 5						
0.5	115- 180	110 - 185	110 – 170	115 – 215	120 – 220	140 – 220
1	120 - 190	130 - 205	140 – 210	150 - 210	155 – 210	180 – 230
1.5	130 - 220	150 - 220	175- 225	175- 225	175- 230	180- 235
2	160 - 230	180 - 250	200 – 260	210 - 270	220 – 275	230 – 300
2.5	180 -260	220 - 270	230 – 290	240 - 290	260 - 305	275 – 325
Site 6						
0.5	170 – 320	180 - 320	190 - 325	205 - 330	220 – 340	230 – 360
1	190 – 350	225 - 345	230 - 350	240 - 370	245 – 375	270 – 400
1.5	210 – 360	230 - 370	265- 395	265- 390	260 – 385	265- 390
2	230 – 370	260 - 430	280 - 430	305 - 440	310 – 440	320 – 460
2.5	260 – 390	290 - 415	310 - 420	320 - 430	330 – 470	365 – 495
Site 9						
0.5	160 - 250	160 – 250	170 - 245	190 – 300	190 – 300	210 – 290
1	180 - 290	200 – 290	210 - 290	215 - 290	220 – 290	245 – 310
1.5	220 - 300	220 – 300	240- 305	240- 305	240- 305	240- 315
2	230 - 310	240 – 320	250 - 330	280 - 350	290 – 355	300 – 380
2.5	250 - 330	280 – 350	305 - 375	310 - 380	320 – 390	340 – 405

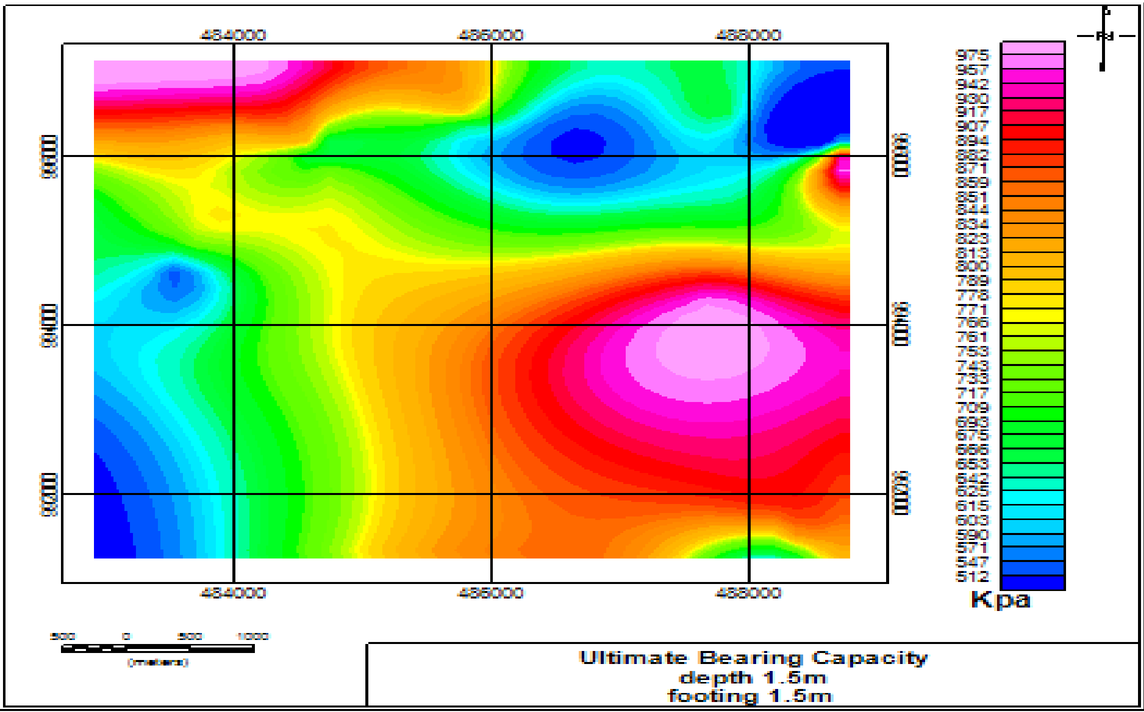
For the zonation average Qult and Qall values obtained from various methods were considered. The footing dimensions considered for zonation purpose are 1.5 x 1.5 m, 2 x 2 m, 2.5 x 2.5 m and 4 x 4 m. For each footing dimension zonation maps were produced for both ultimate bearing capacity (Qult) and allowable bearing capacity (Qall), separate. The primary purpose of preparing these bearing capacity zonation maps is to provide a spatial distribution of bearing capacity range over the area which may provide firsthand information on rough estimates on bearing capacity. However, systematic detailed estimation of bearing capacity would still be required for the design purpose. The estimates provided through above mentioned zonation need to be considered as indicative only.

6.3.1 Bearing capacity zonation for 1.5 x 1.5 m footing at 1.5 m depth

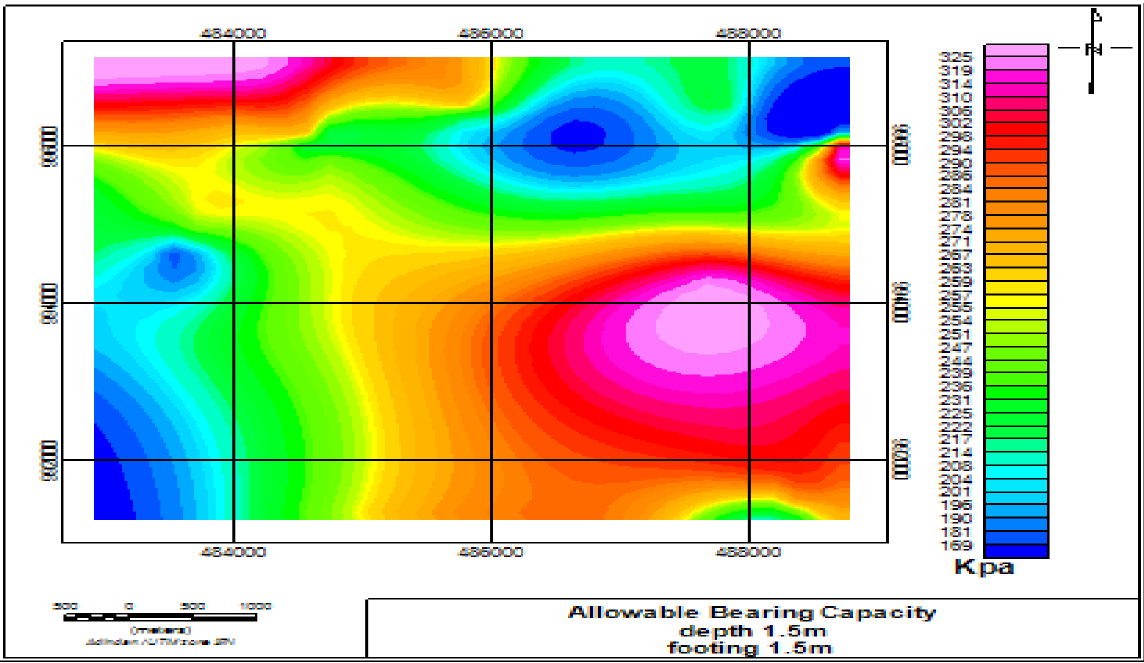
The bearing capacity zonation for 1.5 x 1.5 m footing at 1.5 m depth is presented in Fig. 6.1. As can be seen from this map (Fig. 6.1) the highest values of bearing capacity are in the North-west and Eastern parts of the study area. The soils in these areas are characterized by high content of fine grain soil and are high plasticity. The ultimate bearing capacity and allowable bearing capacity values ranges from 930 to 975 kPa and 310 to 325 kPa, respectively. The lowest value of bearing capacity is in the North-east and South-west parts of the study area. The soils in these areas are characterized by fine gained soil with low to high plasticity. The ultimate bearing capacity and allowable bearing capacity value ranges 512 to 590 kPa and 159 to 195 kPa, respectively. The central part of the study area demonstrates intermediate values of bearing capacity. The soil in this particular area is characterized by fine gained soil of low plasticity. The ultimate bearing capacity and allowable bearing capacity value range from 766 to 840 kPa and 250 to 270 kPa, respectively.

6.3.2 Bearing capacity zonation for 2 x 2 m footing at 1.5 m depth

The bearing capacity zonation for 2 x 2 m footing at 1.5 m depth is presented in Fig. 6.2. As can be seen from this map (Fig. 6.2) the highest value of bearing capacity is in the North-west and South-east parts of the study area. The soil in this particular area is characterized by high content of fine grain soil with high plasticity. The ultimate bearing capacity and allowable bearing capacity value ranges from 925 – 970 kPa and 309 to 324 kPa, respectively. The lowest value of bearing capacity is in the North-east and South-west parts of the study area. The soil in this particular area is characterized by fine gained soil with low to high plasticity. The ultimate bearing capacity and allowable bearing capacity value ranges from 505 to 580 kPa and 169 to190 kPa, respectively. The central part of the study area demonstrates intermediate value of bearing capacity. The soil in this particular area is characterized by fine gained soils with low plasticity. The ultimate bearing capacity and allowable bearing capacity value ranges from 760 to 800 kPa and 255 to 270 kPa, respectively.



A. Ultimate Bearing Capacity

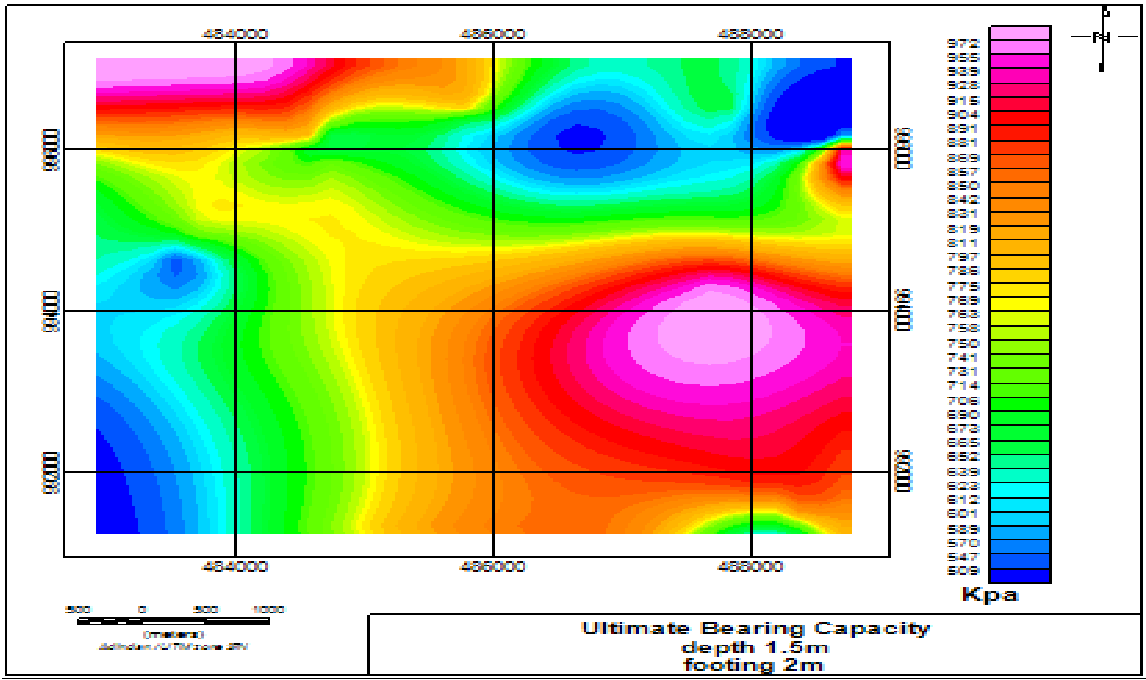


B. Allowable Bearing Caapacity

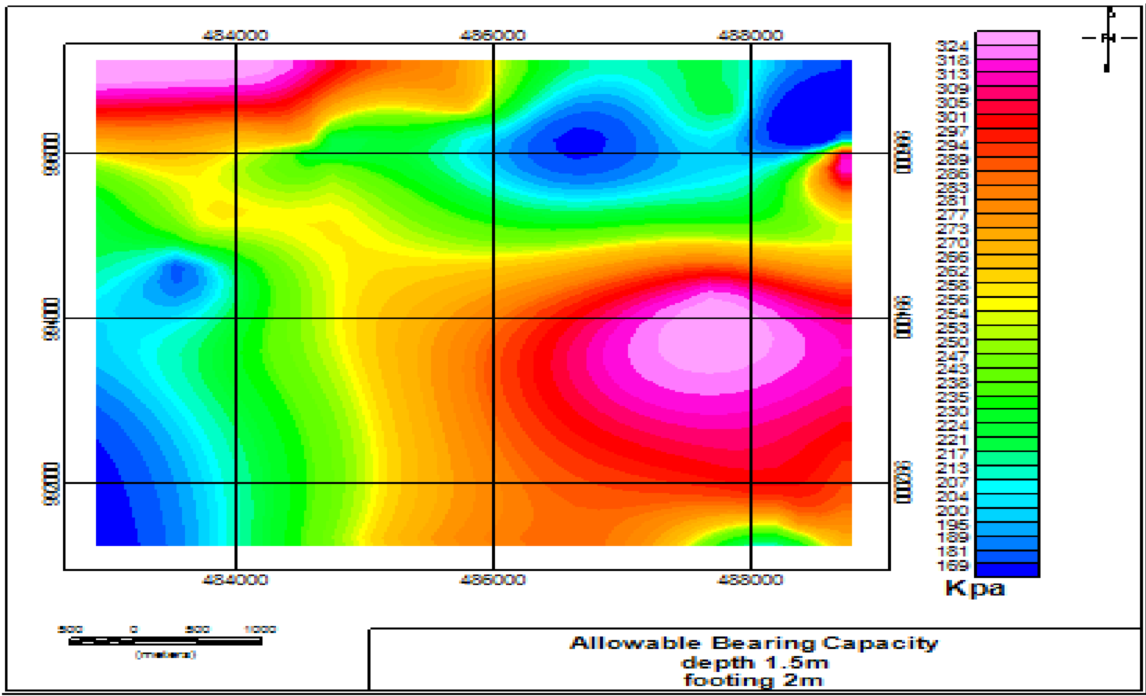
Fig. 6.1 Bearing capacity zonation of the study area (Footing 1.5 x 1.5 m, Depth 1.5 m)

