ENGINEERING GEOLOGICAL CHARACTERIZATION AND SUITABILITY ANALYSIS OF SUBGRADE SOIL – A CASE STUDY ON SELECTED SECTION OF AMBO –WOLISSO ROAD PROJECT, SOUTH WESTERN ETHIOPIA

Addis Ababa University

College of Natural and Computational Sciences

School of Earth Sciences

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The school of Graduate Studies of Addis Ababa University In partial fulfillment of the requirements for the Degree of Masters of Science in Engineering Geology

BY

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A THESIS SUBMITTED TO THE SCHOOL OF GRADUATE STUDIES OFADDIS ABABA UNIVERSITY IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTERS OF SCIENCE IN ENGINEERING GEOLOGY

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Abstract

The performance of a pavement is highly controlled by the underlying roadbed soil geotechnical characteristics. A subgrade provides support for upper layers of a pavement structure that are constructed upon it and the traffic load. The purpose of geotechnical characterization of a subgrade is to evaluate the quality of the subgrade based on various results of insitu and laboratory test and sound geotechnical principles that would impact the pavement performance.

In the present study the subgrade soils are characterized on selected section of Ambo-Wolliso road located in south western Ethiopia, Oromia region. The geographic location of the study area is bounded by UTM coordinate of 8^{o} 56' N 37^{o} 52' E and 8^{o}49' N 37^{o}53' E. the objective of the study were 1, to characterize the subgrade material based on the engineering geological parameters 2, to assess the suitability and bearing capacity of the subgrade soil 3, to suggest some counter measures for the unsuitable part of the subgrade soil.

To achieve this objective, a total of 16 subgrade soil samples have been taken at 1km regular interval and tests were conducted at project geotechnical laboratory for the determination of Atterberg limit, specific gravity, Natural moisture content, grading, Maximum dry density and Optimum moisture content, California bearing capacity and swell value and hydrometer test is conducted at Addis Ababa university Engineering Geological laboratory.

Finally, interpretation have been made with the data obtained from field works and laboratory investigation, supported by previous studies, research and standard manual of the subgrade characteristics. From the laboratory investigation it has been concluded that about 37.5 % of the subgrade soil is found to be unsuitable on the bases of bearing capacity. Hence, remedial measures such as removal and replacement, proper drainage design and blending with locally available materials are proposed.

Key words: Subgrade soil, Suitability Analysis, Expansive soil, Volcanic ash

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DECLARATION

I hereby declare that this thesis is my original work that has been carried out under the supervision of Dr. Zemenu Geremew, School of Earth science, Addis Ababa University during the year 2017 as part of the 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 of institution for the award of any degree or diploma and all sources of materials used for the thesis have duly acknowledged.

Biruk Mitiku

Signature ____________________________

Place and date of submission: School of Graduate Studies, Addis Ababa University

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Acronyms

AASHTO	American Association state Highways & transportation officials
ASTM	American Society of Testing and Materials
ATU	Albert Transportation and Utilities
CBR	California Bearing Ratio
DCP	Dynamic Cone Penetration
EGS	Ethiopian Geological survey
ERA	Ethiopian Road Authority
Fig	Figure
FHWA	Federal Highway Administration
GI	Group Index
LL	Liquid Limit
LS	Linear Shrinkage
MDD	Maximum Dry Density
m.a.s.l	Meter above sea level
NP	Non Plastic material
OMC	Optimum Moisture Content
PI	Plasticity Index
PL	Plastic Limit
SPT	Standard Penetration Test
TRL	Transport Research Laboratory
UCS	Unconfined Compressive Strength Test
USCS	Unified Soil Classification System
UTM	Universal Transverse Mercator
XRD	X-Ray Diffraction

Chapter one Introduction

1.0 Background of the study

The performance of a pavement is highly controlled by the underlying roadbed soil geotechnical characteristics. A subgrade provides support for upper layers of a pavement structure, which are constructed upon it and the traffic load. The purpose of geotechnical characterization of a subgrade is to evaluate the quality of the subgrade based on various results of insitu and laboratory test and sound geotechnical principles that would impact the pavement performance. As such, it is very important to have sufficient information about the variability, depth, areal extent and engineering properties of the subgrade throughout the proposed road length.

The basic properties of a soil are largely influenced by the environment and past geological activates, in which it is formed and exists (Bell, 2005).

The type and frequency of exploration required for specific road project primarily depends on the geology and environment on which the road alignment crosses. The type of road project is also important factor. In Ethiopia the most critical component in design and construction of the road is the subgrade strength, which can be obtained by a number of ways but often in Ethiopia laboratory CBR is used to determine the load support characteristics of the subgrade (ERA-b, 2013). In addition classification of a soil, moisture density relation (degree to which the soil can be compacted) expansion characteristics, susceptibility to pumping determines the general suitability of the subgrade soil (Guyer, 2009).

Many of the soil problems are usually related to specific geotechnical materials (Chen, 1999). Moisture within the pavement system nearly always has detrimental effect on pavement performance (FHWA, 2006). There are many different sources for moisture in the subgrade soil, this include, ground water, surface infiltration, soil capillarity, topography, rainfall and drainage condition. These will affect the future support rendered by the subgrade by reducing its strength and stiffness, promote contamination of coarse granular material due to fine migration and can cause swelling.

Two special conditions that often need to be checked for natural subgrade soils are the potential for swelling clays and collapsible silts (FHWA, 2006).

Swelling soils exhibit large changes in soil volume with changes in soil moisture. The potential for volumetric swell of a soil depends on the type and amount of clay, its relative density, the compaction moisture, permeability, location of the water table, presence of vegetation and trees, and overburden stress. Swell potential also depends on the mineralogical composition of fine-grained soils. Montmorillonite (smectite) exhibits a high degree of swell potential, illite has negligible to moderate swell characteristics, and kaolinite exhibits almost none. A one-dimensional swell potential test is used to estimate the percent swell and swelling pressures developed by the swelling soils.

Collapsible soils exhibit abrupt changes in strength at moisture contents approaching saturation. When dry or at low moisture content, collapsible soils give the appearance of a stable deposit. At high moisture contents, these soils collapse and undergo sudden decreases in volume. Collapsible soils are found most commonly in loess deposits, which are composed of wind blown silts. Other collapsible deposits include residual soils formed as a result of the removal of organics by decomposition or the leaching of certain minerals (calcium carbonate). In both cases, disturbed samples obtained from these deposits will be classified as silt, Loess, unlike other non-cohesive soils, will stand on almost a vertical slope until saturated. It has a low relative density, a low unit weight, and a high void ratio. A one dimensional collapse potential test is used to identify collapsible soils (ERA-b, 2013).

