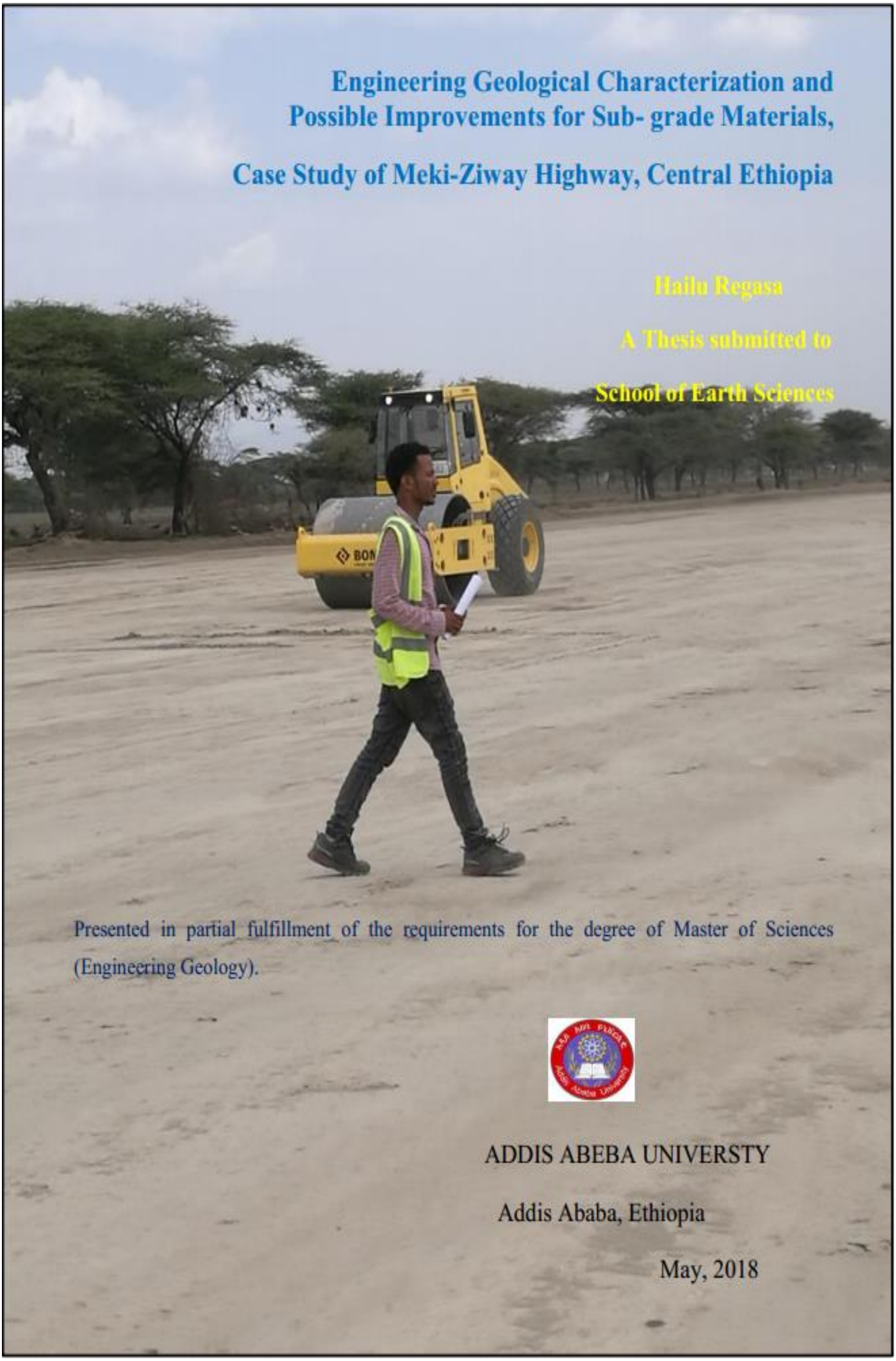

Engineering Geological Characterization and Possible Improvements for Sub- grade Materials,

Case Study of Meki-Ziway Highway, Central Ethiopia

Hailu Regasa

A Thesis submitted to

School of Earth Sciences

Presented in partial fulfillment of the requirements for the degree of Master of Sciences (Engineering Geology).

ADDIS ABEBA UNIVERSTY

Addis Ababa, Ethiopia

May, 2018

Signature page

Addis Ababa University
School of Graduate Studies

This is to certify that the thesis prepared by **Hailu Regasa**, entitled: "**Engineering Geological Characterization and Possible Improvements for Sub- grade Materials, Case study of Meki-Ziway Highway, 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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Dr. Zemenu Geremew Signature _______________Date _______________

Advisor

Declaration

I hereby declare that this thesis is my original work that has been carried out under the supervision of Dr. Zemenu Geremew Yigzaw, School of earth science Addis Ababa University during the year 2017/18 as part of Master of Science Program in Engineering Geology in accordance with the rule and regulation of the institute. I further declare that this work has not been submitted to any other University or institution for the award of any degree or diploma and all sources of material used for the thesis have duly acknowledge.

Hailu Regasa

Signature:

Place and Date of submission: School of Graduate Studies, Addis Ababa University
May 2018

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Hailu Regasa

ABSTRACT

Engineering Geological Characterization and Possible Improvements for Sub- grade Materials, Case study of Meki-Ziway Highway, central Ethiopia

Hailu Regasa

Addis Ababa University, 2018

The present study was conducted to characterize the subgrade of Meki-Ziway High way, which is located in Oromia Regional National State along the main Ethiopian rift system.

Pre-field, field and post field activities including literature, survey primary data and sample collection and laboratory test were conducted to achieve the objectives of this research work. Representative samples of subgrade materials were subjected to laboratory tests for examining their engineering characteristics. Grain size analysis and Atterberg limits tests, proctor test, California bearing ratio and swell tests were conducted for 26 disturbed subgrade soil samples. For other Seven (7) disturbed soil sample, linear shrinkage tests had carried out.

The test results showed that sub-grade soils are classified as CH, CL, MH, ML, SC and SM in Unified soil classification system; A-7-5, A-7-6, A–6, A–5, A–4, A-2-4, A-2-5 and A-2-7 on the American Association of State Highway and Transportation Officials. The test results realize that about 23.19 % of the subgrade sections of the project route have low bearing capacity and due to their low strength, they are not capable of withstanding the stresses, which will imposed on them, unless their quality is improved. Since the subgrade soils along this section is fine grained and too thick, chemical stabilization method was selected for the present research based on the characteristics of sub-grade soil. Accordingly, two (2) disturbed samples were collected and tested for Atterberg limit, proctor test, California bearing ratio and swell by mixing them with Quicklime at different proportion of 2%, 4% and 8%. The improvement of subgrade soil sample increased with increasing ratio of lime as well as curing periods. In general, increased the ratio was more significant affect than that of increased curing periods. It was favorable that 4% of the chemical is suitable to adequate improvement of subgrade soil.

Keywords: Sub-grade, Pavement, lime stabilization.

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List of Acronyms

AASHTO	American Association of State Highway and Transportation Officials
ASTM	American Society for Testing and Materials
BS	British standard
CBR	California Bearing Ratio
CDA,	Cumulative Difference Average value
ERA	Ethiopian Road Authority
FAO	Food and Agricultural Organization
FHWA	Federal Highway Administration
GI	Group Index
GPS	Geographical positioning system
MDD	Maximum Dry Density
Max	Maximum
MER	Main Ethiopian Rift
Min	Minimum
NCHRP	National Cooperative Highway Research Program
OMC	Optimum Moisture Content
PL	Plastic Limit
PI	Plasticity Index
LL	Liquid Limit
LS	Linear shrinkage
USCS	Unified Soil Classification system

Chapter one

Introduction

1. Background.

Geotechnical soils and material characterization is important components for the design and construction of the road projects. During characterization visual descriptions, sampling and testing of natural sub-grade soil material along the project route has been carried out. In the process of characterization, different techniques and procedures were applied for the interpretation of the sub -grade soil condition.

Data was collected in the field, soil samples were tested in the laboratory and the results were obtained to determine the design parameters for the pavement structure. Hence, prior to construction, the sub grade soils of the road should be sampled and investigated for the suitability of load bearing behavior.

A pavement is a hardwearing surface having materials laid down on an area subjected to sustain mainly the vehicular traffic, such as a road or highway. The performance of pavements depends upon the quality of subgrades and sub bases. Therefore, subgrade material characterization plays a vital role in the design, construction, and maintenance of engineering structures especially in pavements.

According to Salter. (1988), the performance of a highway pavement is influenced to a very considerable extent by the sub grade material and ‘density’ is a good indicator of the strength of granular sub-grades, it is also necessary to investigate the modulus of the sub-grade, since these measures represent different natural characteristics.

Pavement evaluation and design analysis relies mainly on information on the stiffness of pavement layers and the modulus of sub-grades as references, in addition to supplementary data on density and moisture content (Huang, 2004).

It is often required to estimate the sub-grade-stiffness or modulus of the pavements, before and after their construction as part of the quality-control measures, and for quality assurance (Chen et al., 2005). Road pavements are designed to limit the stress created at the subgrade level by the traffic travelling on the pavement surface so that the subgrade is not subject to significant deformations.

Therefore, to get reliable test results for evaluation of these engineering properties and minimize errors and deviation with regard to the inherent variability of the sub grade soils, proper sampling and testing procedures are of paramount importance.

1.2. Description of Modjo Hawassa Express way Project

The Modjo-Hawassa expressway project is predominantly located in Oromia Regional National State (ORNS) and serves the Southern Nations, Nationalities and Peoples Regional State (SNNPRS). The road generally traverses in a southward direction and it is a branch south from the Addis Ababa-Adama expressway.

The Modjo-Hawassa expressway project will improve the connectivity of high-potential farming and tourism areas, while forming part of the Addis Ababa-Moyale-Nairobi-Mombasa Corridor, which is of strategic importance to diversify Ethiopia's international links and seaport outlets. The overall Modjo-Hawassa expressway will have a length of 202.km from the proposed new Modjo interchange at Addis Ababa – Adama expressway which is located east of the existing Modjo Dry Port and traverses southward while the towns along the route are accessed at Koka, Alemtena, Ziway, Bulbula, Langano, Arsi Negele, Shashemene and Hawassa.

The project is among the road sector development program envisaged by the Ethiopian Roads Authority (ERA). The Meki -Ziway Highway construction was financed through a Loan from the Export-Import Bank of Korea from the resources of the Economic Development Cooperation Fund (EDCF) of the Republic of Korea.

1.3 Statements of the problem

Modjo – Hawassa project is a dual carriageway highway that provides a high speed, free flowing connection to improve the connectivity of high-potential farming and tourism areas, while consolidating the Corridor Addis Ababa-Moyale-Nairobi-Mombasa, of strategic importance to diversify Ethiopia's international links and sea outlets.

Construction of civil structure requires acquiring information about the geology, engineering properties of soils and rocks, geological structures and hydrogeology of the site. The presences of expansive soil causes swelling and shrinking and collapsible soils cause collapsing of subgrade material. In construction sites, such soil type do have significant influence on planning, structural design, construction and maintenance costs, performance and engineering life of roads and highways. These soils are susceptible to volume change in response to fluctuations in groundwater table and moisture content following seasonal climatic variations.

This can causes severe damage to infrastructures unless proper measures are taken in the design and construction phases. Therefore, identifying the problem related to subgrade soil is very important to improve their strength and increases engineering life of roads.

Besides, to this, roads fail during construction or immediately after construction before their design life or even before its liability periods after construction, due to volume change of the subgrade soil. Collapsible soil result from a unique condition in which "bridges " of fine materials within a frame work of coarser and particles become when wet and collapse under the load.

The result of collapsibility of subgrade soil is mostly manifested by the development of a deeply rutted and often uneven road surface and significant deterioration of the riding quality of the road which is common on the Modjo Hawassa old road and there serious problem related to ground cracks/ fracturing which cuts Modjo –Hawassa old road and gully erosions are also seen.

1.4 General objective:

The General objective of this research is to characterize the engineering properties of Modjo –Hawassa subgrade soil along the Expressway corridor and suggest possible mitigation measures for identified problems based on ERA specification and other standards.

1.4.1 Specific objectives

The present study was aimed to achieve the following specific objectives.

- ❖ To characterize the sub-grade soils based on the specifications as proposed by ERA.
- ❖ To assess the suitability and bearing capacity of the sub-grade soils and material.
- ❖ To work out possible improvements for subgrade material.
- ❖ To suggest countermeasures for unsuitable parts of the sub-grade soils and material.

1.5 Methodology and Material used for the study

In order to achieve the above objectives, the following Materials and methods were employed: A visual site visit was made to know the performance of the subgrade soil along the route corridor of the new Modjo- Hawassa expressway, lot II Meki Ziway section.

During the present study, the material such as Compass, GPS, sieve, Casagrad's device, Standard proctor test mold, CBR testing mold and CBR penetration machines are among materials used.

A literature review was undertaken to get good understanding of engineering properties of expansive soils and collapsible soils and problems associated with such subgrade soils.

Secondary data collected from previous work and primary data generated from field and laboratory testing during the present study were used with different standards and specifications to characterize the sub-grade material.

GPS, meter, photo camera, Cassagrande's, stock of sieve linear shrinkage mold, standard proctor mold and CBR mold within CBR penetrator machine are among the material used during the present study.

Generally, the present study was based on three phases of activities, which were performed in order to come up with the goals of the objectives. These phases are desk study, field works and post fieldwork.

1.5.1 Reconnaissance study

The activities that are taken under this phase were:. Reviewing and compiling literatures, papers and related books in order to get a conceptual framework on general methods of subgrade characterization.

Collecting, reviewing and compiling of available maps, such as topographical, and geological, hydrogeological and other maps of the study area. Besides, secondary data such as meteorological data were collected from relevant sources.

1.5.2 Field work

Fieldwork is a phase in which the researcher contributes his actual effort to collect primary data that will be latter used for the interpretation and analysis of the result.

During the fieldwork, the following activities were carried out. Collection of field data from field measure and records. Collection of representative samples for laboratory test.

A 26 representative samples were collected with different interval for different purpose. Following the Ethiopian Roads Authority (ERA) site investigation manual (2013b), soil samples for sub-grade material characterization were taken at 0.5km interval for identification test and 1km interval for CBR tests.

1.5.3Post fieldwork.

After the collection of field data, the parameters that characterize sub-grade materials were determined in laboratory on representative collected samples.

Based on AASHTO and ERA standards, the sub-grade properties including, grain size analysis, specific gravity, Atterberg limits, linear shrinkage, and optimum moisture content, maximum dry density, California bearing ratio (CBR) and loaded swelling potential were determined and used to characterize the sub-grade materials.

Finally, suitability analysis of the materials for pavement construction was done according to different design parameters.

Based on the laboratory test results; the problematic subgrade soil was identified and possible counter measurements were determined. The flow chart of general Methodology applied for the present study is presented in fig. 1.1

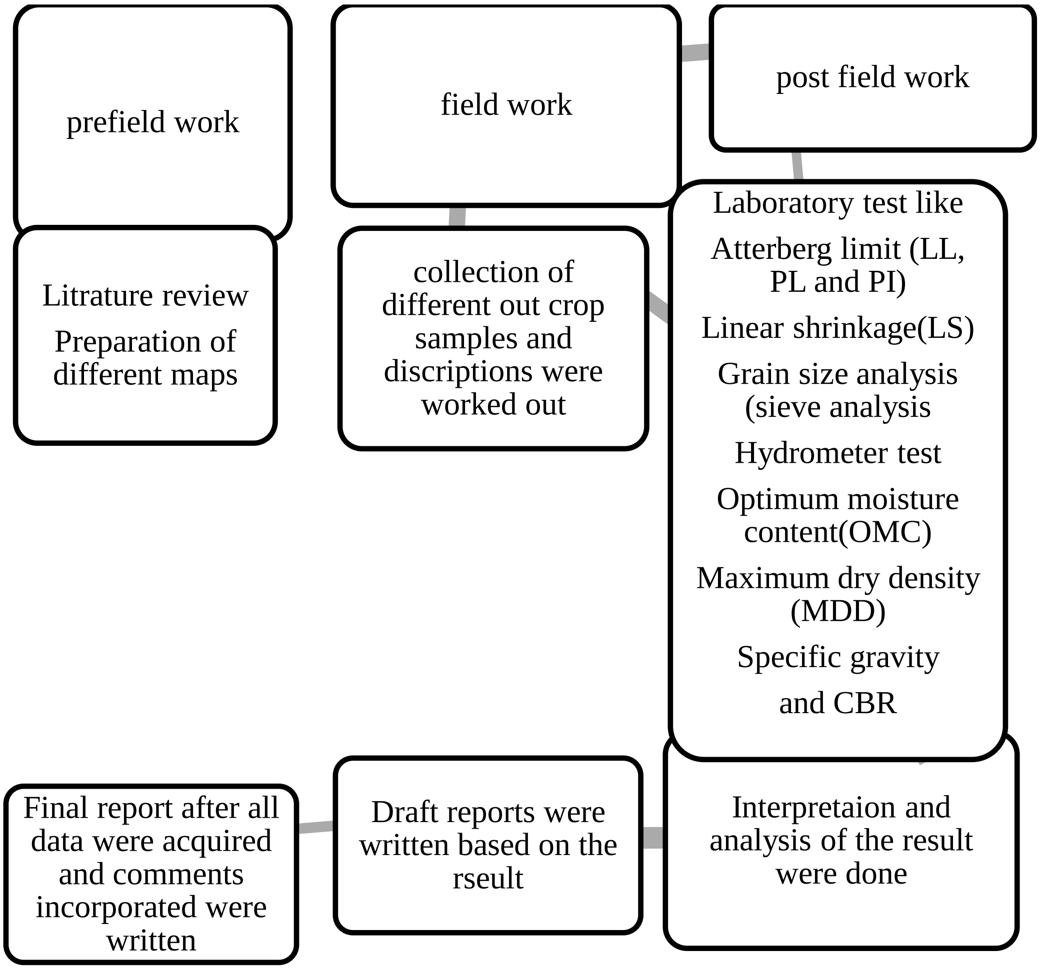

Fig1.1 flow chart of general methodology adopted for the present study.

1.6 Significances of the research

The durability and stability of all the engineering structures are directly dependent on soil type and its response under loading condition. The present study indicates the performance of subgrade soil along the route corridor of Modjo Hawassa expressway, lot two Meki-Ziway section. Besides, it gives the possible improvement methods for the identified problematic section of subgrade soil along this route alignment.

1.8 Limitations

During the present study; disturbed sampling method was adopted and this might affected the quality of the result. Further, every effort was made to perform the present study in a scientific and logical manner under the constraints of frequent failure of CBR testing machine, Liquid limits apparatus, limited materials testing equipment's and variability of the soils moisture contents.

1.9 Scheme of the presentation

The research project represents systematically the detailed sub-grade characterization and possible improvement of sub grade for Meki-Ziway High way as case study. It takes account of the following listed chapters with their contents.

Chapter 1. It covers the introduction with the background, justification of the problem as well as the objectives, methodologies, outcomes of the present study, Significances of the research, limitation of the research and finally scheme of the presentation.

Chapter 2. This chapter present reviews of the literature on sub-grade and methodology in different literature to characterize sub-grade soil.

Chapter 3 present the general over view of the study area. This chapter contains the locations, the accessibility, and length of the route, climate and rainfall, physiography, seismicity of the study area, Geological set up of the study area

Chapter 4. It presents sub-grades characterization and Interpretation. It covers the sub-grade soil investigation including field investigation, sampling, description and laboratory testing result, soil classification, analysis and interpretation of field and laboratory test investigations.

Chapter 5 deals about chemical stabilization of sub grades soils.

Chapter 6. This chapter presents the conclusion and recommendations that include the researcher's scientific and logical recommendation

Chapter 2

Literature Review

2.1 Introduction

Reviews of literatures were carried out exhaustively to get adequate knowledge related to the subject of this study. The reviews presented in the subsequent part primarily focusing on soil formation, unsuitable soils, soil classification, sub grade bearing strength classes, delineation of homogenous sub grade materials, problems associated with expansive soils, and endorsements for construction on problematic soils. The literature sources were used previous works, project documents, reference books, journals and manuals of other countries with similar physical characteristics to Ethiopia.

2.2 Soil Formation

According to Hunt (2005), soils of several origins and modes of occurrence or deposition have characteristic terrain features of landform, drainage patterns, and vegetation, which used as indicators to provide the basis for their identification, the anticipation of structural features such as stratification, and the estimation of their characteristic engineering properties.

Most significantly, soils origin and mode of occurrence affect their gradation characteristics as well as their stresses. Soil properties reflect residual parent rock type. For instance, relatively thin deposits of inactive clays are normal in most sedimentary and sialic igneous rocks; thick deposits of inactive clays are normal in foliated rocks in moist climates; active clays develop from basic rocks and marine shale in moist climates (Hunt, 2005).

Ferromagnesian produce clays and Fe_2O_3, which in humid climates give residual soils a strong red color, or when concentrations are weaker, yellow or brown colors. Muscovite (white mica) is relatively stable and is often found unaltered, since it is one of the last minerals to decompose (biotite, black mica, decomposes relatively rapidly).

Quartz, the predominant oxide, essentially undergoes no change, except for some slight solution, and produces silt and sand particles (Hunt, 2005). Composed chiefly of quartz grains, most sandstone develop a profile of limited thickness, which results from the decomposition of impurities, or untethered materials, such as feldspar.

The formation of clay particles from rocks can take place either by the build-up of the mineral particles from components in solution, or by the chemical breakdown of other minerals.

2.3 Subgrade soil.

According to Jyoti, (2013), Subgrade soils are crucial element of pavement structures, and inadequate subgrade performance is the cause of many premature pavement failures. In addition, the author explain that, clay subgrades soil may provide inadequate support, particularly when they are saturated. Soils with high plasticity index may also shrink and swell substantially with changes in moisture conditions. These variations in volume can cause the pavement to shift or heave with changes in moisture content, and may cause a reduction in the density and strength of the subgrade, accelerating pavement deterioration.

2.3 Problematic sub-grade soil.

2.3.1 Expansive soils

Expansive soil refers to a soil that has the potential for swelling and shrinking due to changing moisture condition. Expansive soils cause more destruction to structures particularly pavements and light buildings than any other natural hazard, including earthquakes and floods**.**

Chen (1988) stated that, the origin of expansive soils related to a complex combination of conditions and processes that result in the formation of clay minerals having a particular chemical makeup, which, when in contact with water, will expand.

Variations in the situations and processes may also form other clay minerals, most of which are non-expansive. Clay soils are difficulties in road construction and are a relatively common problem in Ethiopia (ERA 2013b).

2.3.1.1 Characteristics of Expansive Soils

Soils usually encounter as sub-grade material during any road construction project and these sub-grade soils may vary from highly expansive to non-expansive in nature. Therefore, these sub-grade soils should be adequately investigated since the stability and performance of the pavements are greatly influenced by the nature of sub-grade soil as it serves as foundations for pavements (Chen, 1988).

According to Rao (2007), Expansive soils absorb water heavily, swell, become soft and lose strength. These soils are easily compressible when wet and possesses a tendency to heave during wet condition and shrink in volume and develop cracks during dry seasons of the year.

Holland and Rechards (1982) out lined that horizontal and vertical movements manifest the seasonal changes in volume of expansive soil.

The horizontal movements lead to fissure opening during drying and closing during wetting whereas the vertical movement leads to cyclic changes in soil levels. The authors described that the magnitude of these movements decreases with depth where there are no seasonal moisture changes.

2.3.1.2 Identification of Expansive Soils

West (1995) considers that the Atterberg limits provide a means of determining the standard of performance and a level of sensitivity to volumetric change due to moisture influx. The Atterberg limits consist of the liquid limit (LL), plastic limit (PL) and similarly the liquidity and plasticity indices (LI and PI).

Tanzanian Pavement and Materials Design Manual (TPMDM, 1999) suggest that, the following stages of investigation programme for identify of expansive soils.

- Routine investigations are those carried out during surveys of all projects.
- Extended investigations include simple additional indicator testing in the laboratory when expansive soils are suspected.

2.3.1.3 Problem associated with Expansive soil

Chen (1988) mentioned that the problem of expansive soils is widespread throughout world: including; Australia, United States, Canada, China, Israel, India and Egypt.

Road Design Manual of Ministry of Transport and Communication Roads Department, Republic of Kenya, Part III (1987) lists the following three basic problems of expansive soils.

- Volume changes: expansive soils shrink and crack when they dry out and swell when they get wet.
- The cracks allow water to penetrate deep into the soil, hence causing considerable expansion. This consequence in deformation of the road surface, then the expansion and the subsequent heave are never uniform.

Furthermore, these volume changes may produce lateral displacements (“creep”) of the expansive clay, if the side slopes are not gentle enough. Seasonal wetting causes the road edges to wet and dry at a different rate than those under the surfacing.

This mechanism in turn causes differential movements over the cross section of the road and associated crack developments, first occurring in the shoulder area, and subsequently developing in the carriageway.

Problems associated with expansive sub grade soils arise mostly from the nature of the soil and drainage facilities available (Rao, 2007). Expansive soils have a low California bearing ratio value in the range of 2 to 4% that they have low strength to support the loads transmitted from the pavement structure and results in excessive deformation beyond permissible limits (Rao, 2007; Seehra, 2008).

2.3.3 Improvement of Expansive Sub-Grade Soils

Even, there are different methods of soil stabilization; Chemical stabilization is good method to improve poor sub-grade soil with fine grain size. Chemicals such as lime improves the low load bearing capacity of poor sub-grade soil and lower the plasticity index and percent swell of unsuitable sub-grade soil, (Chen,1988; Littleand Nair,2009, McKeen,1956).

Expansive soils do not meet the requirements for what and may need improvements to their engineering properties in order to transform them in to effective construction materials.

This is often accomplished by physical or chemical stabilization or modification of these problematic soils (Littleand air, 2009).

According to ERA (2013 a), the minimum acceptable strength of a stabilized material depends on its position in the pavement structure and the level of traffic. It must be sufficiently strong to resist traffic stresses but upper limits of strength are usually set to minimize the risk of reflection cracking.

Accordingly, to improve the weak sub grade soil, there should be selection of the treatment methods and this selection of the stabilizing methods can be based on the plasticity and particle size distribution of the material to be treated.

There are different practice to stabilize problematic subgrade soil materials: Arora (1992)

- Thermal stabilization
- Electrical stabilization
- Bituminous stabilization
- Mechanical stabilization
- Stabilization by grouting
- Stabilization by geotextile and fabrics

2.3.3.1 Cement Stabilization

The cement content controls whether the characteristics of the mixture are dominated by the properties of the original soil or by the hydration products. As the proportion of cement in the mixture increases, so the strength increases and increases with time. (ERA 2013 a).

2.3.3.2 Lime Stabilization.

Chen, (1988); Little and Nair, (2009), states that lime and cements are the main chemical stabilizers and the selection of those chemicals has been done based on their index properties as and Grain size distribution (percent passing the no.200 sieve). Accordingly, the authors suggest that the sub grade soil having, PI > 10 and clay content (2µ) > 10% should be treated by lime. Adding different percent of lime with expansive soil brings in changes in Atterberg limit.

In different kinds of literature, it is showed that increase in both liquid and plastic limits and there is a reduction in the Plasticity Index. In addition to these lime treated soil result in increases of the amount OMC and decreases the amount of MDD (Bayat et al., 2013).

2.4 Collapsible soil

According to Khaled (2012), Collapsible soils are defined as unsaturated soils that can sustain substantially high-applied vertical stress without showing significant change in volume. When wetted, collapsible soils susceptible to large and sudden reduction in volume.

According to ERA (2013b), collapsible soils are distributed in some part of Ethiopia, among them, Ziway, Shashemene and Hawassa that are manifested by the occurrence of ground crack and potholes during heavy rain due to hydro- compaction.

Naema, A. A. (2015), explain that, the Collapsible soils are generally low-density, fine-grained combinations of clay and sand left by mudflows that have dried, leaving tiny air pockets and when the soil is dry, the cemented materials are strong enough to bond the sand particles together. However, when natural soil becomes wet, moisture alters the cementation structure and the soil's strength compromised, causing collapse or subsidence.

2.4.1 Characteristics of collapsible soil

Many researchers reported that lack of knowledge in the construction industry with respect to identification, behavior/ characteristics and treatment of collapsing soils led to many cases of foundation problems.

According to Houston, et al. (2002), collapsible soils can show large volume variation upon moistening, with or and sometimes without additional loading and thus posing significant challenges to the geotechnical profession. Pereira et al. (2000) concise the factors that produce collapse as follows:

- An open, partially unstable
- A high enough net total stress that will cause the structure to be metastable.
- A cementing agent that stabilizes the soil in the unsaturated state.
- The addition of water to the soil, which causes the bonding or cementing agent to be reduced and the inter-granular contacts to fail in shear, resulting in reduction in total volume of the soil mass.

According to Mitchell and Soga (2005), collapsible soils are commonly related with an open structure formed by sharp grains, low initial density, low natural water content, low plasticity, relatively high rigidity and strength in the dry state, and often by particle size in the silt to fine sand range. In most cases, they contain over sixty(60%) of fines and have a porosity of 50% to 60%, liquid limit of about 25 and plastic limit ranging from 0 to 10.

As Dudley, H. (1970 cited in Naema, A. A. 2015), the major soil types that lead to substantial collapse included soil classified as CL and ML soils, and the dry density and plasticity of collapsible soils are together low and accordingly the liquid limit, plasticity index and natural moisture content are usually lower than 45%, 25% and 10%, respectively.

2.4.2 Identification of collapsible soils

Gibbs et al., (1962) discussed that the identification and prediction of the collapse soil have proven difficult because of no single criterion that can be applied to all collapsible soils. Routine and sophisticated tests do not always reliably indicate the existence of collapsible soils because of many different configurations, which hold the bulky grains in place. (Table2.1of the indicates collapse potential of the collapsible soil)

Habibagahi et al,. (2004) printed out that, some criteria for identifying collapsible soils have been described as having low density, high porosity (more than 40%) and low saturation (less than 60%); open partially unstable structure and unsaturated fabric; high silt content (more than 30% and sometimes more than 90%) and sand size with a small amount of clay.

Table 2.1 Classification of collapsible soils, Jennings and Knight (1975).