6.3.3 Bearing capacity zonation for 2.5 x 2.5 m footing at 1.5 m depth

The bearing capacity zonation for 2.5 x 2.5 m footing at 1.5 m depth is presented in Fig. 6.3. As can be seen from this map (Fig. 6.3) the highest value of bearing capacity is in the North-west and South-east parts of the study area. The soil in this particular area is characterized by high content of fine grained soil with high plasticity. The ultimate bearing capacity and allowable bearing capacity value ranges from 925 – 972 kPa and 315 to 325 kPa, respectively.

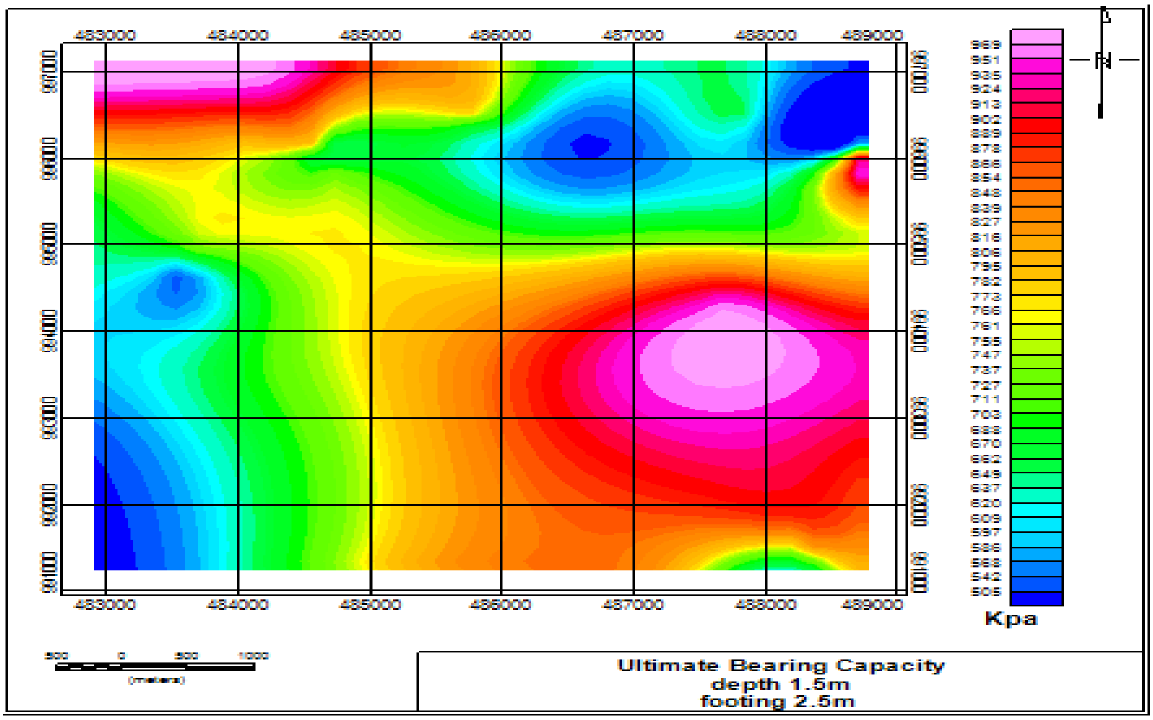


A. Ultimate Bearing Capacity

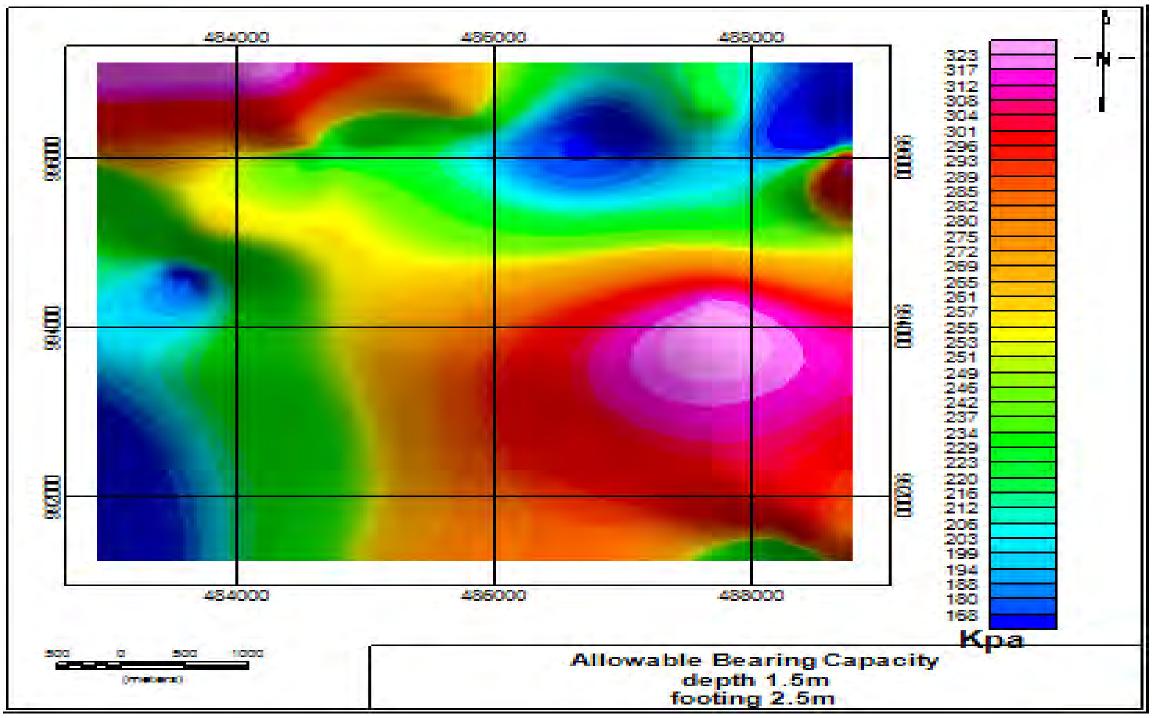


B. Allowable Bearing Capacity

Fig. 6.2 Bearing capacity zonation of the study area (Footing 2 x 2 m, Depth 1.5 m)



A. Ultimate Bearing Capacity



B. Allowable Bearing Capacity

Fig. 6.3 Bearing capacity zonation of the study area (Footing 2.5 x 2.5 m, Depth 1.5 m)

The lowest value of bearing capacity is in the North-east and South-west parts of the study area. The soil in this particular area is characterized by fine gained soils of low to high plasticity. The ultimate bearing capacity and allowable bearing capacity value ranges from 505 -580 kPa and 159 – 190 kPa, respectively. The central part of the study area demonstrates intermediate value of bearing capacity. The soil in this particular area is characterized by fine gained soil of low plasticity. The ultimate bearing capacity and allowable bearing capacity value ranges from 770 to 800 kPa and 255 to 270 kPa, respectively.

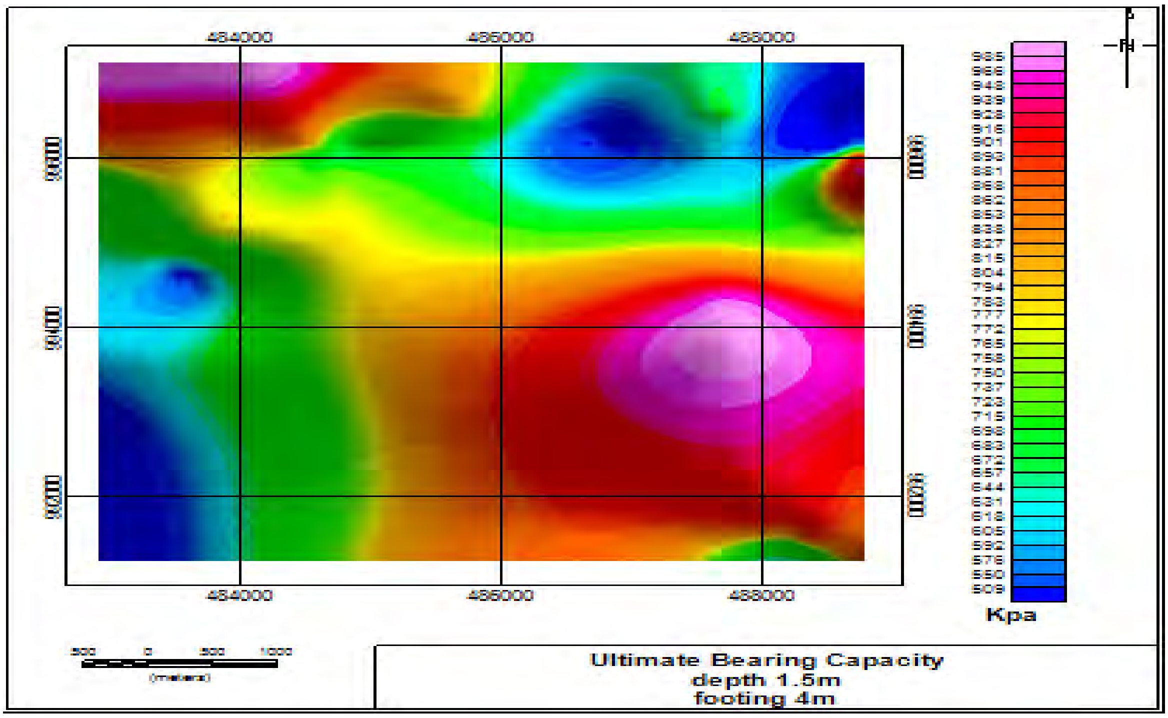
6.3.4 Bearing capacity zonation for 4 x 4 m footing at 1.5 m depth

The bearing capacity zonation for 4 x 4 m footing at 1.5 m depth is presented in Fig. 6.4. As can be seen from this map (Fig. 6.4) the highest value of bearing capacity is in the North-west and South-east parts of the study area. The soil in this particular area is characterized by high content of fine grained soil with high plasticity. The ultimate bearing capacity and allowable bearing capacity value ranges from 950 to 990 kPa and 315 to 330 kPa, respectively. The lowest value of bearing capacity is in the North-east and South-west parts of the study area. The soil in this particular area is characterized by fine gained soil of low to high plasticity. The ultimate bearing capacity and allowable bearing capacity value ranges from 510 – 570 kPa and 170 – 190 kPa, respectively. The central part of the study area demonstrates intermediate value of bearing capacity. The soil in this particular area is characterized by fine gained soil of low plasticity. The ultimate bearing capacity and allowable bearing capacity value ranges from 770 – 850 kPa and 260 to 270 kPa, respectively.