1.1. Problem statement

Ambo-Wolliso road project is an upgrading low level gravel surface rural road to high level asphalt surface road. The existing road is extremely rough and there are potholes, corrugations, ruts and serious water erosion problem along several sections of the project road. Therefore, for any road projects to be laid out on the ground a detailed geotechnical property of the soil needs to be carried out. It is for this reason that geotechnical properties of the subgrade material condition was analyzed and evaluated for the suitability of the existing material on selected section of Ambo-Wolliso has satisfied the standard specification set by Ethiopian

Road Authority (ERA) and project standard specification for subgrade. On the bases of the laboratory results, potentially problematic soils were identified and some remedial measures were suggested to overcoming the problems.

1.2 Research objective

1.2.1 General objective

The general objective of this research was to characterize the subgrade soil on selected section of the Ambo- Wolisso road and to evaluate its suitability for use as a road foundation.

1.2.2 Specific objectives

- To characterize the subgrade material based on engineering geological parameters.

- To assess the suitability and bearing capacity of the sub-grade soil

- To suggest some countermeasures for unsuitable parts of the sub-grade soil

1.3 Methodology

In order to achieve the objectives of this research work, the following methodology was adopted.

1.3.1 Desk study

- Identify sources of information, reviewing and collection of available information like standard manuals, papers, books, maps (geological and topographical) relating to climate, geology, Physiography of the research area.

1.3.2 Field work

- Field visit to over view the geology of the study area and describing the soil and rock condition.
- Collection of representative soil samples from the road carriage way at subgrade level at 1km interval.

1.3.3 Post Field work

- Conducting soil laboratory tests such as
    - Atterberg limits (liquid limit and plastic limit)
    - Grain size analysis(sieve analysis and hydrometer test for fine grained soils)
    - Modified proctor test
    - Specific Gravity
    - California Bearing Ratio (CBR)
    - CBR swell
    - Natural Moisture Content (NMC)
- Classification of the subgrade soil material based on laboratory test result by using AASHTO M145 standard.
- Characterization the subgrade soil by comparing and relating different parameters of engineering properties of soil.
- Interpretation of the result of the test and the index properties, bearing capacity and swelling potential of the subgrade soil along the road alignment.
- Data processing and presentation were done using software like MS Excel

1.4 Outcome of the Research

- Characterized the subgrade soils from km 5 to 20 of Ambo-Wolisso road project
- Suitable and unsuitable subgrade soil from the study area are identified based on the ERA and project standards
- Expansive soil and silty soils were the recognized problems in the study road section
- Removal and replacement and blending with clay material are recommended for identified problems

1.5 Scope and limitation of the Research

The research was performed in scientific principle and logical manner. The study has been supported by different types of literature, ERA manual and standards and laboratory tests. However, the findings of the research are limited to selected section of the road project and the type and number of tests conducted is limited based on available resources. Therefore, the recommendation given should be considered for the study section only.

1.6 Scheme of presentation

The presentation of this thesis work is organized in six chapters. The first chapter gives brief description of the research background, objectives, scope and methodology employed. Chapter two presents conceptual background related to subgrade soil characteristics, parameters to be considered and important detail from previous studies. Chapter three describes briefly the study area in terms of its geological set up, topography, and climate condition. Chapter four describes the laboratory test procedures followed and the results obtained. The fifth chapter presents the analysis and discussion of results with respect to ERA standards and previous studies. Finally, conclusions and recommendations drawn from the research are presented in chapter six.

Chapter Two **Literature Review**

2.1 Introduction

The geotechnical components of a pavement system includes, asphalt surface, unbound granular base, unbound granular sub base and the subgrade or Road bed. Geotechnical characteristics of a subgrade soils have major influence on the performance of a pavement. Subgrade provides platform for construction of pavement structure and its purpose is to support the pavement structure without deformation that would impact the pavement performance (FHWA, 2006).

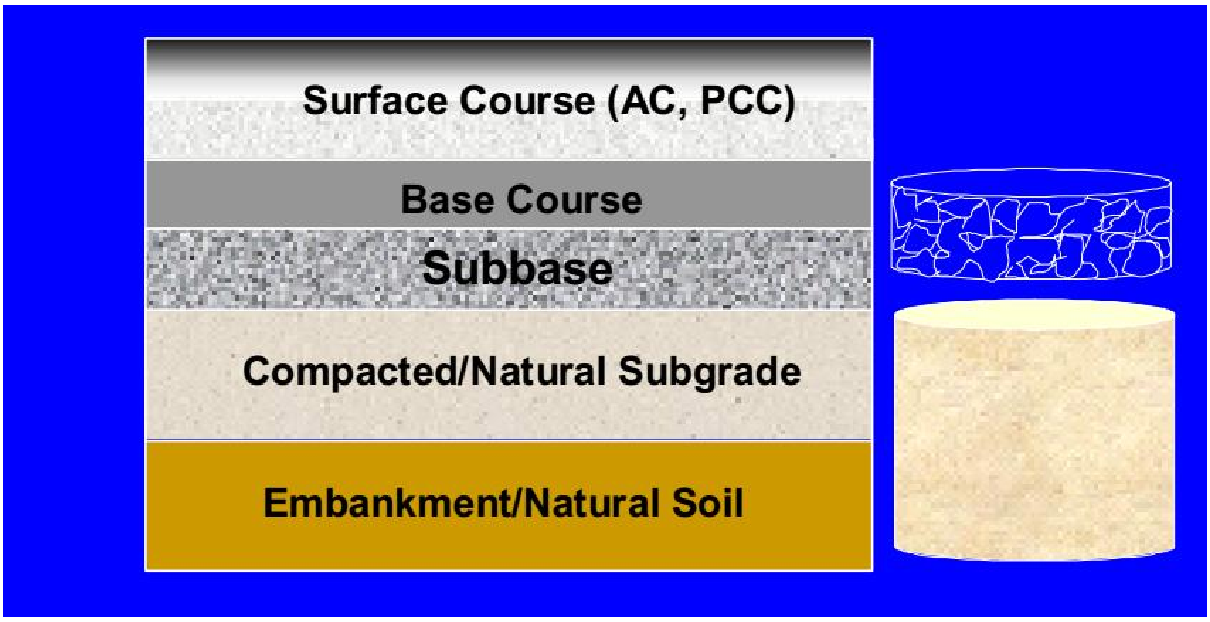

Fig: 2.1 Basic components of a typical pavement system (source: FHWA, 2006).

The main objectives of a ground investigation are generally to identify and classify the soil types into groups of materials that exhibit broadly similar engineering behavior and to determine the parameters that are required for engineering design calculations (Bell, 2005). The property of a soil frequently is influenced to a significant extent by climate regime in which it is formed and exists.

Construction works of Ambo - Wolliso road (2013), it was explained in the contract document that unsuitable material has the following physical properties:

I. Peat material from swamps, marshes and bogs that contain excessive amount of logs, trees, stumps and other material

II. Clay material having a liquid limit exceeding 60%, or plasticity index exceeding 30% or CBR value less than 3% or swell value more than 2%.