Collapse Potential (%)	Severity of Collapse Problem
0-1	No problem
1-5	Moderate trouble
5-10	Trouble
10-20	Severe trouble
>20	very Severe trouble

2.4.3 Problem associated with collapsible sub-grade soils

According to Briscolland. R and Chown. R (2001), many soils can cause failure in geotechnical engineering because of their property of expansion, collapsing, disperse, undergo excessive settlement and have distinct lack of strength. Such characteristics may be attributable to their composition, the nature of their pore fluids, their mineralogy or their fabric and soils are moisture sensitive in that increase in moisture content is the primary triggering mechanism for the volume reduction of these soils.

Jotisankasa. A. (2005) describes that the collapsible soils normally possess porous textures with high void ratios and have relatively low densities. They often have adequate void space in their natural state to hold their liquid limit moisture content at saturation.

At their natural low moisture content, these soils possess high apparent strength but they are susceptible to large reductions in void ratio upon wetting.

2.4.4 Improvement of collapsible Sub-Grade Soils

According to Sheckle, D. (1988), various methods to remove the collapse potential from potentially collapsible subgrades had utilized, but essentially heavy compaction of the wet material is necessary.

The author explains that, in assessing mitigation measures, the issue of shallow or the deep deposits should be considered, and that distinction is often not clear-cut. Some mitigation measures such as compaction with rollers or tampers, will almost certainly not produce results for deep deposits, and may or may not work for shallow deposits. In any case, each situation should be evaluated and often several methods used before satisfactory results are achieved.

According to Houston.S, (1988), identification of the collapsible soil and severity of settlement potential is the initial information needed for the mitigation design. The achievability of possible mitigation approaches at a given project site depend on the structure conditions and the level of risk that the project owner is willing to accept.

More options will generally be available to new construction compared to existing structures where there are limitations to mitigation options.

Naema. A. (2016) conclude that, removal of some thickness of collapsible soil and replacing it by cohesion less soil and altering surface and subsurface drainage patterns of water on collapsible soil improve the stability of collapsible soil formation. According to this author, the soil partially replaced with compacted cohesion less soil reduces the foundation settlement by about half and increased bearing capacity by about (80-100%).

2.5 Soil classification

The Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes, AASHTO designation: M 145-91 described a procedure for classifying soils into seven groups (A-1 to A-7) based on laboratory determination of particle-size distribution, liquid limit, and plasticity index. The AASHTO Classification System is mostly applicable in highway works to check the suitability of material used in pavement structures, subgrades, embankments, sub bases and base courses.

Evaluation of soils within each group is made by means of a “group index”, which is a value calculated from an empirical formula. Group index = (F - 35) [0.2 + 0.005(LL – 40)] + 0.01(F - 15) (PI – 10) in which, F = percentage passing 0.075-mm (No. 200) sieve, expressed as a whole number.

$$GI = (F-35)\,[0.2+0.005(LL-40)] + 0.01(F-15)\,(PI-10) \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \; 2.1$$

LL = liquid limit PI = plasticity index

Group classification including group index should be useful in determining the relative quality of the soil material for use in earthwork structures, particularly embankments, subgrades, sub-bases, and bases. Some of the classification systems are the AASHTO Classification System, Unified Soil Classification System.

All classifications of soil for engineering purposes use particle size distribution analysis and Atterberg Limits (LL and PL) as delimiting parameters. The subgrade soils materials have been classified in the field and to verify this first classification for this research work, ASHTO Classification System (AASHTO M 145) and Unified Soil Classification System (ASTM D 2487) have been used.

As it is described by FHWA (2006), the first term of Equation 2.1 is the partial group index determined from the liquid limit. The second term is the partial group index determined from the plasticity index. Following are some rules for determining group index:

If Equation 2.1 yields a negative value for GI, it is taken as zero and the group index calculated from Equation 2.1 is rounded off to the nearest whole number.

The group index of soils belonging to groups A-1-a, A-1-b, A-2-4, A-2-5, and A-3 will always be zero. When the group index for soils belonging to groups A-2-6 and A-2-7 is calculated, the partial group index for PI should be used.

$$GI = 0.01(F-15)(PI-10) \qquad \text{……………………………………….2.2}$$

Under average conditions of good drainage and thorough compaction, the supportive value of a material can expected as an inverse ratio to its group index, i.e. a group index of zero indicates a “good” subgrade material and a group index of 20 or more indicates a poor subgrade material. The group index is used in the grouping of soils and the higher the value of the group index, the poorer is the subgrade (Arora, 1997).

Based on the particle size analysis result the AASHTO Soil Classification categorizes soils into two major groups, the granular materials with 35% or less passing the No. 200 sieve and the silty-clay material with more than 35% passing the No. 200 sieve. While from the Unified Soil Classification System soils can be classified into two main groups as coarse-grained soils those having 50% or less passing the No.200 sieve and the fine-grained are those having more than 50% passing the No. 200 sieve.

2.6 Sub-grad bearing strength classes

For a rational approach to pavement design, the most important characteristic of the subgrade is its elastic modulus. However, the measurement of this modulus requires fairly complicated and time-consuming tests.

ERA (2013b) emphasizes a category of soil should include the soils of the same type having consistent geotechnical characteristics (Grading, Atterberg Limits, Compaction and particularly CBR). Usually, the number of soil categories will not exceed 4 or 5 for a given road project. It is advisable to avoid introducing short sections along the alignment with numerous changes in the soil categories as this can make the construction operations overly complicated. For pavement design, the road sections should be defined in accordance with subgrade strength classes (table2.2).

Table 1.2 Subgrade Strength Classes ERA Flexible Pavement design manual (2013a)

Subgrade Strength Classes	CBR range %
S1	<3
S2	3,4
S3	5,6,7
S4	8-14
S5	15-30
S6	>30

Almost all types of soil, ranging from sandy clays through to broken rock, can be used for embankment construction and pavement support, the main limitation being the ease with which the material can be handled and compacted. However, materials with CBRs less than 5 are usually very difficult to work as subgrade. If they must be used, select subgrade materials must cover them or capping layers (ERA, 2013b). Based on the Tanzanian Pavement and Material Design Manual (1999), the subgrade shall be classified according to its CBR strength as shown in Table2. 3

Table 2.2 Subgrade strength classes (Tanzania Design Manual, 1999).

Subgrade strength classes	Wet or moderate climatic zones 4 days soaked CBR value	Dry climatic zones (both requirements shall be met)		Number of Blows
		Tested at OMC	4 days soaked value	
S15	Min 15	Min 15	Min 7	95 BS-Heavy
S7	7-14	7-14	3-14	93 BS-Heavy
S3	3-6	3-6	2-6	100 BS-Heavy

2.7 Delineation of homogeneous sub-grade section

ERA (2013a) suggests a road section for which a pavement design is undertaken should be subdivided into subgrade areas, where the subgrade CBR can be reasonably expected to be uniform, i.e. without significant variations.

Nevertheless, it is not practical to create delineation between subgrade areas that would be too precise, and indeed this could be the source of confusion during construction.

The soils investigations should delineate subgrade design units based on geology, pedology, drainage conditions and topography, and consider soil categories, which have consistent geotechnical characteristics.

Generally, it is sensible to avoid short design sections along the alignment.

Where the subgrade CBR values are very inconstant, the design should consider the respective benefits and costs of short sections and of a conservative method based on the worst conditions over longer sections. It is important to differentiate between localized poor (or good) soils and general subgrade areas. Normally, localized poor soils will be removed and replaced with suitable materials. In the CDA, the term Z_x is defined as the difference in the cumulative area values at a given distance (x) between the actual response and the average response (AASHTO, 1993).

For the present study the delineation of homogenous sections of the road alignment has been done based on the cumulative difference, method presented in Appendix 8 of Tanzanian Pavement and Materials Design Manual (1999) and in Appendix J of AASHTO (1993) Pavement Design Guide. Detail calculations of the unit delineation by cumulative sums method values has shown in Appendix I.

Design CBR (CBR_{design}) has made based on the newly developed method by Virginia department of Transportation in 2000 to calculate design CBR value for the present study.

According to Virginia Department of Transportation Pavement Design Guide (2000), design CBR is a factor of the number of CBR test results available in homogeneous sections. Accordingly;

- ✓ For five tests or less tests in one homogeneous section, the design CBR shall be the average of these tests multiplied by two-thirds, rejecting any obviously extreme value.
- ✓ For more than five tests, the highest and lowest CBR values are rejected and the Design CBR value shall be the mathematical average of remaining CBR test values multiplied by 2/3

Design CBR = Average CBR x 2/3…………………………………… ……… eq. 2.3

2.8 Previous work

Nibret Chane, (2011) "Geotechnical classification of subgrade materials on Aposto – Wondo Negele road" by identification of the presence of expansive soil and characterization of its geotechnical properties (Atterberg limits, grading, MDD and OMC, CBR and swell values) under adverse condition. After the laboratory and field test the researcher conclude that at least 82% of the sub-grade soils in the study area are found to be suitable for bearing stratum and construction materials.

Habtamu Solomon (2011) has characterized the subgrade expansive soils in the Northeastern Ethiopia, to evaluate the performance of chemically stabilized problematic soil along the road path. By having different field and laboratory work on geotechnical characteristics of expansive soil, he concluded that expansive soil is poor in its performance as sub grade materials. Consequently, he recommends some measures such as removing-back filling with other competent Martials and using some chemical stabilization methods depending on economic feasibility and availability of filling material at economic distance.

Abinet G/medhin (2006) has conduct research on Engineering geological problems and counter measurements for flexible placement on expansive soil on Jimma road project.

He suggests that attempt have been made within a frame work of identification of expansive soil, problem associated with pavements, factors responsible for failure of pavements and possible remedial measures for road construction on expansive soils.

Yitagesu Desalegn (2012) has conducted research on correlation between DCP and CBR for locally used subgrade material. Accordingly, a study was carried out to find the correlation between Cone Penetrometer (DCP) with CBR values that best suit the type of soils in Ethiopia. From the tests, the Atterberg limits (PI, LL, PL), In situ density, classification, California bearing ratio, insitu Moisture Content and results were acquired.

Zeleke Tadesse (2013) has conducted research on pavement distress along Addis Ababa ring road to assess causative factors responsible for pavement distress and to evaluate rehabilitation measures that may help to reduce such distress of pavement and increases period of its serviceability. From his research, the most distress causing factors are rock and soil types, geology, drainage, Elevation (Altitude), Slope terrain, Curvature, Slope aspect and Structural set up (Fault line) of the area. Most of these factors are related with geology of sub-grade materials.

Biniam Alemu (2010) Engineering geological characterization of Granular sub-base using lime a case of Ambo –Gedo road projects. He has characterized that sub grade and borrow material are the two major tasks in pavement design which determine the pavement quality and pavement type.

In addition, he recommends that, determination of initial consumption of lime on the granular sub-base, and evaluation of the stabilized sub-base are the key and lime is the stabilizing agents to improve the engineering properties of the moderately plastic sub-base where quality granular material is not available in haul distance based on economic and engineering assessment.

On other hand, ETHIO-Infra Engineering PLC (2014) has conducted the Soils and Materials investigation as a part of the review of the Concept Design performed by the Design Consultant. Accordingly, soil classification was did and fall predominantly under A-7-5/6 and A-4. Soil groups are clayey soils that cover about 38.6% of the route corridor. The A-4 soil group covers about 36.4% of the route corridor and they are mostly pumiceous silty soils and the remaining A-6, A-2 and A-5 cover about 25.0% of the route corridor. Laboratory test results indicates that, Plasticity Index (PI) varies between 4% and 24%, CBR at 93% maximum dry density for modified showed highly variable CBR values ranging from weak sub grade soils with CBR of 4 and good sub grade material with high CBR values up to 68.

Chapter 3

General over view of the study area

3.1Introduction

The formation of soils depends on the prevailing environmental factors, geologic and physiographic setup of an area. Roads forms an integral and important part of any transportation system. A city's road network should be efficient in order to maximize economic and social benefits. They play a significant role in achieving national development and contributing to the overall performance and social functioning of the community.

Therefore, usage of roads will be very significant if there is no seaport in the country, just like the case in Ethiopia. Roads enhance mobility take people out of isolation and helps in bringing prosperity in general.

3.2 Location of the study area

The study area is located in the central main rift of Ethiopia in the Oromia National Regional State, UTM coordinates of 8^{0}09'11.8" Northing, 38^{0}49' 452.5" Easting and 7^{0} 55'36.8" Northing; 38^{0} 42'57.7". The proposed study area Meki-Ziway High way project is part of the Modjo-Hawassa High way project that is under construction.

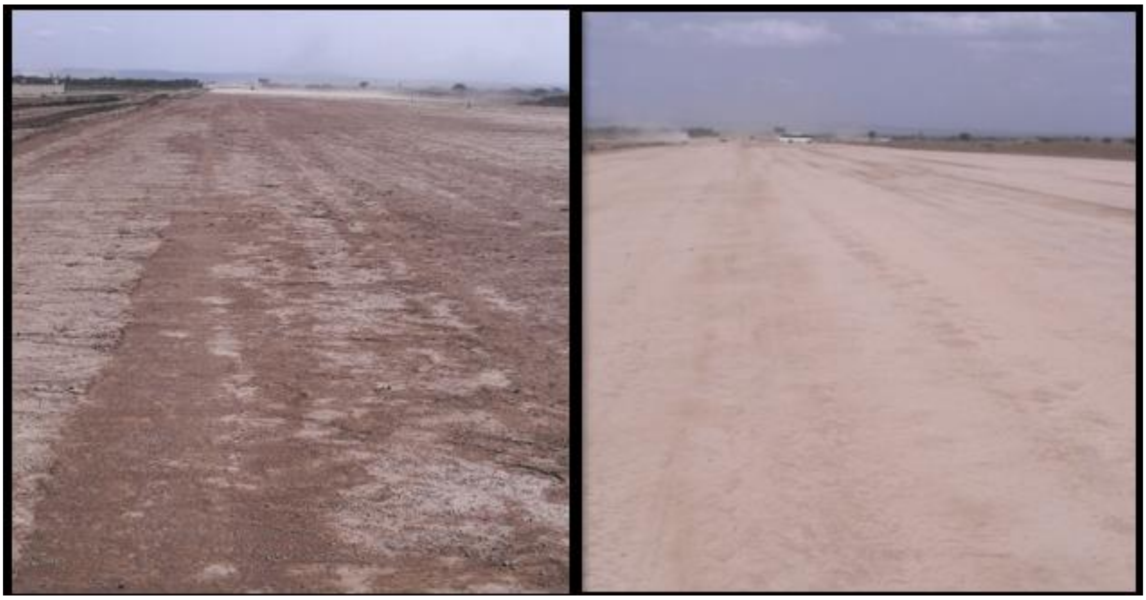

Fig 3.1 indicate the flat topography along the Modjo-Hawassa expressway.

Meki-Ziway Road Project starts before Meki town in Oromia National Regional State (ONRS) and terminates west of Ziway town with a total length of 37km.

The road way traverses a flat topography 3 km west of the existing Modjo-Hawassa road. (See fig3.1 and 3.2).

The roadway lies entirely to the west side of the existing Modjo-Hawassa old road (leaving Meki and Ziway to its east and connected to these towns with link road through grade-separated controlled access / intersection. According ERA, (2015), over 95% of the total road route length, fall in flat plain topography with poor drainage.

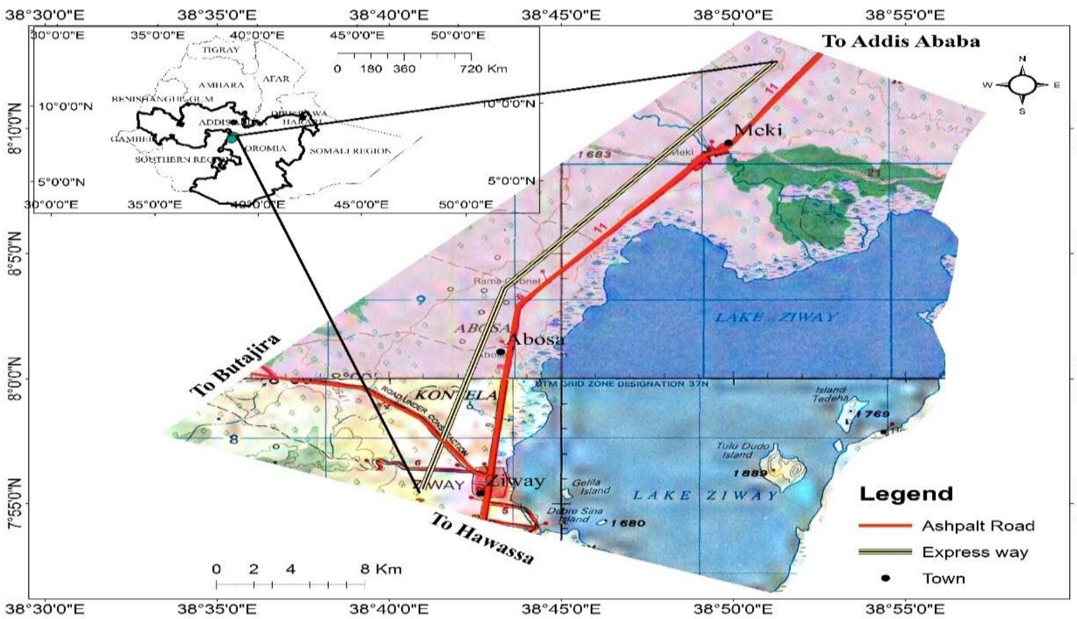

Fig 2.2 Location map of the study area

3. 3 Land use land cove

Based on the field assessment the project area dominantly covered by cultivated lands. The agricultural crops like maize that is common in the area. The other land cover types of the area are water bodies, shrub lands and few woodlands with dominancy of acacia plants and some settlements.

The project area is predominantly located in the range of Acacia-Comiphora ecosystem and the vegetation types ranges from tropical woodland to bushed grassland on the rift floor. Dense acacia woodland used to be the dominant natural vegetation in the road project and surrounding environment. Fig 3.3 indicates land use land cover of the present study area.

Fig 3.3 Photo shows distribution of vegetation (a) and agricultural activities (b) in the study area.

The majority of the proposed road route corridor (97%) traverses in smallholder mixed crop cultivations (main crops being Maize, Sorghum, and Wheat etc.) (Fig3.4).

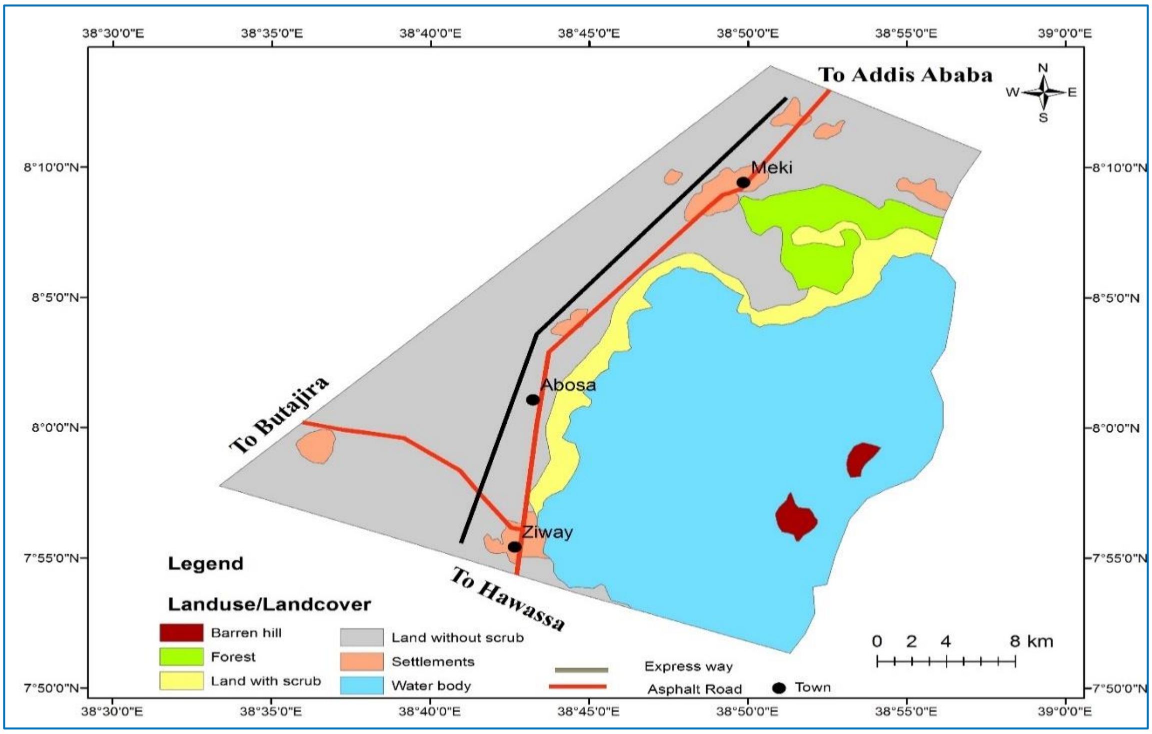

Fig 3.4 Land use land cover map of the study area.

3.4 Landform and Geomorphology

The project area lies within the main Ethiopian rift system. The road alignment traverses dominantly through flat terrain.

From GPS data from field, the elevation of the project area is in the range of 1600m to1962m a.s.l and decrease gradually from Meki to Ziway direction. Topographic and geomorphic features of the study are shown in fig 3.5.

According to Daniel Gemechu (1977) climatic zone classification based on elevation, the study area falls in subtropical (1500-2300m) climatic zone (Fig 3.5.)

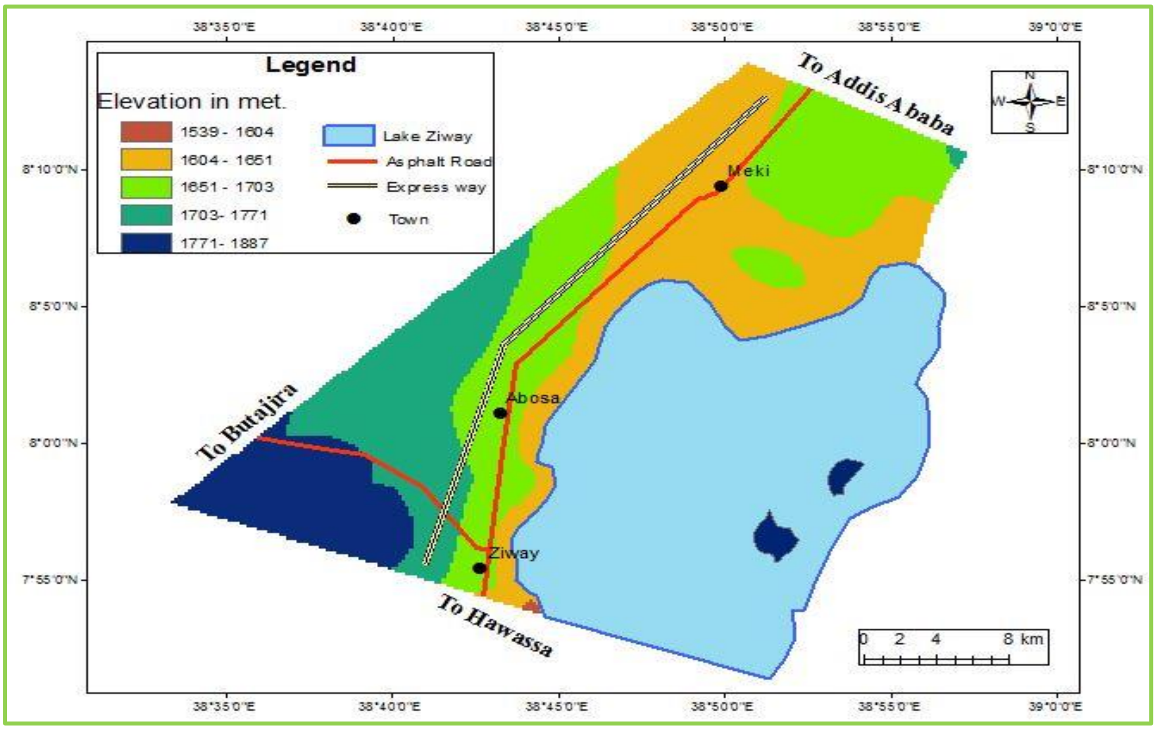

Fig 3.5 Elevation map of the study area

Climate

According to ERA (2013a), climate can affect the nature of the soils and rocks encountered at subgrade level The Principal effects of climate on pavement design are in relation to sub-grade moisture condition, drainage requirement, surfacing type selection and design and the selection of construction materials (AASHTO, 1993).

Gissila etal, (2004), prints that, Ethiopia has three climatological rainy seasons: June–September (called Kiremt), October–January (Bega), and February–May (Belg).

The rainfall data collected from Meki Metrological station showed that the area gets 1171.2 mm annual average precipitation (data for the period 1986-2015(Fig.2).The highest monthly average precipitation recorded in the area was 489.3 mm in July 2012. The area is characterized by uni-modal rainfall pattern with only one distinct rainy season (i.e. June to September) (Fig 3.6).

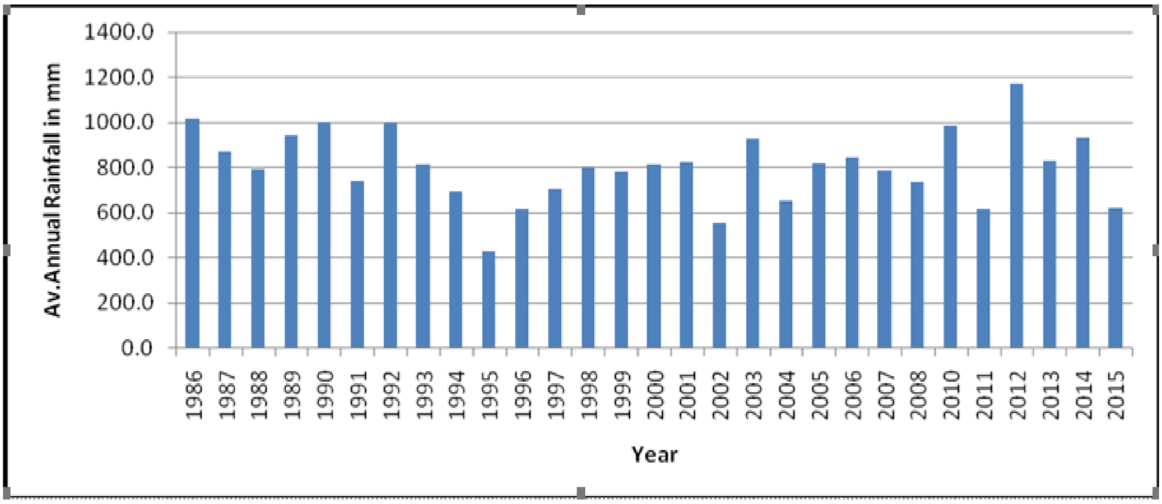

Fig 3.6 Annual average rainfall of Meki station from 1986-2015.

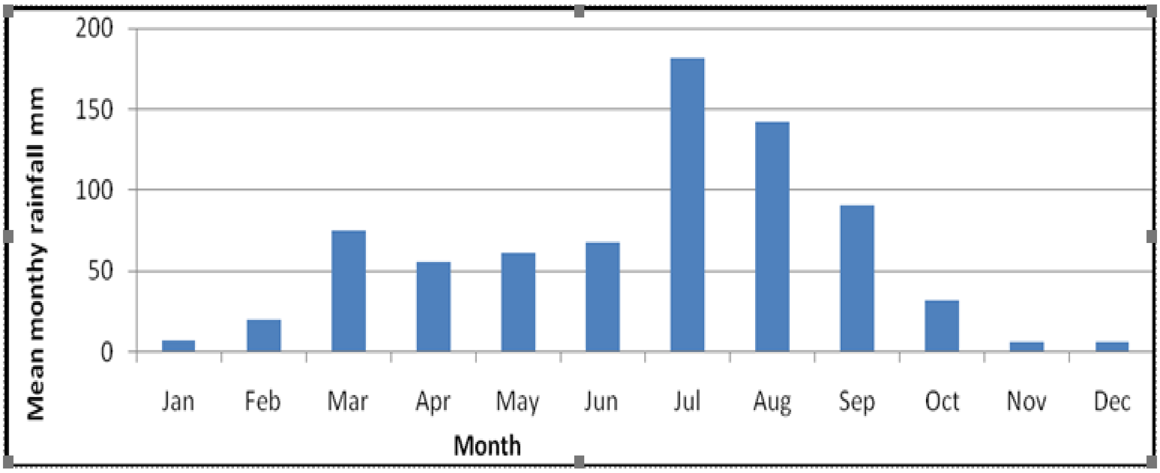

Fig 3.7 Mean monthly rainfall of Meki station from 1986-2015.

The temperature data collected indicates that, the area gets 9.4c^0 and 30 c^0 average minimum and maximum temperature respectively (Fig3.8).

Year	1986	1987	1988	1989	1990	1991	1992	1993	1994	1995	1996	1997	1998	1999	2000
Av.Max	27.9	28.0	27.9	27.6	27.7	28.2	28.2	28.3	29.0	28.3	28.1	28.3	28.7	29.8	28.2
Av.Min	10.5	13.3	12.6	12.2	12.2	12.8	12.6	12.2	12.3	12.8	11.9	12.8	12.3	11.8	12.2

Year	2001	2002	2003	2004	2005	2006	2007	2008	2009	2010	2011	2012	2013	2014	2015
Av.Max	28.8	29.0	29.1	29.9	29.8	30.0	31.2	28.0	29.3	28.3	29.6	30.0	29.2	29.6	29.9
Av.Min	11.6	11.5	10.1	9.4	8.6	10.2	10.3	11.9	12.2	12.6	11.9	11.4	11.5	11.6	11.7

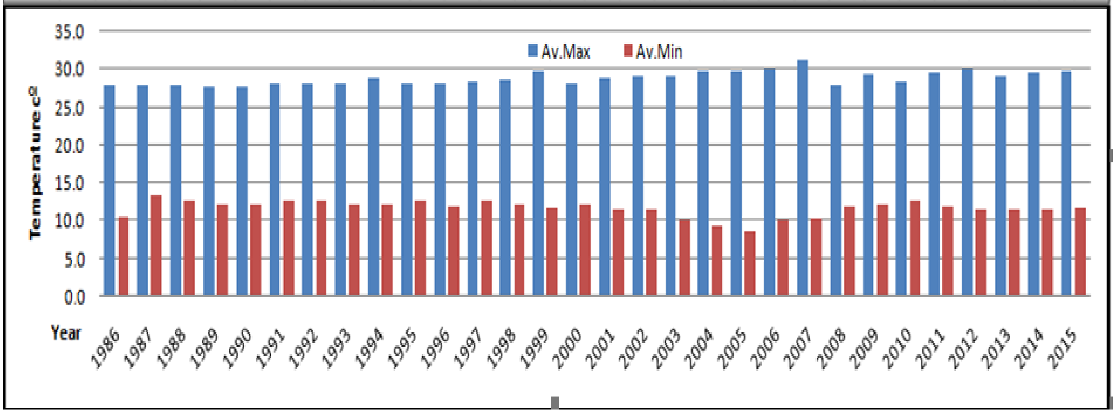

Fig 3.8 Mean Maximum and minimum temperature of Meki station (1986-2015).