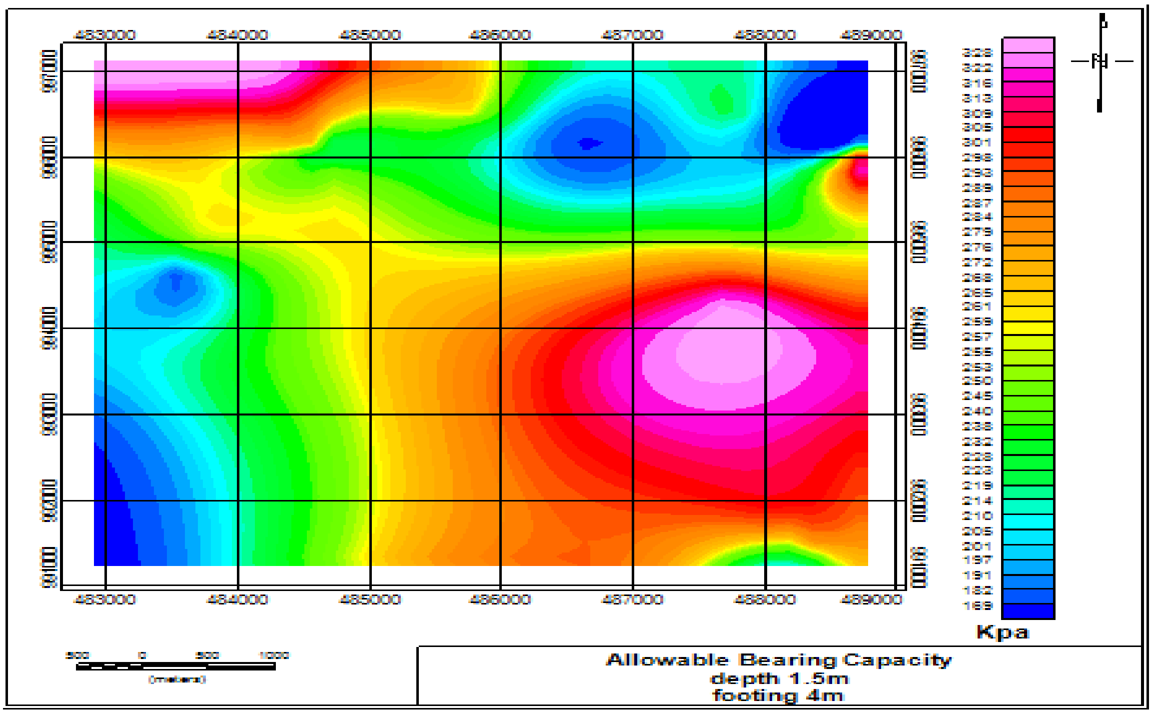
6.4 Foundation Settlement Analysis

In the design of any foundation, one must consider the safety against bearing capacity failure as well as against excessive settlement of the foundation. In the design of most of the foundations, there are specifications for allowable levels of settlement. The foundation soil in its nature is not susceptible for immediate settlement hence only the settlement from primary consolidation is considered in this study. In order to select the appropriate consolidation parameters, laboratory consolidation test result were considered. The test results on foundation settlement are already presented in chapter 5.

From the results the estimated total settlement is not within the limits of acceptable settlement. The results showed that settlement at Site 1, Site 3 and Site 5 varies in a range from 89 to 245 mm, 41 to 124 mm and 74 to 136 mm, respectively. For mat foundation, a maximum allowable average settlement up to 100 mm is tolerated for framed structures (U.S Army Corps of Engineers, 1990). For isolated footing, a maximum allowable average settlement up to 50 mm is tolerated for framed structures. As per Ethiopian Buiding code allowable settlement for sands is 50 mm whereas for clay soils it is 75 mm.



A. Ultimate Bearing Capacity



B. Allowable Bearing capacity

Fig. 6.4 Bearing capacity zonation of the study area (Footing 4 x 4 m, Depth 1.5 m)

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CHAPTER SEVEN CONCLUSION AND RECOMMENDATIONS

7.1 Conclusion

In Addis Ababa city there is a general increase in building construction with ever increasing population at a very faster rate. In recent years several high rise buildings, mass housing projects and other infrastructures are being constructed by private companies and government agencies. For the safety and stability of building structures it is essential to undertake systematic geotechnical characterization and foundation condition evaluation. The performance of any building will entirely depend on the foundation conditions and its behavioral characteristics in terms of its bearing capacity and the settlement potential.

The present research study was conducted in North-Eastern part of Addis Ababa, Bole Sub City Bole Ayat area. The study area is geographically bounded in between 997239.541m N - 990136.243m N latitude and 482785.03m E- 489343.598m E longitude at UTM- Zone -37N. The total area covered by the study area is 40 km^2. The main objective of the present research work was to characterize the building foundation material and to assess its bearing capacity and settlement potential in the study area.

In order to achieve the objectives of the present study, systematic literature review of previous works, secondary data collection, filed observation and laboratory tests were conducted. Representative samples were also collected and analyzed. Secondary data in the form of borehole logs, test reports and other documents on site investigations were procured from several companies and organizations. Test pit logging, sampling and testing were also undertaken to generate primary data. The secondary data was processed and all relevant parameter data/ information necessary for bearing capacity and settlement analysis was obtained. For the purpose of characterization and foundation analysis the study area was divided into 9 sites; Site 1 to Site 9. This was mainly done so that the data can be well managed and systematic analysis can be made.

In the present study area two prominent type of soils and two types of rock units are present. The soil types are; light to dark grey silty clay/ clayey silt soil and soft dark, silty clay (Black Cotton) soil. Among the rocks; weak tuff layer and weak to medium strong ignimbrite rocks are present.

From the present research study following may be concluded;

Sites 1, 4 and 7 are characterized by light to dark grey, stiff to very stiff and highly plastic silty CLAY/clayey SILT soil. In general the results showed that these soils are very high plastic and are fine grained expansive in nature. This soil was encountered underlying by pyroclastic ash and tuff deposits and reaches to the maximum drilled depth. The thickness of this soil layer varies from 2 to 10.7 m as observed from the logs of various boreholes. Further, Sites 2, 3,5,6,7,8 and 9 are characterize by black, soft to stiff consistency soils with very loose to medium dense relative density, medium stiff, highly plastic silty CLAY (Black cotton) soils. The thickness of this soil layer varies from 0.3 to 3m in the study area, as observed from the logs of various boreholes and within the test pits made during the present study.

The results showed that the moisture content of light to dark grey soils ranges from 30 to 45%. The unit weight of light to dark clay soils ranges from 15 to 21 kN/m^3. Further, the liquid limit (LL) and plasticity index (PI) for these soils varies from of 87 to 113% and 47 to 80%, respectively. Also, the free swell (FS) for these soils varies from 90 to 180 %. The void ratio also indicates the softness or stiffness of a soil. Void ratio of light to dark grey soil ranges from 0.0473 -0.0845. Thus, based on LL, PI and FS values light to dark soils in the study area are problematic. The results in general show that these soils are very high plastic and are fine grained expansive in nature. As per Unified Soil Classification System (USCS) these soils may be placed under MH and CH types of soils. Further, in situ Standard Penetration Test (SPT) results indicate that the N value for these soils varies from 6 to 25 blows which represents that light to dark grey soils in the study area are medium stiff to very stiff soils.

As per the results, soft dark clay soils may be placed mainly under ML, MH and CL types of soils (USCS Classification system) (Fig. 5.3). The natural moisture content varies from 26 to 61%. Further, the liquid limit and plasticity index for these soils varies from of 37 to 89 % and 7 to 47 %, respectively. The free swell varies from 90 to 190 %. Thus these soils show very high potential for volume change. Further, in situ Standard Penetration Test (SPT) results indicate that the N value varies from 4 to 17 blows which suggest that these soils are soft to stiff in its consistency and are very loose to medium dense with respect to its relative density

The weak tuff layer in general is very weak, dark grey, moderate to highly weathered, fractured, friable welded or medium dense to very dense tuff. The rock quality designation

(RQD) values for week tuff rock as observed from various borehole log reports shows that it is less than 25 %, indicating very week rock.

The ignimbrite rock layer in general is light grey in color¸ slightly to moderately weathered, closely jointed/ fractured, medium grained and weak to medium strong. The rock quality designation (RQD) values as observed from various borehole log reports shows that it varies from 0 to 80 % and in majority of cases it is less than 50%.

As per the borehole log data, groundwater was not encountered in any of the boreholes. The investigation revealed that there was no ground water within the depth of investigation and is well below the depth of drilling (maximum drilled depth 15m). Thus, groundwater will not have any effect on shallow founded buildings in the study area.

For the present study, bearing capacity was estimated for 66 locations and necessary soil data required for assessment was taken from boreholes logs and test reports. Besides, estimation on bearing capacity was also made from SPT data for which average N values were taken from 12 locations. The bearing capacity was estimated through various analytical techniques as proposed by; Terzaghi (1943), Hanson (1970), Meyerhof (1951) Vesic (1973), National building code standards (EBCS-7(1995) and Semi-empirical (SPT) approach. The bearing capacity computations were done by using program written in Microsoft Excel. For the present study the bearing capacity was only compute for square footings. Also, it was assumed that only vertical static loads will be acting on the foundation and ground surface is horizontal. It is also assumed that ground water table is deep and it may not affect the foundation soils. For the present study ultimate bearing capacity (Qult) and allowable bearing capacity (Qall) were computed by taking variable depths (0.5 to 4 m) and different footing dimensions (0.5 x 0.5 m, 1 x 1 m, 1.5 x 1.5 m, 2 x 2 m and 2.5 x 2.5 m). In order to compute allowable bearing capacity (Qall) a factor of safety of 3 was considered for the present study.

The overall evaluation of bearing capacity of foundation soils at shallow depths in the study area as computed by Terzaghi (1943), Hensen (1957, 1970), Vesic (1973), Meyerhof (1963) and EBSC-7 methods reveals that ultimate bearing capacity in general increase with depth and with increasing footing dimensions. The results also showed that Terzaghi, EBCS and SPT methods provided bearing capacity values which are comparable within close range. Similarly, Meyerhof, Hansen and Vesic methods also showed comparable bearing capacity values within close range, although these values are higher than the former ones. Further, bearing capacity zonation by taking 1.5m foundation depth is also forwarded through the present study. The allowable bearing capacity (Qall) range for various sites in the study area can be summarized as;

- ✓ Site 1 - 125 to 420 kPa
- ✓ Site 2 - 110 to 370 kpa
- ✓ Site 3 - 100 to 410 kpa
- ✓ Site 4 - 100 to 370 kpa
- ✓ Site 5 - 115 to 325 kpa
- ✓ Site 6 - 170 to 495 kpa
- ✓ Site 9 - 160 to 405 kpa

Settlement estimation was conducted based on consolidation results of laboratory tests and EBSC-7, (1995) method. The results showed that settlement at Site 1, Site 3 and Site 5 varies in a range from 89 to 245 mm, 41 to 124 mm and 74 to 136 mm, respectively. From the results the estimated total settlement is not within the limits of allowable settlement.

7.2 Recommendation

Based on the present study results following recommendations are forwarded;

The present study was conducted by generating a geotechnical data base from disorganized historical borehole log and laboratory test results. It was a very tedious and time consuming effort to filter and process these data for required input parameters. Therefore, it is required that the enterprises/ organizations in similar tasks should be committed in building a quality data base and provide support to such endeavors.

In the present study settlement potential was calculated based on available borehole data however the required data was not complete. Therefore, for better understanding of settlement potential future studies need to be accomplished by incorporating all available historical borehole data from different organizations.

In the present study bearing capacity was calculated at different depths ranging from 0.5 to 4m depth only, depending on available important soil properties data. However, if bearing capacity for high rise building needs to be determined, computations should be made beyond 4m. Further in the present study, isolated foundations were only considered to support the proposed structures depending on the super structure anticipated loads. Therefore, mat foundations can be studied in the future studies.

Presence of ground water near the foundation level in a cohesive soil layer reduces the bearing capacity of the foundation layer and it causes differential settlement when there is swelling and shrinking due to moisture fluctuation. Clay soils, like black cotton soils, are seasonally affected by drying, shrinkage and cracking in dry and hot weather, and by swelling in the following wet weather to a depth which will vary according to the nature of the clay and the climatic condition of the region. Since ground water is not observed up to the depth of investigation, moisture fluctuation in the foundation layer could only take place from surface water. Therefore, it is needed to improve the drainage conditions for the expansive soil in layer 1 in the present study area.

Addis Ababa is situated in the seismically active Ethiopian rift margin which is tectonically active area therefore, proper seismic factor should be considered in the building foundation design.

The results from the present study provide a spatial distribution of bearing capacity range over the area which may provide firsthand information on rough estimates on bearing capacity. However, systematic detailed estimation of bearing capacity would still be required for the design purpose. The estimates provided through present study need to be considered as indicative only.