III. Any material which is sufficiently wet and soft to prevent it from being trafficked or excavated by normal bulk earth moving material.

The material to be used for replacement must have the following property

I. The CBR should be greater than 7% at 95% of modified AASHTO (T-180)

II. PI not greater than 20

III. Swell should be less than 1.5% when tested in accordance with AASHTO (T-180) and 2.5kg surcharge.

The chemical and physical properties of material are determined by carrying out different tests on samples of a soil in a laboratory. Tests such as moisture content, CBR, Grain size analysis, density, Atterberg limit etc, uses to assess the engineering properties of a soil.

According to SABA Engineering PLC (2002), parameters determined from laboratory tests are used for the following construction application.

- The finding of a site investigation can be supplemented by further testing as constructions proceed.
- Criteria for the acceptance of a material used in construction.
- Data acquired from classification tests are applied to the identification of soils of soil strata.
- Laboratory test are needed as part of the control measures which are applied during construction of earth works on for ensuring that the design criteria are met.

Further, the laboratory tests used for the evaluation of soil properties from reliable test procedures has led to a closer understanding the natural and probable behavior of soils as engineering material. Some of the resulting advantages in the realm of civil engineering construction have been discussed as follows (SABA Engineering PLC, 2002):

- Increase economy in the use of soil as construction materials

- Reduction of uncertainties in the analysis of foundation and earth works
- Exploitation of difficult sites
- Economy in the design due to the use of lower factor of safety
- Erection of structures , and below ground construction, which would not have been feasible without this knowledge

2.2 Expansive soils

The aerial coverage of expansive soils in Ethiopia is estimated to be 10 million hectare (Lyon associates, 1971; as cited by Daniel Nebro, 2002). They are widely spread in central part of Ethiopia following the major truck roads like Addis Ababa-Ambo, Addis Ababa-Wolliso, Addis Ababa– Debrebirhan, Addis Ababa-Gohatsion, Addis Ababa-Modjo are covered by expansive soils. Also areas like Mekele and Gambella are covered by expansive soils.

The constituents of the parent material during the early and intermediate stages of the weathering process determine the type of clay formed. The nature of the parent material is much more important during these stages than after intense weathering for long periods of time (Chen, 1988).

The parent materials that can be associated with expansive soils are classified into two groups. The first group comprises the basic igneous rocks and the second group comprises the sedimentary rocks that contain montimorillonite as a constituent. The basic igneous rocks are comparatively low in silica, generally about 45 to 52%. Rocks that are rich in metallic base such as the pyroxenes, amphiboles, biotite and olivine fall within this category. Such rocks include the Gabbros, Basalts and Volcanic Glasses (Chen, 1988).

The sedimentary rocks that contain montimorillonite as constituent include shale and clay stones. Limestone and marls rich in magnesium can also weather to clay (Chen, 1988).

The weathering process by which clay is formed includes physical, biological and chemical process. The most important weathering process responsible for the formation of montmorillonite is the chemical weathering of parent rock mineral. The parent material generally consists of ferromagnesium mineral, calcic feldspars, volcanic glass, volcanic rocks and volcanic ash. The formation is aided in alkaline environment, presence of magnesium ion and lack of leaching. Such condition is favorable in semi-arid regions with relatively low rain fall or seasonal moderate rainfall particularly where evaporation exceeds precipitation. Under these conditions enough water is available for the alteration process but the accumulated cations will not be removed by rain water (Chen, 1988).

Clay mineralogy is a fundamental factor controlling expansive soil behavior. Clay minerals can be identified using a variety of techniques. The techniques that can be used are (Chen, 1988; Nelson and Miller, 1992):

- X-ray diffraction
- Differential thermal analysis
- Dye adsorption
- Chemical analysis
- Electron microscope resolution

But these methods are not suitable for routine tests because of the following reason;

- They are time consuming;
- They require expensive test equipment; and
- The results can only be interpreted by specially trained technicians.

The swelling and shrinkage of clay soils is determined by the type of clay mineral present in the soil. Arora (2004) discussed the following points regarding different types of clay minerals and their basic structural units,

- Kaolinite mineral: the basic structural unit consists of aluminum sheets combined with the silica sheet and the structural units join together by hydrogen bond, which develops between the oxygen of silica sheet and the hydroxyls of aluminum sheet. As the bond is fairly strong, the mineral is stable. Therefore, water cannot easily enter in between the structural units and cause expansion.
- Holloysite mineral: is a clay mineral which has the same basic structural units as kaolinite, but in which the successive structural units are more randomly packed and are separated by single molecular layer of water. The

property of hollysite depends upon this water layer. If the water layer is removed by drying, the properties of the mineral drastically change.

- Montmorillonite mineral: this clay mineral has the basic structural unit consists of aluminum sheets sandwiched between two silica sheet and the two structural units are joined together by a link between oxygen of the silica sheets. The link is due to natural attraction for cations in the intervening space and due to vander waal forces. The negatively charged silica sheets attract water in the space between two structural units. It may also cause dissociation of the mineral into individual structural units of thickness. For this reason, the soil containing a large amount of the mineral montmorillonite exhibits high shrinkage and high swelling characteristics.
- Illite mineral: the basic structural unit is similar to that of the mineral montmorillonite. However, the mineral has properties different from montomorillonite due to the following reasons
    1. There is always a substantial amount of the isomorphous substitution of silicon by aluminum in silica sheet. Consequently, the mineral has a large negative charge than that in montomorillonite.
    2. The link between different structural units more firmly than in montorillonite.
    3. The lattice of illite stronger than that of montmorillonite, and is, therefore, less susceptible to cleavage
    4. Illite swells less than montmorillonite. However, swelling is more than in kaolinite
    5. The space between different structural units is much smaller than in montmorillonite, as the potassium ions just fit in between the silica sheet surfaces

Bowles (1992) discussed about the types of clay soils, such as montmorillonite, illites, kaolinites and hayllosites for the range of their liquid limit (LL) and Plasticity index (PI) values. He further discussed concerning these clay minerals that montmorillonites have plasticity index of 150 and above, illites have PI values in the range of 30 to 50% and the kaolinites minerals have values of plasticity index in range of 15 % to 20%.

Anon (1981), suggested that the plasticity index provide an indication of volume change potential, as shown in the Table 2.1. A degree of overlap was allowed Anon (1981), because determination of plasticity is carried out on remolded soil, these grades of volume change potential do not consider the influence of soil texture, moisture content, soil suction, pore water chemistry or stress history.

Table: 2.1 Relation between the swelling potential of clays and the plasticity index (Anon, 1981)

Swelling potential	Plasticity index
Low	0-15
Medium	10-35
High	20-55
Very High	35 & above

In addition, Daksanamurthy and Raman (1973) presented a single index method for identifying expansive soils using only liquid limit. They suggested four classes of clays according to their liquid limits as shown in Table 2.2 (Amer and Mattheus, 2006).