3.6 Drainage patter

As it was presented in Fig (3.10), in the present study area, there is no more drainage system except Meki River that flows to Ziway Lake.

According to Halcrow (2007), the Central Rift Valley consists of a chain of lakes, streams and wetlands and the route corridor lies on the flat plane that can reduce the erosion. Even though, there is no effective drainage system in the area. As a result, settlements, agricultural activities are damaged and there is the formation of swamps in the rain season. Besides, gully erosion developments are also the active geodynamic processes in the area. (Fig 3.9).

Fig 3.9 Formation of gully erosion in the study area.

Being it is a closed basin; the central rift valley is one of the environmentally very vulnerable areas in Ethiopia. The main river in the study area is Meki River, which flow to Ziway Lake. (Fig 3.10).

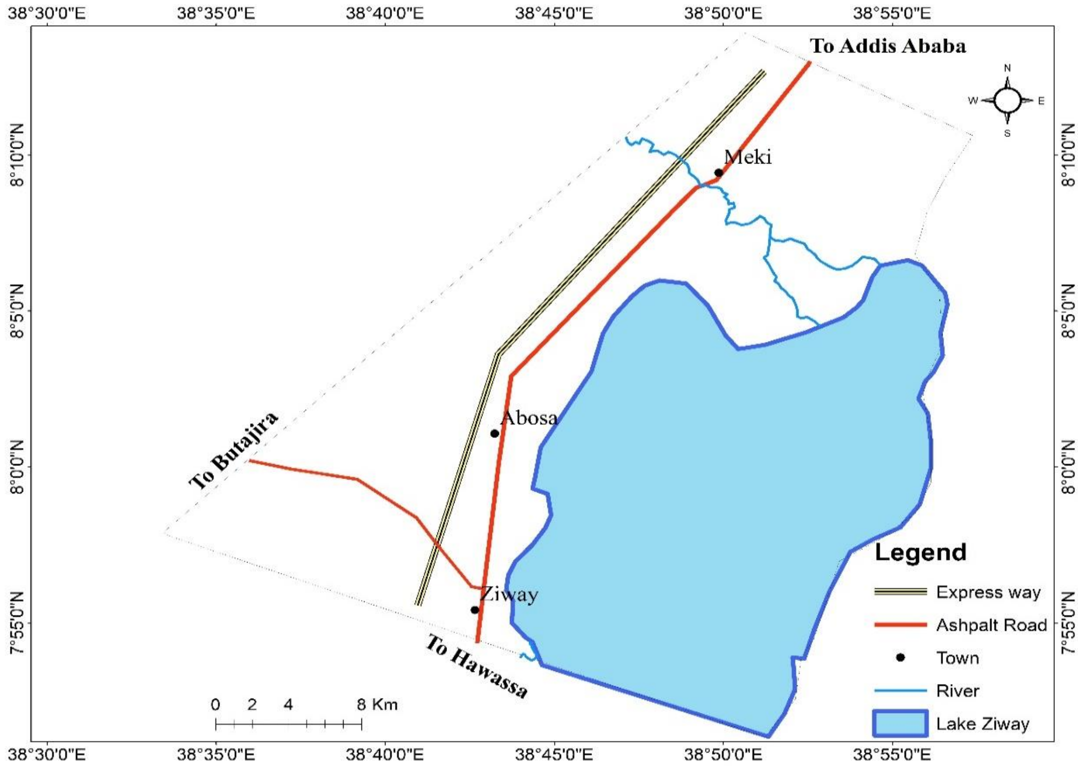

Fig 3.10 Drainage map of the study area

3.8 Regional Geology

The Ethiopian Rift (ER) is part of the east African Rift System (EARS) and comprises a series of rift zones extending from the afar triple junction at the Red Sea and Gulf of Aden intersection to the Kenya rift. It constitutes the northernmost part of the East African Rift System (EARS), connecting the EARS with the afar triple junction and is an area characterized by active extensional tectonics and volcanism. (WoldeGabriel et al., 1990).

According to WoldeGabriel et al., (1990), the Main Ethiopian Rift divided in to three sub sectors: northern, central and southern sectors and bordered by the Ethiopian plateau to the west and Somalia plateau to the east. It is a NNE–SSW to N–S trending trough 80 km wide in its central portion and 1000 km long.

According to Corti (2009), the MER volcanic stratigraphy was summarized as a lower basalt unit with trachy basalts and subordinate silicic flows from 11 to 8 Ma old followed by a widespread ignimbrite cover ranging in age from 7 Ma in the northern sector to 2 Ma to the south and up to 700 m thick.

The Main Ethiopian Rift (MER) is a roughly NE trending sector of the East African Rift system that includes a series of rift segments extending from the Afar Triple Junction at the Red Sea-Gulf of Aden intersection to the Kenya Rift (Fernandes et al., 2004) (fig 3.11).

According to this author, most of the ignimbrite layers believed to have formed by catastrophic eruptions related to the collapse of large calderas, such as the 3.5-Maold Munesa caldera now buried beneath the Ziway - Shala lakes.

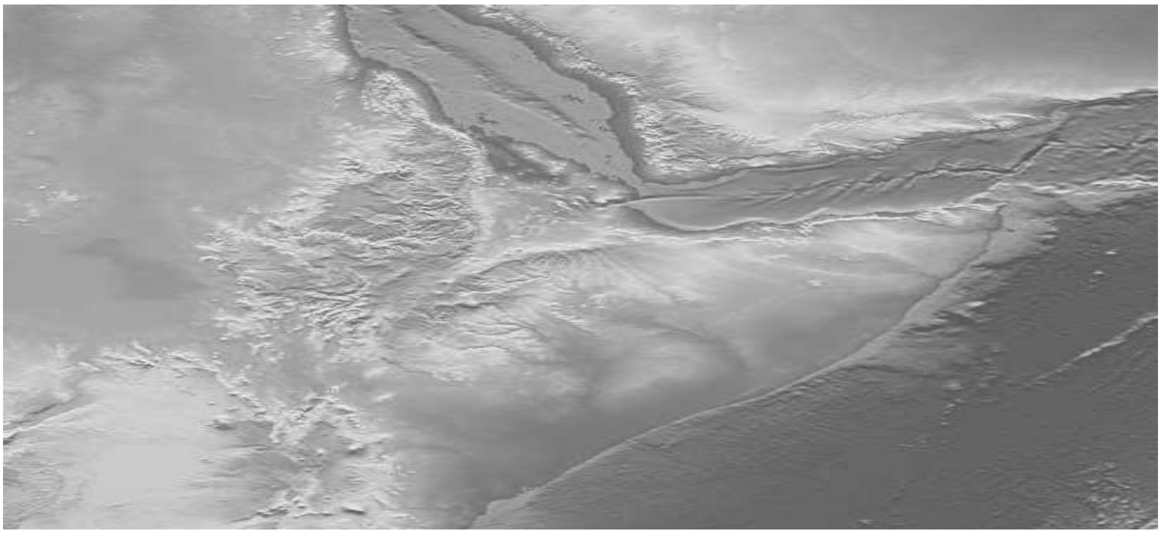

Fig 3. 11Geodynamic setting of the Main Ethiopian Rift. Adopted from Fernandez et al., (2004)

The subsequent Quaternary volcanic unit, which outcrops throughout the MER, is the Wonji Group associated with the oblique Wonji fault belt (Mohr1962). It includes basalt flows and scoria cones, and large silicic central volcanoes with calderas.

3.8.1 Nazareth group and Dino Formation

According to Dipaola (1972), the volcanic rock occupies the western escarpment and it includes basalts and ignimbrites of the plateau. The Nazareth group includes ash flow, tuffs, pantellritic ignimbrites and un-welded tuffs while the Dino formation made up of Dino ignimbrites. These rocks outcrop at the NW part of the plateau in the study area. The rift ignimbrites are highly faulted and outcrop in most parts of the rift. This formation covers mainly the Tora- Koshe- Dugda Ridge

3.8.1.1 Basalts and associated flows of the rift floor

This unit consist recent basalts, which is located close to the western escarpment, in the Butajira-Siltie area. The formation of this group is from Pleistocene to Holocene and includes basaltic hyaloclastics and recent basalts outcropping in the rift floor.

The hyaloclastics consist of fine glassy material; generally yellowish to brown in color containing small boulders of basaltic lava. This flow consists of a lot of scoria.

This lava field (Cinder cones and lava flows) are aligned from Silitie in the south to Shershera in the north of Butajera. They look to have come out along a regional fault.

3.8.1.2 The Central Rift volcanic complexes

As Abiyu Kebede (2007), explains in his thesis, this portion consists of several individual volcanoes and volcanic complexes. The groups are characterized by rhyolytic lava flows and domes associated with the rift floor ignimbrites with age ranging from 0.24 to 0.02 million years. The ignimbrites and pumice are the result of gas rich silicics magma.

The alkaline and per-alkaline silicics are the last volcanic products and are located in the slopes of Bericho, Bora, O-Itu koshe, Alluto volcanoes, and Gadamotta Volcanoes northeast, of Lake Ziway.

3.8.1.3 Aluto volcanics

According to Tenalem Ayenew, (1998), the Aluto group representing multiple flows dominated by pumice, ash recent obsidian flows. Coarse pumice interbedded with sediments encountered in drill holes at Aluto caldera represent sub aqueous ignimbrite flows.

3.8.1.4 Gadamotta Slope Deposit

Tenalem Ayenew, (1998) Explain that Gedamota slope deposit is composed of very thin alluvio-colluvial, volcanic clastic and fluvio-deltaic deposit, resting erosively on older deposits. On the Gedamotta slopes, it represented by alluvial sand and gravel, overlain by alluvially reworked fine-grained greyish tuffs, which are located on the west of Lake Ziway. Gedamota rhyolite is on early Pleistocene alkaline and per-alkaline rhyolitic lava flow and bedded tuff. The Gedamotta ridge is the relict of a large Caldera. After the collapse, the center of the caldera has been margin exists. The eastern side might have collapsed and has been subsequently buried

3.8.1.5 Volcano-sedimentary rocks and lacustrine sediments

According to Carolin et al, (1999), rift sediments predominantly comprise volcanoclastic sediments, tuff and associated lacustrine sediments.

The author also explains that, rift sediment consists of layers of alternating silt and clay with volcanoclastic sediments, sands, ashes, transported pumice, silt, clay and diatomite and sedimentation initiated in the Ziway basin before 0.3 to 0.2 million years which is the age for the initiation of the Ziway Basin and continued until present.

The sediments have been deposited during a wide time interval from the end of Pliocene until recent, which is suggested by their considerable thickness and by the fact that in many places they underlie young volcanic products and are often deeply affected by regional faults.

Besides volcano-sedimentary and lacustrine deposits, alluvium is widely observed in valleys. WoldeGabriel et al., (1990) also explain that, the central sector of the MER and its shoulders are made of vulcanite and pyroclastic rocks, whereas volcano-lacustrine and fluvio-lacustrine deposits cover large areas of the rift floor.

3.8.2 Local Geology

From field observation, the local geology along the route aliments is dominated by volcanic ash like pumaceous material and transported soil (see fig 3.12).

Fig 3.12 Photo of pumice deposit taken from 74+500 of the route alignments.

According to Benvenuti et al., (2002) the central sector of the Main Ethiopian Rift and its shoulders are made of volcanites and pyroclastic rocks, whereas large area of the rift floor are covered by volcano-lacustrine and fluvio-lacustrine deposits. Quaternary sediments like lacustrine origins are the most exposure in the study area. Lacustrine beds are interbedded with plio-pleistocene pyroclastic in the lake region (Mengesha et al., 1996).

According to Lloyd (1980, as cited in Mengesha et al., 1996), most of the Central main Ethiopian Rift is covered by lacustrine beds that are interbedded with plio-pleistocene pyroclastic material.

3.2.1 Sand and silt of volcanics

The largest portion of the present study area is predominantly sand and silt with abundant components of volcanic origin (refer to fig3.13).

The sediments have a varied lithology related to their mode of origin whether they are in active lake deposits, water deposits, coarse sediments introduced by floods, and volcanic ejects.(Tenalem Ayenew,1998).

The Meki Formation, made up of alluvial and fluvio-lacustrine deposits, records the fluctuations in the Meki River base level (Benvenuti et al., 2002).

Fig 3.13 Photo of Silt deposits at 77+00km in the study area near the route alignment.

3.2.2 Pyroclastic ash

The central sector of Main Ethiopian Rift outcrops lithologies consist of plio-pleistocene volcanites (pyroclastic products of felsic composition) (fig3.14) and sediments and the sediments in large consists of lacustrine deposits (Benvenuti et al., 2002). Upper quaternary fluvio-volcano lacustrine fancies and Colluvial deposits that represent weathered/remobilized volcanic rocks and silicic tephra characterize pyroclastic ash.

Fig 3,14pyroclastic products of felsic composition exposed at 86+00km in the study area.

Geological structure

The main Ethiopian rift has accommodated the active extension between the Nubian and Somalian plates since the Late Miocene (Ebinger 2005).

This area recorded a typical evolution of continental rifting, from fault-dominated rift morphology in the early stages of the continental extension (transtension) to magma-dominated extension during break-up (Agostini et al. 2011) and Accocella (2013).

Some types of secondary structures are faults, joints etc. In the study Area, both secondary and primary structures such as bedding and faults have seen (fig3.15). Moreover, geological and structural maps of the present study is given below (fig3.16).

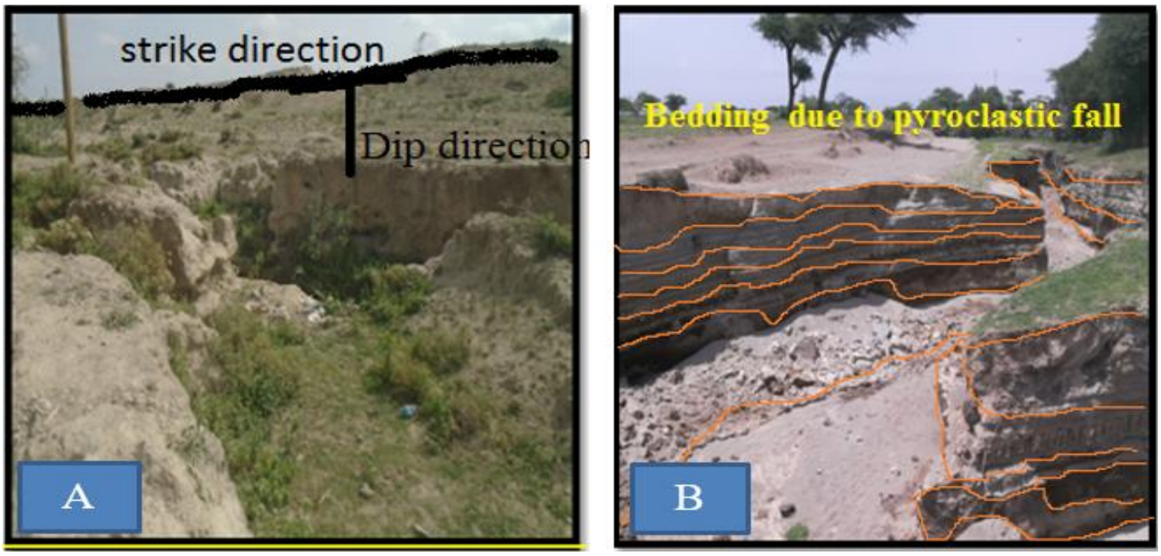

Fig 3.15 Fault (a) and Bedding of the pyroclastic fall (b) located at NS of near the study area.

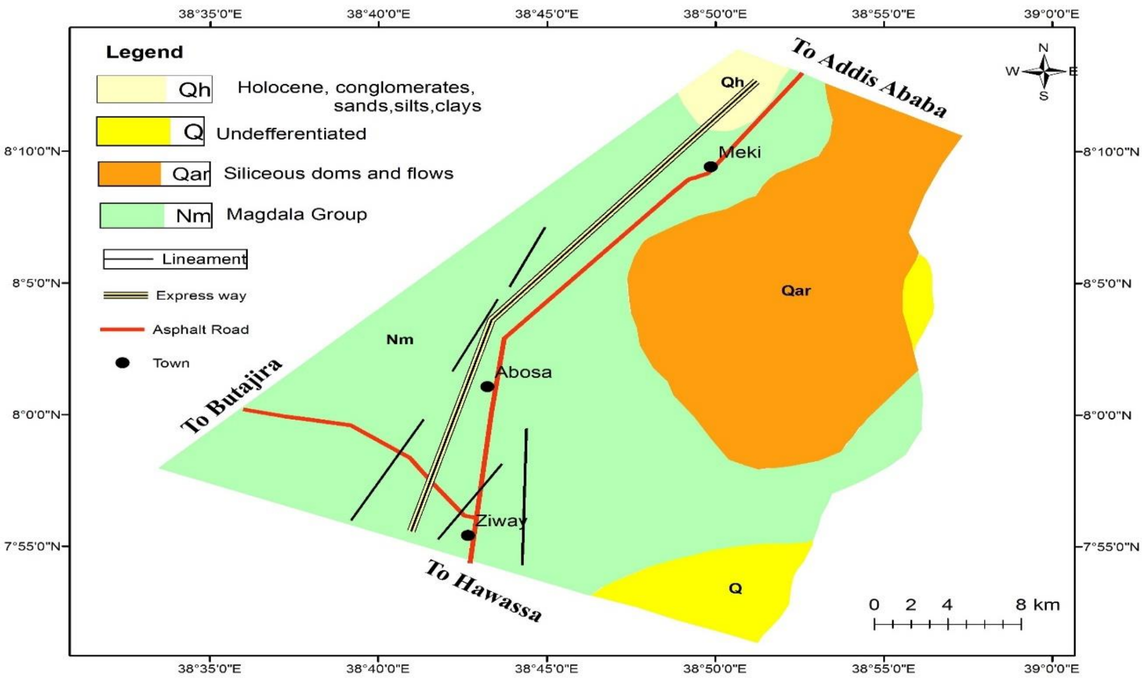

Fig 3.16 Geological map of the study area after (Mengesha et al., 1996).

3.9 Geo hazards

Geo-hazards can pose significant constraints to the construction and operation of road infrastructure, and are therefore critically important to route selection in all terrain types. In Ethiopia, geology and tectonic setting, combined with climate and hydrology, create conditions that encourage the occurrence of a range of geo-hazards. These occur naturally and are often also triggered or exacerbated by road construction.

Geo hazards classified as endogenous and exogenous geological hazards. Endogenous geological hazards originated from internal parts of the earth due to different tectonic effects. The endogenous geological hazards are, ground crack hazards, Seismic hazards, volcanic hazard.

3.9.1 Ground crack

According to Ayalew et al. (2004) ground crack or ground fissures, sinkholes, collapsing surfaces, or surface faults are defined as long, linear tensile structures occurred at the land surface with or without vertical offsets.

As an expression of openings that are products of the same processes, they usually occur parallel or semi-parallel with one another. Moreover, they commonly cross ephemeral streams at nearly right angles, intercept runoff, and redirect much or all of the flow downward.

Although not always and every-where, many ground cracks propagate upward from the subsurface and the surface expression of an opening is the last result of the release in accumulated strain.

Ground cracks (fissures) documented in various locations in the Ethiopian Rift Valley (Asfaw 1998) and represent a very dangerous geological hazard. Ground fissures parallel to the rift axis was reported to the west of Lake Ziway (Asfaw 1998).

The mechanism of the formation of the ground cracks is uncertain and the occurrence of fissures is random and unexpected. The conformity of the youngest fault system and fissures indicates a tectonic origin of this phenomenon.

The association of ground cracks and vertical displacement along normal faults is known elsewhere in the world (Holzer 1984). Fissure formations are likely related to the tectonic opening of cracks within solid rocks in the basement of the rift floor and subsequent subsurface erosion of loose material overlying the bedrock.

Holzer (1984), explain that as the soft sediments plastically accommodate the extension in the first phase, the ground cracks may remain unobserved until a devastating collapse of their thin roof.

The solid rocks deeper within the cracks are always tectonically fractured with no signs of water erosion. The fissure development is accompanied by surface and shallow subsurface processes such as piping, hydro compaction and changes in the volume of material. This process causes rapid and unexpected subsidence of the surface and collapses of sidewalls of the fissure. The ground crack in the present study is summarized in the following fig 3.17

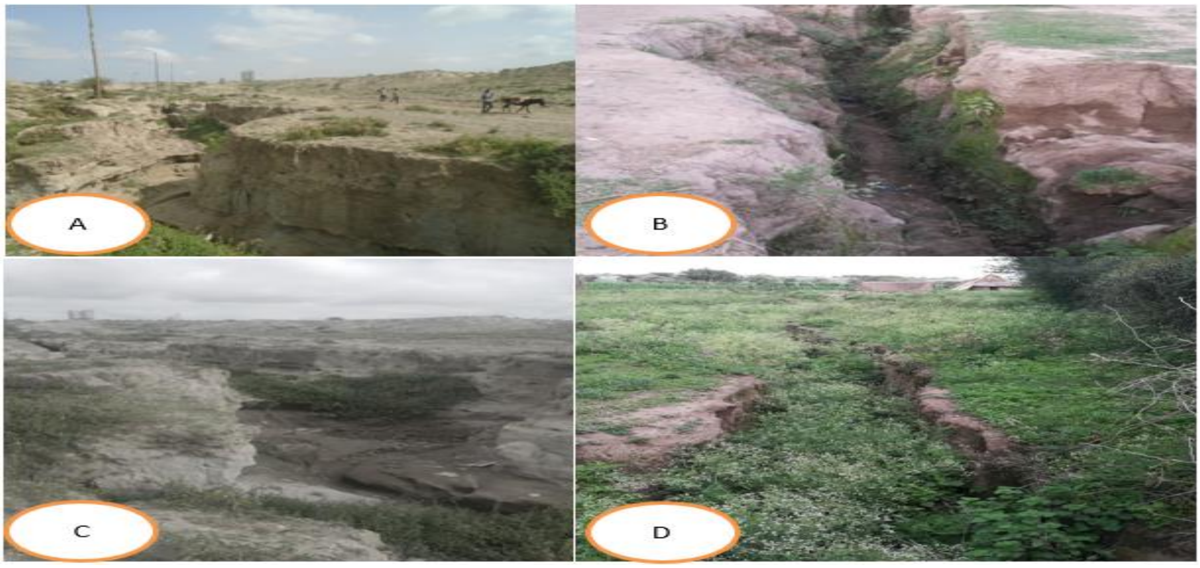

Fig 3.17 A, B, C and D show some cracks found at study area and running toward the new highway road route alignments and orthogonal to it.

3.10 Seismicity

Seismic survey has not been carried out in the project area or in the immediate vicinity. However, a number of seismic surveys by different local and foreign researchers (e.g. IGSSA of Addis Ababa University, Ethiopia-Afar Geoscientific Lithospheric Experiment (EAGLE, 2001-2003), etc.) have been carried out for several years in the MER.

As a result, earthquake epicenters are mainly located in the MER, where tectonics and volcanism is still active.

As indicated, on the Seismic zone map of Ethiopia (fig 3.18), project road is entirely located in Therefore, it is advisable to check the stability of the structure against seismic effects and appropriate seismic design considerations should had been taken into account during the structural design stage by following the seismic design procedure as specified in ERA Bridge Design Manual 2013.

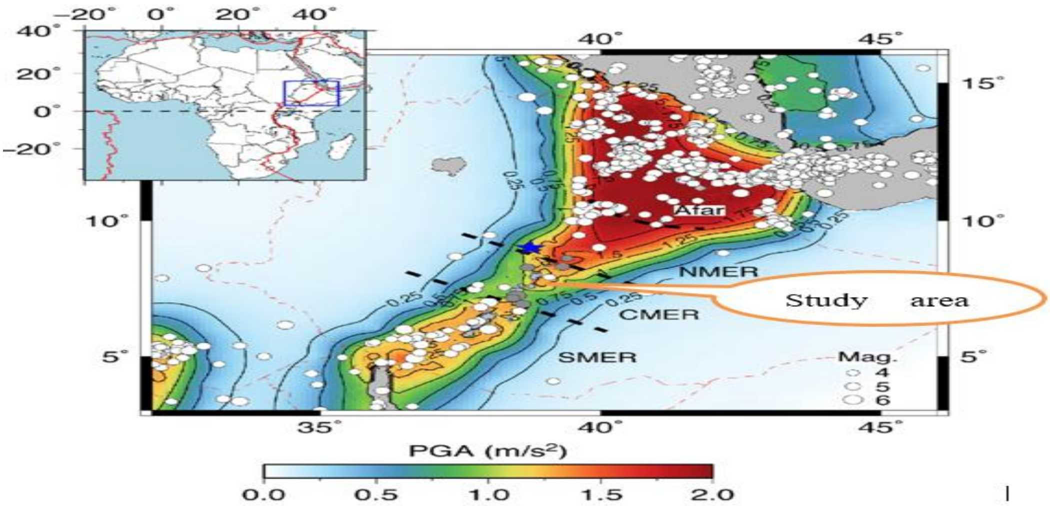

Fig 3.18 Seismic hazard zoning map of Ethiopia adopted from Atalay (2017)

Chapter4.

Sub-grade characterization, test result and interpretation

4.1 Introduction

Subgrade is the upper part of a roadbed and it is a compacted layer of the naturally occurring soil material of existing road alignment or replacement material, which acts as the foundation of a road. Hence, the subgrade of a road alignment should be strong enough to withstand the load from the pavement structures built on it, traffic load, and effects of moisture fluctuation, erosion and geohazards like earthquake.

Thus, the primary investigation work of a road project is evaluating the subgrade conditions to gain the essential data used for detail characterization to check the suitability of the subgrade or its sections. If the natural subgrade has good quality, it made ready to give service after the required earthwork has carried out. Whereas weak or poor natural subgrade needs, improvement and the problematic soil stabilized using mechanical or chemical methods or it totally replaced by another good subgrade material.

Therefore, the intent of the present research was, to characterize the sub-grade material for the construction of the pavement structure and propose remedial measures for unsuitable portions of the subgrade material. Identification of unsuitable soils and characterization of their anticipated behavior is thus an important parameter for site selection, design, and construction projects (AASHTO 1993). This chapter presents the field and the laboratory investigations that have seen made on the subgrade of the project road, and interpretation of the investigation results.

4.2 Sub-grade Soil investigation

The purposes of the sub-grade material investigation include;

Assessment of depth and nature of the sub-grade soil characteristics along the project route.

Assessment of suitability of the sub-grade soils as to incorporate to the pavement design.

Classify the sub-grade soils in to homogenous sections, to determine the design CBR values, identifying the location, depth and nature of unsuitable sub-grade soil.

The sub-grade soil investigation carried out comprises field and laboratory works and discussed in the following sub-sections.

4.2.1 Field Investigation for Sub-Grade Soil

The entire fieldwork of this thesis was assessing the overall engineering geological and geotechnical condition of the route corridor and the project area.

The methods that implemented during the investigation are visual identification and description of the subgrade soil extension, average of digging test pits manually up to 1m depth from the roadbed level, logging of test pits, photographing and collecting reasonable amount of representative samples for laboratory testing.

4.2.2 Visual Investigation

It is necessary to group the subgrade soils of a road alignment in to homogenous sections separating them based on their strength property or their CBR values This data and the traffic load data are used to determine the thicknesses of pavement structures or layers when pavement design is executed using particularly the CBR-Method (the empirical pavement design method). ERA (2013a).

Sub-grade soils with similar soil type were grouped together and their extent was determined. The project area generally lies on flat terrain in a warm and humid climate region. Gully erosions is common in the eastern part of the study area. But near the route alignment, erosion effect is minimal along flat terrain sections and development of thick transported soil is facilitated in the chainage all most of the study area (4.1)

The identification was made based on some of their physical properties, such as, color, and grain size distribution of their constituents. Since these properties, logs of test pits and other index properties of the subgrade soils are indicators of their strength properties, the visual surveying and description of the subgrade soil extensions provide essential information for grouping the subgrade in to homogenous sections.

Thus, the type of sub-grade soil encountered along the route corridor is found to be mainly dependent on topography and geology of the project area.

From the visual inspection of the sub-grade soil during the site visit, in most of the cases, light brownish, dark brownish to yellowish silty clay type and in few sections of the road alignment, red brownish silty clay soil are observed(fig 4.1). Since the study area is fall in the pyroclastic fall deposits, it is dominated by volcanic ash.