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Annex 1 **Summary of results for Ultimate bearing capacity (Qult) for various Site 1**

Sites	Depth (m)	Footing dimension (m x m)					
		0.5 x 0.5	1 x 1	1.5 x 1.5	2 x 2	2.5 x 2.5	4 x 4
		Range of Ultimate bearing capacity (Qult) (KPa)					
Terzagi	**0.5**	507 - 764	508 – 761	486 – 746	512 -745	539 -824	572 – 861
	1	527 - 794	553- 786	526 – 771	547 – 785	559- 839	612 – 901
	1.5	547 - 804	558 – 811	561 – 816	572 – 835	584 – 854	617 – 911
	2	567 - 824	578 - 841	581 - 851	612 - 885	634 – 919	687 – 971
	2.5	607 – 844	638 – 861	611- 861	627- 895	654 – 949	717- 1001
Henson	**0.5**	675 – 1024	661-1012	634 – 987	606 – 905	597 – 921	567- 847
	1	695- 1054	706- 1037	674 – 1012	641 – 945	617 – 946	607- 914
	1.5	715 - 1064	711 - 1062	709 – 1057	666 – 995	642 – 961	612 – 924
	2	735 – 1084	731 - 1092	729- 1092	706 - 1045	692 – 1026	682 – 984
	2.5	775 -1104	791 – 1112	759- 1102	721-1055	712 – 1056	712-1014
Vesics	**0.5**	631 – 966	629- 967	608 – 952	640 - 737	620 -965	604 – 933
	1	651- 996	674 -992	648- 977	665- 977	641 – 985	644 – 973
	1.5	671 - 1006	679 -1017	683 - 1022	685 - 1027	666 -1000	649 – 983
	2	691-1026	689 - 1047	703 - 1057	725- 1077	716 -1065	719- 1043
	2.5	731-1046	749 – 1067	733 – 1067	740- 1087	736- 1095	749 - 1073
Meyerhof	**0.5**	651 - 996	658 - 999	668 – 1005	673 - 1013	682 – 1021	691 - 1031
	1	671 - 1014	693 – 1016	695 – 1023	702 - 1033	714–1039	732 - 1043
	1.5	691 - 1034	698 - 1041	706 - 1053	714 - 1070	722–1087	747- 1138
	2	711 - 1054	718 – 1073	726 – 1081	755 – 1089	766 – 1099	787 - 1139
	2.5	755 - 1074	778 - 1093	779 - 1102	784 - 1119	792 – 1138	801 - 1151
EBSC-7	**0.5**	471 - 682	473 – 731	576- 735	584 - 739	584–768	586 - 773
	1	491–712	515 – 756	516 -760	519 - 764	520–797	555 - 845
	1.5	511 - 772	520–781	530 - 791	537 - 804	543–817	563 - 858
	2	531 - 792	540 – 811	550 – 826	577 – 854	593–872	613 - 889
	2.5	561 - 812	600 - 831	610 - 836	611 - 864	613–884	621 - 893

Annex 2 **Summary of results for Allowable bearing capacity (Qall) for Site 1**

Sites	Depth (m)	Footing dimension (m x m)					
		0.5 x 0.5	1 x 1	1.5 x 1.5	2 x 2	2.5 x 2.5	4 x 4
		Range of Allowable bearing capacity (Qult) (KPa)					
Terzagi	**0.5**	125 – 198	132 - 192	133 - 205	135 - 207	145– 256	157 – 266
	1	145 – 218	177 – 205	152 – 225	165–228	175 –271	197 – 289
	1.5	165 - 228	182 - 230	187 - 234	190 - 278	194 - 284	205 – 303
	2	185 - 248	202 – 255	207 – 269	213 – 289	227–327	235–343
	2.5	201 - 272	240 - 275	240 - 279	257 - 309	266–335	287 – 357
Henson	**0.5**	174 - 300	202 - 306	206- 318	209 - 322	211- 316	198 – 304
	1	194 – 330	232 - 334	231 - 339	219 – 330	212 – 318	201 – 306
	1.5	216 - 340	237 - 345	236 - 352	222 - 331	214–320	204–308
	2	236 – 360	257 - 365	269 - 387	270 – 397	278–408	289–408
	2.5	276 - 380	286 - 384	293 - 396	299 - 407	308–415	321–426
Vesics	**0.5**	132 - 270	160 - 291	166 - 293	173 - 297	165 – 301	167 – 308
	1	152 – 300	205 – 314	206 – 315	208 – 319	206 – 315	205–312
	1.5	172 - 310	210 - 339	227 - 340	228 - 342	222- 333	216 – 327
	2	192 - 330	230 - 369	257 - 375	248 - 392	272 – 398	286–403
	2.5	230 - 350	260 - 389	287 - 395	295 - 402	304 – 413	326–435
Meyerhof	**0.5**	165 - 309	185 - 311	196 - 326	209 – 336	213 – 344	224 – 362
	1	185 – 329	220 – 330	221 – 343	223 - 343	234 – 352	239 – 367
	1.5	205 - 339	225 - 345	235 - 351	238 - 356	240 – 362	249–379
	2	225 - 359	245 - 375	255 – 386	263 - 403	290–423	299- 436
	2.5	265 - 379	285 - 395	295 - 406	302 - 413	312 – 433	321–453
EBSC-7	**0.5**	117 - 212	126 - 216	134 – 218	143 - 224	156–227	165 - 239
	1	137 – 242	166 - 227	167 - 231	172 - 233	176–245	184 - 265
	1.5	157 - 252	171 - 257	176 - 263	179 - 268	181–272	187 - 286
	2	177 – 272	191 - 287	196- 298	212 – 318	245 – 335	267 – 337
	2.5	197 - 292	208 - 308	224 - 312	227 - 328	251 – 357	274 - 359

Annex 3 **Summary of results for Ultimate bearing capacity (Qult) for Site 2**

Sites	Depth (m)	Footing dimension (m x m)					
		0.5 x 0.5	1 x 1	1.5 x 1.5	2 x 2	2.5 x 2.5	4 x 4
		Range of Allowable bearing capacity (Qult) (KPa)					
Terzagi	**0.5**	331 - 642	328 - 637	308 - 623	336 - 614	363 - 674	400 – 697
	1	351 - 672	373 - 662	348 - 648	371 - 654	383 - 699	440 – 737
	1.5	371 - 682	378 - 687	383 - 693	396 - 704	408 - 714	445 – 747
	2	391 - 702	398 - 717	403 - 728	436 - 754	458 - 779	515 – 807
	2.5	431 - 722	458 -737	433 - 738	451 - 764	478 - 809	545 – 837
Henson	**0.5**	423 - 821	429 - 815	413 - 806	402 - 731	403 - 750	390 – 699
	1	443 - 851	474 - 840	453 - 831	437 - 771	423 - 775	430 – 739
	1.5	463 - 861	479 - 865	488 - 876	462 - 821	448 - 790	435 – 749
	2	483 -881	499 -895	508 - 911	502 - 871	498 - 855	505 – 809
	2.5	523 - 901	559 -915	538 - 921	517 - 881	518 - 885	535 – 389
Vesics	**0.5**	429 - 789	424 - 784	403 - 770	423 - 750	429 - 773	430 – 735
	1	449 -819	469 - 809	443 - 795	458 - 790	449 - 798	470 – 775
	1.5	469 - 829	474 - 834	478 - 840	483 - 840	474 - 813	475 – 785
	2	489 - 849	494 - 864	498 - 875	523 - 890	524 - 878	545 – 845
	2.5	529 - 869	554 - 884	528 - 885	538 - 900	544 - 908	575 – 875
Meyerhof	**0.5**	427 - 839	428 - 811	414 - 800	438 - 788	463 - 846	491 – 860
	1	447 -840	473 - 836	454 - 825	473 - 828	483 - 871	531 – 900
	1.5	467 – 850	478 - 861	489 - 870	498 - 878	508 - 886	536 – 910
	2	487 - 870	498 - 891	509 - 905	538 - 928	558 - 951	606 – 970
	2.5	527 - 890	558 - 911	539 - 915	553 - 938	578 - 981	636 – 1000
EBSC-7	**0.5**	318 - 621	319 - 622	302 - 610	325 - 445	348 - 501	371 – 496
	1	338 - 651	364 - 647	342 - 635	360 - 485	368 - 526	411 – 536
	1.5	358 – 661	369 - 672	377 - 680	385 - 535	393 - 541	416 – 546
	2	378 - 681	389 - 702	397 - 715	425 - 585	443 - 606	486 – 606
	2.5	418 - 701	449 - 722	427 - 725	440 - 595	463 - 636	516 – 636

Annex 4 **Summary of results for Allowable bearing capacity (Qall) for Site 2**

Sites	Depth (m)	Footing dimension (m x m)					
		0.5 x 0.5	1 x 1	1.5 x 1.5	2 x 2	2.5 x 2.5	4 x 4
		Range of Allowable bearing capacity (Qult) (KPa)					
Terzagi	**0.5**	79 - 188	74 - 178	52 – 161	72 – 144	91 - 198	103 - 199
	1	99 - 214	119 - 203	92 – 186	107 – 184	111 - 223	143 - 239
	1.5	119 - 224	124 - 228	127 – 231	132 – 234	136 - 238	148 - 249
	2	139 - 244	144 - 258	147 – 266	172 – 284	186 -303	218 - 309
	2.5	179 - 264	204 - 278	177 – 276	187 – 294	206 - 333	248 - 339
Henson	**0.5**	102 - 232	103 - 233	87 – 222	94 – 183	104 - 223	104 - 199
	1	122 - 262	148 - 258	127 – 247	129 – 223	124 - 248	144 - 239
	1.5	142 - 272	153 - 283	162 – 292	154 – 273	149 - 263	145 - 249
	2	162 - 292	173 - 313	182 – 327	194 -323	199 - 328	215 - 309
	2.5	202 - 212	233 - 333	212 – 337	209 – 333	219 - 358	245 - 339
Vesics	**0.5**	99 - 220	99 - 220	84 – 210	101 – 190	113 - 231	113 - 211
	1	119 -250	144 - 245	124 – 235	136 – 230	133 - 256	153 - 251
	1.5	139- 260	149 - 270	159 – 280	161 – 280	158 - 271	158 - 261
	2	159 - 280	169 - 300	179 – 315	201 – 330	208 - 336	228 - 321
	2.5	199 - 300	229 - 320	209 – 325	216 – 340	228 - 366	258 - 351
Meyerhof	**0.5**	110 - 259	107 - 236	88 – 220	106 – 202	124 - 255	133 - 253
	1	130 - 289	152 - 261	128 – 245	141 – 242	144 - 280	173 - 293
	1.5	150 - 279	157 -286	163– 290	166 – 292	169 - 295	178 – 303
	2	170 - 259	177 - 316	183 – 325	206 – 342	219 - 360	248 - 363
	2.5	210 - 239	237 - 336	213 -335	221 – 352	239 - 390	278 - 393
EBSC-7	**0.5**	76 - 168	72 - 164	50 – 146	68 – 128	86 - 180	101 - 176
	1	96 - 198	117 - 189	90 – 171	103 – 168	106 - 205	141 - 216
	1.5	116 - 208	122 - 214	125 – 216	128 – 218	131 - 220	146 – 226
	2	136 - 228	142 - 240	145 – 251	168 – 268	278 - 285	216 - 286
	2.5	176 - 248	202 - 260	175 – 261	183 – 278	298 - 315	246 - 316

Annex 5 **Summary of results for Ultimate bearing capacity (Qult) for Site 3**

Sites	Depth (m)	Footing dimension (m x m)					
		0.5 x 0.5	1 x 1	1.5 x 1.5	2 x 2	2.5 x 2.5	4 x 4
		Range of Ultimate bearing capacity (Qult) (kPa)					
Terzagi	**0.5**	609 - 790	607 - 795	594 – 790	628 – 790	651 - 859	692 - 908
	1	629 - 820	652 - 820	634 – 815	658 – 830	671 - 884	732 - 948
	1.5	649 - 830	657 - 845	669 – 860	683 – 880	696 - 899	737 – 958
	2	669 - 850	677 - 875	689 – 895	723 – 930	746 - 964	807 - 1018
	2.5	709 - 870	737 - 895	719 – 905	738 – 940	766 - 994	837 - 1048
Henson	**0.5**	551 - 1032	553 - 1040	539 – 1037	487 – 951	508 - 966	498 - 915
	1	571 - 1062	598 - 1065	579 – 1062	522 – 991	528 - 991	518 - 955
	1.5	591 - 1072	603 - 1090	614 – 1107	547 - 1041	553 - 1006	523 - 965
	2	611 - 1092	623 - 1120	634 – 1142	587 - 1091	603 - 1071	593 - 1025
	2.5	651 - 1112	683 - 1140	664 – 1152	602 - 1101	623 - 1100	623 - 1055
Vesics	**0.5**	535 - 1016	535 - 1012	536 – 1004	536 – 986	526 - 1005	505 - 992
	1	555 - 1046	572 - 1037	554–1044	554 – 1024	546 – 1030	547 – 1016
	1.5	575 - 1056	583 - 1062	589–1069	589 - 1074	571 - 1045	552 - 1026
	2	595 - 1076	603–1092	609 – 1104	629 - 1124	621 – 1110	622 - 1086
	2.5	635 - 1096	635 - 1112	639 – 1114	649 - 1134	640 - 1140	652 – 1116
Meyerhof	**0.5**	562 - 1041	565 - 1039	566 – 1030	577 - 1031	582 - 1098	593 - 1138
	1	582 - 1071	601–1064	601 – 1065	602 - 1071	602–1123	631 – 1178
	1.5	602 - 1081	606 - 1089	612 – 1103	617 - 1121	622 - 1138	636 - 1189
	2	622 - 1101	626 - 1119	622–1138	657 - 1161	672 - 1143	696 - 1238
	2.5	662 - 1122	686 - 1139	652–1148	698 - 1171	698 - 1173	714 - 1269
EBSC-7	**0.5**	397 - 772	399 - 779	423 – 799	434–801	445 - 800	456 - 812
	1	418 - 802	442 - 804	443 –806	454–809	462 - 811	465 – 834
	1.5	438 - 812	447 - 818	456 – 825	460 – 839	464 - 852	476 - 893
	2	458 - 832	467 - 848	478–852	483–867	493 - 889	499 – 887
	2.5	498 - 852	502 -868	504–873	506–889	512 - 894	513 - 878