Table: 2.2 Relation between the swelling potential of clays and the liquid limit (Amer and Mattheus, 2006)

Swelling potential	Liquid limit
Low	$20<LL\leq 35$
Medium	$35<LL\leq 50$
High	$50<LL\leq 70$
Very High	$LL>70$

2.3 Pyroclastic Materials

The general geotechnical characteristic regarding pyroclastic materials is discussed in detail by Bell (2005) as follow:

- Pyroclastics usually give rise to extremely variable ground conditions owing to wide variations in their strength, durability, and permeability.

Ashes tend to be weak and are often highly permeable. Those that are metastable are likely to undergo hydrocompaction on saturation. Moreover, ashes deposited on slopes are frequently prone to sliding, yet the irregular shapes of their constituent particles, which can therefore interlock, may enable very steep slopes to be excavated that can stand up, at least in the short term.

- The strength and behaviour of tuffs depend on their degree of indurations. However, the durability of some basaltic tuffs is poor or very poor, and they may be susceptible to frost.

- An ignimbrite is a pyroclastic rock consisting predominantly of pumiceous material that shows evidence of having been formed from a hot and concentrated pyroclastic flow. Once deposited, induration may be brought about by welding of viscous glassy fragments, by devitrification of glassy material, by deposition of material from escaping gases, and by compaction. Accordingly, ignimbrites have a wide variety of geotechnical characteristics, which are attributable to their modes of eruption, transportation, and deposition. At one extreme they are weak materials that behave as soils in the engineering sense; at the other extreme they are strong hard rocks in which extensive sets of essentially vertical cooling joints are developed. In fact, non-durable, intermediate, and highly durable ignimbrites have been recognized. The non-durable ignimbrites are characterized by low densities, high porosities, and low unconfined compressive strengths (5 MPa or less).

2.4 Overview of AASHTO design manual

According to AASHTO pavement design manual (1993), the basis for material characterization is elastic or resilient modulus. For roadbed materials, laboratory resilient modules test (AASHTO T 274) should be performed on representative samples in stress and moisture condition simulating those of the field condition.

Further, on the same manual environmental effects discussed are; seasonal temperature and moisture changes that can affect the strength, durability and load carrying capacity of the subgrade and this affect the pavement performance.

In addition, AASHTO (1993) suggests determination of Homogenous sections using the CBR at 95% of the MDD (Maximum dry density) and analysis of Unit delineation by cumulative differences. Homogeneous section delineation by cumulative differences is computed using the following parameters (AASHTO, 1993 appendix J)

- Average CBR value
- Sampling interval
- Total project length
- Total number of samples taken in project

The revised AASHTO manual in (2000), indicates the following points as the guideline for subgrade characterization

- The spacing of the samples collection for the material characterization should be at the range of 150m to 450m during construction phase in consideration the geological variability
- The strength in terms of CBR value and swelling and shrinkage potential of the swelling soil subgrade material should be determined.
- Optimum moisture content and maximum dry density of the material should be determined before the design

AASHTO design manual (2004), introduce soil classification for subgrade characterization and accordingly, the AASHTO soils classification includes seven basic groups (A-1 to A-7) and twelve subgroups. Organic soils are classified as A-8. Of particular interest is the Group Index, which is used as a general guide to the load bearing ability of a soil. The group index is a function of the liquid limit, the plasticity index and the amount of material passing the 0.075mm sieve. Under average conditions of good drainage and thorough compaction, the supporting value of a material may be assumed as an inverse ratio to its group index, i.e. a group index of '0' indicates a "good" sub-grade material and a group index of '20' or more indicates a poor sub-grade material.

Using the AASHTO M 145 soil classification group index is calculated by the following formula:

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

Where F= percent of fine passing sieve size 0.075mm

LL = Liquid Limit

PI = Plastic Index

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)

Rules related to interpreting the GI are:

- If the equation yields a negative value, GI is taken as zero.
- The group index calculated from the equation is rounded off to the nearest whole number.
- The group index of a soils belonging to groups A-1-a, A-1-b, A-2-4 and A-3 will always be zero.

2.5 Overview of ERA Pavement Manual

According to ERA-b (2013), environmental information on the climate, topography, geological aspect of the project site is a basic requirement to understanding engineering characteristics of the area and is pre-request for the planning, design and construction of a road.

The existence of diverse topography, geology, and climate condition in Ethiopia often create different geotechnical problem specific to a certain region. The geotechnical problems range from

- Land slide in rugged mountains

- Problem soils in flat area

ERA-b (2013) discuss the following points with regard to Land slide;

- Landslide occurs when shear stress (driving force) increase and redaction in shear strength (resisting force).

- Geology and Topography parameters are regard as landslide causes. Hence, topography is responsible to cause landslide due to:

- The steepness and shape of the slope
- The location of tension cracks and sign of movements and

The geology is responsible to cause landslide by one or combination of the following factors:

- The rock type, weathering grade, jointing and fracture pattern
- Presence of faults and shear force
- The direction and angle of dip and joints in underlying bedrock compared to the angle and orientation of slopes, particularly if bed rock is exposed or is at shallow depth beneath the surface. The extent of the joints and the presence of clay filling also has the an influence
- The sequence of underlying strata, particularly if this include weak or impermeable layers
- Presence of colluviums and unconsolidated material

Rainfall, ground water, earth quake and manmade activity like road cuts and embankment are triggering factors. These factors are responsible to increase pore pressure, increase the weight of the material on slope above the point of gravitational equilibrium, increase pressure within a zone of weakness in material underlying a slope or decrease the coefficient of friction on particular sliding surface and road cuts can increase in the height and steepness of the slope or if embankment placed on slopes can trigger land slide.

According to ERA-b (2013) the following soil types are problematic;

I. Expansive soil; typically clay soil that undergo large volume change to moisture change in the soil.

II. Collapsible soils; soils that undergo a significant, sudden and irrecoverable decrease in volume upon wetting. This type of soil dominantly contains salts, and sand with some clayey material and rock fragments. They are usually associated with area of moisture deficiency, such as those in arid and semi- arid region.

The first indication of the possibility of collapsible materials is a very low density and moisture content, because of the large number of voids. Another simple indicator test for collapsible materials is to excavate a pit in the material and then replace the material. If the material is collapsible, the soil structure will be disturbed and the replaced material will be insufficient to fill the pit (Brink, 1979).

The result of collapse of the subgrade 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. It is seldom associated with significant cracking unless the bituminous surfacing is very stiff.

By its nature, a collapsible soil needs to be wetted up and heavily compacted in order to disrupt the collapsible fabric. Conventional compaction plant has been shown to be only moderately effective in removing the collapse potential to any significant depth, even after the addition of compaction water. However, modern high energy impact compaction techniques using large impact rollers (25 kJ), with or without the addition of water, have proved most effective in reducing the collapse potential to a significant depth (Paige, 2008).