Besides exogenic, processes such as ground crack and gully erosion are also observed in the study area that is discussed in chapter 3 of this paper (refer to fig 3.15 and 3.17)

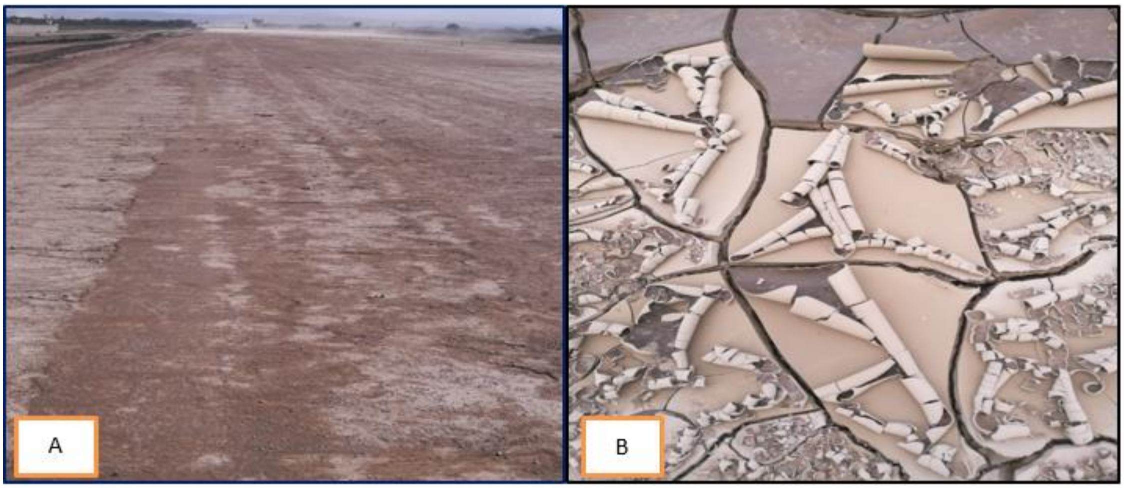

Fig 4.1 Typical sub-grade soils in the road project Km 76+ 800 LHS (a) and mud crack and sheet like structure from Km 89+00(b).

The descriptions of the sections of the subgrade soil extension from the visual assessment of the road alignment are given below in Table 4.1

Table 3.1 station and depth of sampling with description.

S no	Chainage km		Visual Description	Depth
	From	to		
1	56+500	58+00	Light yellowish silty clay soil	85cm
2	58+00	59+500	Dark gray silty clay soil	90cm
3	59+500	61+00	Light yellowish silty clay soil	1.20cm
4	61+00	62+500	Light yellowish silty clay soil	90cm
5	64+00	65+500	Gray to darkish silty clay soil	1.30cm
6	65+500	67+00	Light yellowish silty clay soil	95cm
7	67+00	68+500	Light yellowish silty clay soil	110cm
8	68+500	70+00	Dark yellowish silty clay soil	90m
9	70+00	71+500	Dark yellowish silty clay soil	90cm
10	71+500	73+00	Light yellowish silty clay soil	80cm
11	73+00	74+500	Light yellowish silty clay soil	73cm
12	76+00	77+00	Light yellowish silty clay soil	88cm
13	77+00	78+00	Light grayish silty clay soil	94cm
14	78+00	79+00	Dark grayish silty clay soil	125cm
15	79+00	80+00	Light grayish silty clay soil	102cm
16	80+00	81+00	Light grayish silty clay soil	67cm
17	81+00	82+00	Light grayish silty clay soil	90cm
18	82+00	83+00	Dark grayish silty clay soil	100cm
19	83+00	84+00	Light grayish silty clay soil	95cm
20	84+00	85+00	Light grayish silty clay soil	110cm
21	85+00	86+00	Light grayish silty clay soil	86cm
23	86+00	87+500	Light grayish silty clay soil	105cm
24	87+500	89+00	Light grayish silty clay soil	115cm
25	89+00	90+500	Light grayish silty clay soil	85cm
26	90+500	92+00	Light grayish silty clay soil	72cm

4.2.3 Subsurface investigation aim

Subsurface investigation was carried out for the present study to know the nature and actual thickness of subsurface material. In order to achieve this goal, eight borehole data were adopted from Saba Engineering P.L.C. The borehole data were took from station 58km, 61km, and 65km, 74+500km, 77km, 79km, 81km, and 82km and subsurface soil profiles were prepared using starter software.(see fig 4.2). The depth of borehole from 8-station fall between 9m to 25m and all soil profile for the eight borehole are presented in annex II.

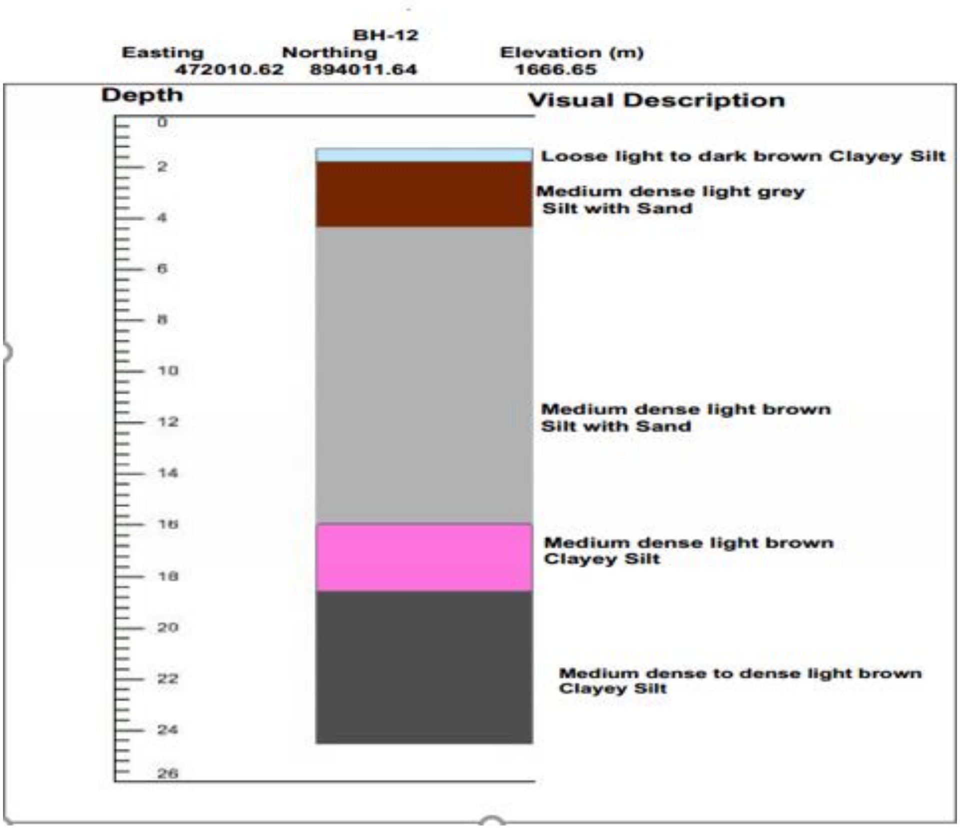

Fig 4.2 soil profile that indicates its thickness from BH12 at km74+500

4.3Test Pitting and Sampling

Regardless of the type of project, sample spacing should be located to obtain the basic knowledge of the engineering properties of the overburden and bedrock formations that will be affected or will have an effect upon the proposed pavement structures (Chen, 1988). Test pitting helps to directly access the subsurface material though it is restricted to shallow depth. It is a valuable method of subgrade investigation since vertical and horizontal variation in soil or rock layers of the subgrade can be observed and sample can easily be taken from each layer. For this research, trial pits have been dug using hand tools like pick-axes and shovels to the depth of average 1m and diameter of 1m from the roadbed level (fig4.3).

The test pits have been located using GPS and photographs of the test pits have been taken. Logging of the test pits have been carried out by including description of the topsoil. The test pit location has been chosen based on the change in soil types or geological units of the route corridor.

Fig 4.3 Sub grade soil sampling at station of 74+500km photo taken on May 17/2017

The general guideline of ERA (2013b) about the sampling frequency for final or detailed investigation, which is 0.5km for identification tests and 1km interval for CBR test, has been applied during sampling. Based on the similarity of the soil, about six samples were collected at1km interval and other 20 samples were collected at 1.5km interval. From the test, 26 disturbed samples were collected to conduct laboratory tests for sub grade soil characterize. Fig 4.4 indicates sample location. All the collected disturbed samples were kept in plastic bags.

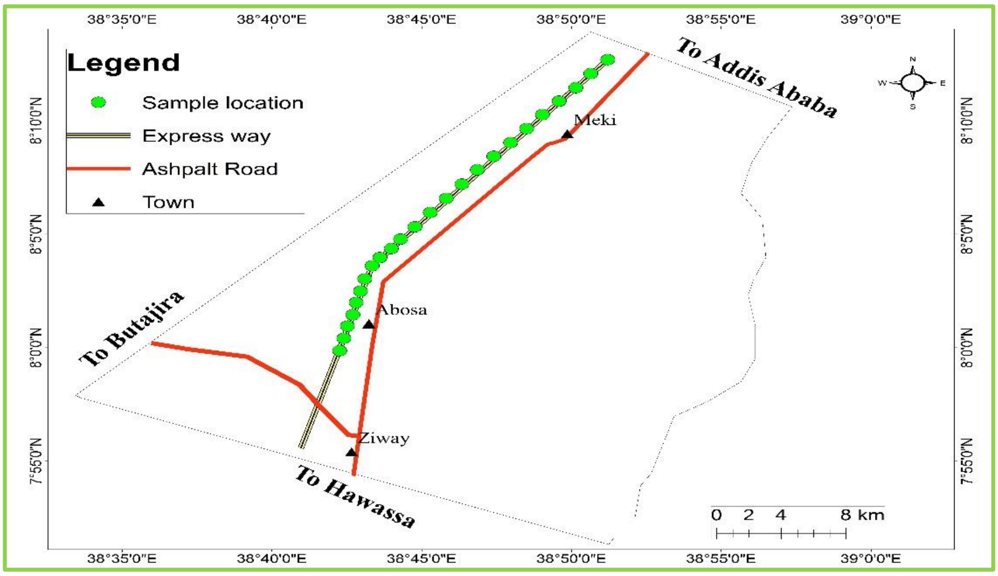

Fig 4.4 sampling location map of Meki –Ziway Highway

4.4 Laboratory investigation for sub-grade soils

After collecting twenty-six (26) subgrade soil samples using appropriate techniques for this thesis work, the following activities were carried out (table 4.2).

Table 4.2 Summary of laboratory tests for natural subgrade.

Type of test	Type of material	No of test	Test designation
Grain size analysis	Subgrade	26	AASHTO T 89 and T 90
Hydro meter test	Subgrade	6	AASHTO T-88-86
Atterberg limit	Subgrade	26	AASHTO T 90
Linear shrinkage	Subgrade	7	BS 1377
Compaction test	Subgrade	26	AASHTO T 99
Specific gravity	Subgrade	6	AASHTO T 99
CBR test	Subgrade	26	AASHTO T 180

4.5. Index Properties

According to Federal High Way Administration (FHWA, 2001), Index Properties are mostly used for the identification and classification of soils. They are used to characterize soils and determine their basic properties such as moisture content, specific gravity, particle size distribution, and consistency and moisture-density relationships.

4.5.1Particle size distribution (AASHTO T-88)

The objective of particle size distribution analysis is to determine the relative percentages of different sizes of particles in the sample. From this, it is possible to determine whether the soil consists of predominantly gravel, sand, silt or clay sizes and which of these sizes is to control the engineering properties of the soil (BS 1377: Part 2, 1990).

In this study the particle size distribution analysis of the disturbed subgrade soil samples has carried out by wet sieve analysis for materials coarser than 0.075mm and by hydrometer test for fine material (finer than 0.075mm) based on AASHTO T-89 and T-88standards respectively. The results of the particle size distribution or grain size analysis tests in the present study is show that, from 26 subgrade soil samples, or about 18km (69%) have fine fraction greater than 35% and the rest km8 (31%) have fine fraction less than 35%. Only 5km (19 %) have fine materials below 50%. The two horizontal lines drown in fig 4.5 indicates the boundary percentage finer than number 200 sieve values between the coarse grained or granular soil materials and fine grained or silty-clay soil materials (the boundary value for AASHTO Soil Classification System is 35% and the boundary value for Unified Soil Classification System is 50%).

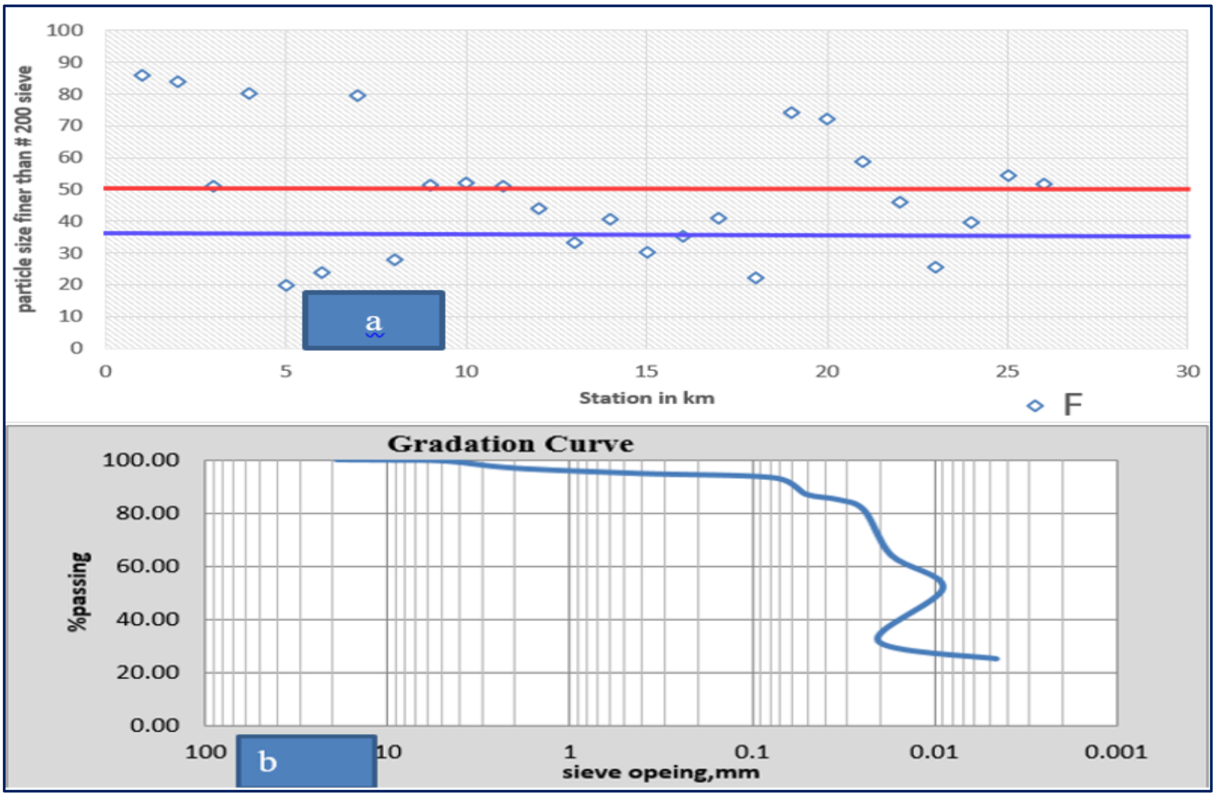

Fig 4.5 a and b Indicates Percentage of finer with station and full gradation curve at 56+500km respectively.

For the present study, six (6) samples are selected and tested by using hydrometer 151H. Accordingly, the percent percentage of sand, silt and clay are determined. The percent of sand, silt and clay are fall between zero (0) and 67 - 81 and 19 - 33 respectively.

Both the AASHTO soil classification and the USCS have been made from the particle size distribution tests realize that the subgrade sections of the route corridor are dominantly silt clay or fine-grained as per AASHTO soils classification and half asper USCS. Accordingly, fine-grained or silt-clay soils are not promising to be used as subgrade of a road without treatment.

4.6 Soil Classification (AASHTO M 145) and (ASTM D 2487)

Soil classification for engineering purpose involves grouping of soils in to classes depending on their similarities in engineering properties. Varies soil classification systems have been developed by different institutions for particular uses.

The results of AASHTO Soil Classification point out that out of 26 subgrade soil samples, three (11.538%) samples or about 3km of road sections are classified under the group A-2-7 (silty or clayey gravel and sand).

Four (15.4%) samples are or about 4km of road sections classified under the group A-2-4, four (15.4%) samples or about 4km of road sections are classified under the group A-4, 5(19%) of the samples or about 5km of road sections are classified under the group A-6, eight (31%) or about 8km of road section are classified under the group A-7-5 and A-7-6 (clayey soils) and others 2 samples are classified under group A- 2-5 and A-5 (table 4.4) and(fig4.6)

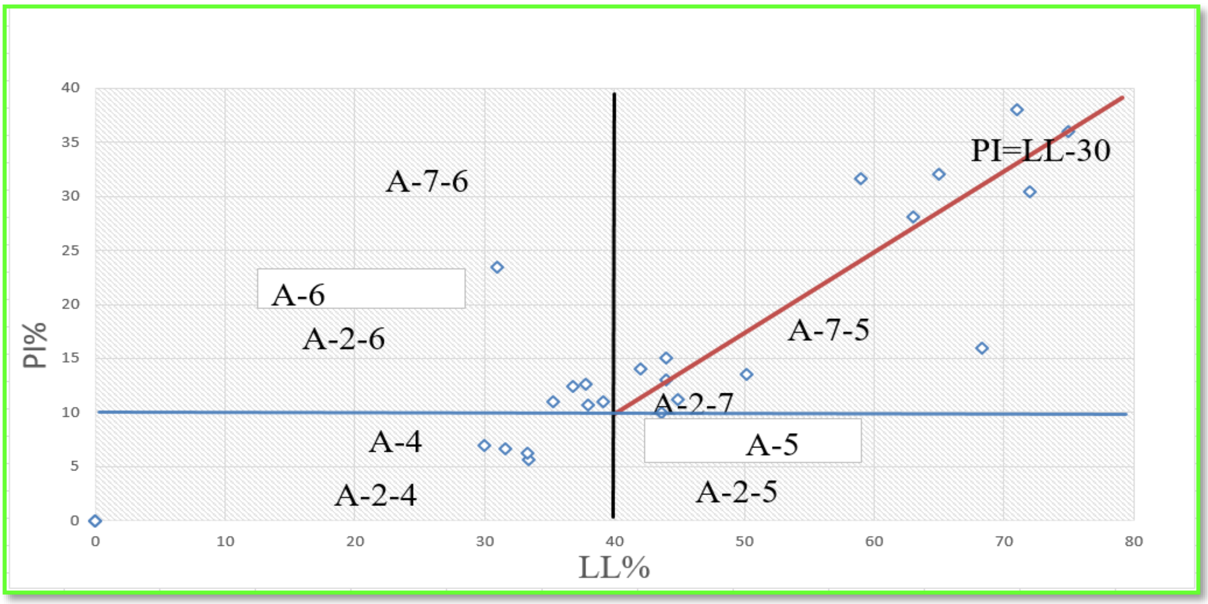

Fig 4.6 Position of natural subgrade soils on the AASHTO plasticity chart as per AASHTO & USCS (after Utah DOT, 1998).

4.6.1 The group index ((GI) AASHTO, 1993).

In general, the quality of performance of a soil as a subgrade material is inversely proportional to the group index. Classification of sub-grade soils using Group index values (GI) is shown in Table 4.3

Table 4.3 summery of group index

Soil Classes	Group Index (GI)	Number of occurrence	(%) occurrences	Remark (AASHTO, 1993).
A-7-6 (25)	GI > 20	1	3.8	Poor for sub grade
A-7-5(23)	GI > 20	6	23.1	Poor for sub grade
	GI< 10	1	3.8	Intermediate
A-6	GI≤5	5	19	Suitable for sub grade
A-5(5)	GI≤5	1	3.8	Suitable for sub grade
A-4(0)	GI≤5	4	15.4	Suitable for sub grade
A-2-4(0)	GI≤5	4	15.4	Suitable for sub grade
A-2-5(0)	GI≤5	1	3.8	Suitable for sub grad
A-2-7(1)	GI≤5	3	11.5	Suitable for sub grade
TOAL		26	100	

Based on the USCS soil classification, twelve (12) subgrade samples are classified as SM, three, as CH, three as ML, three as CL, two as SC and three as MH. Their percentage of occurrence and degree of suitability as subgrade is shown in Table 4.4.

From the results of the classification of the subgrade soils of the project road alignment based on the USCS the dominant soil types of the natural subgrade are clay and silt.

The constituents of considerable sections of the natural subgrade of the route corridor are CH and MH, high plastic clays and high plastic silts. Their percentage of occurrence and degree of suitability as subgrade is shown in Table 4.5.

Table 4.4 AASHTO soil classification for the subgrade soil samples of the project road.

S. no.	Soil classes		No of sample	Percentage occurrence	Rating as a subgrade
1	A-2	A-2-4(0)	4	30	Excellent
		A-2-5(0)	1		Excellent
		A-2-7(1)	3		Good
2	A-4(0)		4	38	Poor to fair
3	A-5(5)		1		Poor
4	A-6		5		Poor
5	A-7	A-7-5(23)	7	32	Very poor
		A-7-6(25	1		Very poor
Total			26	100	

Table 4.5 Unified soil classification for the subgrade soil samples of the project road.

S.no	Soil classes	Number of samples	% of occurrence	Rating as a subgrade	Specification ERA FPDMVol1 (2013a)
1	CH	3	11.54	Poor to fair	
2	CL	3	11.54	Poor to fair	
3	MH	3	11.54	Poor	
4	ML	3	11.54	Poor to fair	
5	SC	2	7.69	Poor to fair	
6	SM	12	46.15	Fair to good	

4.6.3 Atterberg Limits (AASHTO T 90)

In the year 1911 Atterberg proposed the limits (liquid limit LL, plastic limit PL and shrinkage limit SL) of consistency in an effort to classify the soils and understand the correlation between the limits and engineering properties like compressibility, shear strength and permeability (Cassagrande, 1932).

He also established four states of consistency (i.e. degree of firmness) of fine-grained soils with the three limits. These states are solid, semi-sold, plastic and liquid.

The change in the state results due to the change in moisture content of the fine-grained soils. The water contents at which the soil changes from the solid to semisolid, semisolid to plastic, plastic to liquid states are shrinkage limit, plastic limit and liquid limit respectively.

These water contents are expressed in percent as the ratio of the amount of water in the specimen and the mass of dry soil specimen. The difference between the liquid limit and the plastic limit is the plasticity index (PI).

The liquid limit and plasticity index distribution of the subgrade soil samples with stations of the project road are given in Fig 4.7

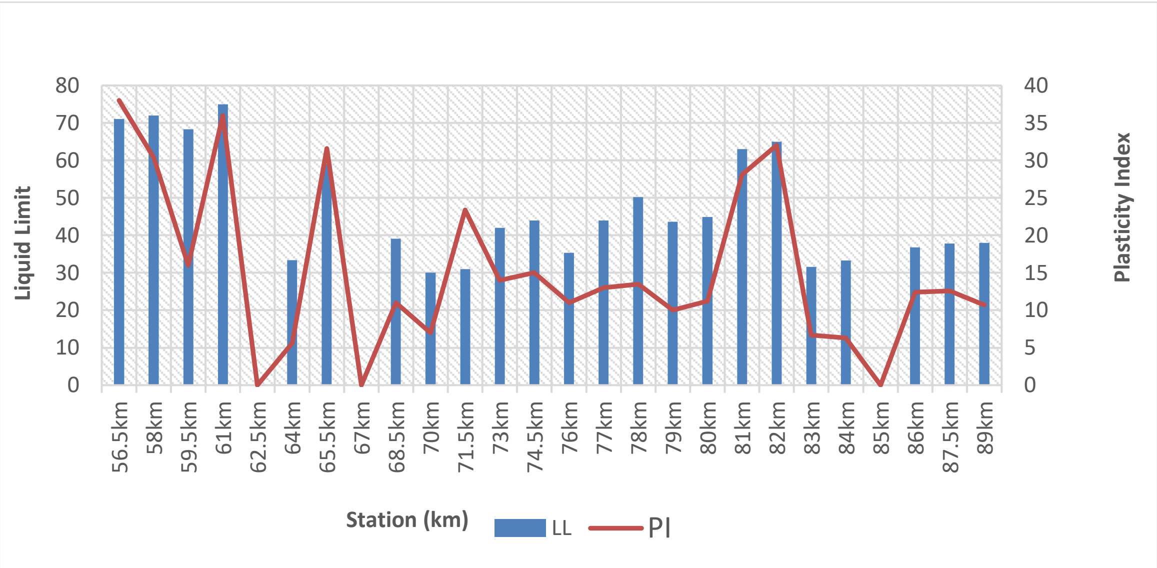

Fig 4.7 Distribution of liquid limit and plasticity index along the route alignment

Holtz and Gibbs. (1956) demonstrated that the plasticity index and the liquid limit are useful indices for determining the swelling characteristics of most clay. Since the liquid limit and the swelling of clays both depend on the amount of water clay tries to absorb, it is natural that they are related.

Swelling potential of the subgrade soils based on liquid limit (table 4.6) shows that 31% (about 8km of road section) samples of the subgrade soil possessing liquid limits from 40% to 60% has high swelling potential. The other 3.8% samples with liquid limits greater than 60 % have very high swelling potential.

As it given in Table 4.7, 12 (46.2%) or 12km of subgrade soil samples with plasticity index of < 12% are classified as low swelling potential. 7km (27%) subgrade soil samples fall in the range of 12% -23% plasticity indices that, they are characterized as medium degree of swelling potential. Five (19%) or 5km subgrade soil samples are with plasticity indices from 23 % to 32% and classified as soils having high swelling potential and two (7.8%) subgrade sample classified as soils having very high swelling potential

AASHTO M 145 (1991) stated that, liquid limits of 40% and above are assumed critical. In addition, plasticity indices of 10 and above are assumed critical. Subgrade soil samples of the route have liquid limits and plasticity index above the limits.

These shows that almost all the soil sections of the natural subgrade can cause adverse effect on the performance of the project road. Therefore, these sections need the application of improvement or remedial measures.

Table 4.6 swelling potential of the subgrade soil samples of the project road based on their liquid limit results.

Swelling potential	specification	LL (%)	No of sample	Percentage occurrence of samples
Low	AASHTO M 145 (1991)	<30	3	12%
Medium	AASHTO M 145 (1991)	30-40	10	38.5%
High	AASHTO M 145 (1991)	40-60	7	26.9%
Very high	AASHTO M 145 (1991)	>60	6	23 %

Table 4.7 swelling potential of the subgrade soil samples of the project road based on plasticity index results.

Swelling potential	Plasticity index (%)	Number of sample	Percentage occurrence of samples
Low	<12	12	46.2%
Medium	12-23	7	27%
High	23-32	5	19%
Very high	>32	2	7.8%

Soil having a liquid limit exceeding 60% or a plasticity index exceeding 30% when determined in accordance with the requirements of AASHTO T-89 andT-90 and ERA (2013c) are unsuitable natural subgrade materials. Comparison of liquid limit values and plasticity index values of the subgrade soils with this specification indicates that, six (6) subgrade soil samples at km59+500, km58+00, km59+500, km61+00, km81+00 and km82+00 have liquid limit values exceeding 60% and have plasticity index values exceeding 30 (table 4.7).

4.6.2.1 Linear shrinkage

The linear shrinkage value is the way of quantifying the amount of shrinkage likely to be experienced by clayey material. Linear shrinkage method covers determination of the total LS from linear measurements on a bar of soil of the fraction of soil passing 0.425mm sieve, originally having the moisture content of the liquid limit.

(BS 1377 (2003)). It is computed as the percentage of change in the length of the soil sample as it shrinks.

$$LS=100(LW-LD)/LW \qquad \textbf{4.1}$$

Linear shrinkage test had conducted for seven (7) subgrade soil samples taken from soil sections of the route, which were expected to have high swelling potential or high degree of expansiveness. Accordingly, subgrade soil samples have linear shrinkage values ranging from 5% to 14% (table4.8).

Table 4.8 Summery of linear shrinkage

S. no	Sample code		L_O	L_D	LS%	Remark	specification
1	HR1	56+500km	140mm	120.0mm	14.3%	Unsuitable	BS 1377
2	HR2	58+00km	140mm	120.5mm	13.2%	Unsuitable	BS 1377
3	HR3	59+500km	140mm	133.0mm	5%	Suitable	BS 1377
4	HR4	61+00km	140mm	121.0mm	13.6%	Unsuitable	BS 1377
5	HR7	65+500km	140mm	128.40mm	8.3%	Suitable	BS 1377
6	HR19	81+00km	140mm	122.0mm	12.8%	Unsuitable	BS 1377
7	HR20	82+00KM	140mm	122.5.0mm	12.5%	Unsuitable	BS 1377

Based on Chen (1988), linear shrinkage values above 8%, have critical degree of expansion. Accordingly, 71.4% of linear shrinkages values are above 8%. Thus, these are unsuitable subgrade soils of the route (table 4.9).

Table 4.9 Linear shrinkage test results summary of subgrade soil samples

S.no.	ST(Km)	Pit no./cod	Linear shrinkage (%)	Remark
1	56+500	HR1	14.28%	Critical (Chen, 1988).
2	58+00	HR2	13.21%	Critical
3	59+00	HR3	5%	Non-critical
4	61+500	HR4	13.57%	critical
5	65++00	HR7	8%	Non critical
6	81+00	HR19	12.81%	critical
7	82+00	HR20	12.5%	critical

4.7 Compaction Test (AASHTO T99)

Compaction is mechanical method of improving the shear strength of soils and it involves pressing soil particle tightly together by expelling air from void spaces between the particles. In laboratory compaction test is done by compacting a soil sample in a standard mold into specified number of layers having about equal thickness by applying a number of blows from a standard rammer or a hammer freely falling from particular height.

For different known moisture contents, the process was repeated and dry density is determined for each trial.

From the relationship between the soil moisture contents and the compacted dry densities of the same compaction effort the maximum dry density (MDD) and the corresponding moisture content or the optimum moisture content (OMC) were obtained. (AASHTO 1993)

In addition to increasing the shear strength of soils, compaction results to a decrease in future settlements and a decrease in permeability. Generally, well-graded soils attain greater dry density than the poorly graded or gap graded soils due to their better particles interlocking.