Annex 6 **Summary of results for Allowable bearing capacity (Qall) for Site 3**

Sites	Depth (m)	Footing dimension (m x m)					
		0.5 x 0.5	1 x 1	1.5 x 1.5	2 x 2	2.5 x 2.5	4 x 4
		Range of Allowable bearing capacity (Qall) (kPa)					
Terzagi	**0.5**	167 - 226	168 - 229	148 - 216	167 – 203	187 - 259	200 - 269
	1	187 - 256	213 - 254	188 - 241	202 – 243	207 - 284	240 - 309
	1.5	213 - 266	217 - 279	223 - 286	227 – 293	232 - 299	245 - 319
	2	233- 286	237 - 309	243 - 321	267 – 343	282 - 364	315 - 379
	2.5	273 - 306	297 - 329	273 - 331	282 – 353	302 - 394	345 - 409
Henson	**0.5**	144 - 309	141 - 307	129 - 299	131 – 257	139 - 295	129 - 271
	1	164 - 339	186 - 332	169 - 324	166 – 297	159 - 320	169 - 311
	1.5	184 - 349	191 - 357	204 - 369	191 – 347	184 - 335	174 - 321
	2	304 - 369	211 - 387	224 - 404	231 – 397	234 - 400	244 - 381
	2.5	344 - 389	271 - 407	254 - 414	246 – 407	254 - 430	274 - 411
Vesics	**0.5**	147 - 299	142 - 298	121 - 286	136 – 268	145 - 308	139 - 292
	1	167 - 329	187 - 323	161 - 311	171 – 308	165 - 333	179 - 332
	1.5	187 - 339	192 - 348	196 - 356	196 – 358	190 - 348	184 - 342
	2	207 - 359	212 - 378	216 - 391	236 – 408	240 - 413	254 - 402
	2.5	247 - 379	272 - 398	246 - 401	251 – 418	260 - 443	284 - 432
Meyerhof	**0.5**	152 - 219	148 - 313	129 - 299	145 – 283	162 - 339	167 - 346
	1	172 - 249	193 - 338	169 - 324	180 – 323	182 - 364	207 - 386
	1.5	192 - 259	198 - 363	204 - 369	205 – 373	207 - 379	212 - 396
	2	212 - 279	218 - 393	224 - 404	245 – 423	257 - 444	282 - 456
	2.5	252 - 299	278 - 413	254 - 414	260 – 433	277 - 474	312 - 486
EBSC-7	**0.5**	109 - 226	101 - 220	77 – 205	93 – 189	109 - 244	107 - 225
	1	129 - 256	146 - 245	117 - 230	128 – 229	129 - 269	147 - 265
	1.5	149 - 266	151 - 270	152 - 275	153 – 279	154 - 284	152 - 275
	2	169 - 286	171 - 300	172 - 310	193 – 329	204 - 349	222 - 335
	2.5	209 - 306	231 - 320	202 - 320	208 – 339	224 - 379	252 - 365

Annex 7 **Summary of results for Ultimate bearing capacity (Qult) for Site 4**

Sites	**Depth (m)**	**Footing dimension (m x m)**					
		0.5 x 0.5	**1 x 1**	**1.5 x 1.5**	**2 x 2**	**2.5 x 2.5**	**4 x 4**
		Range of Ultimate bearing capacity (Qult) (kPa)					
Terzagi	**0.5**	339 - 613	342 - 621	314 - 620	333 – 623	353 - 655	399- 683
	1	359 - 643	378 - 642	354 – 644	368 – 644	373 – 680	497 - 723
	1.5	379- 653	384 - 667	389- 679	393– 694	398– 709	410– 753
	2	399 – 673	401- 697	409 – 701	443–736	458 - 769	480 - 773
	2.5	423 - 691	431 - 705	439 - 711	458 – 746	468 - 787	487 - 798
Henson	**0.5**	407 - 808	413 - 819	427 - 823	431 – 809	401 - 763	382 - 749
	1	427 - 838	448 - 844	453 - 854	433 – 812	411 – 777	389 – 753
	1.5	447- 848	453 - 854	467- 868	437– 817	419- 788	394– 755
	2	467 - 868	473 - 879	487 - 899	457 – 852	459 - 852	464 - 815
	2.5	497 - 888	513 - 899	507–901	497– 902	479 - 882	494 - 845
Vesics	**0.5**	396 - 767	397 - 773	401 - 785	402 – 786	401 - 785	398 - 782
	1	419 - 797	437 - 798	412 – 801	421 – 801	405 – 805	400 – 798
	1.5	439 –819	442 - 823	447– 839	446– 843	430- 820	412–803
	2	459 – 839	462 – 853	467 - 874	486 – 893	480 - 885	482 - 863
	2.5	479 – 859	489 - 873	497 - 884	501- 903	500 - 902	502 - 893
Meyerhof	**0.5**	411–805	406 - 807	397 - 797	415 – 803	433 – 801	446 - 825
	1	431– 835	451 - 832	434 – 820	445 – 827	455 – 821	459 – 839
	1.5	451–845	456 - 857	469–865	470– 877	472– 889	478– 926
	2	461 – 865	476 – 887	489 – 895	490 – 905	499 - 913	501 – 935
	2.5	501 – 885	496 - 932	519 - 905	529 – 915	543 - 934	557- 956
EBSC-7	**0.5**	297 – 718	331- 720	335 - 725	337 – 735	339 - 766	342 - 782
	1	317 – 748	336 – 745	337 – 748	338 – 758	340 – 779	344 -798
	1.5	337–758	341- 765	347- 778	348– 795	350 - 811	354–860
	2	357–778	361 - 795	387 - 828	397 – 838	367 – 843	397 – 857
	2.5	397 – 798	421 - 815	425 - 838	437 – 858	448 - 863	452 - 883

Annex 8 **Summary of results for Allowable bearing capacity (Qall) for Site 4**

Sites	**Depth (m)**	**Footing dimension (m x m)**					
		0.5 x 0.5	**1 x 1**	**1.5 x 1.5**	**2 x 2**	**2.5 x 2.5**	**4 x 4**
		Range of Allowable bearing capacity (Qult) (kPa)					
Terzagi	**0.5**	90–180	97 - 185	108 - 187	108 – 192	109 - 198	111 - 209
	1	103–208	122 – 200	121 – 203	122 – 209	123 - 215	123 - 240
	1.5	123– 218	127- 221	129 - 226	131– 231	132- 236	136– 251
	2	143–238	147 - 251	149 - 261	171–281	182 - 301	206 - 311
	2.5	183 – 258	200 - 261	201 - 271	186 - 191	202 - 330	236 - 344
Henson	**0.5**	101 – 236	100 - 224	108 - 226	109 - 223	109- 223	103 - 223
	1	121 – 261	142 – 249	142 – 249	142 - 239	131 - 247	125 - 241
	1.5	141– 271	147- 274	155 - 289	145– 272	139- 262	131– 251
	2	161 - 291	167–304	175 - 319	190 - 322	189 - 326	201 - 311
	2.5	201 – 311	217 - 324	217 - 329	205 - 332	209 - 356	231 - 341
Vesics	**0.5**	90 - 216	105 – 211	105 - 211	105 - 213	106 - 233	106 - 233
	1	108 – 246	130 – 236	130 - 236	130 - 240	132 - 258	132 - 257
	1.5	128 - 256	135 – 261	149- 279	148– 281	143- 273	137– 267
	2	148 – 276	155 - 291	169 - 326	192–331	193 - 323	207 - 327
	2.5	188 - 296	216 - 321	199 - 336	207–341	213 - 353	237 - 357
Meyerhof	**0.5**	107 – 239	121 - 134	123 - 136	118–123	121 - 256	121 -258
	1	127 – 269	146 – 259	147 - 261	146 - 263	143 - 281	154 - 298
	1.5	147– 279	151– 284	156- 288	156- 292	157- 296	159– 308
	2	167– 299	171 – 314	171- 223	201 - 342	207–344	259 - 368
	2.5	207–319	231 - 334	231 - 334	216 - 352	227 - 374	289 - 398
EBSC-7	**0.5**	90 - 207	91 - 208	90 - 208	90 - 204	99 - 230	90 - 238
	1	92 – 237	106 - 228	106 – 228	107 - 215	106 – 255	121 - 278
	1.5	103–247	111 - 253	115– 259	116- 265	118- 270	126– 286
	2	123 – 267	131 – 273	140 - 294	161 - 315	168 – 320	196 - 246
	2.5	163 – 287	171 - 293	171 - 301	176 - 325	188 - 350	226 - 276

Annex 9 **Summary of results for Ultimate bearing capacity (Qult) for Site 5**

Sites	Depth (m)	Footing dimension (m x m)					
		0.5 x 0.5	1 x 1	1.5 x 1.5	2 x 2	2.5 x 2.5	4 x 4
		Range of Allowable bearing capacity (Qult) (kPa)					
Terzagi	0.5	421- 523	415 - 515	395 - 504	427 - 550	449 – 565	566 – 703
	1	441 – 553	457 – 540	435- 529	457 – 575	469 – 590	526 – 663
	1.5	461 – 563	462 - 565	470 - 574	482 - 590	494 – 605	531 – 653
	2	481- 583	482 – 595	490 – 609	532 – 655	544 – 670	601 – 713
	2.5	521 - 603	542 - 615	520 - 619	552 - 685	564 – 700	631 – 743
Henson	0.5	511 - 545	509 - 640	515- 661	511 - 650	493 – 628	492 – 595
	1	531 – 575	554 – 665	555 – 686	531 – 675	513 – 653	512 – 635
	1.5	551 - 685	559 – 690	590 - 731	556 -690	538 – 668	517 – 645
	2	601 – 705	579 – 720	610 – 766	606 – 715	588 – 733	587 – 705
	2.5	641 - 725	639 – 740	640 - 776	626 - 745	608 – 763	617 – 735
Vesics	0.5	505 - 652	509 - 640	608 - 651	532 - 680	519 – 661	512 – 645
	1	525 – 672	554 – 665	738 – 676	552 – 705	539 – 686	552 – 685
	1.5	545 - 682	559 - 690	573 - 711	577 - 717	564- 701	557 – 695
	2	565 – 702	579 – 720	593 – 746	627 – 782	614 – 766	627 – 755
	2.5	605 – 722	613 - 740	623 - 756	647 - 812	634 – 796	657 – 785
Meyerhof	0.5	529 - 679	528 - 673	515 - 671	554 - 704	563 – 717	590 – 746
	1	549 – 709	573 – 698	555 – 696	574 – 729	583 – 742	630 – 786
	1.5	569 - 719	578 - 723	590 - 731	599 - 744	608 – 757	635 – 796
	2	589 – 539	598 – 753	610 – 766	649 – 809	658 – 822	705 – 856
	2.5	629 - 559	658 - 773	640 - 776	669 - 839	678 – 852	735 – 886
EBSC-7	0.5	398 - 498	391 - 495	372 - 484	409 - 525	417 – 540	438 – 556
	1	418 – 528	436 – 520	412 – 509	429 – 550	437 – 565	478 – 596
	1.5	438 - 538	441 - 545	447 - 554	454 - 565	462 – 575	483 – 606
	2	458 - 558	461 – 575	467 – 589	504 – 630	512 – 640	553 – 666
	2.5	498 - 578	521 - 595	497 - 599	524 - 660	532 – 670	583 – 696