III. Dispersive soils; soils deflocculates in the presence of relatively pure water to form colloidal suspensions and therefore highly susceptible to erosion and piping. Normally, they contain a higher content of sodium in the pore water than other soils. However, there is no significant difference in the clay content of dispersive and non-dispersive soils. Although soils with high exchangeable sodium such as; Na montmorillonite clays tend to be more dispersive than the others.

Paige (2008), discusses the following countermeasures for avoiding dispersive soil damage in the road environment. Avoid its use in fills as far as possible and remove and replace it in the subgrade. It is important to manage water flows and drainage in the area well. As the presence of sodium as an exchange cation in the clays is the major problem, treatment with lime or gypsum will allow the calcium ions to replace the sodium and reduce the problem. It is also important that the material is compacted at 2 to 3% above optimum moisture content to as high a density as possible (Donaldson, 1975; Elges, 1985).

IV. Colluvial soils; all soils which have been transported by gravity forces and deposited in valleys, swales or other low laying topographic features, often with the aid of water flows. They include slope – wash deposits, scree (tales), and landslide debris. The soils are usually characterized by being a mixture of particles of contrasting sizes from highly plastic clay to boulders. Colluvial soils are likely highly permeable and compressible

According to ERA Geometric design manual (2013), terrain class determined by counting the number of 5 meter contours crossed by a straight line connecting the two ends of the road section according to the following definitions:

Flat: 0-10 five meters contours per km. The transverse ground slopes perpendicular to the ground contours are generally bellow 3%.

Rolling: 11-25 five meter contours per km. The transverse ground slopes perpendicular to the ground contours are generally between 3% and 25%.

Mountainous: 26-50 five meter contours per km. The transverse ground slopes perpendicular to the ground contours are generally above 25%.

Escarpment: Escarpments are geological features that require special geometric standards because of the engineering problems involved. They are characterized by more than 50 five meter contours per km and the transverse ground slopes perpendicular to the ground contours are generally greater than 50%.

Soils can be classified on laboratory based process and soils with similar engineering characteristics can uniformly be grouped (ERA, 2013). AASHTO classification system (M145) is used in most cases of site investigation in Ethiopia (ERA-b, 2013). AASHTO classification is useful to determine the relative quality of the soil material for use in earth work structure, particularly embankment sub-grades, sub-bases and bases (ERA-b, 2013).

According to ERA-a (2013) pavement design manual, subgrade strength is assessed in terms of California Bearing Ratio (CBR) and this depends on the

I. Type of soil,

II. Its density and

III. Its moisture content.

Further ERA-b (2013) manual define, three classifications of sub-grade conditions to estimate the sub-grade strength for design. These conditions are;

Category 1: The water table is high enough during the rainy season to govern the moisture content.

The design strength must then be based on this assumption by testing samples compacted to the target density at optimum moisture content but then tested after a period of soaking.

Category 2: The water table is deep, but rainfall can influence the subgrade moisture content under the road.

These conditions occur when rainfall exceeds evaporation and transpiration for at least two months of the year

Category 3: Deep water table and arid climate.

For soil in this category it is recommended that for design purposes a value of 80% of the optimum moisture content obtained by ASTM Test Method D 698, reflecting the probability that the subgrade will lose some moisture and gain strength after construction.

According to ERA–a (2013), Flexible pavement design manual, six strength classes assigned as shown in the table 2.3 for subgrade based on its CBR value.

Table 2.3 subgrade strength class according to ERA-a, 2013

Class	CBR range (%)
S1	<3
S2	3,4
S3	5,6,7
S4	8-14
S5	15-30
S6	>30

In addition, the manual discusses, for a road section for which a pavement design is undertaken should be subdivided into subgrade area where subgrade CBR expected to be uniform without significant variation. For natural roadbed soil with CBR’s less than 2% special treatments are required.

Further, ERA technical specification manual (2013), discussed regarding the material minimum requirements for construction of a subgrade soils as follows:

- A soaked CBR at 95% MDD of not less than 5% and swell value at 100%MDD not more than 2% (with two surcharge rings). The MDD must determine in accordance with the requirement of AASHTO T 180 method D
- Liquid limit not exceeding 60% and a plasticity index not exceeding 30% and should determine in accordance with the requirement of AASHTO T 89 and T 90

The ERA pavement design manual, (2013) also explains regarding problems associated with construction over expansive soils due to seasonal changes. Seasonal wetting cause soils at the edge of the pavements to wet and dry out at rates differing from further under the bituminous surfacing. This mechanism causes deferential movement over the road way cross section and associated crack will development, beginning at the shoulder towards the carriageway.

Accordingly, the manual recommend economical mitigation measures as follows:

- Excavate and replacement of expansive soil by non expansive soil
- Design earthwork to control subsequent moisture changes and consequent volume changes through:
    - Sealing of a shoulders
    - Replacement of the upper layer of expansive soil
    - Provision of a minimum cover (cupping layer)
- Keeping expansive soils moist during construction and cover with earthworks prior to any drying
- Compacting the expansive soil with normal density requirement only and should not be above the limit
- Using fill material over the expansive soil which are impermeable soils with a PI of greater than 15%.
- Avoiding side drains in area expansive soil and where this is not possible, they should be pleased away from the toy of the side slopes.

2.6 Overview of Kenyan and Tanzanian pavement design manual

According to Kenyan Road Design Manual (1989), a subgrade material should fulfill the following criteria

- CBR at 100% MDD (standard proctor) and 4 days soaking should be greater than 5%

- Swelling at 100% MDD (standard proctor) and 4 days soaking should be less than 2%

- Organic content (percentage by weight) should be less than 3%.

The manual strongly recommend the incorporation of improved subgrade material rather than stabilization and removal of unsuitable material, because improved subgrade material can

- Increase the overall bearing strength of the subgrade and reduce in the thickness of the sub-base materials.
- Protect the upper layers from adverse weather condition (water soaking)
- Reduce the strength variation of subgrade material and facilitate the proper compaction of pavement layers.

In the manual improvement refers soil modification by using locally available suitable materials. That can be on textural changes that result measurable strength changes within short term. However, stabilization refers when a significant long term reactions occur due to pozzelans reaction. Stabilizing materials can be industrial by products such as; Cement, Lime and flay ash or naturally occurring pozzelans.

According to Tanzanian pavement design manual (1999), subgrade investigation should be planed and conducted in a manner that classifying all material according to their suitability in load bearing layers within the design depth and the investigation shall be extended to bellow design depth as required to determine problems that need special consideration. This includes:

- Presence of problem soils
- Unfavorable subgrade condition
- Features associated with slope and embankment stability

In addition to this, Atterberg limit, density/ moisture relation, grain size analysis and hydrometer test and CBR test shall be carried out to characterize the subgrade soils and the subgrade materials are classified according to its strength into 3 strength classes under different condition as shown in table 2.4.