For this particular research work, compaction test of 26 subgrade soil samples have been performed following AASHTO T 99 standard testing method (standard compaction) and the results are presented in annex I

The OMC and MDD of the subgrade soil samples range from 15% to 32.7% and 0.9g/cc to 1.59g/cc respectively. (See fig 4.8 and fig 4.9)

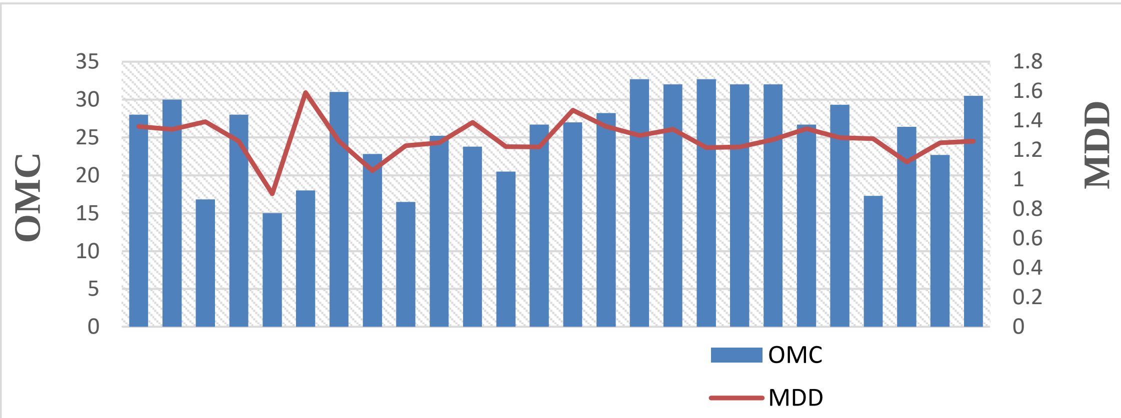

Fig 4.8 OMC and M DD of subgrade soils of the route alignment.

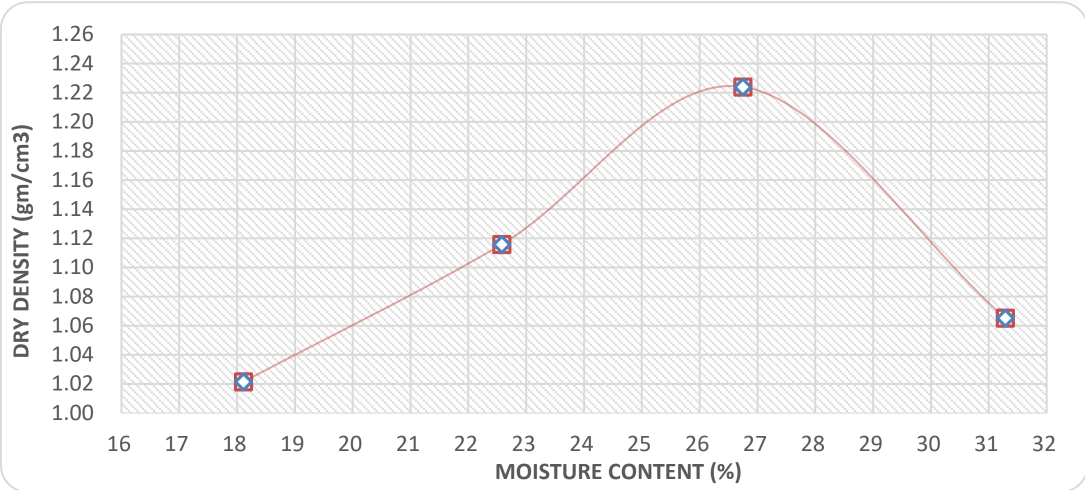

Fig 4.9 Graph of Maximum dry density at km 73+00

From 26-subgrade soil sample, three section (km 62+500, km67+00 and km86+00), have low MDD, which is 0.9, 1.1 and 1.1 respectively.

The moisture content and plasticity index of the sub grade soils of the present study area are presented in annex I From test result, it was concluded that highly plastic material has high moisture content. Based on ERA specification, the degree of compaction of the soils is controlled by the particle size distribution and the plasticity of the soil.

Low dry densities and high optimum moisture contents characterize the subgrade soil samples of the project route corridor.

4.8 California Bearing Ratio ((CBR) AASHTO T193)

O.J. Porter for the California Highway Department originally developed CBR test during the 1920s. It is a load-deformation or a stress-strain test that conducted on disturbed soil sample in laboratory and on undisturbed soil sample or on soil in-situ.

The strength of the road subgrade for flexible pavements is commonly assessed in terms of the California Bearing Ratio (CBR) and this is dependent on the type of soil, its density, and its moisture content, ERA (2013a).

The California Bearing Ratio test is conducted for evaluating the suitability of the subgrade and the materials used in sub base and base of a flexible pavement. The plunger in the CBR test penetrates the specimen in the mould at the rate of 1.25mm per minute.

The loads required for a penetration of 2.5mm and 5.0mm are determined. The penetration load is expressed as a percentage of the standard loads at the respective penetration level of 2.5mm or 5.0mm.

The CBR value is determined corresponding to both penetration levels. The greater of these values is used for the design of the pavement (Arora, 1997).

The ERA (2013b) recommends to conduct standard compaction tests and to measure CBR and CBR swell after 4 days soaking, on sample(s) molded at 95% maximum dry density (Standard Compaction) and optimum moisture content (Standard Compaction) for natural subgrade soils of a road project.

For the present study, CBR tests were carried out on 26-subgrade soil (fig 4.10). The CBR values at 95% MDD (Standard Compaction) of the subgrade soils vary from 2% to 30%. In general, for subgrades soil with CBR values less than five (5), special treatment should be given according to ERA (2013b). A laboratory report of three-point CBR test results were presented in annex I

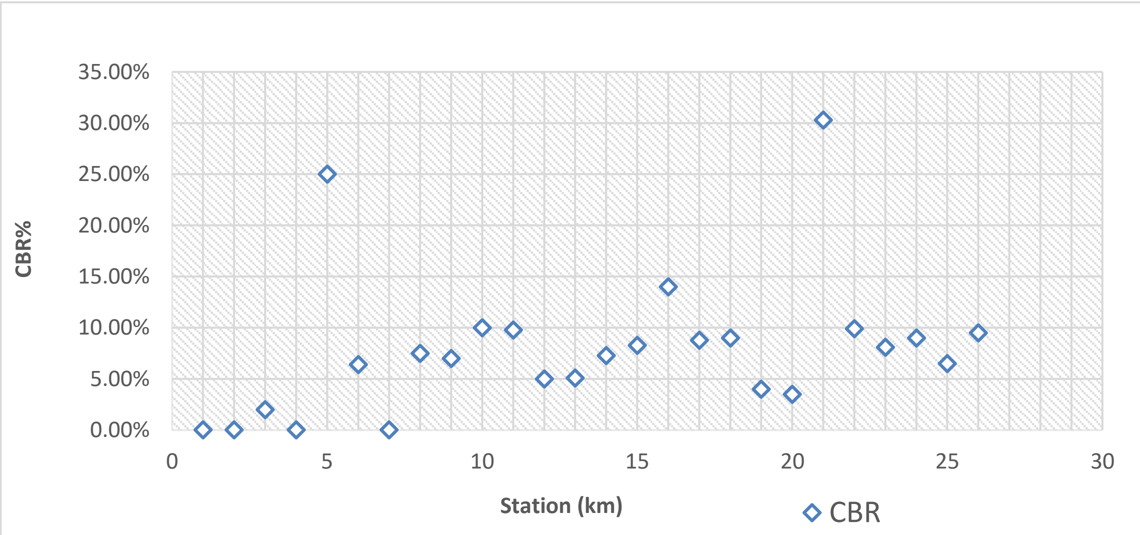

Fig 4.10 Chart of CBR values of subgrade soil along the route alignment

Laboratory soaked CBR were carried out on subgrade soil samples compacted to a dry density of 95% standard compaction, at standard moisture content and appropriate surcharge.

ERA (2013) states in-situ subgrade sections with CBR value less than 5% are generally unsuitable. Such types of subgrade sections are recommended to be replaced by suitable material up to preferred depth or chemically stabilized. When the in-situ subgrade soils are evaluated based on their CBR values, samples taken from km 58+000, km 59+500 and km 61+00, km 65+500, km81+000, km82+000 or their corresponding sections from km 58+000to km 65+500 and from km81+000 to km82+000 have CBR values less than 5%; therefore, they are unsuitable.

4.8.1 The CBR swell (AASHTO T193)

The percent swell value computed during CBR soaking also used to evaluate the swelling potential of soils to check the suitability of subgrade, sub base and base materials of flexible pavements.

Tripod with dial indicator placed on top of the mold and initial dial reading was made. At the end of 96 hours, a final deal reading was made on the soaked specimen and the swell as a percentage of the initial sample length is calculated (AASHTO T 193). All the percent swell values of the subgrade soil samples are between0.32percentage and 2.89%. The CBR values vs CBE swell is shown in fig 4.11

According to ERA, (2013a) natural subgrade soils with CBR swell exceeding 2% are unsuitable. As per this specification, about five (5) subgrade soil samples of the project road are unsuitable. The test result of CBR swell is presented in annex I,

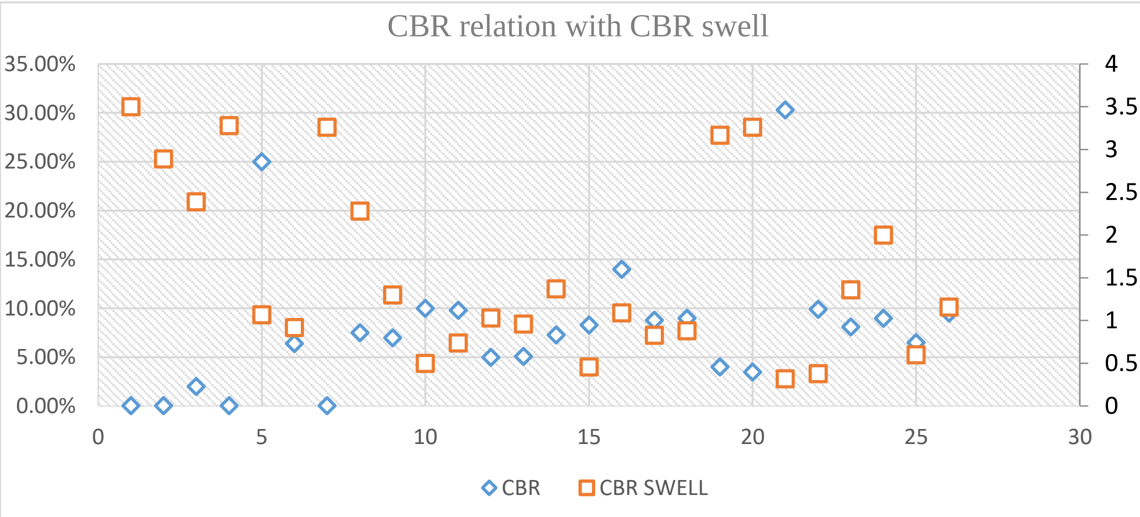

Fig 4.11 Distribution of CBR vs CBR swell values of subgrade soil along route alignment.

All the CBR swell values of the subgrade, soil samples of the present study are between 0.32% and 3.5%. In accordance with this specification, five (5) or km 58+000, km 59+500 and km 61+00, km 65+500, and km82+000 of the project road subgrade soil sections are undesirable

4.9 Delineation of homogeneous section

For the present study the delineation of homogenous sections of the road alignment has been done based on the cumulative difference, method presented in Appendix 8 of Tanzanian Pavement and Materials Design Manual (1999) and in Appendix J of AASHTO (1993) Pavement Design Guide.

Detail calculations of the unit delineation by cumulative sums method values has shown in table 4.14, Appendix I.

The limits of homogenous sections were defined from the main slope changes of the graph shown in Fig 4.12

In addition, the homogenous sections of the road alignment delineated are five as shown in Table 4.15 Design CBR (CBR_{design}) has made based on the newly developed method by Virginia department of Transportation in 2000 to calculate design CBR value for the present study, which presented in section 2.7

The two-thirds factor provides the necessary safety margin to compensate for any non-uniformity of the soil, and for any low-test results not considered when computing the average of the CBR sample values. Also four days of soaking, as specified in the CBR test method (AASHTO, 1993), does not necessarily give the minimum CBR strength of some soils. Thus, the two-thirds factor would compensate for all such variations.

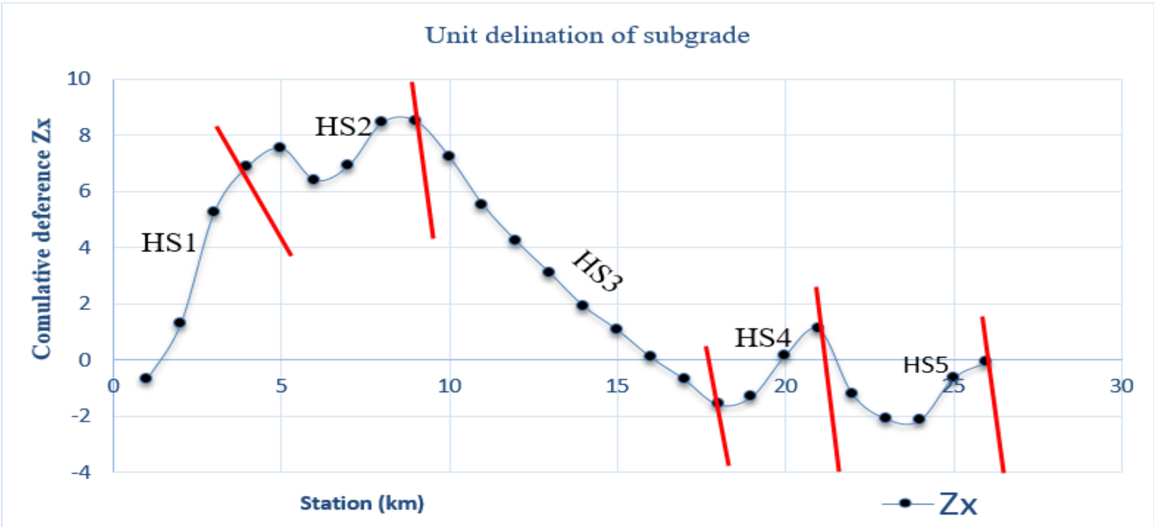

Fig 4.12 Subgrade unit delineation of the route alignment.

Table 4.10 sub grade homogenous section

Homogenous section	Chainage (km)	CBR values	$CBR_{design\%}$	ERA 2013Subgrade strength class
HS1	56+500 - 62+500	4.4%, 3.5%, 2%, 4.0%, 25%	2.3	S1
HS2	62+500 - 68+500	6.4%, 4.0%, 7.5%, 7%	4.15	S2
HS3	68+500 - 80+00	10%, 9.8%, 5%, 5.1%, 7.3%, 8.3%, 14%, 8.8%,9%	5.6	S3
HS4	80+00 - 83+00	4.0%, 3.5%, 30.3%,	8.4	S4
HS5	83+00 – 89+00	9.9%, 8.1%, 9.00%, 6.5%, 9.5%	5.7	S3

4.10 Overall Characterization of sub-grade

In the present study field, investigation has been done based on visual investigation and test pit. Accordingly, 26 disturbed sample was collected for laboratory test purpose.

Gradation test has been done for soil classification, Atterberg limit, linear test; Procter test, CBR and CBR sell test were conducted for general subgrade characterization.

Subgrade soils were classified as A-2-4, A-2-5, A-2-7, A-4, A-5, A-6, A-7-5 and A-7-6 according to AASHTO and CL, CH, MH, ML, SC and SM in USCS soil classification systems. About 27% subgrades soil samples are problematic soil based on index properties and subgrade soil strengths (CBR values). The next 73% subgrade soil samples are suitable. In general, five (5) or km 58+000, km 59+500 and km 61+00, km 65+500, and km82+000 of the project road subgrade soil sections are undesirable based on ERA(2013) standards. As a result, such subgrade soil section need proper improvement

Chapter 5

Chemical stabilization of sub grade

5.1 Introduction

Geotechnical properties of problematic soils such as soft fine-grained and expansive soils are improved by various methods. The problematic soil was removed and replaced by a good quality material or treated using mechanical and/or chemical stabilization.

According to Guyer, J. (2 011); US Army, (1994) Soil stabilization is the alteration of one or more soil properties, by mechanical or chemical means, to create an improved soil material possessing the desired engineering properties. As it was discussed in section 4.10 that, about 27% of the route corridors lies on unsuitable subgrade soil.

According to NCHRP (2009), Long-term performance of pavement structures significantly impacted by the stability of the underlying soils. Although stabilization is an effective alternative for improving soil properties, the engineering properties derived from stabilization vary widely due to heterogeneity in soil composition, differences in micro and macro structure among soils, heterogeneity of geologic deposits, and due to differences in physical and chemical interactions between the soil and candidate stabilizers.

5.2 Selection of chemical Stabilizing Agent

In fact, there are different methods of subgrade improving techniques, such as remove and replacing, grouting and chemical stabilization. But different literatures suggests that soil with fine grain sizes which classified as A-4, A-5, A-6, A-7, high PI value, low CBR high CBR swell and etc. are suitable for stabilization with lime and coarse grain size soil are suitable for stabilization with cement. As a result, lime stabilization techniques was followed for the present study since some of the route alignments of Modjo-Hawassa road projects, Meki-Ziway section fall under A-7 as per AASHTO classification system, high value of PI, low CBR values and high CBR swell.

According to Chen, (1988); Little and Nair, (2009), lime and cement are the main stabilizer agent since soil stabilization started and Proper selection of stabilizer depends on geotechnical properties of existing natural subgrade soil. Ranges of options are available for selecting soil stabilizer agents most of which are based on the soil classification following either the AASHTO or Unified soil classification system.

A simple and currently accepted method of stabilizer selection is based on soil index properties; plasticity index and percent passing the no. 200 sieve (Little and Nair, 2009). Once the stabilizer agent is selected, detailed laboratory tests to determine strength and performance characteristics of soils are required.

Therefore, based on AASHTO classification, soil types A-4, A-5, A-6, A-7 and some of A-2-6 and A-2-7 are suitable for stabilization with lime. When the chemical stabilization or modification of subgrade soils considered as the most economical or feasible alternate, the following criteria should be considered for chemical selection based on index properties of the soils (U.S. Air Force Academy, 1976.)

Accordingly, chemical Selection for soil Stabilization is given as following.

1) Lime: If PI > 10 and clay content (2μ) > 10%.

2) Cement: If PI ≤10 and < 20% passing No. 200.

In the present study, chemical stabilizer that is locally manufactured quick lime that was bought from DERBA MIDROC CEMENT FACTORY P.L.C was used for the stabilization of the sub-grade soil. Laboratory tests conducted on quick lime-soil mixtures to evaluate the performance of chemical additives on the sub-grade soils at varying proportions and with different curing periods.

The initial lime consumption was determined according to ASTM D6276-99a (1999) also known as “Eades and Grim” test. Laboratory tests with quick lime proportions of 2%, 4%, and 8% with 7, 14 and 28 days of curing periods were conducted.

The performance of the locally manufactured quick lime on the unsuitable sub-grade soil under investigation was evaluated based on test results of Atterberg limits (liquid limit, plastic limit and plasticity index), moisture-density relation of the sub-grade soil, and CBR and percent swell of CBR.

The results showed that the sub-grade soil is homogeneous along the trial road section and with depth and classified within the same A-7-5 (25), A-7-5(23) on the AASHTO and MH and CH on the USCS soil classification systems respectively (Refer to Chapter 4, fig 4.6).

Two representative soil samples along the road section were hence selected to evaluate the effects of a locally manufactured quick lime on such unsuitable sub-grade soil.

The percent pass 75μm sieve, plasticity index and classification of the two representative samples of the sub-grade soil under investigation are shown in Table 5.1

Table 5.1 Percent Pass 75μm, Plasticity Index and Classification of the Selected Soil Samples

ST	Depth in m	% pass no. 200 (75μm) sieve	Atterberg Limits			Classification	
			LL	PL	PI	AASHTO	USCS
km81+ 00	1-1.50	74	63	35	28	A-7-5(25)	MH
km82+00	1.2-1.80	71	65	33	32	A-7-5(23)	CH

5.4 Mixing Procedures

Mixing procedure for the present study was carried out based on ASTM D 698 test procedure. The proposed three percent lime (2%, 4% and 8%) thoroughly mixed with subgrade material and it allowed mellowing for 2 days. Treated subgrade materials Sprinkled during the mixing and mellowing operation, to achieve adequate hydration and proper moisture content. After mellowing, resume mixing until a homogeneous, friable mixture is obtained. After the completion of mellowing time, the same test procedure like untreated soil test was conducted and cured for 7, 14 and 28 days.

5.5 Initial Chemical Consumption Determination

The aim of this test is to identify the amount of lime necessary to satisfy immediate lime-soil reactions and also provide a sufficient quantity of calcium to maintain a high residual pH and sustain significant long-term pozzolanic reactions (ASTM D6276).

Particularly for quick lime, a PH of 12 is required to solubilize silicates and aluminates from the clay matrix and fine silt soil. During the present study, the initial consumption of lime was determined according to ASTM D6276 (Eades and Grim test).

The PH test result provided the initial lime consumption for stabilization and was estimated to be 2% for the clay soil. The dosages obtained by PH test have been checked by other tests, which include Atterberg limits, Moisture density relation, CBR, and percent swell of CBR by applying quick lime proportions of 2%, 4%, and 8% and the following fig 32 indicates initial consumption of lime by percent.

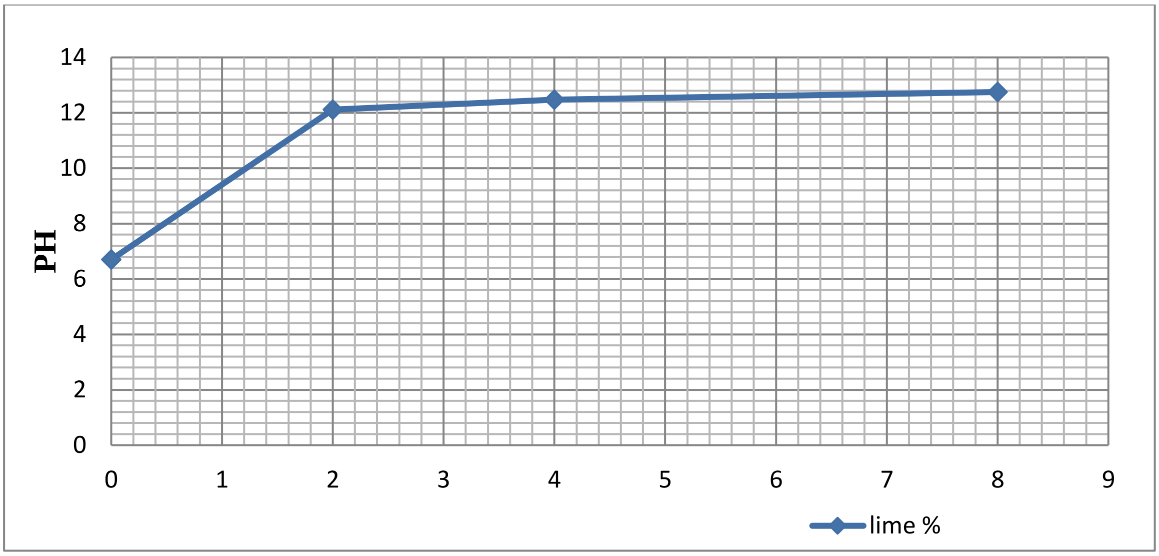

Fig 5.1 determining initial consumption of lime

5.5 Soil test results of treated soil samples and interpretation

In the present study, the subgrade soil test was conducted to evaluate the improvements of unsuitable sub-grade soil in its index properties, moisture density relation, California bearing ratio and CBR swell treated with different lime ratios. The soil samples were first air-dried and properly pulverized.

Atterberg limits test was conducted on soil samples passing no 40 sieve where as other tests were conducted on soil samples passing no four sieve as per AASHTO T87-86(2000) standards.

5.5.1Atterberg limits test (AASHTO T89)

The most important and principal aims of the present study was to evaluate the changes of liquid limits, plastic limits and plasticity index with addition of quick lime to unsuitable soil samples. To do that, liquid limit and plastic limit tests were conducted on quick lime soil mixtures according to consistency test of AASHTO T89 and T90, respectively.

Soil samples were first air dried, pulverized, and then sieved with no 40 sieve. Soil passing no 40 sieve was mixed with different proportion of chemical additives at optimum moisture content and kept for curing packed in plastic bags to protect loss of moisture. The proportion of quick lime used was 2%, 4%, and 8%.

The Atterberg limits tests of quick lime-soil soil mixtures were determined after 7, 14 and 28 days of curing to estimate the influence of time on Atterberg limit values. The test results of Atterberg limits for quick lime treated soil are presented in Table 20

Table 5.2 Atterberg limits test result of quick lime treated soil

ST				Curing Period	2%lime			4%lime			8%lime		
	LL	PL	PI		LL	PL	PI	LL	PL	PI	LL	PL	PI
Km 81	63	35	28	7 days	32.9	28.1	4.8	31.7	26.3	5.4	32.9	27.3	5.6
	-	-	-	14 days	30.2	22.6	7.6	32.7	25.5	7.2	33.8	27.2	6.6
				28 days	41	30.5	10.5	41.2	30.0	11.2	36.6	28.5	8.1
Km 82	65	33	32	7 days	32	27.2	4.8	32.8	28.4	4	33.2	27.4	5.8
	-	-	-	14 days	31	24.5	6.5	34	26.7	7.3	34.6	28.1	6.5
	-	-	-	28 days	39	28	11	42	31.9	10.1	38.3	30.1	8.2

<u>The performance of chemicals</u>

After completion of chemical stabilization test, the results of the chemical performances were as follows. The test summery of the quick lime mixed within unsuitable subgrade soil were presented in annex I, table 20

<u>Changes on Plasticity Index</u>

The addition of 2%, 4%, and 8% of quick lime and curing for 7 days has decreased the PI of the subgrade soil by 82.9%, 80.7%, and 79.3%, respectively. Furthermore, the addition of 2%, 4% and 8% quick lime and extending the curing period to 14 days has decreased the PI by 72.9%, 74.4% and 76.5%, respectively. In addition, extending curing period to 28 days has decreased PI by 67.2, 65.5%, and 74.68% respectively (fig5.3).

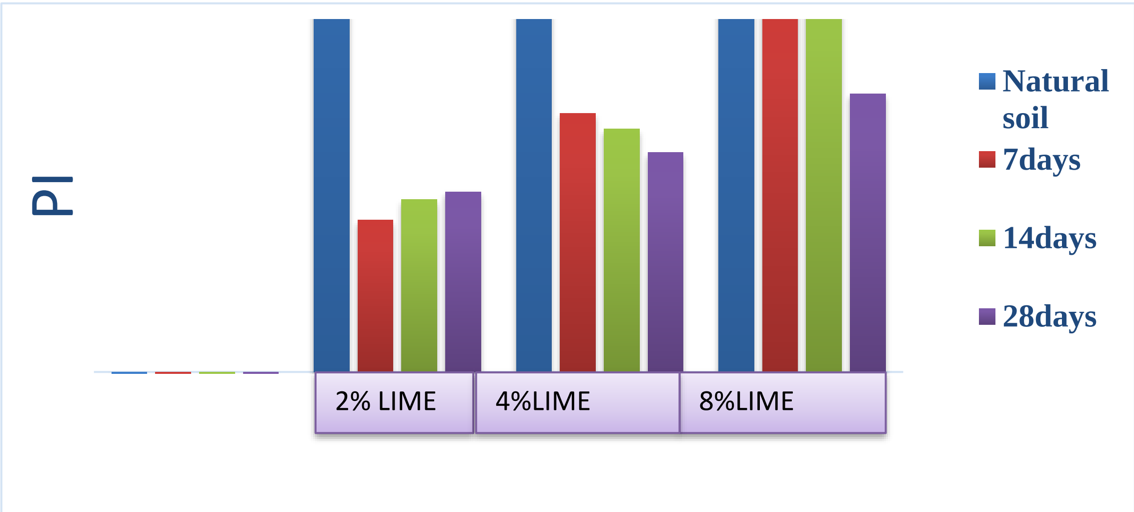

Fig 5.3 shows ST (Km81+00) plasticity index condition with different lime percent at different curing periods.

5.5.2 Moisture Density Relation (AASHTO T99)

Air-dried and pulverized soil passing no four sieve was used to determine moisture-density relation of the soil mixed with varying proportions of the chemical additives.

The soil was mixed with ratios of 2%, 4%, and 8% of quick lime, Further; standard proctor test was carried out according to AASHTO T99. Moisture content versus dry density graph was produced and optimum moisture content (OMC) and maximum dry density (MDD) were determined from the graph. The test results are shown in Table 5.3

Table 4 Moisture density relation test result of quick lime treated Soil

ST	**Parameter**	**Natural Soil**	**Lime Treated Soil**		
			2%	**4%**	**8%**
Km81+00	OMC%	32	33	33.6	34
	MDD g/ cm^3	1.27	1.25	1.2	1.2
Km82+00	OMC %	32	33.4	33.7	34.4
	MDD g/ cm^3	1.22	1.142	1.14	1.14

The addition of 2%, 4%, and 8% of quick lime and curing for 7 days has increases OMC of subgrade soil by 3%, 5% and 6% respectively. As observed from table 5.3 above, in both sample, OMC increase while MDD decrease. Alike variations in OMC and MDD observed in literature in case of lime stabilized subgrade soil that shows lime treated soil attain higher densities than natural soil (Achampong et al., 2013, Bayat et al., 2013).

California Bearing Ratio ((CBR) AASHTO T193)

The same soil sample passing no four sieve was mixed with the chemical additives at optimum moisture content and compacted in CBR molds at maximum dry density. Quick lime and soil mixtures were cured for 7, 14 and 28 days of curing periods to estimate the influence of time on CBR value. CBR tests were conducted after the curing periods at the worst condition soaked for 96hrs or 4days. The CBR test results are shown in Table 5.4

Table 5.4 CBR test result of quick lime treated Soil.