Annex 10 **Summary of results for Allowable bearing capacity (Qall) for Site 5**

Sites	Depth (m)	Footing dimension (m x m)					
		0.5 x 0.5	1 x 1	1.5 x 1.5	2 x 2	2.5 x 2.5	4 x 4
		Range of Allowable bearing capacity (Qult) (kPa)					
Terzagi	0.5	106 - 139	102 - 138	81 - 121	100 - 106	119 - 161	132 - 177
	1	126 - 169	147 - 163	121 - 146	135 - 146	139 - 186	152 - 202
	1.5	146 - 179	152 - 188	156 - 191	160 - 196	164 - 201	177 - 217
	2	166 - 199	172 -218	176 - 226	200 - 246	214 - 266	227 - 282
	2.5	206 - 219	232 - 238	206 - 236	215 - 256	234 - 296	247 - 312
Henson	0.5	132 - 178	136 - 184	121 - 173	125 - 140	134 - 182	127 - 175
	1	152 - 218	181 - 209	161 - 198	160 - 180	154 - 207	147 - 200
	1.5	172 - 228	186 - 234	196 - 243	185 - 230	179 - 222	172 - 215
	2	192 - 248	206 - 264	216 - 278	225 - 280	229 - 287	222 - 280
	2.5	232 - 268	266 - 289	246 - 288	140 - 190	249 - 317	242 - 310
Vesics	0.5	136 - 191	136 - 184	121 - 173	139 - 158	157 - 212	166 - 225
	1	156 - 221	181 - 209	161 - 198	174 - 198	177 - 237	186 - 250
	1.5	176 - 231	186 - 234	196 - 243	199 - 248	202 - 252	211 - 265
	2	196 - 251	206 - 264	216 - 278	239 - 298	252 - 317	261 - 330
	2.5	236 - 271	266 - 284	246 - 288	254 - 308	272 - 347	281 - 360
Meyerhof	0.5	138 - 187	134 - 183	116 - 167	132 - 149	143 - 193	140 - 191
	1	158 - 217	179 - 208	156 - 192	167 - 189	163 - 218	160 - 216
	1.5	178 - 227	184 - 233	191 - 237	192 - 239	188- 233	185 - 231
	2	198 - 247	204 - 263	211 - 272	232 - 289	238 - 298	235 - 296
	2.5	238 - 267	264 - 283	241 - 282	247 - 299	258 - 328	255 - 326
EBSC-7	0.5	92 - 126	88 - 129	74 - 114	91 - 98	109 - 151	116 - 162
	1	112 - 156	133 - 154	114 - 139	126 - 138	129 - 176	136 - 187
	1.5	132 - 166	138 - 179	149 - 184	151 - 188	154 - 191	161 - 202
	2	152 - 186	158 - 209	169 - 219	191 - 238	204 - 256	211 - 267
	2.5	192 - 206	218 - 229	199 - 229	206 - 248	224 - 286	231 - 297

Annex 11 **Summary of results for Ultimate bearing capacity (Qult) for Site 6**

Sites	Depth (m)	Footing dimension (m x m)					
		0.5 x 0.5	1 x 1	1.5 x 1.5	2 x 2	2.5 x 2.5	4 x 4
		Range of Allowable bearing capacity (Qult) (kPa)					
Terzagi	0.5	639 - 964	637 - 862	624 - 958	668 - 1005	682 - 1022	724 - 1062
	1	659 - 994	682 – 987	664 – 983	688 – 1030	702- 1047	764 – 1102
	1.5	679 - 1004	687 - 1012	699 - 1028	713- 1045	727 - 1062	769 - 1112
	2	699 – 1024	707 - 1042	719 – 1063	763 – 1110	757 – 1127	839 – 1162
	2.5	739 - 1044	760 - 1062	749 - 1073	783 - 1140	777 - 1157	869 - 1192
Henson	0.5	820 - 1239	843 - 1247	898 - 1243	848 - 1190	764 - 1144	727 - 1074
	1	850 - 1245	866 - 1263	858- 1268	868 – 1215	784 – 1169	767 – 1114
	1.5	870- 1280	875 - 1290	893 - 1313	893 - 1230	809 - 1184	772 - 1124
	2	890 - 1300	895 - 1316	913 – 1348	943- 1295	859 – 1269	842 – 1184
	2.5	910 - 1315	925- 1337	953 - 1358	963 - 1325	879 - 1299	872 - 1214
Vesics	0.5	801 - 1189	798 – 1184	785 - 1188	818- 1219	793 - 1178	773 - 1128
	1	821 – 1219	842 – 1209	825 – 1213	838 – 1244	813 – 1203	813 – 1168
	1.5	841 - 1229	847 – 1234	860 - 1258	863 - 1259	838 - 1218	818 - 1178
	2	861 – 1249	867 – 1264	880 – 1293	913 – 1324	888 – 1283	888 – 1238
	2.5	901 - 1269	927 – 1284	910 - 1303	933 - 1354	908 - 1313	918 - 1268
Meyerhof	0.5	871 - 1232	847 – 1245	893 - 1224	894 - 1287	956 - 1371	979 - 1322
	1	851 – 1262	872 – 1270	853- 1259	874 – 1303	936 – 1346	939 – 1362
	1.5	871 - 1282	877- 1295	888 - 1304	899- 1318	911 - 1331	944 - 1372
	2	891 – 1302	897- 1325	908 – 1339	949 – 1383	961 – 1396	1014 – 1432
	2.5	931 - 1322	957 - 1345	938 - 1349	969 - 1413	981 - 1426	1044 - 1462
EBSC-7	0.5	648 - 932	609 - 907	670 - 894	629 - 935	638 - 946	665 – 969
	1	668 – 962	654 – 932	630 – 919	649 – 960	658 – 971	705 – 1009
	1.5	648 - 952	659 – 957	665 - 964	674 - 975	683 - 986	710 - 1019
	2	668 – 972	679 – 987	685 – 999	724 – 1040	733 – 1051	780 – 1079
	2.5	708 - 992	739 - 1007	715 - 1009	744 - 1070	753 - 1081	810 - 1109

Annex 12 **Summary of results for Allowable bearing capacity (Qall) for Site 6**

Sites	Depth (m)	Footing dimension (m x m)					
		0.5 x 0.5	1 x 1	1.5 x 1.5	2 x 2	2.5 x 2.5	4 x 4
		Range of Allowable bearing capacity (Qult) (kPa)					
Terzagi	0.5	183 - 291	179 - 283	158 - 275	192 - 308	197 - 314	211 - 320
	1	203 – 321	224 – 312	198 - 297	212 – 333	217 – 339	251 – 360
	1.5	223 - 331	229 – 337	233 - 342	237 - 348	242 - 354	256 - 370
	2	243 - 351	249 – 367	253 – 377	287 – 413	292 – 419	256 – 430
	2.5	283 – 361	309 - 387	283 - 387	387 - 443	312 - 449	356 - 460
Henson	0.5	233- 372	239 -376	222 - 367	234 -370	224- 354	212 -324
	1	253 – 402	284 -401	262 -392	254- 395	244 – 372	252-364
	1.5	273 - 412	289 - 426	297 - 437	279 - 410	269 - 394	257 – 374
	2	293 -432	309 - 456	317 -472	329 - 475	319-459	327 – 434
	2.5	313 – 452	369 -476	347 -482	349 -505	339 -489	357 – 464
Vesics	0.5	225- 344	221 – 341	211 – 349	242 – 379	234-366	227 -342
	1	245 – 374	266 – 366	251 – 374	262 -405	254 -391	267-382
	1.5	265 - 384	271 – 391	286 – 419	287 - 419	279 - 406	272- 392
	2	285 -404	291 - 421	337- 484	337-484	329-471	342 -452
	2.5	325- 424	351- 441	357 -514	357-514	349 -501	372 -482
Meyerhof	0.5	243 – 375	241 -377	221-364	254-389	298- 403	269 – 407
	1	263 - 405	286 -402	261 -389	274-414	278 - 428	309 – 447
	1.5	283 - 415	291 - 427	296 - 434	299 - 439	303 - 443	314 – 457
	2	303-435	311 -457	316 -469	349 -504	353 -508	384 - 517
	2.5	343 -455	371-477	346-479	369-534	373-538	414- 547
EBSC-7	0.5	171 – 267	166- 265	146 -251	179 -285	182 - 288	191- 289
	1	191- 297	211 - 290	186 -276	199 - 310	202 – 313	231 – 329
	1.5	211 - 307	216 – 315	221 - 321	224 - 325	227 – 328	236 - 339
	2	231 - 327	236 – 345	241- 356	274 -390	277- 393	306- 399
	2.5	271 – 347	296 -365	271 - 366	294 -420	297 - 423	336 – 429

Annex 13 **Summary of results for Ultimate bearing capacity (Qult) for Site 9**

Sites	Depth (m)	Footing dimension (m x m)					
		0.5 x 0.5	1 x 1	1.5 x 1.5	2 x 2	2.5 x 2.5	4 x 4
		Range of Allowable bearing capacity (Qult) (kPa)					
Terzagi	**0.5**	602 - 732	597 – 729	578 - 716	606 - 715	633 - 785	658 - 833
	1	622 - 762	642 – 754	618 - 741	641 - 755	653 - 810	698 - 873
	1.5	642 - 772	647 – 779	653 - 786	666 - 805	678 - 825	703 – 883
	2	662 - 792	667 – 809	673 - 821	706 - 855	728 - 890	773 - 943
	2.5	702 - 812	727 – 829	703 - 831	721 - 865	748 - 920	803 - 973
Henson	**0.5**	755 - 959	753 – 962	747 - 947	702 - 868	688 - 887	653 - 812
	1	775 - 989	798 -987	787 - 972	737 - 908	708 - 912	693 - 852
	1.5	795 - 999	803 – 1012	812 - 1017	762 - 958	733 - 927	698 – 892
	2	815 - 1019	823 - 1042	832 - 1052	802 - 1008	783 - 992	768 - 952
	2.5	855 - 1039	883 - 1062	842 - 1062	817 - 1018	803 - 1022	798 - 982
Vesics	**0.5**	728- 933	728 - 928	707 - 947	724 - 901	716 - 926	696 - 902
	1	748 - 963	773 - 953	747 - 940	759 - 941	736 - 951	736 - 942
	1.5	768 - 973	778 - 978	782- 985	784 - 991	761 - 966	741 – 952
	2	788 - 983	798 - 1008	802 -1020	824 - 1041	811 - 1031	811 - 1012
	2.5	828 - 1003	858 - 1028	832- 1030	839 - 1051	831 - 1061	841 - 1042
Meyerhof	**0.5**	752 - 947	752 - 949	735 - 944	759 - 942	782 - 1009	808 - 1051
	1	772 - 977	797 - 974	775 - 969	794 - 982	802 - 1034	848 - 1091
	1.5	792 - 987	802 - 999	810 - 1014	819 - 1032	827 - 1049	853- 1101
	2	812 -997	822 - 1029	830 - 1049	859 - 1082	877 - 1114	923 - 1161
	2.5	852 - 1017	882 - 1049	860 - 1889	874 - 1092	897 - 1144	953 - 1191
EBSC-7	**0.5**	541 - 709	543 - 706	528 - 693	550 -687	572 - 751	593 - 782
	1	561 - 739	588 - 731	568 - 718	585 - 727	592 - 776	633 - 822
	1.5	581 - 749	593 - 756	603 - 763	610 - 777	617 - 791	638 - 832
	2	601 - 769	613 - 786	623 - 798	650 - 827	667 - 856	708 - 892
	2.5	641 - 789	673 - 806	653 - 808	665 - 837	687 - 886	738 - 922

Annex 14 **Summary of results for Allowable bearing capacity (Qall) for Site 9**

Sites	Depth (m)	Footing dimension (m x m)					
		0.5 x 0.5	1 x 1	1.5 x 1.5	2 x 2	2.5 x 2.5	4 x 4
		Range of Allowable bearing capacity (Qult) (kPa)					
Terzagi	**0.5**	-	-	142 - 192	162 - 178	181 - 235	189 - 244
	1	-	-	182 - 217	197 - 218	201 - 260	229 - 284
	1.5	-	-	217 - 262	222 - 268	226 - 275	234 - 294
	2	-	-	237 - 297	262 - 318	276 - 340	304 - 354
	2.5	-	-	267 - 307	277 - 328	296 - 370	334 - 384
Henson	**0.5**	-	-	195 - 269	194 - 229	199 - 269	277 - 347
	1	-	-	235 - 294	229 - 269	219 - 294	237 - 307
	1.5	-	-	270 - 339	254 - 319	244- 309	232 - 297
	2	-	-	290 - 374	294 - 369	294 - 374	302 - 357
	2.5	-	-	320 - 384	309 - 379	314 - 404	332 - 387
Vesics	**0.5**	-	-	185 - 258	201 - 240	208 - 282	302 - 267
	1	-	-	225 - 283	236 - 280	228 - 307	242 – 307
	1.5	-	-	260 – 328	261 – 330	253 - 322	247 – 317
	2	-	-	280 - 363	301 - 380	303 - 384	317 – 377
	2.5	-	-	310 - 373	316 - 390	323 - 414	347 – 407
Meyerhof	**0.5**	-	-	195 - 268	213 - 254	230 - 309	240 – 317
	1	-	-	235 - 293	248 - 294	250 - 334	280 – 357
	1.5	-	-	270 – 338	273 – 344	275 - 349	285– 367
	2	-	-	290 - 373	313 - 394	325 - 414	355 – 427
	2.5	-	-	320 - 383	328 - 404	345 - 444	385 – 457
EBSC-7	**0.5**	-	-	126 - 184	143 - 169	160 - 223	167 – 227
	1	-	-	166 - 209	178 - 209	180 - 248	207 – 267
	1.5	-	-	201 - 254	203 - 259	205 - 263	212–277
	2	-	-	221 - 289	243 - 309	255 - 328	282 – 337
	2.5	-	-	251 - 299	258 - 319	275 - 358	312 – 367