The manual further states that, the problematic soil in the subgrade or unfavorable material along the design road alignment should undergo improvement as; all subgrade shall be brought to design strength of a minimum CBR of 15% by constructing one or more improved subgrade layers. After the improvement the subgrade material should be re- categorized into one of the three strength classes.

According to the manual, the uses of improved subgrade layers have the following advantages:

- Protection of earthworks below
- Provision of running surface for the traffic during construction
- Improve compaction of pavement layers above
- Provision of homogeneous subgrade strength
- The improved subgrade acts as a filter layer between pavement layers and poorer soils below
- Economic use of local material

Table 2.4 Tanzanian subgrade strength class

Subgrade class	CBR (%)			Density for determination of CBR_{Design} [% MDD]
	Wet or moderate climate zones (4 days soaked value)	Dry climate zones (both requirement shall be meet)		
		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- Light
CBR <3% (CBR <2% in dry climate zone) require special treatment				

Additionally, the manual state that, Material to be use for subgrade improvement could be a suitable soil or dump rock. Soils used in improved subgrade layers shall be non expansive, non dispersible and free from any deleterious matter and Dump rocks can be used as improved subgrade when construction in sufficiently thick layers. Construction of improved subgrade made of dump rock and shall be finished of by filling in the voids in surface with subgrade soils meeting the requirements.

2.7 Previous works

HYD GEOTECHNICAL AND ENGINEERING SERVICE PLC, (2016). The company has conducted subgrade investigation from km 20 to 31 in the mountainous section of the Ambo – Wolliso road project. The subgrade investigation mainly includes;

- drilling four boreholes to the depth of 14m to 10m below the ground level
- 15 in situ SPT field testes
- 16 soils samples (12 disturbed and 4 undisturbed)
- Laboratory testing of classification tests, specific gravity, moisture content, unit weight(dry/wet) and unconfined compressive strength tests

From the field and laboratory investigation the following results and conclusions are drawn in the report

- The study subgrade soils are characterized mainly by non plastic inorganic silts (ML) and silty sand (SM) based on USCS and A-4 (silty soil) based on AASHTO classification. It has also found A-2-4 (fine sandy soil), A-7-5 and A-7-6 (clay soils) during investigation.
- On basis of SPT values, strength parameters were defined and most of the soils are found very stiff and medium dense. Besides, from the SPT value the bearing stress of the soils were calculated considering the depth of measurement and variable width of the ground and bearing capacity ranging from 139kPa to 499kPa is found
- Finally the unconfined compressive tests (UCS) results of the undisturbed samples exhibits higher value of 421kPa for A-2-4 samples; while the A-4 gives a value ranges from 281kPa to 328kPa.

Muler Kennedy (2007), “IMPROVEMENT OF RED COFFEE SOIL USING VOLCANIC TUFFS FROM NGURUNGA, KENYA”. He characterize the natural red coffee soil of fair to poor engineering property and volcanic ash was mixed the same red coffee soil samples by using different laboratory tests. Finally, he compared the pure material subgrade properties with the subgrade volcanic ash mixtures and he obtained an improvement on subgrade material at 12% volcanic ash content as, decreasing the plasticity index and linear shrinkage and increasing CBR value. The obtained values met the requirements to be used as sub-grade materials as per Road Note 31, Road Note 21 and the Kenya Road Design manual.

Misgana Oljira (2014) “Engineering Geological characterization and suitability Analysis of subgrade materials- A Case Study of Sembo-Shola Gebeya- Gindeber Road, central Ethiopia”. He characterizes subgrade soils in the study area based on laboratory investigations, supported by standard manuals. He has been concluded that about 41% on plasticity, 62% on CBR, 64% on CBR swelling, 46% on group index of the subgrade soil in his study area are found to be unsuitable and all are highly comprisable. Finally, remedial measures such as removal and replacement, mechanical compaction, proper drainage, chemical treatment, blending with existing soils and proper pavement thickness design are proposed on his thesis.

Nibret Chane (2011) “Geotechnical characteristics of subgrade material on Aposto – Wondo – Negele road” evaluate the geotechnical characteristics of the subgrade soils of the study area on the basis of laboratory Atterberg limit, Grading, MDD, OMC, CBR and CBR swelling testes. From the laboratory investigation, he has concluded that about 82% of the subgrade soils are suitable for bearing stratum and construction material. The recommendations he has drawn for unsuitable section is:

- Mechanical stabilization
- Removal and replacement
- In situ treatment
- Rock fill with Geo-textile and underground drain

Thus for the present study, the various literatures have been utilized to provide the basis for characterizing the subgrade soils of the study area. Such characterization has aided in the evaluation of the study area and the remedial measures that have been taken.

Chapter Three THE STUDY AREA

3.1 Location and Accessibility

Ambo-Wolisso road Upgrading project is located in western part of Ethiopia in the Oromia Regional state. The Ambo-Wolliso road is a link that connects Addis Ababa Nekemte and Addis Ababa-Jimma Trunk Roads. As the Project name indicates, the Road connects the towns of Ambo and Wolliso-the capitals of West Shewa and South West Shewa Zones of Oromia Regional State, respectively. The total length of the Road section under present study is 15km and passes on elevations ranging from 2278m to 3132m above mean sea level. The geographic positions of the road section under study falls between 8° 56' N 37° 52' E and 8° 49' N 37° 53' E (Figure: 3.1).