ST	Curing Period	Natural Soil	quick Lime Treated Soil		
			2%lime	4%lime	8%lime
Km81+00	7 days	4	4.5	4.7	5
	14 days	4	8.4	8.9	10.9
	28 days	4	14.5	20.5	22.5
Km82+00	7 days	3.5	4.1	4.8	5.6
	14 days	3.5	8.7	9.1	11
	28 days	3.5	15.2	21.4	23.3

Changes in CBR

The addition of 2%, 4%, and 8% quick lime and curing for 7 days has increased the CBR of unsuitable from 4% of the natural soil to 12.5%, 17.5%, and 25%, respectively. Furthermore, the addition of 2%, 4% and 8% quick lime and extending the curing period to 14 days has increased the CBR to 52.4%, 55% and 63.3%, respectively.(refer fig 5.5)

Fig 5.4 shows ST (Km81+00) CBR condition with different percent of lime different at curing time

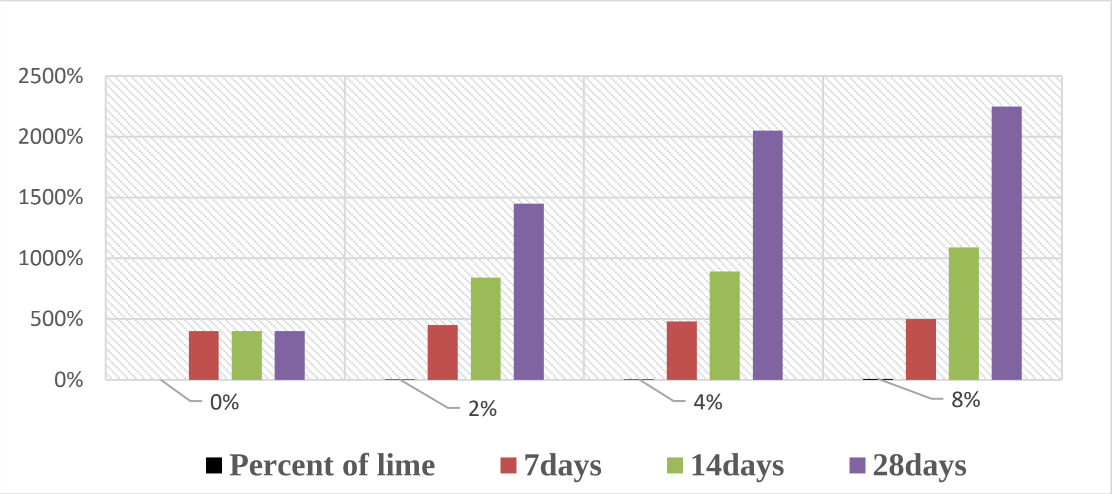

Fig 5.4 Percent of CBR (AASHTO T193)

Quick lime and soil mixtures compacted in CBR molds at optimum moisture content with maximum dry density were gauged for swelling characteristics before and after soaking to evaluate the percent swell.

The test helps to evaluate the change in volume of the sub-grade soil after treatment with the chemical additives at different ratios and with varying curing periods when exposed to moisture (Table 5.5).

Table 5.5 Percent swell (CBR) test result of quick lime treated Soil

ST	Curing Period	Natural Soil	Quick lime treated soil		
			2%lime	4%lime	8%lime
Km 81+00	7 days	3.2	0.35	0.33	0.30
	14 days	3.2	0.26	0.24	0.22
	28 days	3.2	0.61	0.58	0.5
Km 82+00	7 days	3.3	0.33	0.32	0.30
	14 days	3.3	0.29	0.25	0.22
	28 days	3.3	0.67	0.60	0.55

Changes in Swell Percent

The swell percent decreases much below 2% with the addition of a minimum of 4% chemical additives. (See fig 5.5 and 5.6).

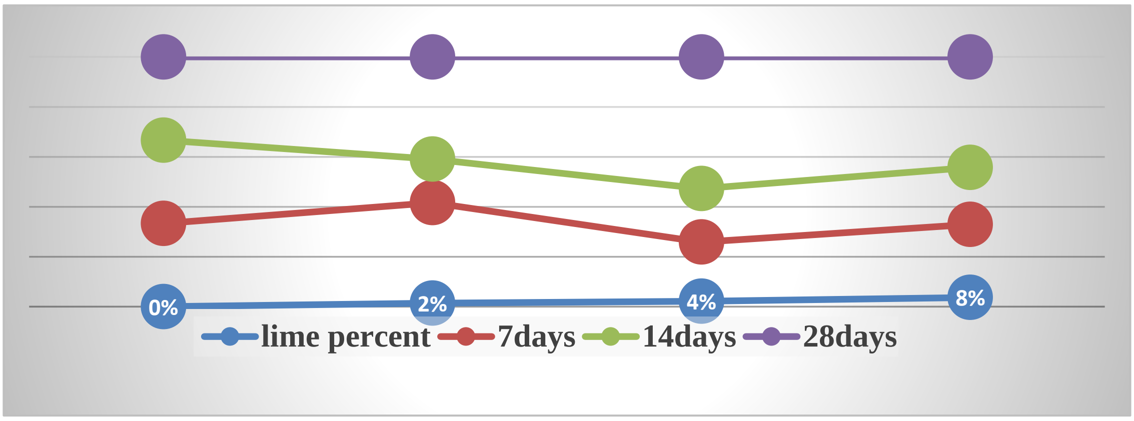

Fig 5.5 shows ST (Km81+00) CBR swell condition with different percent of lime different at curing time

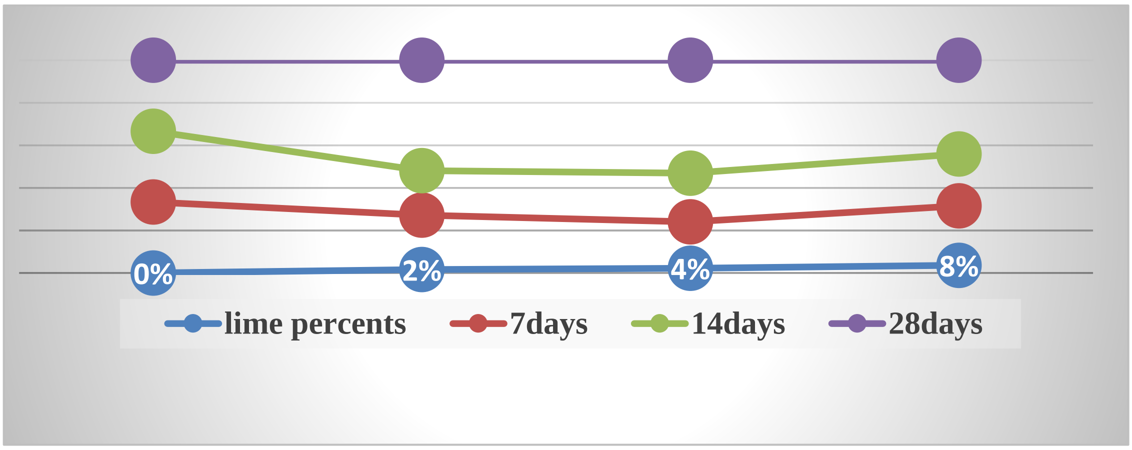

Fig 5.6 shows ST (Km82+00) CBR swell condition with different percent of lime different at curing times

5.5.2 Overall effect of Chemical stabilization on sub-grade soils

The chemical stabilizer quick lime has significantly changed the properties of the sub-grade soil. The changes attained due to different dosages of the chemical additives and the length of the curing periods was observed to vary accordingly. This proves that both proportion variation and length of curing periods have impact on the effectiveness of the chemicals to stabilize the sub-grade soil. In most of the cases, however, the effect of proportion of the chemical additives is more significant than that of the length of the curing periods.

Chapter 6

Conclusions and recommendation

7.1 Conclusion

The present study was carried out on Modjo - Hawassa High way project phase II, which Links Meki Town with Ziway. The study area is found in the eastern part of Oromia Regional state. The road project lies on the thick deposits of volcanic sediments such as sandy materials, volcanic ash and pumaceous materials.

The main objective of the present study was to characterize the sub-grade soil for its general suitability for pavement design and to work on possible improvement using the appropriate methods.

In order to achieve the objectives of the present study, literature review of the previous works, field observations and laboratory tests were conducted. In addition, the researcher has followed the systematic methodologies. Twenty- six (26) sub-grade soil samples and other two sample to mix lime were collected at different interval and tests conducted on 26 samples for the determination of Particle size distribution, hydrometer analysis, Atterberg limits, linear shrinkage and MDD, OMC and CBR and CBRswell values. Tests had conducted on both natural sub-grade soil and two (2) treated soil samples to evaluate the improvements achieved by the addition of the chemical agents. Parameters such as liquid limit, Plastic Limit, Plasticity index, proctor test, California bearing ratio (CBR) and CBR swell had been did for treated soil.

The test results for natural/ untreated sub-grade soil indicated that, 77% of soils possess Liquid limit having value less than or equal to 60%, whereas only 23% soils possess liquid limit values higher than 60%. Similarly19% of soils possess PI having value greater than 30%, whereas 81% soils possess PI values less than or equal to 30%. About 11.53% of soils are non-plastic. The group index which is a function of liquid limit, plasticity index and percentage passing 0.075mm have been calculated for the subgrade materials and about 27% of total samples shows high group index (GI>20%), which depict the subgrade materials to be unsuitable for subgrade materials according to AASHTO, 1993. Further, the maximum dry density for sub-grade soil samples varies between 0.9 and 1.59gm/cc. Similarly, the moisture content varies between 15% and 33% & the CBR values for the sub grade materials range from 2% to 30%. In addition, the linear shrinkage lies between 5% and 14.28%.

The geological formations and associated structures had also reviewed to interpret the geotechnical properties of the sub-grade soils in the project area.

Based on the analysis and interpretations of the test results, the sub-grade soils are classified as CH, CL, MH, ML, SC and SM in the USCS; A-7-5, A-7-6, A–6, A–5, A–4, A-2-4, A-2-5 and A-2-7 on the AASHTO classification system.

The unsuitability came from the low bearing capacity and higher volume change property of the soils with varying moisture content. For the unsuitable sub-grade soils, economically and practically feasible chemical stabilization method had determined in order to improve the soil engineering properties.

In order to give appropriate stabilization measures, the sub-grade material in the study area categorized as, unsuitable sub-grade material, whose CBR values at 95% of MDD is less than 5%, maximum plasticity index of 30%, Maximum CBR swell values of 2%. To do this, locally manufactured lime stabilization method had selected based on the thickness and grain size of sub-grade soils and the initial lime consumption had determined by Ph. test.

According to the test results, addition of hydrated lime has demonstrated significant improvements in the poor sub-grade soil under the present study. The engineering properties of the sub-grade soil has shown improvements with increasing dosages and with prolonged curing periods. In almost all the cases, the PI and swelling properties has decreased and the CBR has increased with respective increasing of the chemical dosages.

The addition of 2% of hydrated lime does not assure the improvement of the sub-grade soil to the desired engineering properties. But the addition of 4% of the chemical hydrated lime has sufficiently stabilize the sub-grade soil with further increment of the curing period the sub-grade soil has further improved.

7.2 Recommendation

Based on the present study results, following recommendation were forwarded:

This research was done solely for the native subgrade material characterization purpose in highway projects. For deep foundations like big bridges in similar road projects, further detail investigation should carried out. However, for minor structures, such as pipe, slab culverts, etc. whose foundation is in the unstable zone, the Dynamic cone penetration (DCP) test should exercise to give immediate solutions at the site.

The site is located near the active Ethiopian rift. Therefore, detailed seismic study should be done and appropriate seismic parameters such as seismic amplification of the soils and liquifability of the materials must had been incorporated in the design.

Along the route alignments, there are number of large ground cracks, which are orthogonal to the road alignment, so that detail study should be done on cause of ground cracks near the present study area.

Certain roadside drainage systems like ditch, culverts and others draining techniques should be made. Because, the chemical reactions affected by the different environmental condition as if erosion, surface runoff, and any water that enter into stabilized layer. Therefore while chemical stabilization is practiced these environmental conditions must be taken into consideration and avoid entrance of excess water to the pavement by providing drainage controlling structure.

Finally, the present study is an indicative and not as such detailed, thus the suitability of the soil is evaluated only in terms of use for subgrade materials, and hence the result should not be directly used for other purpose such as building, dam and reservoir site selection.

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ANNEXURE I laboratory test result summery of natural subgrade soil

S.no	ST	Sieve Analysis % pass			Atterberg Limit				Soil Classification			Procter Test		CBR at 95% MDD	CBR Swelling	ERA Strength class	Specific Gravity
		Sieve no															
		2	40	200	LL	PL	PI	LS	AASHTO	GI	USCS	MDD	OMC				
1	56.5km	100	99.5	85.9	71	32.7	38		A-7-5(38)	38	CH	1.360	28	4.4%	3.5	S2	2.35
2	58km	100	96.8	83.7	72	41.6	30.4		A-7-5(31)	31	CH	1.340	30	3.5%	2.89	S2	2.3
3	59.5km	100	100	51	68.3	53	16		A-7-5(8)	8	MH	1.392	16.80	2.00	2.39	S1	2.35
4	61km	100	98.5	80	75	39	36		A-7-5(34)	34	MH	1.260	28	4.0%	3.28	S2	2.35
5	62.5km	97.5	88.00	19.9	0	0	NP	-	A-2-4 (0)	0	SM	.903	15.00	25%	1.07	S5	-
6	64km	98.9	95.7	23.7	33.4	27.8	5.6	-	A-2-4(0)	0	SM	1.590	18.00	6.4	0.92	S3	-
7	65.5km	98.5	95.6	79.5	59	27.4	31.6		A-7-6 (27)	27	SM	1.260	31.00	4.0%	3.26	S2	-
8	67km	96.2	93.3	27.8	0	0	NP	-	A-2-4 (0)	0	SM	1.060	22.80	7.5%	2.28	S4	-
9	68.5km	100	99.6	51.2	39.1	29	11		A-6 (0)	0	ML	1.230	16.50	7%	1.3	S4	-
10	70km	99.8	97.2	52.1	30	23	7	-	A-4 (0)	0	CL	1.250	25.20	10%	0.5	S4	-
11	71.5km	99.8	96.6	51	31	8	23.4	-	A-4 (0)	0	CL	1.389	23.80	9.8%	0.74	S4	-
12	73km	99.9	98.2	44.1	42	28	14	-	A-7-6 (4)	4	SM	1.223	20.50	5%	1.03	S3	-
13	74.5km	97.4	80.2	33.3	44	29	15	-	A-2-7 (1)	1	SM	1.220	26.70	5.1%	0.96	S3	-
14	76km	99.3	85.7	40.5	35.3	24.4	11	-	A-6 (0)	0	SC	1.471	27.00	7.3%	1.37	S3	2.33
15	77km	99.6	87.4	30.3	44	31	13	-	A-2-7 (0)	0	SM	1.359	28.20	8.3%	0.46	S4	2.23
16	78km	99.1	89.1	35.2	50.2	36.7	13.5	-	A-2-7 (1)	1	SM	1.300	32.70	14%	1.09	S4	2.32
17	79km	99.1	83.8	41	43.6	33.6	10	-	A-5 (5)	5	SM	1.340	32.00	8.8%	0.83	S4	2.32
18	80km	98.3	77'9	22.3	44.9	33.7	11.2	-	A-2-5 (0)	0	SM	1.217	32.70	9%	0.88	S4	2.38
19	81km	98.2	94.2	74	63	34.9	28.1	-	A-7-5(23)	23	MH	1.220	32	4.0%	3.17	S2	2.38
20	82km	96.9	89.9	72.1	65	32.5	32		A-7-5(25)	25	CH	1.270	32.00	3.5%	3.26	S2	2.28
21	83km	98.5	86.8	58.6	31.6	24.9	6.7	-	A-4 (0)	0	ML	1.345	26.70	30.3%	0.32	S6	2.28
22	84km	94.8	79.6	45.8	33.3	27	6.3	-	A-4 (0)	0	SM	1.285	29.30	9.9%	0.38	S4	2.36
23	85km	97.4	86.8	25.4	0	0	NP	-	A-2-4 (0)	0	SM	1.278	17.30	8.1%	1.36	S4	2.32
24	86km	86.8	68.1	39.6	36.8	24.4	12.4	-	A-6 (0)	0	SC	1.120	26.40	9.00%	2.0	S4	2.338
25	87.5km	98.7	86.7	54.5	37.8	25.2	12.6		A-6 (0)	0	CL	1.249	22.69	6.5%	0.60	S3	-
26	89km	98	86.8	51.7	38	27.3	10.7		A-6 (0)	0	ML	1.260	30.50	9.5%	1.16	S4	-

Detail calculations of the unit delineation by cumulative sums method values

S. no	St(km)	CBR	Interval no (n)	Interval distance (Δxi)	CID $\sum(\Delta xi$)	Average interval response (CBR)	Actual interval area (ai)	Cumulative area (∑ai)	Zx value
1	56+500	3.5	1	1.50	1.500	3.5	1.75	1.75	1.75-1.75×1.5= -0.875
2	58+00	2.89	2	1.5	3	3.195	4.79	6.54	6.54 -1.75×3 = 1.29
3	59+500	2.39	3	1.5	4.500	4.39	6.585	13.125	13.125 -1.75×4.500 = 5.25
4	61+00	3.28	4	1.5	6.00	2.835	4.2525	17.3775	17.3775 -1.75×6.00 = 6.8775
5	62+500	1.07	5	1.5	7.5.00	2.175	3.2625	20.64	20.64 -1.75×7.500 = 7.515
6	64+00	0.92	6	1.5	9.00	0.995	1.4925	22.1325	22.1325 -1.75×9.50 = 6.3825
7	65+500	3.26	7	1.5	10.50	2.09	3.135	25.2675	25.2675 -1.75×10.50 = 6.8925
8	67+00	2.28	8	1.5	12.0	2.77	4.155	29.4225	29.4225 -1.75×12.0 = 8.4225
9	68+500	1.3	9	1.5	13.50	1.79	2.685	32.1075	32.1075 -1.75×13.50 = 8.4825
10	70+00	0.5	10	1.5	15.00	0.9	1.35	33.4575	33.4575 -1.75×15.00 = 7.2075
11	71+00	0.74	11	1.5	16.500	0.62	0.93	34.3875	34.3875 -1.75×16.500 = 5.51
12	72+500	1.03	12	1.5	18.00	0.885	1.3275	35.715	35.715 -1.75×18.00 = 4.215
13	74+00	0.96	13	1.5	19.500	0.995	1.4925	37.2075	37.2075 -1.75×19.500 = 3.0825
14	76+00	1.37	14	2	21.5	1.165	2.33	39.5375	39.5375 -1.75×21.5 = 1.9125
15	77+00	0.46	15	1	22.5	0.915	0.915	40.4525	40.4525 -1.75×22.5 = 1.0775
16	78+00	1.09	16	1	23.5	0.775	0.775	41.2275	41.2275 -1.75×23.5 = 0.1025
17	79+00	0.83	17	1	24.5	0.96	0.96	42.1875	42.1875 -1.75×24.5 = -0.6875
18	80+00	0.88	18	1	25.5	0.855	0.855	43.0425	43.0425 -1.75×25.5 = -1.5825
19	81+00	3.17	19	1	26.5	2.025	2.025	45.0675	45.0675 -1.75×26.5 = -1.3075
20	82+00	3.26	20	1	27.5	3.215	3.215	48.2825	48.2825 -1.75×27.5 = 0.1575
21	83+00	0.32	21	1	28.5	1.79	1.79	50.9725	50.9725 -1.75×28.5 = 1.0975
22	84+00	0.38	22	1	29.5	0.35	0.35	50.4225	50.4225 -1.75×29.5 = -1.2025
23	85+00	1.36	23	1	30.5	0.87	0.87	51.2925	51.2925 -1.75×30.5 = -2.0825
24	86+00	2.0	24	1	31.5	1.68	1.68	52.9725	52.9725 -1.75×31.5 = -2.1525
25	87+500	3.5	25	1.5	33	2.75	4.125	57.0975	57.0975 - 1.75×33 = -0.6525
26	89+00	2.89	26	1.5	34.5	3.195	4.7925	60.2925	60.2925 -1.75×34.5 = -0.0825

Moisture content and plasticity index of the sub grade soils of the present study

St no	**ST**	**MDD**	**OMC**
1	56.5km	1.360	28
2	58km	1.340	30
3	59.5km	1.392	16.80
4	61km	1.260	28
5	62.5km	.903	15.00
6	64km	1.590	18.00
7	65.5km	1.260	31.00
8	67km	1.060	22.80

9	68.5km	1.230	16.50
10	70km	1.250	25.20
11	71.5km	1.389	23.80
12	73km	1.223	20.50
13	74.5km	1.220	26.70
14	76km	1.471	27.00
15	77km	1.359	28.20
16	78km	1.300	32.70
17	79km	1.340	32.00
18	80km	1.217	32.70
19	81km	1.220	32
20	82km	1.270	32.00
21	83km	1.345	26.70
22	84km	1.285	29.30
23	85km	1.278	17.30
24	86km	1.120	26.40
25	87.5km	1.249	22.69
26	89km	1.260	30.50

Moisture contents vs plasticity index

St no	ST	OMC	PI
1	56.5km	38	28
2	58km	30.4	30
3	59.5km	16	16.80
4	61km	36	28
5	62.5km	NP	15.00
6	64km	5.6	18.00
7	65.5km	31.6	31.00
8	67km	NP	22.80
9	68.5km	11	16.50
10	70km	7	25.20
11	71.5km	23.4	23.80
12	73km	14	20.50
13	74.5km	15	26.70
14	76km	11	27.00
15	77km	13	28.20
16	78km	13.5	32.70
17	79km	10	32.00
18	80km	11.2	32.70
19	81km	28.1	32
20	82km	32	32.00
21	83km	6.7	26.70
22	84km	6.3	29.30
23	85km	NP	17.30
24	86km	12.4	26.40
25	87.5km	12.6	22.69
26	89km	10.7	30.50

Test result of CBR within stations.

St no	ST	CBR
1	56.5km	4.4%
2	58km	3.5%
3	59.5km	2.00
4	61km	4.0%
5	62.5km	25%
6	64km	6.4
7	65.5km	4.0%
8	67km	7.5%
9	68.5km	7%
10	70km	10%
11	71.5km	9.8%
12	73km	5%
13	74.5km	5.1%
14	76km	7.3%
15	77km	8.3%
16	78km	14%
17	79km	8.8%
18	80km	9%
19	81km	4.0%
20	82km	3.5%
21	83km	30.3%
22	84km	9.9%
23	85km	8.1%
24	86km	9.00%
25	87.5km	6.5%
26	89km	9.5%

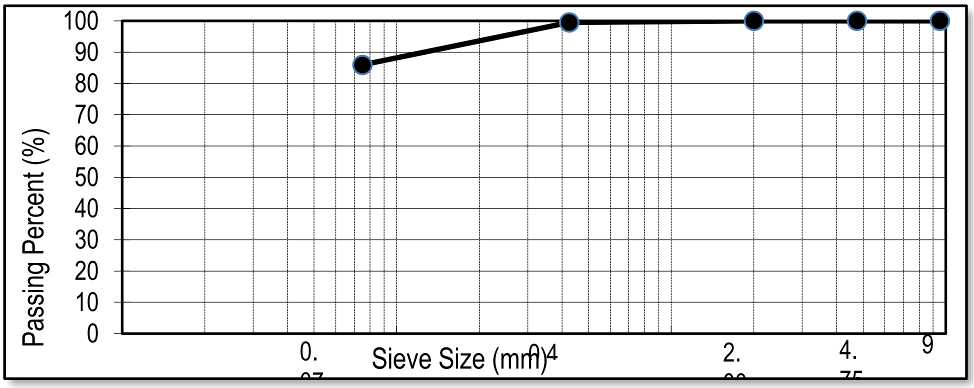
Test result of CBR Swell within stations.

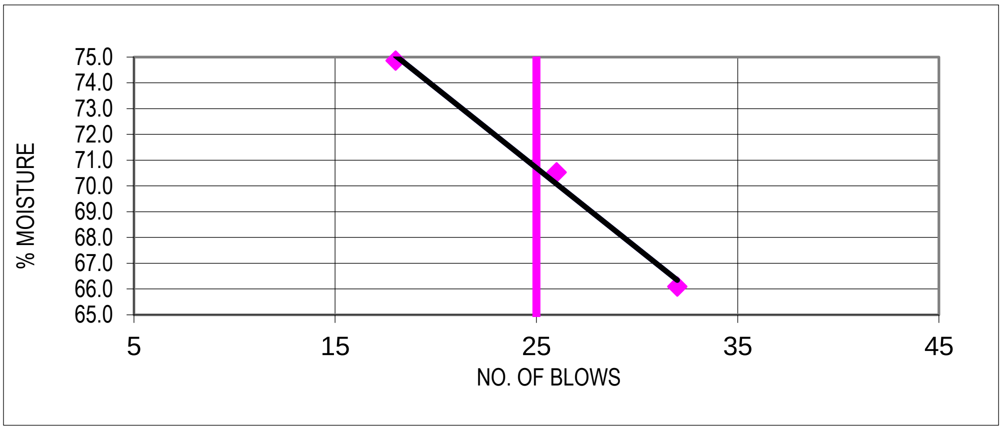
St no	ST	CBR Swell
1	56.5km	3.5
2	58km	2.89
3	59.5km	2.39
4	61km	3.28
5	62.5km	1.07
6	64km	0.92
7	65.5km	3.26
8	67km	2.28
9	68.5km	1.3
10	70km	0.5
11	71.5km	0.74
12	73km	1.03
13	74.5km	0.96
14	76km	1.37
15	77km	0.46
16	78km	1.09
17	79km	0.83
18	80km	0.88
19	81km	3.17
20	82km	3.26
21	83km	0.32
22	84km	0.38
23	85km	1.36
24	86km	2.0
25	87.5km	0.60
26	89km	1.16