Appendix

BH.no	Easting	Northing	Lab. Description	Atterberg limit		% Gravel	% Sand	% Silt	% Clay	% Fs	Group sympol
				LL	PL						
1	483741.8600	994902.3900	High plastic clayey gravelly SILT	97	49	22.6	9.9	47.5	20.0	110	MH
2	483127.0500	995979.1400	High plastic silty CLAY	102	56	0.0	4.4	25.6	70.0	130	CH
3	483207.7100	995979.1400	High plastic silty CLAY	105	56	0.0	0.3	41.7	58.0	100	CH
4	484039.9700	996104.8100	High plastic gravelly CLAY	100	57	29.4	2.9	9.7	58.0	100	CH
5	484198.4400	996799.8200	High plastic silty CLAY	98	48	0.3	3.0	14.7	82.0	130	CH
6	484627.9600	996001.7500	High plastic silty CLAY	107	53	0.8	3.0	32.2	61.0	130	CH
7	484600.1900	996144.1000	High plastic silty CLAY	111	66	0.0	2.2	35.8	62.0	110	CH
8	484692.6100	995772.8700	High plastic silty CLAY	99	53	1.6	3.8	38.6	56.0	120	CH
9	484652.6200	996244.3300	High plastic silty CLAY	90	43	0.0	2.5	31.5	66.0	110	CH
10	484638.0200	996416.3100	High plastic silty CLAY	93	38	7.7	1.2	55.1	36	105	MH
11	484948.4900	996423.1400	High plastic silty CLAY	101	45	3.0	1.7	18.0	80.0	120	CH
12	484971.6500	996064.3500	High plastic silty CLAY	103	57	0.0	1.3	18.7	80.0	160	CH
13	484807.2500	996875.4100	High plastic silty CLAY with gravel	97	52	8.7	2.3	6.00	83.0	100	CH
14	484874.7400	996824.0600	High plastic silty CLAY	103	57	14.4	2.8	36.8	46.0	180	CH
15	484810.9200	996823.8800	High plastic silty CLAY	103	57	0.0	1.8	20.2	78.0	130	CH
16	484957.3500	992866.4800	High plastic silty CLAY	102	55	0.3	3.5	38.2	58.0	120	CH
17	484952.0100	992952.0600	High plastic silty CLAY	87	40	0.0	2.1	27.9	70.0	100	CH
18	483660.8500	994602.1500	High plastic silty CLAY	97	52	0.2	2.5	19.3	78.0	140	CH
19	483335.512	994718.51	High plastic silty CLAY	109	60	0.0	1.6	40.4	58.0	110	CH
20	483353.241	994721.31	High plastic silty CLAY	100	59	0.0	3.7	14.3	82.0	110	CH
21	483332.476	994670.545	High plastic silty CLAY	93	45	5.9	3.5	30.6	60	130	CH
22	483349.707	994680.251	High plastic silty CLAY	106	58	0.0	3.0	35	62	140	CH
23	483390.396	994671.683	High plastic silty CLAY	104	52	0.0	1.6	37.4	61	150	CH
24	483407.604	994681.43	High plastic silty CLAY	113	67	0.0	2.2	19.8	68	140	CH
25	483415.512	994715.83	High plastic clayey SILT with sand	108	63	15.2	1.6	45.2	38	100	MH
26	483391.897	994712.781	High plastic silty CLAY	97	47	0.0	1.0	51.0	48	115	MH
27	483482.773	994661.109	High plastic silty CLAY with few gravel	99	65	13.0	44.4	16.1	66	120	CH
28	483486.87	994684.563	High plastic clayey SILT/ silty CLAY with few gravel	101	53	11.9	2.1	43.0	43	130	CH-MH
29	483537.975	994666.479	High plastic silty CLAY	103	54	0.0	1.1	16.9	82	130	CH
30	483547.37	994689.881	High plastic silty CLAY with few gravel	98	48	12.5	1.3	28.2	58	130	CH
31	483444.862	994622.649	High plastic silty CLAY	92	45	1.0	3.9	23.0	72	140	CH
32	483467.991	994632.697	High plastic silty CLAY	102	53	3.6	3.5	10.9	82	105	CH
33	483502.372	994624.311	High plastic silty CLAY	100	52	0.7	5.0	63.2	36	140	CH

34	483525.501	994634.36	High plastic clayey SILT	104	59	0.0	0.8	63.2	36	120	MH
35	483470.021	994502.171	High plastic silty CLAY	97	54	0.4	1.8	31.8	66	110	CH
36	483446.014	994497.179	High plastic clayey SILT	97	57	0.0	1.5	95.7	2.8	100	MH
37	483497.838	994502.904	High plastic clayey SILT	106	61	0.0	1.2	97.8	1.0	170	MH
38	483510.319	994516.62	High plastic clayey SILT	101	57	0.0	0.6	95.3	4.0	170	MH
39	483142.7900	991319.4800	High plastic silty CLAY	95	51	0.0	2.0	9.9	88.0	160	CH
40	483198.7000	990875.8100	High plastic clayey SILT	101	57	0.1	0.6	58.8	40.5	140	MH
41	483087.15	990316.5800	High plastic clayey SILT	101	53	0.0	1.9	96.1	2.0	150	MH
42	483210.3600	990301.1100	High plastic clayey SILT	105	60	0.0	1.6	57.4	41.0	160	MH
43	483365.1200	990599.2600	High plastic clayey SILT	106	61	0.0	0.6	94.8	4.6	120	MH
44	483382.2000	990359.0600	High plastic silty CLAY	87	51	0.8	6.4	37.8	55.0	100	MH
45	484619.0900	990734.1200	High plastic clayey SILT	98	47	0.0	1.0	95.0	4.0	140	MH
46	483229.4500	990626.1600	High plastic clayey SILT	94	50	0.0	2.7	95.3	2.0	140	MH
47	483009.9100	990713.2700	High plastic silty CLAY	102	61	0.0	1.3	38.7	60.0	95	CH
48	484050.7600	991167.9500	High plastic clayey SILT	95	52	0.0	2.6	89.4	8.0	90	MH
49	484169.8900	990441.4100	High plastic clayey SILT	101	56	0.0	0.5	61.4	20	120	MH
50	484158.4300	991625.3100	High plastic silty CLAY	101	57	0.6	0.6	45.3	54	150	CH
51	485428.6600	993160.3900	High plastic clayey SILT	99	59	1.1	1.1	93.8	4	150	MH
52	485419.4800	992996.5000	High plastic clayey SILT	109	65	0.0	1.4	50.6	48	170	MH
53	485266.6500	992992.8000	High plastic silty CLAY	91	51	0.0	1.2	25.8	73	140	CH
54	485231.7600	992916.5400	High plastic silty CLAY	96	56	0.3	2.8	29.9	67	150	CH
55	486051.6200	993311.5400	High plastic clayey SILT	93	47	0.0	1.1	97.9	1	130	MH
56	486089.1100	993255.7900	High plastic clayey SILT	107	61	0.0	1.4	96.6	2	120	MH
57	486082.6300	993026.4000	High plastic clayey SILT	97	57	0.0	1.5	95.7	2.8	100	MH
58	486170.2700	993348.6900	High plastic silty CLAY	102	61	0.0	1.3	38.7	60.0	95	CH
59	486210.9400	993386.2800	High plastic silty CLAY	92	45	1.0	3.9	23.0	72	140	CH
60	486229.0400	993183.8600	High plastic silty CLAY	102	61	0.0	1.3	38.7	60.0	95	CH
61	485803	996392	Silty clay	98.3	39.1	0.0	1.2	14.6	84.1	170	ML
62	486570	996106	Silty clay	98.8	72.1	0.0	9.2	37.3	53.3	190	ML
63	486562	996117	Silty clay	80.6	46.3	0.0	5	29.7	65.3	145	ML
64	485778	996391	Silty clay	95.08	78.2	0.0	4.3	55.3	40.4	185	ML
65	485843	996492	Silty clay	98.2	61	0.0	20	45.4	34.6	180	ML
66	485840	996483	Silty clay	80	56	0.0	1.2	37.6	61.3	170	ML
67	486326.6504	9910873.2610	Non plastic SILT with sand	NP	NP	0.0	16.60	77.40	6.00	-	ML
68	486337.7335	990893.4604	Low plastic SILT with sand	45	13	0.0	16.60	77.40	6.00	-	ML
69	486373.7887	990884.5482	Non plastic SILT with sand	NP	NP	0.0	15.60	72.20	12.00	-	ML
70	486382.2658	990862.8328	Non plastic SILT with sand	NP	NP	0.0	22.30	70.60	6.00	-	ML
71	486377.7636	990826.6271	High plastic clayey SILT	80	29	0.0	25.90	64.40	8.00	-	MH
72	486372.0401	990794.9047	High plastic clayey SILT	75	26	0.0	3.20	65.30	31.00	-	MH

73	486370.5230	990766.6530	Low plastic clayey SILT	45	11	0.0	1.90	70.10	28.00	-	ML
74	486364.6682	990731.5599	High plastic clayey SILT	70	22	0.0	7.50	74.50	18.00	-	MH
75	486359.9529	990710.2572	Non plastic SILT with sand	NP	NP	0.0	2.10	84.90	13.00	-	ML
76	486349.7453	990690.2410	Non plastic silty SAND with gravel	NP	NP	0.0	27.40	66.60	6.00	-	SM
77	486312.2953	990697.5909	High plastic Silty GRAVEL	50	16	0.0	43.60	36.10	4.00	-	GM
78	486304.1769	990720.6490	Low Plastic SILT with sand	44	15	0.0	10.60	33.50	4.00	-	ML
79	486304.9306	990752.7270	Low Plastic SILT gravel with sand	35	7	0.0	16.40	69.50	6.00	-	GM
80	486307.6522	990779.3155	Low plastic Sand SILT with gravel	38	13	0.0	16.80	39.30	5.00	-	ML
90	486282.8723	990886.4530	High plastic clayey SILT	90	43	0.0	12.70	57.30	6.00	-	MH
91	486274.9749	990906.4700	Non plastic SILT with sand	NP	NP	0.0	28.20	64.50	7.00	-	ML
92	486239.7925	990919.9818	Low plastic SILT with sand	48	17	0.0	16.30	82.70	1.00	-	ML
93	486228.4516	990898.7592	High plastic clayey SILT	89	50	0.0	2.60	56.80	40.00	-	MH
94	486326.6504	9910873.2610	Non plastic Sandy SILT	NP	NP	0.0	42.20	56.60	1.00	-	ML
95	486337.7335	990893.4604	Low plastic gravelly CLAY with sand	47	30	0.0	14.40	42.50	11.00	-	CL
96	486373.7887	990884.5482	Low plastic silty GRAVEL with sand	42	20	0.0	18.30	26.20	2.00	-	GM
97	486382.2658	990862.8328	Low plastic clayey SILT	52	15	0.0	5.60	84.00	10.00	-	MH
98	486377.7636	990826.6271	Non plastic SILT with sand	NP	NP	0.0	17.00	78.60	3.00	-	ML
99	486372.0401	990794.9047	Low plastic Sandy SILT with gravel	44	11	0.0	21.00	61.90	2.50	-	ML
100	486223.3940	990874.0648	Low plastic gravelly CLAY with sand	47	30	0.0	14.40	42.50	11.00	-	CL
101	486218.0273	990838.0382	Low plastic silty GRAVEL with sand	42	20	0.0	18.30	26.20	2.00	-	GM
102	486215.4338	990814.7867	High plastic clayey SILT with sand	51	17	0.0	16.50	61.70	21.00	-	MH
103	486209.5931	990780.0874	Low plastic sandy CLAY	42	17	0.0	45.30	42.60	11.80	-	CH
104	486272.7449	990780.5262	Low plastic silty SAND with gravel	40	8	0.0	16.80	31.50	6.00	-	CM
105	486268.1208	990757.1572	Low plastic sandy SILT	37	9	0.0	39.10	53.50	4.30	-	ML
107	486204.5569	990748.6150	Low plastic clayey GRAVEL with sand	28	11	0.0	17.40	25.80	2.00	-	GC
108	486207.5149	990724.3408	Low plastic clayey GRAVEL with sand	26	11	0.0	12.50	26.50	2.00	-	GC
109	486251.1703	990715.1484	Low plastic silty GRAVEL with sand	43	12	0.0	9.90	15.80	6.00	-	GM
200	486263.4150	990735.2460	Low plastic silty GRAVEL with sand	47	17	0.0	15.20	29.50	3.00	-	GM
201	486202.3627	990682.1618	Low plastic sandy SILT with gravel	NP	NP	0.0	26.40	47.20	6.80	-	ML
202	486191.5768	990659.7303	Low plastic clayey GRAVEL with sand	28	11	0.0	18.80	26.90	1.00	-	GC
203	486231.4940	990678.0209	Low plastic silty GRAVEL with sand	49	12	0.0	25.80	33.60	6.00	-	GM
204	486227.0585	990643.7683	Low plastic clayey GRAVEL with sand	22	7	0.0	18.20	15.30	2.00	-	GC
205	486182.7201	990640.4832	High plastic SILT Sand	69	17	0.0	35.80	39.40	9.00	-	SM
206	486178.4695	990608.4891	High plastic SILT sand with gravel	56	13	0.0	29.00	38.70	9.00	-	SM
207	486181.7960	990585.8406	High plastic sandy clayey SILT	61	16	0.0	18.70	51.00	13.00	-	MH
208	488042.05	991754.78	High plastic silty GRAVEL with sand	58	20	0.0	16.20	17.60	1.00	-	CH
209	488020.01	991702.25	High plastic claye SILT with sand	52	19	0.0	16.70	56.90	25.00	-	MH
210	488002.06	991702.25	High plastic silty SAND with gravel	54	31	0.0	5.20	14.80	2.00	-	GM
211	488043.83	991557.73	Low plastic silty GRAVEL	48	13	0.0	7.00	11.30	11.90	-	GC