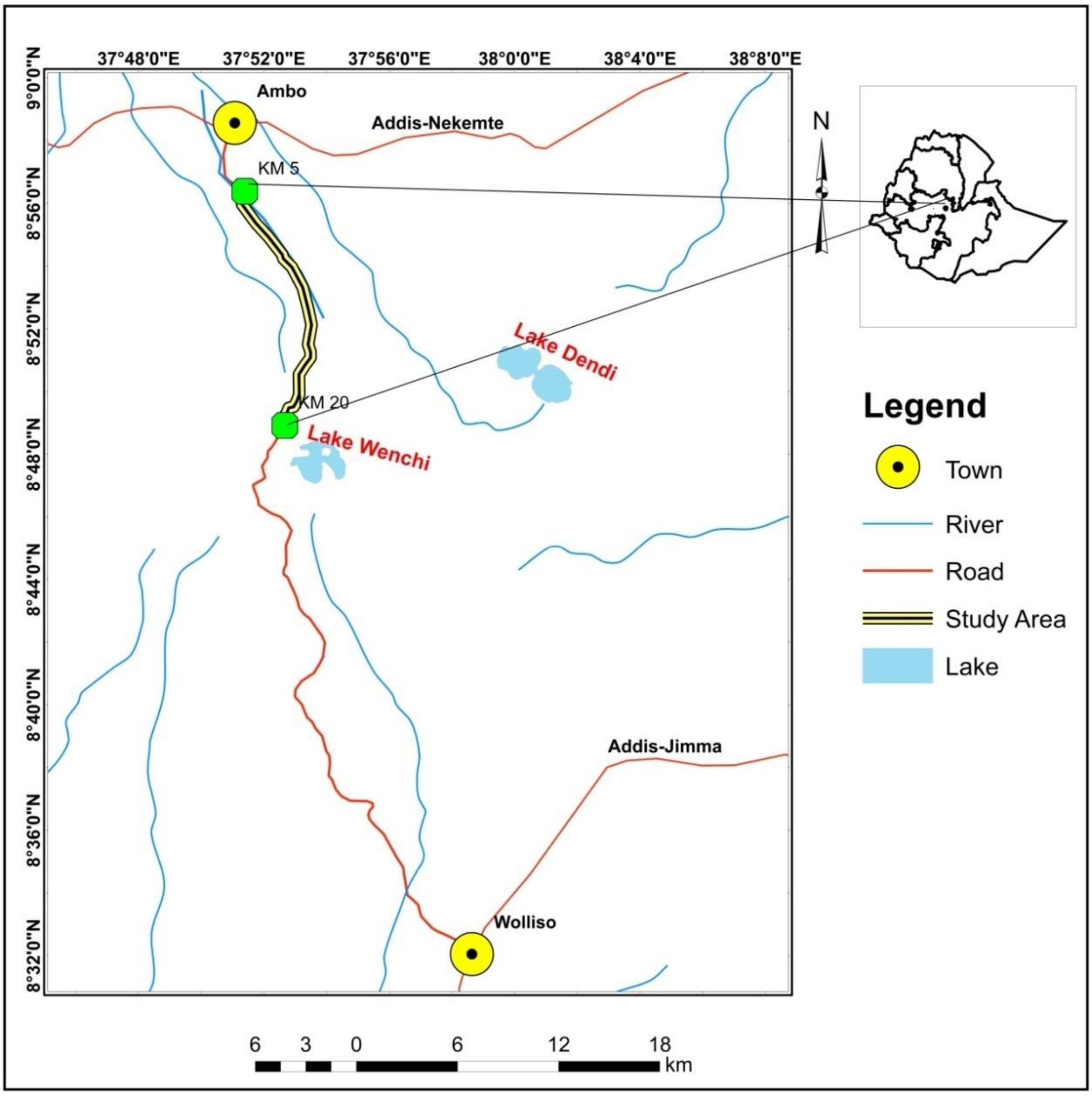

Fig: 3.1 Location map of the study area

3.2 Land use/Land Cover

On the basis of observations and data collected during the field visit, the land use and land cover patterns along the project alignment is summarized as follow; approximately 80% (12km) of the project road traverses farmed areas whereas 6.5%(1km) passes through villages, and 13.5% (2 km) goes through areas dominantly covered by bush lands or woodlands.

According to environmental and social impact assessment report of the project (2012), in the high land/*Dega* agro-climate part of the project area, the cropping pattern is characterized by cereals-enset mixed farming system. Cereals, dominantly wheat and barley, are mixed with Enset plantation in which Enset is mostly co-dominant with the cereal crops. This farming system also contains root crops mainly potatoes at significant levels. Approximately 9km (60%) length of the study area, i.e. from about km 11 to km 20, is located in this agro-climatic zone. In the middle altitude/*Weina Dega* agro-climatic zone, the cropping pattern is characterized by cereals, which is dominated by teff and maize and other important crops include sorghum, noug and potatoes. About 6km (40%) sections of the study area, i.e. from about km 5 to km 11, are situated in the *Weina Dega* agroclimatic zone.

3.3 Soils

Predominantly three types of soil formations cover the extent of the study area. These include black clay, black brown clay and volcanic ash. These are the residual and volcanic (pyroclastic) formations. The residual soils are soils formed by in situ weathering and decomposition of rocks. The volcanic formations are those transported and deposited by the action of volcanic activity.

3.4 Climatic conditions

Climatic conditions in Ethiopia are largely governed by altitudinal variations that are controlling rainfall distributions to some degree and the temperature variation to a very large extent (Atlas of Ethiopia, 1988). Based on Mean Seasonal Precipitation and Mean Seasonal Temperature variations, three operational seasonal periods are commonly known in Ethiopia. These are named as "***Bega"***,

"Belg" and "***kiremt"*** and are occurring in months of October - January, February - May and June - September, respectively. As per the metrological data in the Atlas of Ethiopia, 1988; the project alignment falls into Warm Temperate Climate I. This is characterized by the heavy rains in the summer and distinct dry months in winter. The elevation covered by this type of climatic group includes range of elevation between 1750 and 3200 above mean sea level.

3.4.1 Rain fall

The mean Annual rainfall of the route corridor ranges from 1200 to 1800mm. The major rainy season occurs in the month of June - September (Kiremt) and ranges from 800 to 1200mm. The other seasonal rain fall amounting to 400 to 600mm occurs in the months between February and May (Ethiopian Metrological Agency, Ambo and Wolliso stations). According to the data obtained from the National Metrological Agency from Ambo and Wolliso stations, the maximum monthly rainfall of Ambo and Wolliso towns are 570.1 mm and 750.5 mm respectively.

3.4.2 Temperature

The mean annual temperature along the route corridor ranges from 15^oC - 20^oC (Ethiopian metrological agency). The hottest months are from April to May and the coldest months are from June to October. According to the data obtained from the National Metrological Agency, the maximum monthly temperatures of Ambo and Wolliso towns are 30.1^oC and 29.6^oC respectively. Besides, the Minimum monthly temperatures of Ambo and Wolliso towns are 7.7^oC and 8.7^oC, respectively.

3.5 Physiographic Characteristics

The study road route alignment generally traverses over contrasted physiographic regions, which were formed during past geotectonic evolution of the Main Ethiopian Rift (MER) formation and later on modified to the present topographic scene by prolonged geo- morphological process, such as weathering and faulting (Eferem Beshawered, 2010). The road route corridor generally traverses over western highlands characterized by mountains terrain and a flat laying topography.

Based on morphological features, the study area can be classified into three distinct physiographic regions (Table: 3.1). These are: Mountains terrain covers 2km (13.3%), rolling terrain 3km (20%) and flat terrain 10km (64.7%).

Table: 3.1 Terrain classes of the study area

Station		Terrain type	Length (m)
From	To		
5+000	5+800	Flat	800
5+800	6+600	Rolling	800
6+600	9+900	Flat	3300
9+900	10+360	Rolling	460
10+360	15+000	Flat	4640
15+000	15+970	Rolling (Altufa kebele town)	970
15+970	17+970	Rolling	2000
17+970	20+000	Mountainous	2030

Mountainous Terrain

This landform falls typically between km 17.97 and 20 and comprises about 13.3% of the study area. Generally, the terrain is characterized by mountain slopes, ridges and valleys totally covered by volcanic ash. The topographic elevation varies between 2900 to 3132 m.a.s.l.