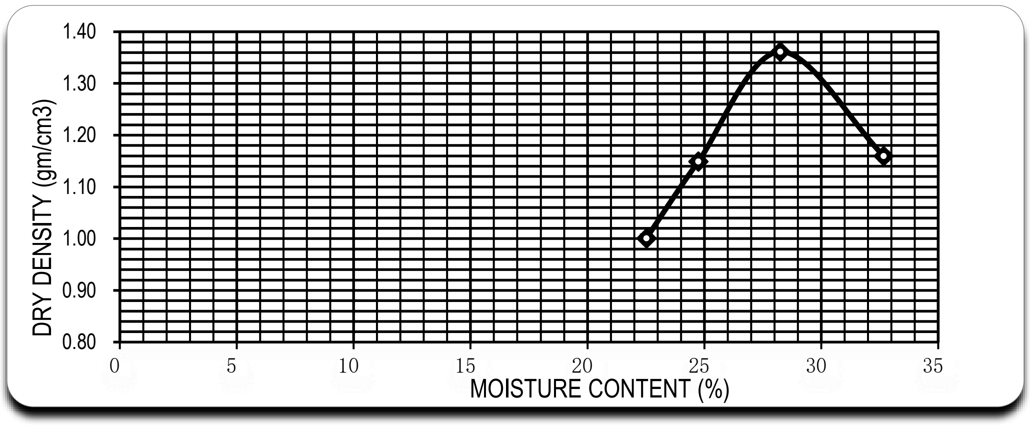
ANNEXURE II

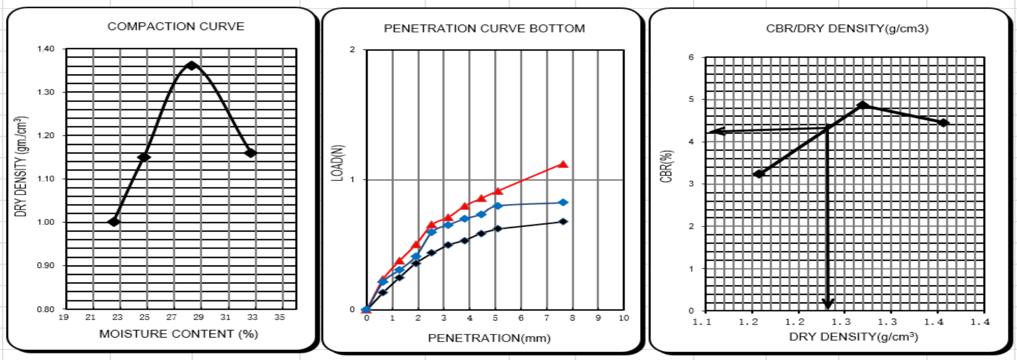
Graphs and soil profile figures

Gradation, PI, MDD, CBR vs Proctor curves for sample from 56+500km



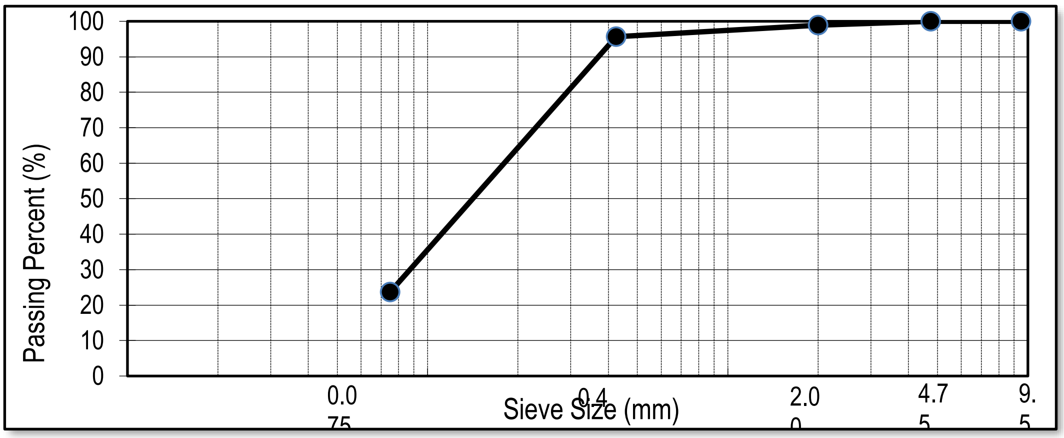


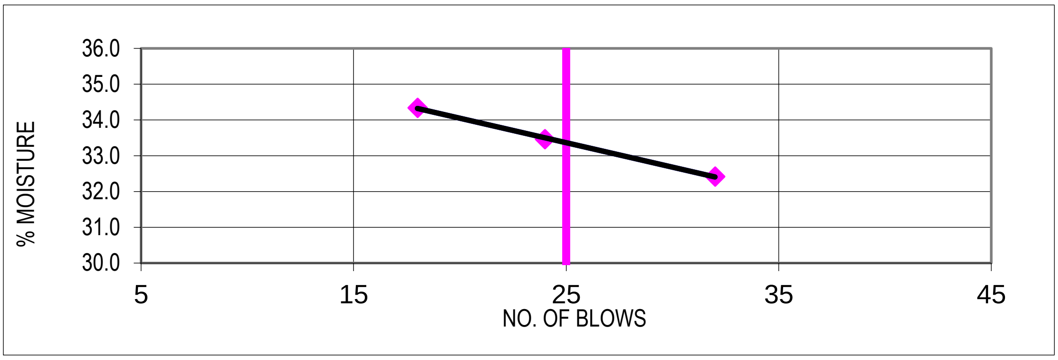


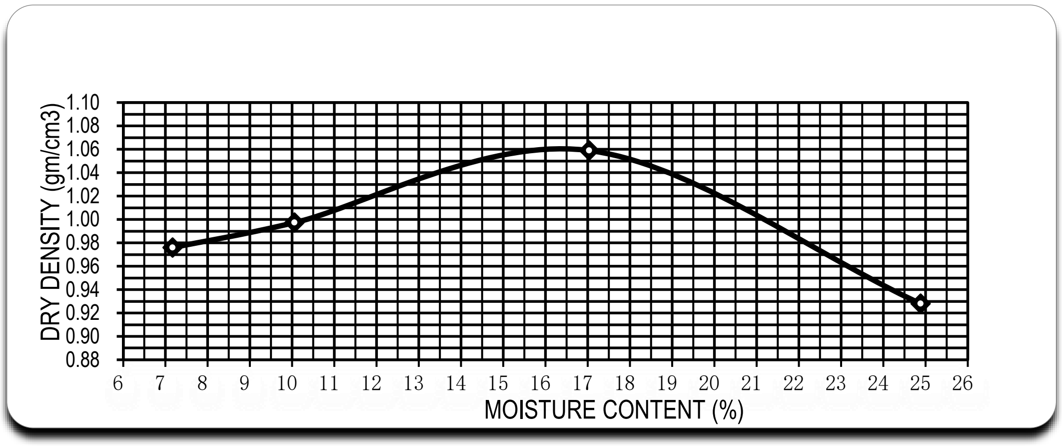


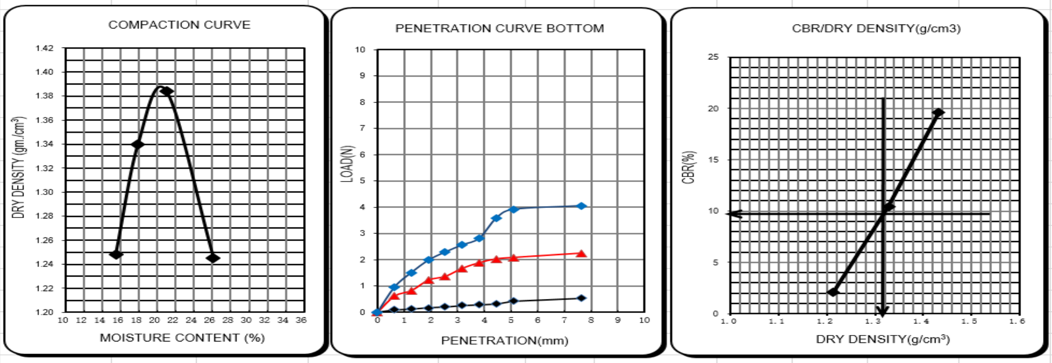
95% M.D.D =	1.292
CBR Value =	4.4%

Gradation, PI, MDD, CBR vs Proctor curves for sample from 64+00km



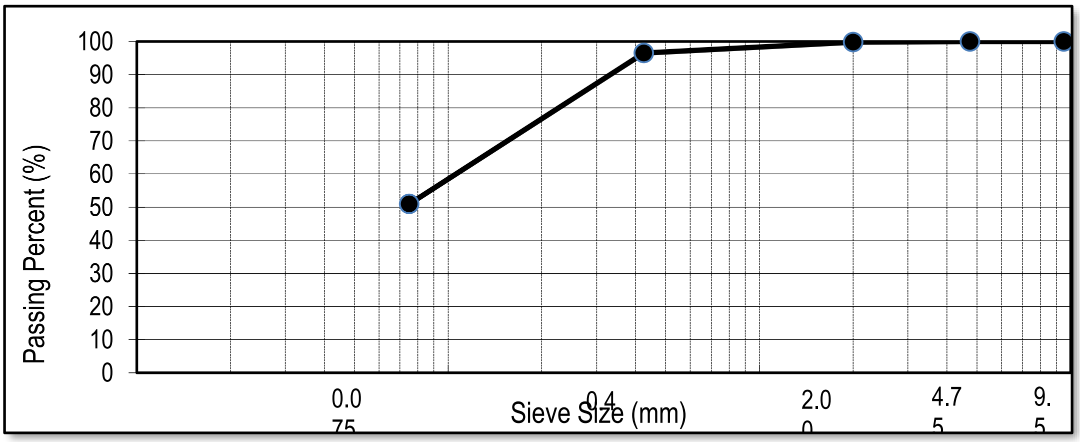


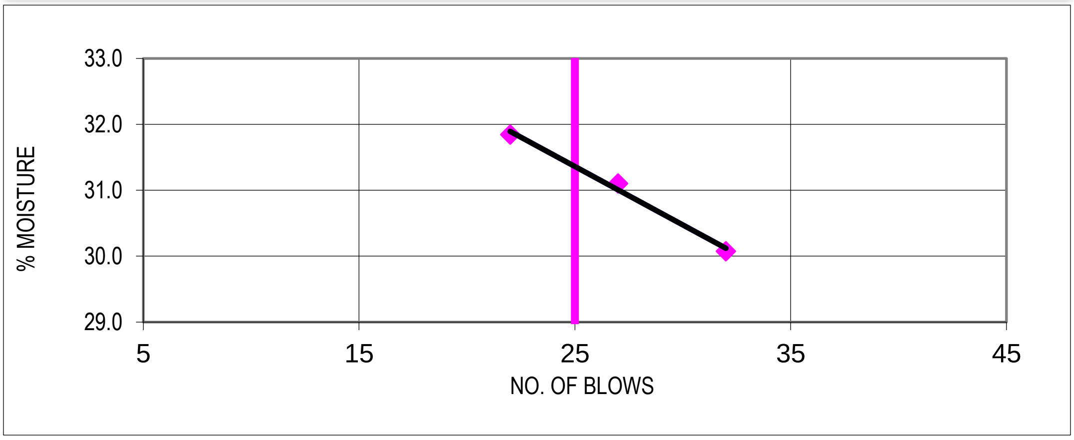


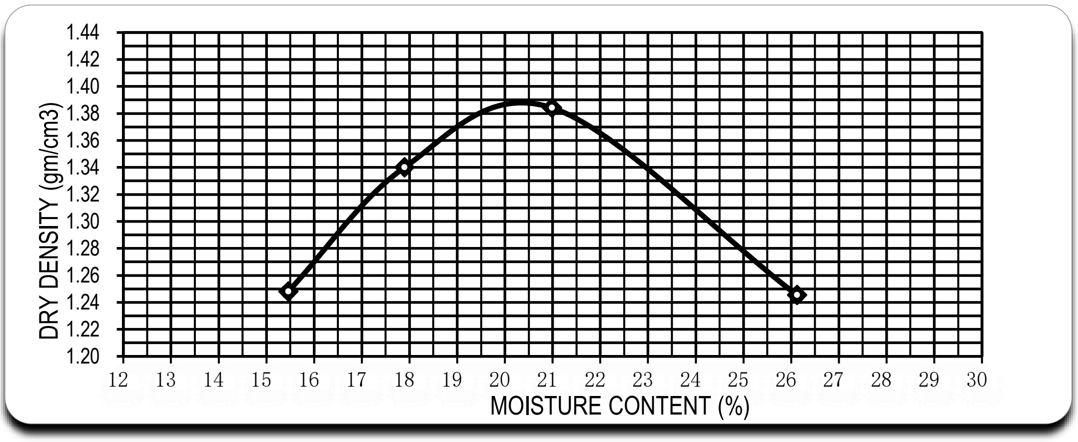


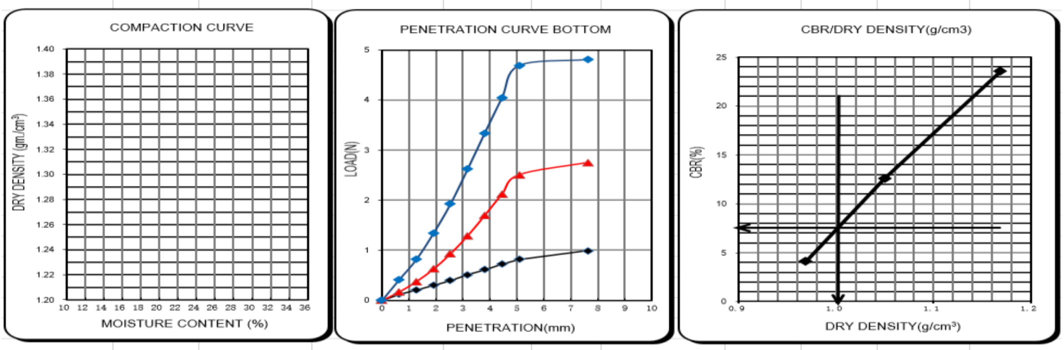
95% M.D.D =	1.320
CBR Value =	9.8%

Gradation, PI, MDD, CBR vs Proctor curves for sample from 71+500



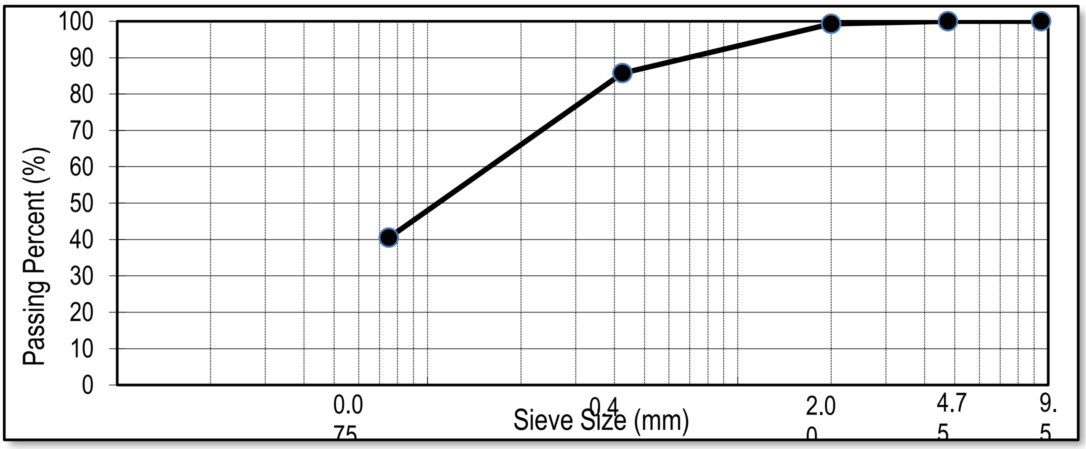


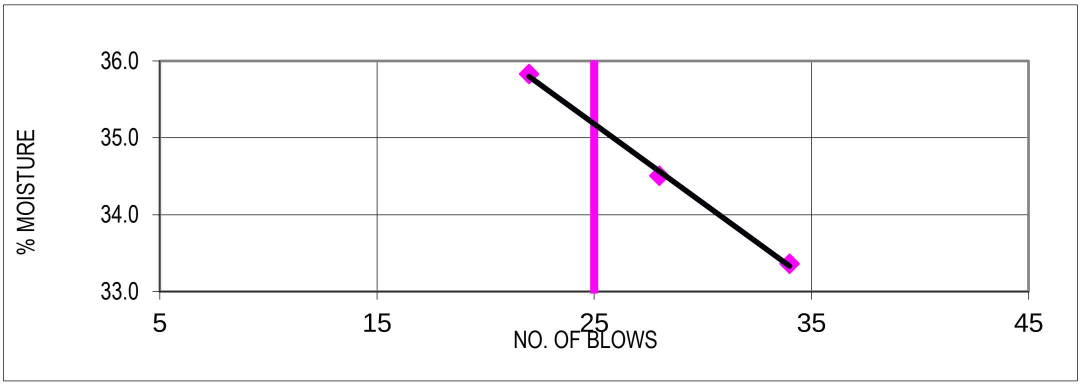


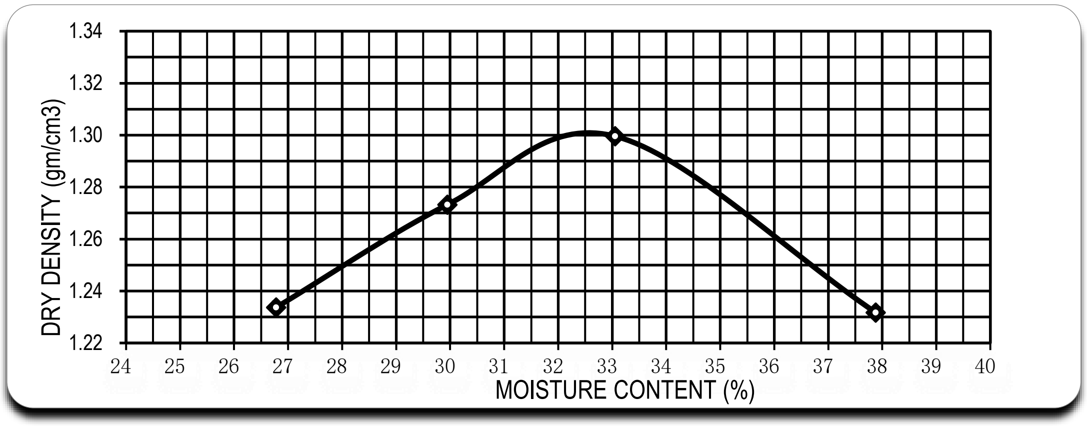


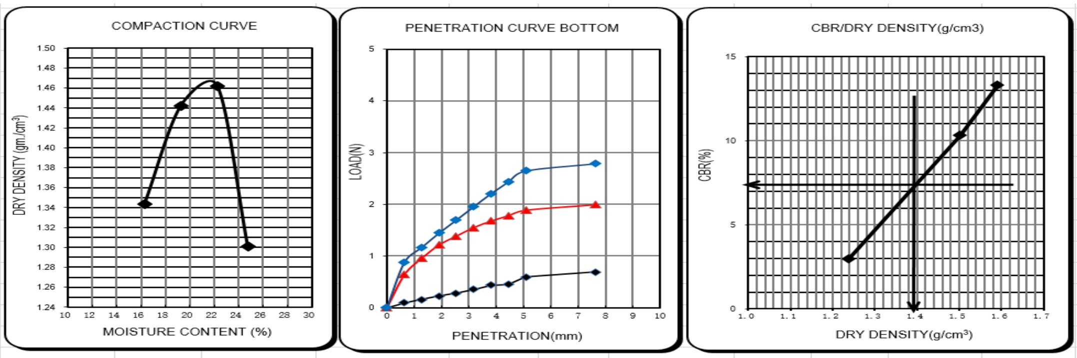
95% M.D.D =	1.007
CBR Value =	7.5%

Gradation, PI, MDD, CBR vs Proctor curves for sample from 76+00km



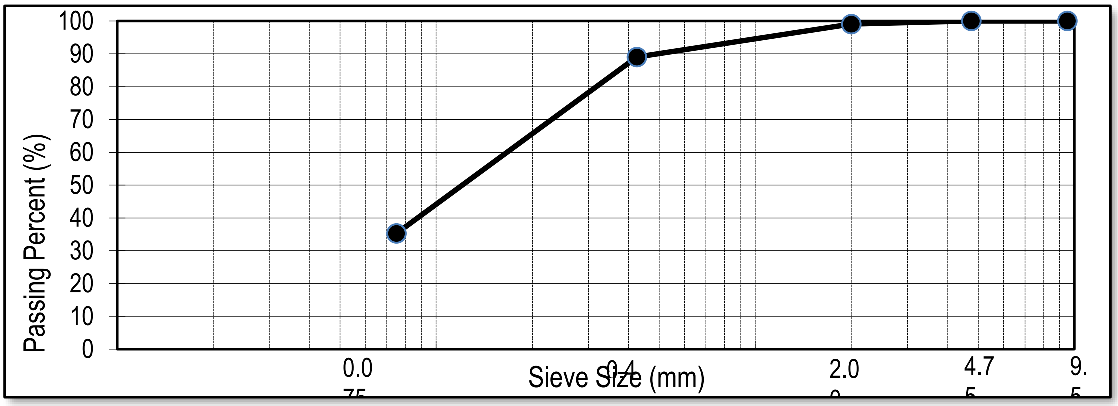


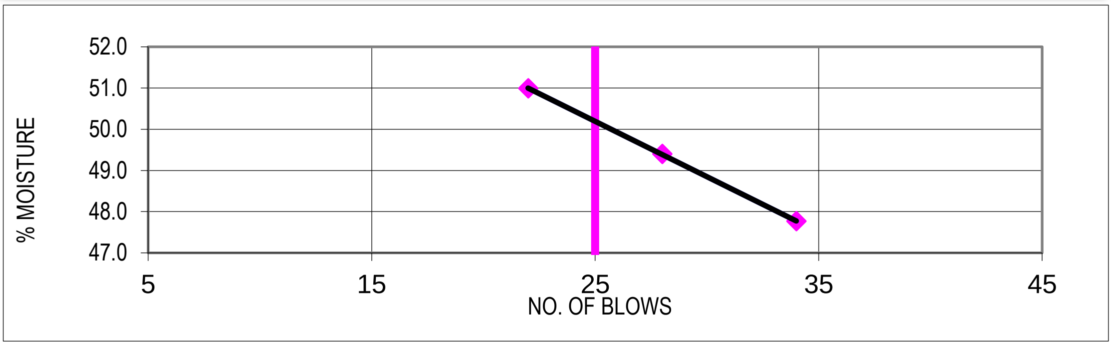


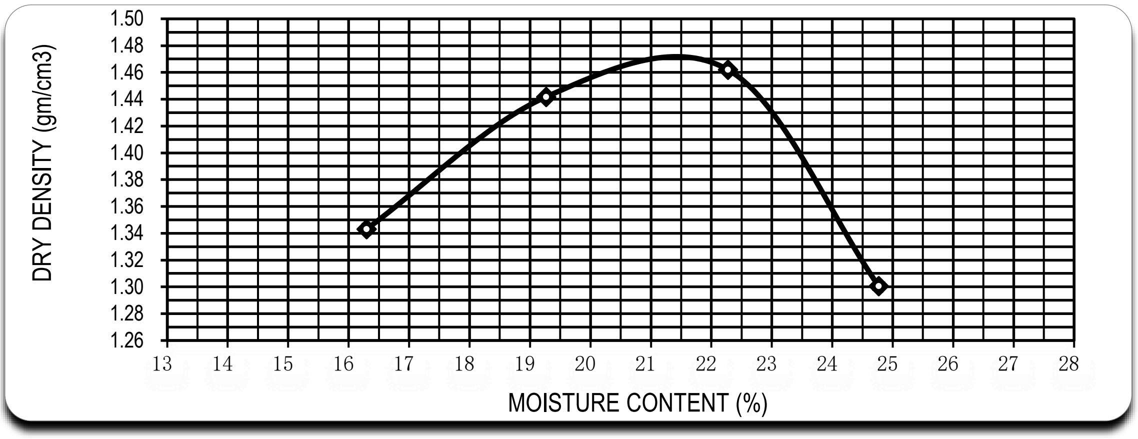


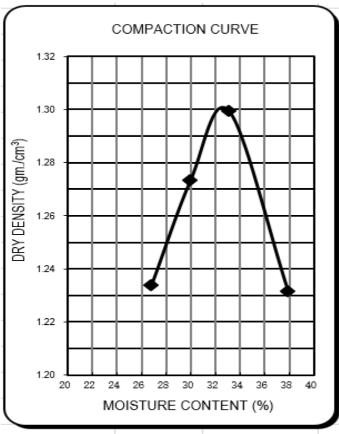
95% M.D.D =	1.397
CBR Value =	7.3%

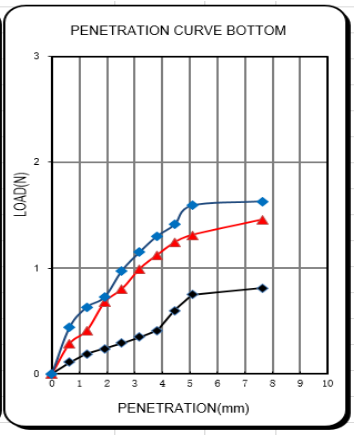
Gradation, PI, MDD, CBR vs Proctor curves for sample from 78+00km

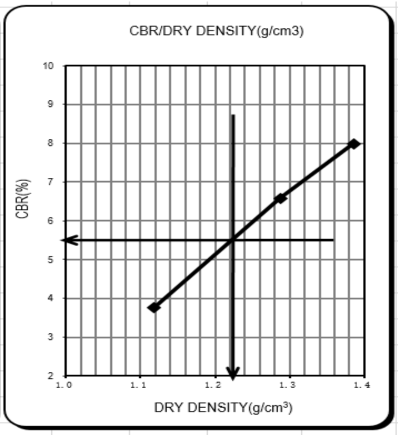






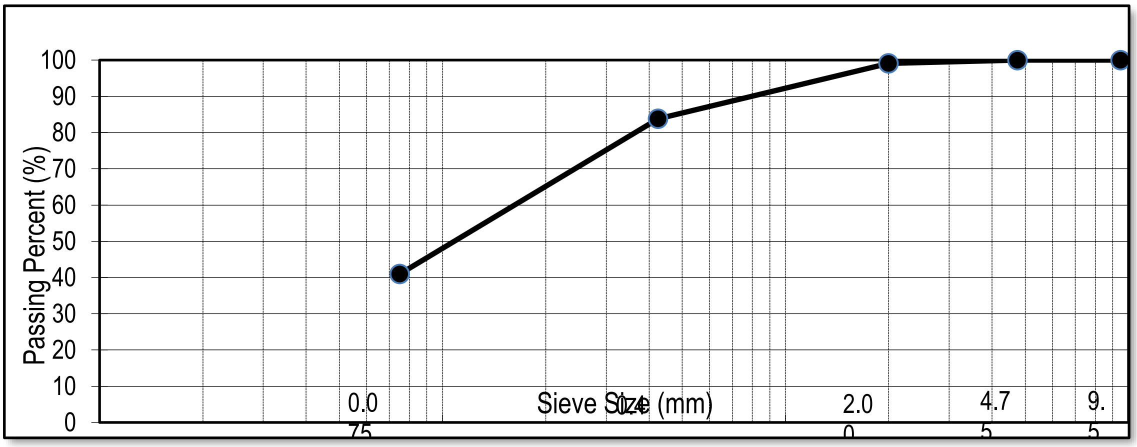


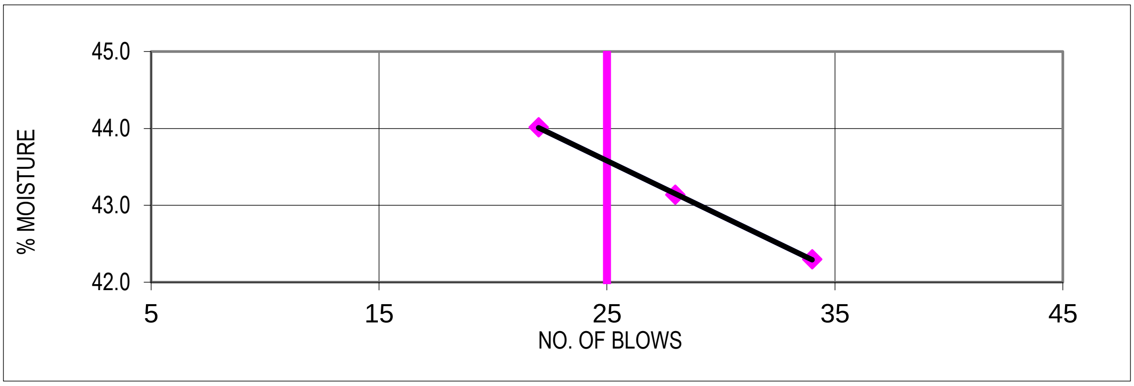


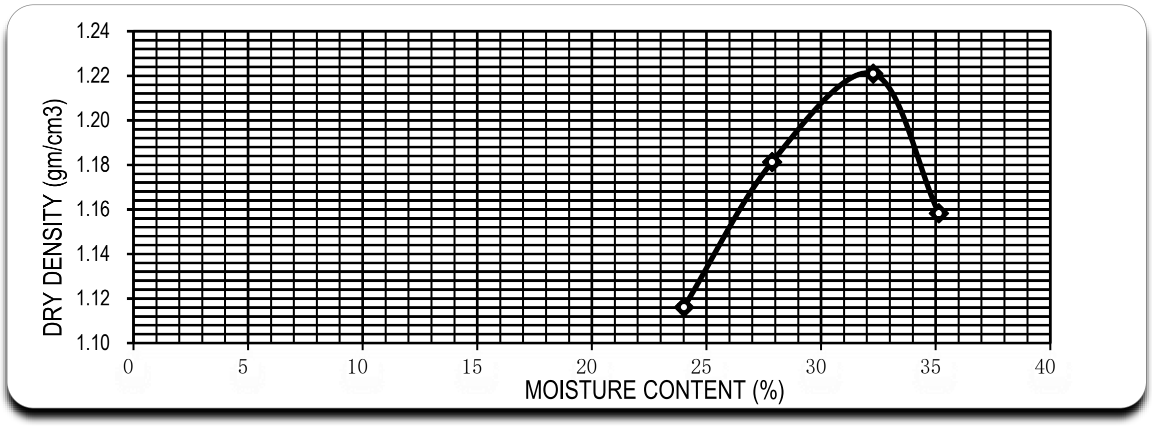


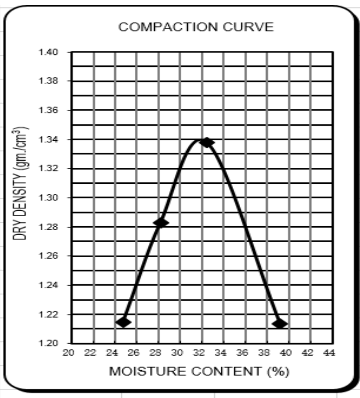
95% M.D.D =	1.235
CBR Value =	14.0%

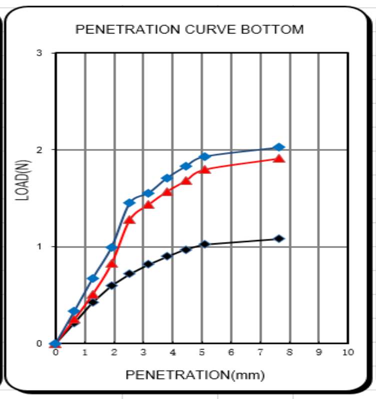
Gradation, PI, MDD, CBR vs Proctor curves for sample from 79+00km

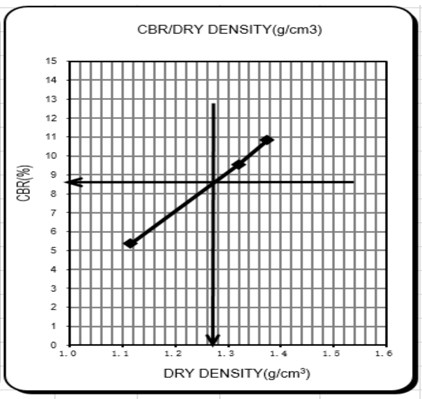






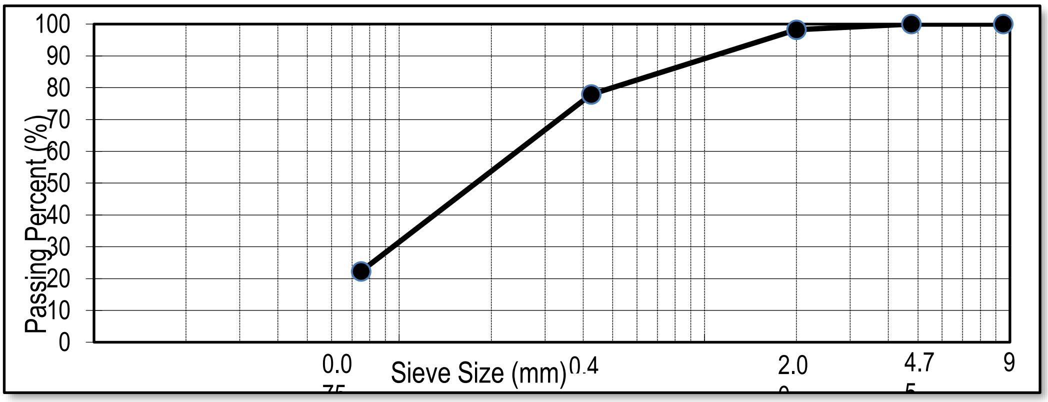


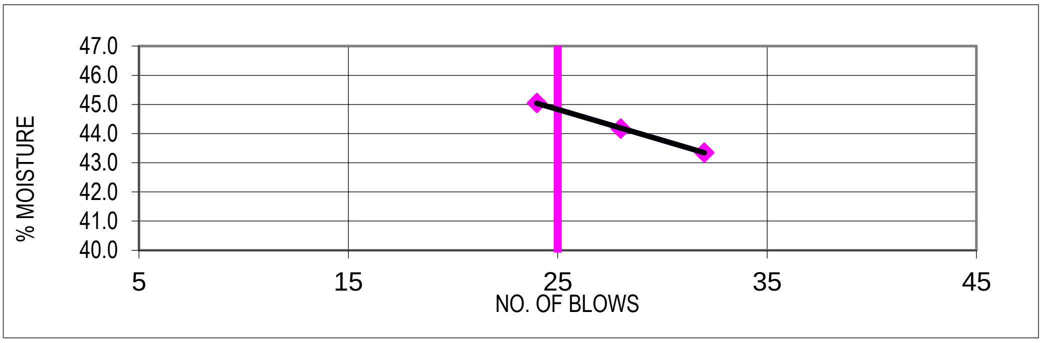


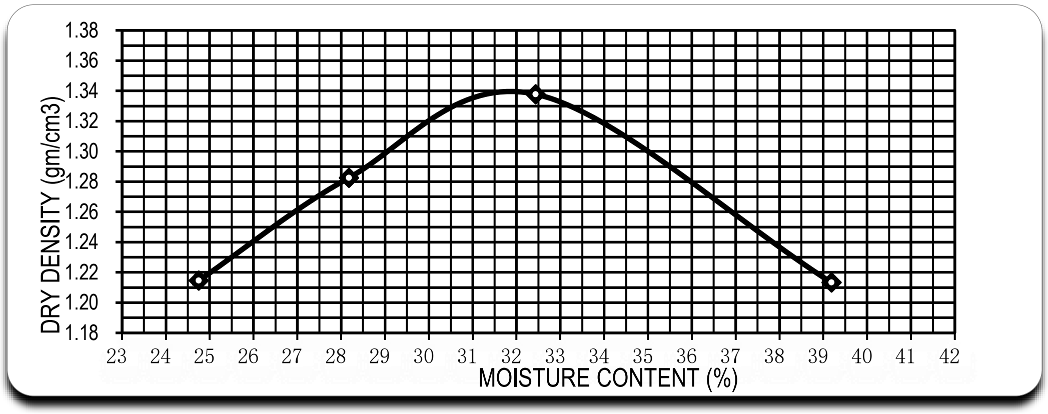


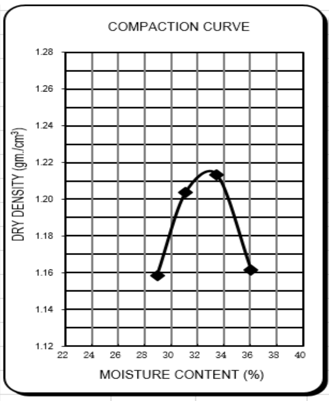
95% M.D.D =	1.273
CBR Value =	8.8%

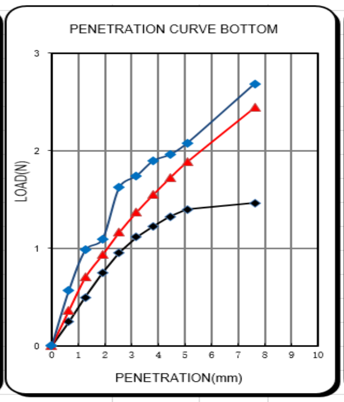
Gradation, PI, MDD, CBR vs Proctor curves for sample from 80+00km