212	488089.68	991520.34	Low plastic clayey SAND with gravel	30	12	0.0	22.20	28.20	4.00	-	GC
213	488144.69	991485.83	Low plastic clayey GRAVEL	26	10	0.0	40.70	25.70	3.10	-	SC
214	488191.91	991448.93	Low plastic clayey SAND with gravel	27	10	0.0	27.50	41.10	2.50	-	SM
215	488260.02	991399.67	Low plastic clayey SAND with gravel	34	15	0.0	5.00	7.30	3.50	-	GC
216	488306.84	991361.23	Low plastic clayey GRAVEL with sand	30	9	0.0	11.80	20.00	4.90	-	GC
217	488199.87	991570.13	Low plastic clayey GRAVEL with sand	29	9	0.0	20.60	26.20	2.30	-	GM
218	488217.86	991516.23	High plastic clayey SILT with sand	54	15	0.0	10.20	10.60	1.00	-	GC
219	488267.03	991577.01	Non plastic sandy SILT	NP	NP	0.0	23.50	46.80	10.00	-	MH
230	488348.53	991416.78	Non plastic sandy SILT	NP	NP	0.0	12.80	24.70	1.00	-	GM
232	488348.33	991331.25	Low plastic clayey GRAVEL	29	10	0.0	71.80	18.00	9.00	-	SM
233	488377.23	991292.66	Low plastic clayey GRAVEL	39	18	0.0	16.50	28.10	5.00	-	GM
234	488483.14	991316.12	Low plastic clayey GRAVEL with sand	2	1.2	0.0	15.80	64.20	20.00	-	ML
235	488510.24	991195.06	Low plastic clayey GRAVEL with sand	3.8	2.6	0.0	22.00	41.20	6.00	-	SM
236	488598.2	991128.45	Low plastic clayey GRAVEL with sand	26	9	0.0	21.0	71.80	6.00	-	MH
237	488217.86	991516.23	Low plastic clayey GRAVEL with sand	30	11	0.0	15.70	27.50	1.00	-	GM
238	488686.41	991064.11	High plastic clayey SILT with sand	63	13	0.0	21.80	50.10	21.00	-	ML
239	488793.99	991085.59	High plastic SILT with sand	52	8	0.0	12.10	71.80	10.00	-	ML
240	488918.74	991255.65	Low plastic SILT with sand	48	11	0.0	12.00	74.00	10.00	-	ML
241	488939.14	991402.13	Low plastic clayey SILT with sand	NP	NP	1.8	22.4	48.8	27.0	NIL	-
242	488886.19	991465.71	High plastic silty CLAY	87	54	0.0	0.9	39.1	60.0	100	CH
243	488737.33	991581.59	High plastic silty CLAY	92	46	0.0	1.6	38.4	60.0	100	CH
244	488428.561	995778.945	High plastic silty CALY	99	48	0.0	0.0	11.0	88.0	100	CH
245	488450.89	995786.52	High plastic silty CALY	99	57	0.0	0.0	7.7	90.0	110	CH
246	488495.24	995779.765	Non plastic sandy SILT	NP	NP	0.0	0.0	71.3	6.0	11	-
247	488517.563	995787.839	High plastic silty CALY	104	59	0.0	0.0	18.7	80.0	120	CH
248	488555.051	995788.083	High plastic silty CALY	99	55	0.0	0.0	38.8	60.0	100	CH
249	488578.73	995779.95	Non plastic sandy SILT	NP	NP	0.0	0.0	65.2	2.5	10	-
251	488608.664	995780.217	High plastic silty CALY	104	54	0.0	0.0	26.7	72.0	100	CH
252	488636.383	995788.8	Non plastic sandy SILT	NP	NP	0.0	0.0	50.9	2.0	30	-
253	488706.1	995796.1	Non plastic silty GRAVEL with sand	NP	NP	0.0	0.0	38.1	1.8	30	-
254	488706.5	995778.1	High plastic silty CALY	105	56	0.0	0.0	13.3	86.0	120	CH
255	488741.5	995781.85	High plastic silty CALY	101	51	0.0	0.0	44.2	52.0	100	CH
256	488764.6	995781.85	Non plastic silty GRAVEL with sand	NP	NP	0.0	0.0	31.8	2.0	10	-
257	488793.16	995790.05	High plastic silty CALY	112	60	0.0	0.0	29.1	70.0	130	CH
258	488815.67	995782.39	High plastic silty CALY	99	49	0.0	0.0	27.2	70.0	105	CH

259	488874.53	995796.23	High plastic silty CALY	105	56	0.0	0.0	21.9	77.0	100	CH
260	488858.34	995777.78	High plastic silty CALY	113	65	0.0	0.0	20.4	79.0	150	CH
261	488705.57	995748.41	High plastic silty CALY	109	54	0.0	0.0	34.4	65.0	130	CH
262	488705.36	995730.43	Non plastic silty GRAVEL with sand	NP	NP	0.0	0.0	32.0	2.0	NIL	-
263	488869.92	995730.43	High plastic silty CALY	105	61	0.0	0.0	12.3	8.0	130	CH
264	488869.99	995748.09	High plastic silty CALY	97	52	0.0	0.0	14.6	82.0	110	CH
265	488344.74	995725.06	High plastic silty CALY with sand	54	26	0.0	0.0	11.0	88.0	80	CH
266	488371.15	995702.83	High plastic silty CALY	108	55	0.0	0.0	7.7	9.0	100	CH
267	488444.74	995696.81	Non plastic silty SAND with gravel	NP	NP	0.0	0.0	71.3	6.0	110	-
268	488457.83	995720.41	High plastic sandy SILT	96	37	0.0	0.0	18.7	80.0	11	MH
269	488493.53	995714.32	High plastic sandy SILT	86	41	0.0	0.0	38.8	60.0	120	MH
270	488493.83	995696.92	High plastic silty CALY	117	64	0.0	0.0	65.2	2.5	100	CH
271	488523.15	995700.89	High plastic silty CALY	100	49	0.0	0.0	26.7	72.0	10	CH
272	488543.15	995700.89	High plastic clayey SILT	103	51	0.0	0.0	50.9	2.0	100	CH
273	488573.63	995714.38	High plastic silty CALY	48	19	0.0	0.0	38.1	1.8	30	ML
274	488573.59	995696.37	High plastic silty CALY	108	64	0.0	0.0	13.3	86.0	30	CH
275	488701.71	995700.37	High plastic silty CALY	97	45	0.0	0.0	44.2	52.0	120	CH
276	488815.71	995700.38	Low plastic sandy SILT	64	16	0.0	0.0	31.8	2.0	100	SM
277	488555.02	996106.02	Non plastic silty GRAVEL with sand	NP	NP	47.2	37.2	14.6	1.0	NIL	-
278	488587.91	996106.02	Non plastic silty GRAVEL with sand	NP	NP	57.8	20.6	20.6	1.0	NIL	-
279	488612.91	996106.02	High plastic silty CLAY	90	45	0.0	1.4	40.6	58.0	120	CH
280	488537.91	996056.02	Low plastic clayey SILT with few sand	44	2	6.6	13.3	64.1	16.0	NIL	ML
281	488555.02	996056.02	High plastic silty CLAY	102	61	0.0	0.8	43.2	56.0	120	CH
282	488587.91	996056.02	High plastic silty CLAY	96	50	0.0	2.3	33.7	64.0	120	CH
283	488562.91	996031.02	Non plastic silty GRAVEL with sand	NP	NP	55.2	16.1	25.8	2.9	NIL	-
284	488612.91	996031.02	Non plastic silty GRAVEL with sand	NP	NP	46.6	18.0	32.6	2.0	NIL	-
285	488537.91	996013.04	High plastic silty CALY	91	52	0.0	2.8	43.2	54.0	100	CH
286	488562.91	996013.04	Low plastic gravelly SILT with	NP	NP	27.6	17.4	50.5	4.5	NIL	-
287	488612.76	996013.30	High plastic silty CLAY	80	43	0.9	13.1	36.0	50.0	100	CH
289	488537.95	995987.75	Non plastic clayey SILT	NP	NP	0.0	1.0	95.0	4.0	NIL	-
290	488588.39	995988.60	Non plastic sandy SILT	NP	NP	2.7	33.5	39.7	24.0	NIL	-

291	488537.90	995957.30	Low plastic sandy SILT	50	11	1	33.3	55.6	10.0	10	ML
292	488588.48	995957.31	High plastic silty CALY	88	42	0.0	2.3	19.7	78.0	80	CH
293	487851	995937	Non plastic sandy SILT	NP	NP	1.8	35.5	58.7	4.0	NIL	-
294	487851	995937	Non plastic sandy SILT	NP	NP	5.0	36.6	58.4	4.0	NIL	-
295	485822	996492	High plastic silty CLAY	89	48	0.0	1.1	38.9	60.0	100	CH
296	487750	996584	High plastic silty CLAY	84	46	1.3	1.6	31.1	66.0	120	CH
297	487412.5456	994244.4	High plastic silty CLAY	93	44	0.0	0.8	54.2	45.0	120	CH
298	487482.96	994231.1	High plastic silty CLAY	64	9	19.0	18.8	59.2	3.0	-	MH
299	487415.91	994180.08	High plastic silty CLAY	97	51	0.0	1.2	42.8	56.0	130	CH
300	487478.46	994147.02	Non plastic silty SAND with gravel	NP	NP	25.1	30.1	40.8	4.0	-	-
301	487415.34	994115.94	High plastic clayey SILT	92	50	0.0	1.7	92.3	6.0	110	MH
302	487560.82	994295.17	High plastic silty CALY	100	49	0.0	1.2	18.4	80.0	100	CH
303	487638.42	994305.64	High plastic clayey SILT	92	50	0.0	1.7	81.3	17.0	-	MH
304	487563.12	994227.47	High Plastic silty CLAY	99	54	0.0	1.3	40.7	58.0	120	CH
305	487637.74	994209.04	High Plastic silty CLAY with sand	89	50	2.6	15.1	17.3	65.0	-	CH
306	487593.41	994187.52	High plastic sandy CLAY	80	30	0.4	16.6	8.7	76.0	90	CH
307	487589.32	994152.02	Non plastic sandy SILT	NP	NP	1.3	26.4	67.3	5.0	15	-
308	487550.34	994108.11	High plastic silty CALY	96	51	0.0	1.0	39.0	60.0	120	CH
309	487642.94	994114.52	High plastic silty CLAY	91	61	0.0	1.7	35.3	63.0	110	-
310	487412.5456	994244.4	High plastic silty CALY	86	39	0.0	3.5	20.9	76.0	120	CH