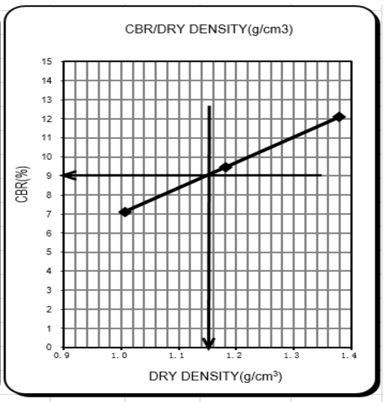






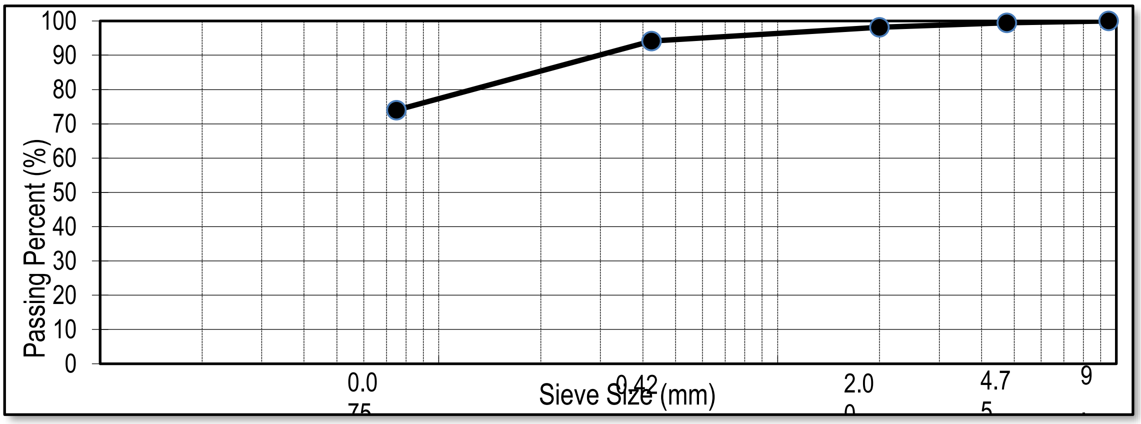


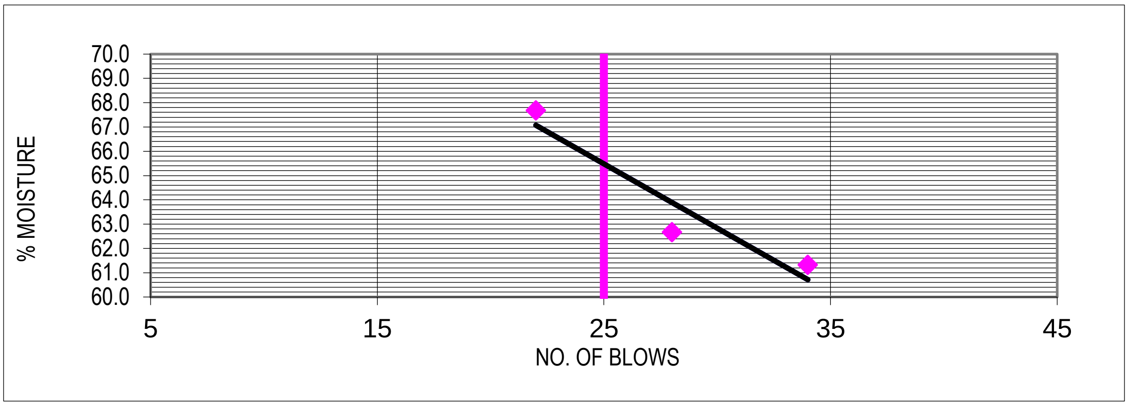


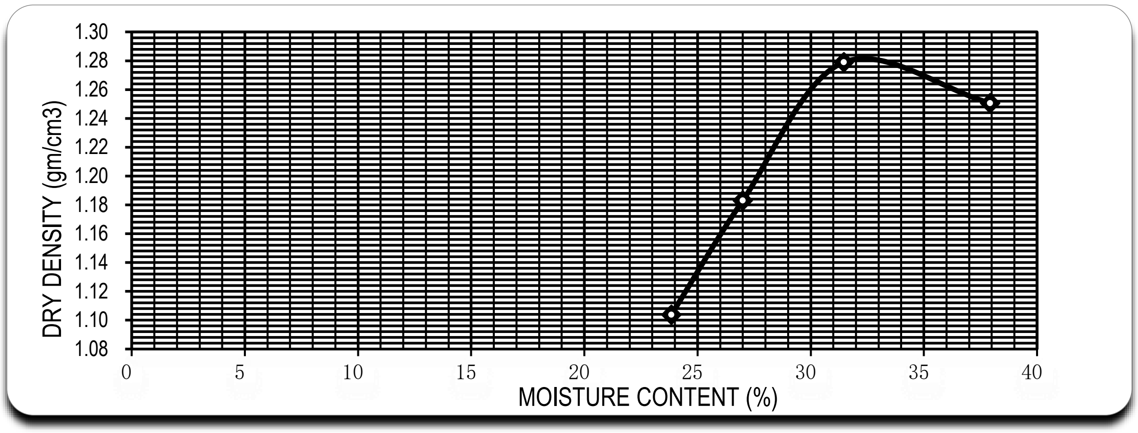


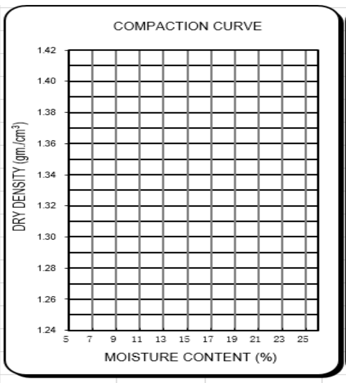
95% M.D.D =	1.156
CBR Value =	9.0%

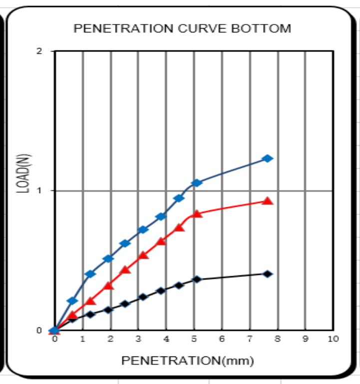
Gradation, PI, MDD, CBR vs Proctor curves for sample from 81+00km

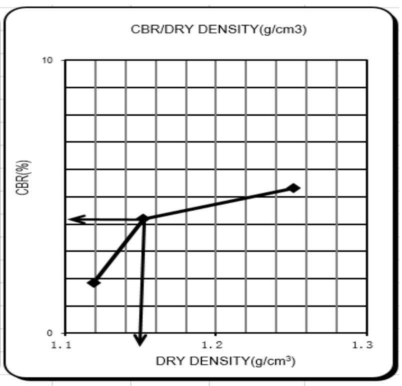






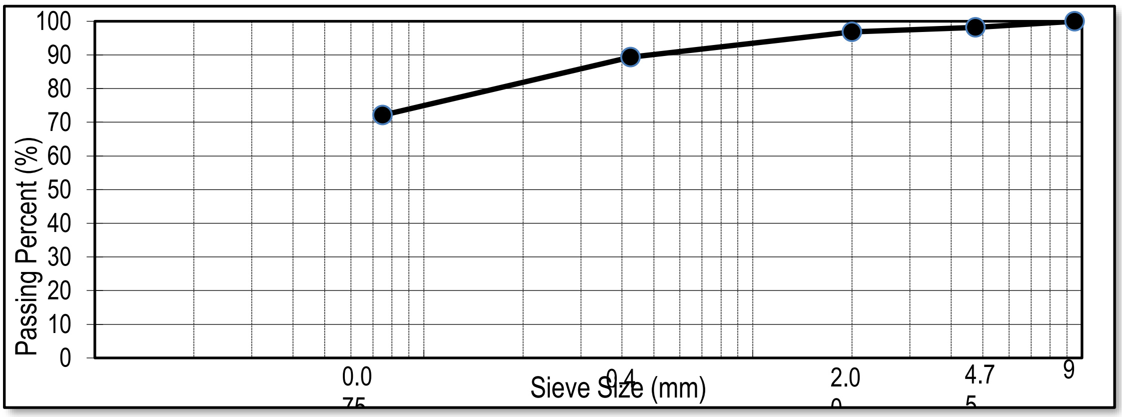


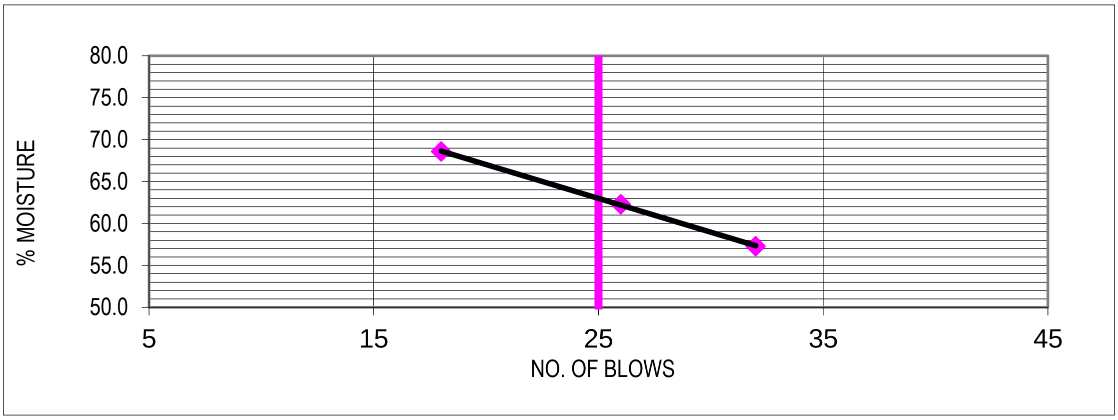


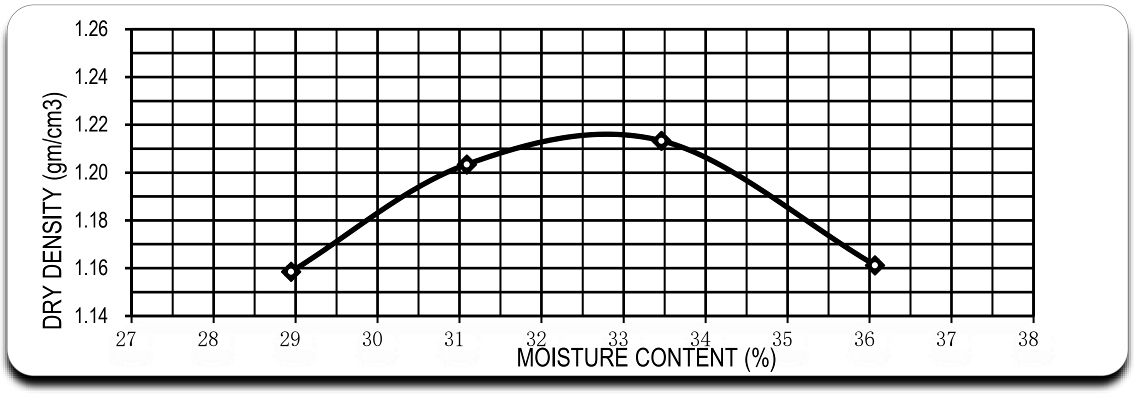


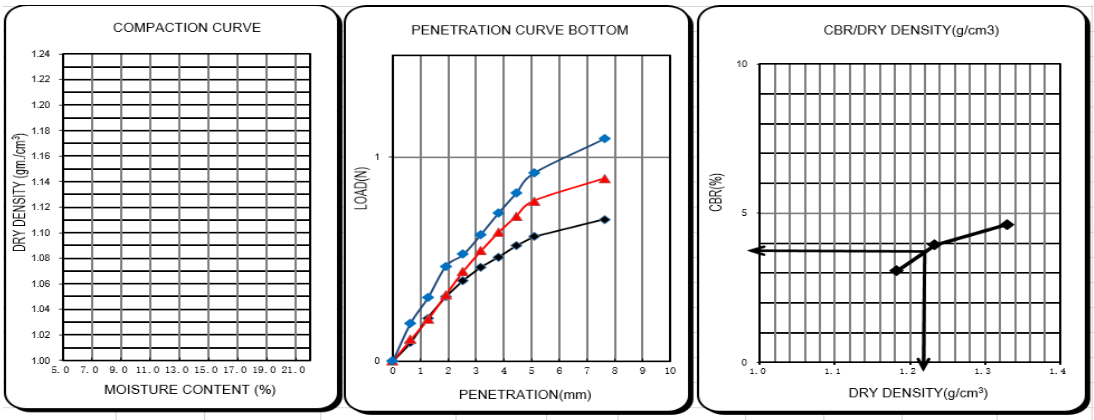
95% M.D.D =	1.159
CBR Value =	4.2%

Gradation, PI, MDD, CBR vs Proctor curves for sample from 82+00km







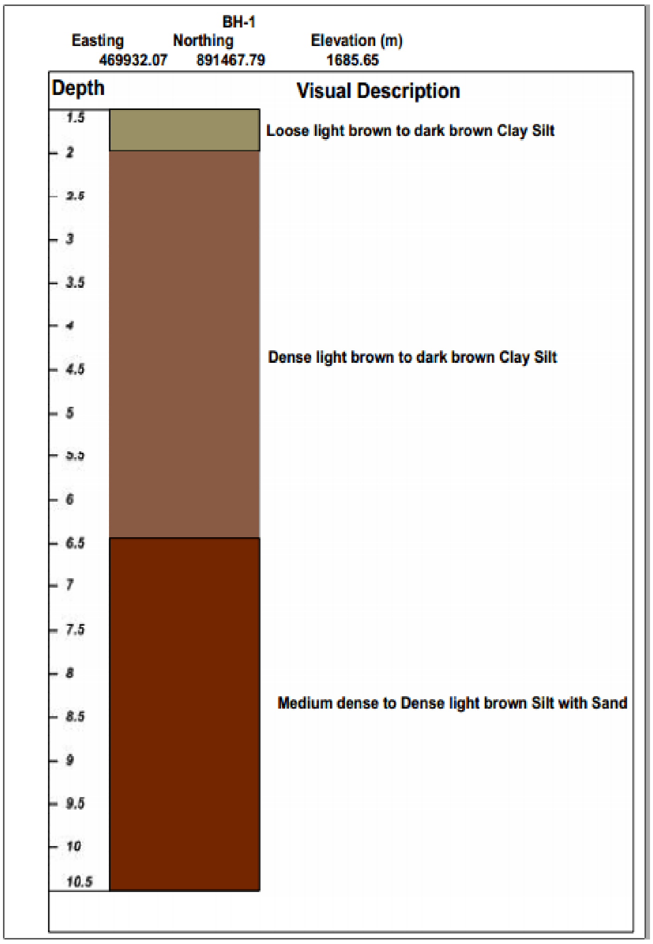


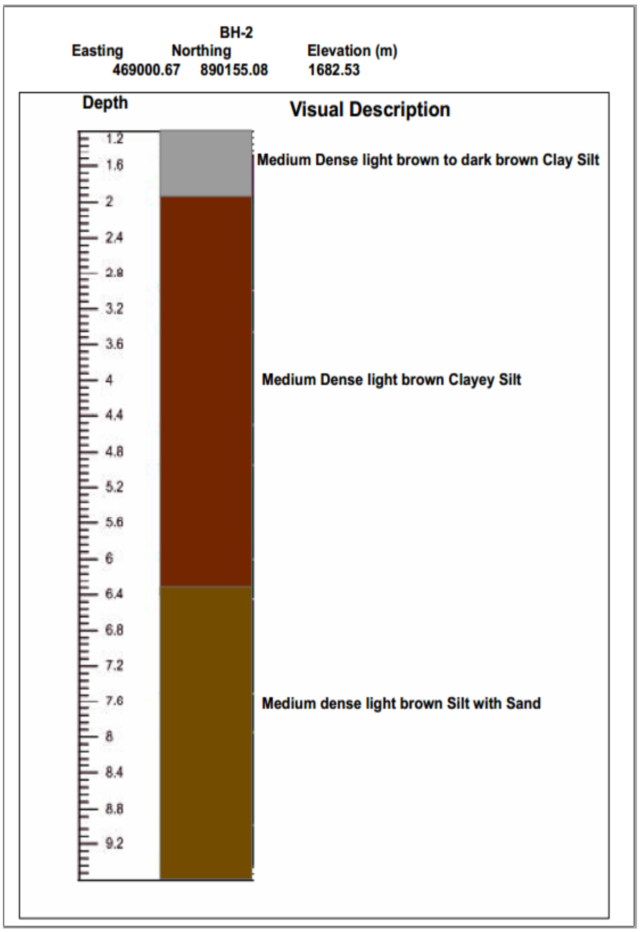
95% M.D.D =	1.207
CBR Value =	3.5%

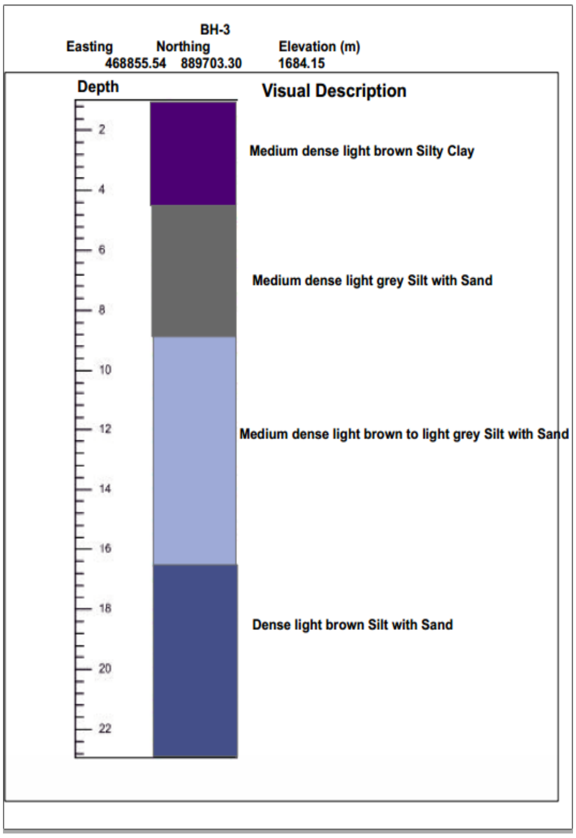
Soil profile from eight bore hole along the Meki –Ziway highway.

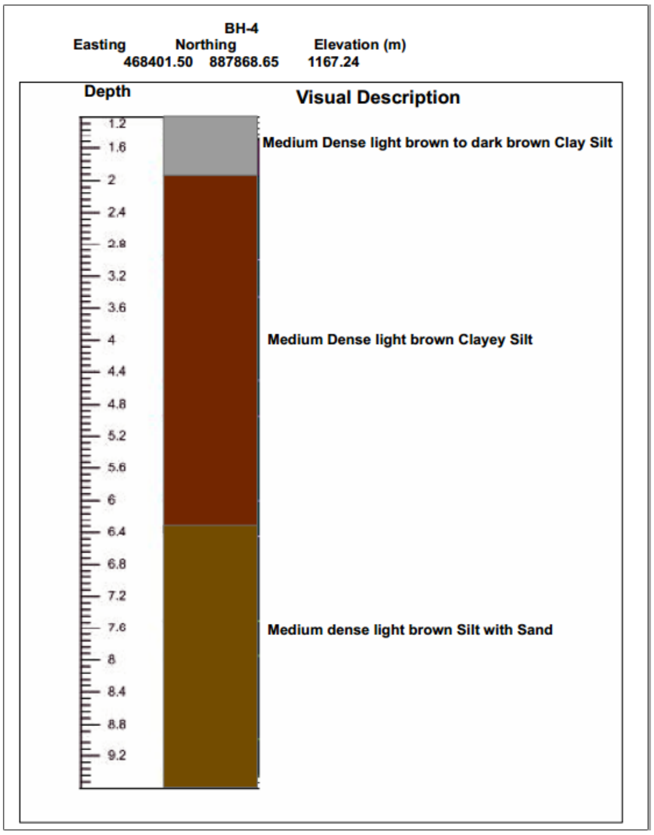
Note.

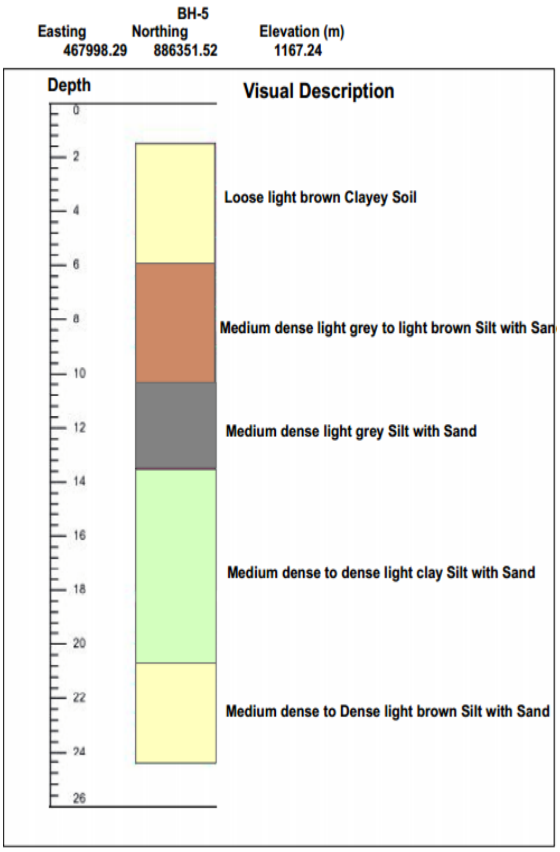
Bore hole code	Station
BH1	77+500km
BH2	79+500km
BH3	80+00km
BH4	81+00km
BH5	82+00km
BH12	74+500km
BH14	65+00km
BH16	59+500km

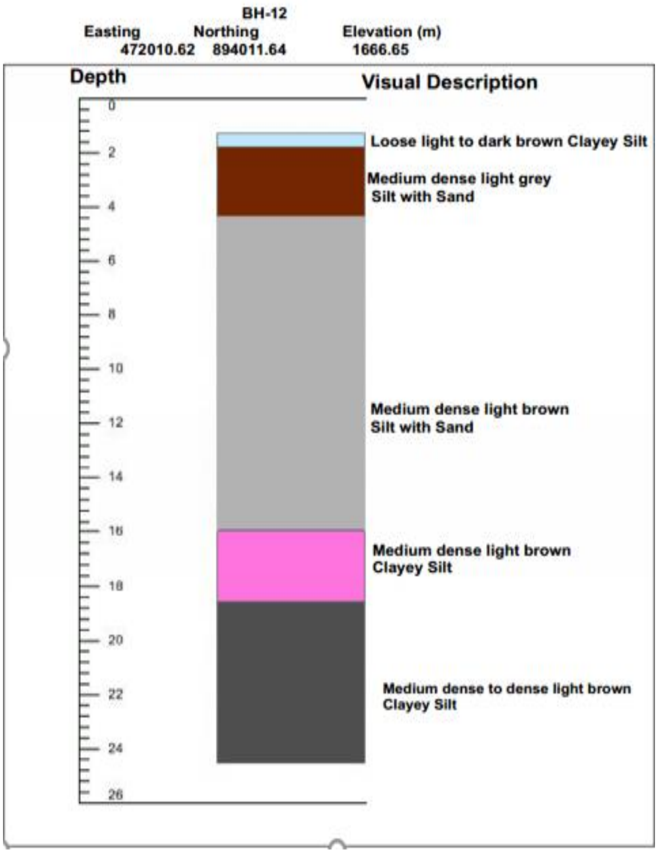


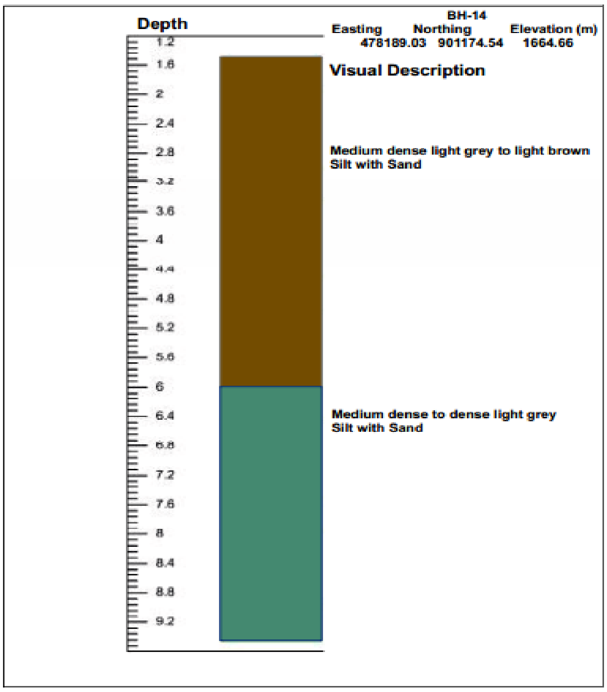


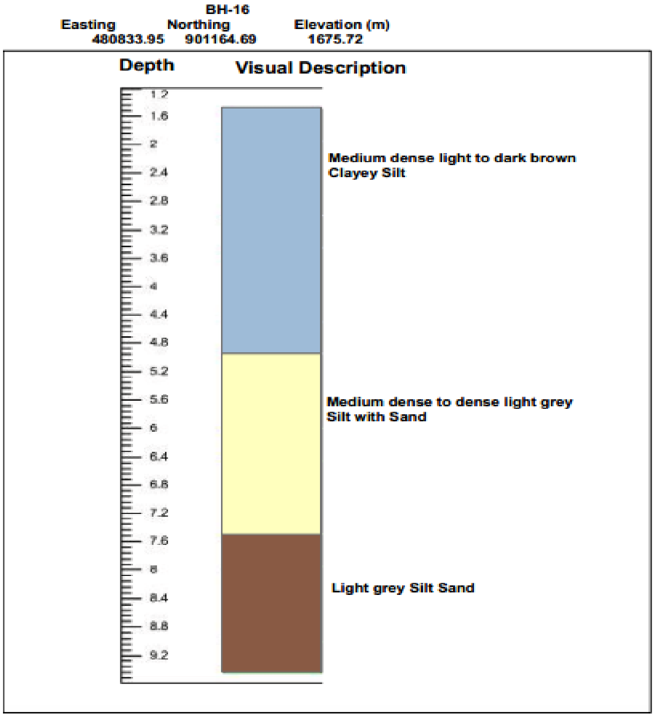

BH-3
Easting 468855.54
Northing 889703.30
Elevation (m) 1684.15
Depth
Visual Description
Medium dense light brown Silty Clay
Medium dense light grey Silt with Sand
Medium dense light brown to light grey Silt with Sand
Dense light brown Silt with Sand


BH-4
Easting 468401.50
Northing 887868.65
Elevation (m) 1167.24
Depth
Visual Description
Medium Dense light brown to dark brown Clay Silt
Medium Dense light brown Clayey Silt
Medium dense light brown Silt with Sand


BH-5
Easting 467998.29
Northing 886351.52
Elevation (m) 1167.24
Depth
Visual Description
Loose light brown Clayey Soil
Medium dense light grey to light brown Silt with San
Medium dense light grey Silt with Sand
Medium dense to dense light clay Silt with Sand
Medium dense to Dense light brown Silt with Sand


BH-12
Easting 472010.62
Northing 894011.64
Elevation (m) 1666.65
Depth
Visual Description
Loose light to dark brown Clayey Silt
Medium dense light grey Silt with Sand
Medium dense light brown Silt with Sand
Medium dense light brown Clayey Silt
Medium dense to dense light brown Clayey Silt


BH-14
Depth
Easting
Northing
Elevation (m)
478189.03
901174.54
1664.66
Visual Description
Medium dense light grey to light brown Silt with Sand
Medium dense to dense light grey Silt with Sand
1.2
1.6
2
2.4
2.8
3.2
3.6
4
4.4
4.8
5.2
5.6
6
6.4
6.8
7.2
7.6
8
8.4
8.8
9.2


BH-16
Easting
Northing
Elevation (m)
480833.95
901164.69
1675.72
Depth
Visual Description
Medium dense light to dark brown Clayey Silt
Medium dense to dense light grey Silt with Sand
Light grey Silt Sand
1.2
1.6
2
2.4
2.8
3.2
3.6
4
4.4
4.8
5.2
5.6
6
6.4
6.8
7.2
7.6
8
8.4
8.8
9.2