Rolling Terrain

This type of terrain is common between flat and mountainous terrain and between flat terrains as shown in Table 3.1. It is covered by black clay soil, black brown clay soils and volcanic ash. The topographic elevation varies between 2300 to 2900m above sea level and covers about 20% of the study area.

Flat Terrain

This physiographic region is characterized by the area in between Ambo and Altufa kebele town, and constitutes about 64.7% of the study area. This region is dominantly covered by black clay soil. Its topography varies from 2278 to 2737 m.a.s.l.

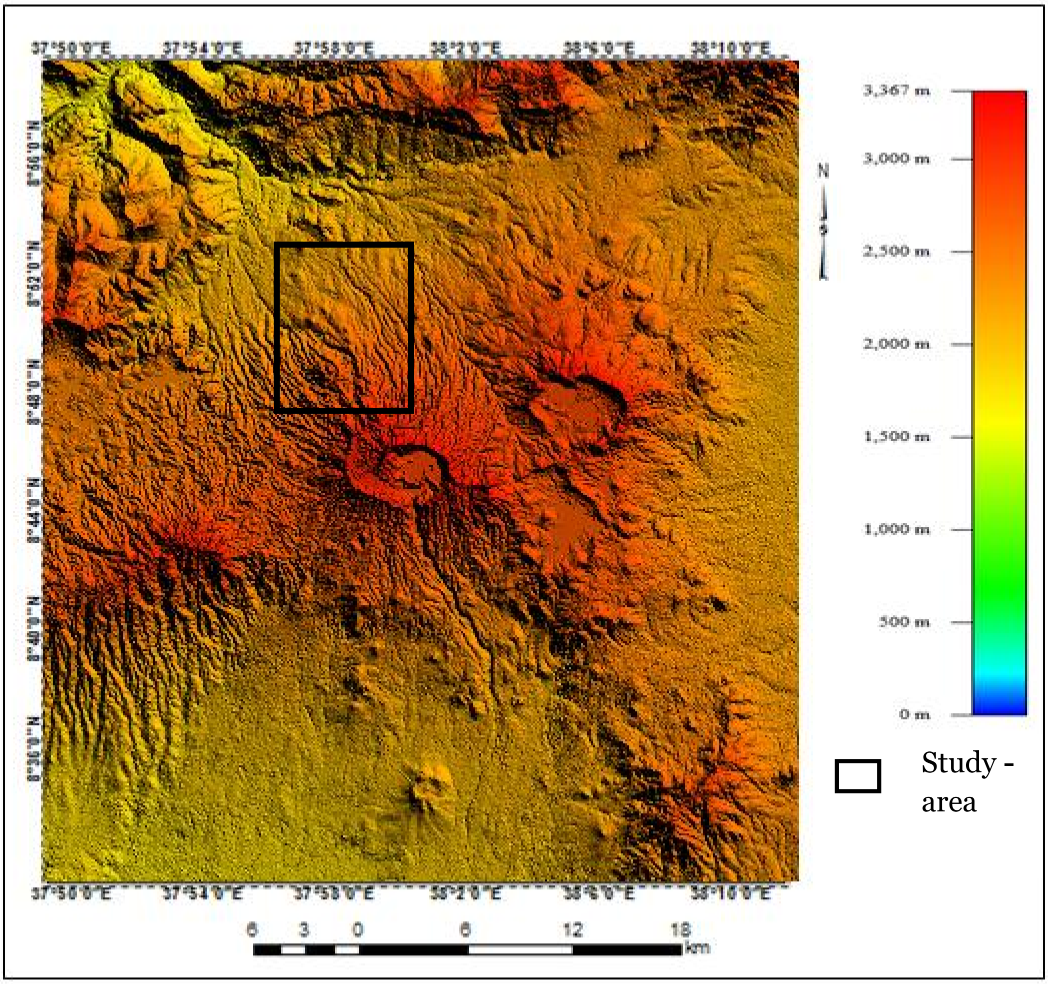

Fig: 3.2 Topography of the Ambo-Wolliso road project

3.6 Drainage and Water Resources

The Ambo-Wolisso Road is located in two major drainage systems. From Ambo town to about 24km, drains towards north contributing to the Guder river system, which is a sub-basin of the Abay river basin. While the section from 24km to end of the project drains towards southeast to the Gibe river system. Thus, the project road crosses the catchment divide between the Abay and Omo-Gibe river basins.

Since the selected section for this study runs between 5km and 20km therefore it drains to Guder river system and the road in the study area is crossed by Teltele River at station 6+125 located in rolling part of the road.

3.7 Regional Geology

The basement rocks of Ethiopia consist of Precambrian igneous and metamorphic containing several orogenic episodes.

For much of the early Paleozoic, Ethiopia was in a state of steady uplift, which caused widespread erosion in most part of the country. Subsidence followed in the Mesozoic with a large shallow sea spreading initially over the Ogaden province eventually extending further north and west as the land continued to subside. General uplift and drying out of lakebeds to leave gypsum followed this sequence and anhydrite precipitates. Regional tectonic activity associated with rifting events in the Red Sea, Gulf of Aden and East African Rift Valley during the late Tertiary caused faulting and fracturing together with widespread volcanism. Vast quantities of basaltic lava were extruded over the western half of Ethiopia. This was accompanied by ash and coarser tephra forming a sequence know as the Trap Series. Quaternary deposits are mainly confined to those associated with large depressions and lakes. Seismic and volcanic activity continues today along the Ethiopian Rift valley system, manifestation of thermal springs are related this events (Kazmin, 1972).

Mesozoic limestone, dolomitic and marl deposits in western and northern Ethiopia occur in Tigray, in the Danakil Alps and in the Blue Nile (Abbay) valley. Equivalent to Adigrat sandstone outcrops of Mesozoic marine materials occur in the central plateau area near Ambo town in the Didessa valley (Kazmin, 1972).

3.8 Geology

The substantial Geological units in the study area are BabichGuder Basalt, and Deadi-Wenchi Pyroclastic Deposit. According to Eferem Beshawered (2010), the geology of the study area is summarized as follows

The basalt is predominantly dark gray in color, fine-grained, aphanitic to porphyritic in texture, moderate to strongly weathered and thinly to thickly jointed and slightly to strongly fractured. Most outcrops are excellent candidates, for natural gravel, few for crushing aggregate and masonry stones depending on their state of weathering, fracturing and jointing patterns (Eferem Beshawered, 2010).

Microscopically, it is composed of 60% plagioclase, 30% pyroxne, 8% opaques and 2% olivine with intergranular and intersertal textures, (Eferem Beshawered, 2010).

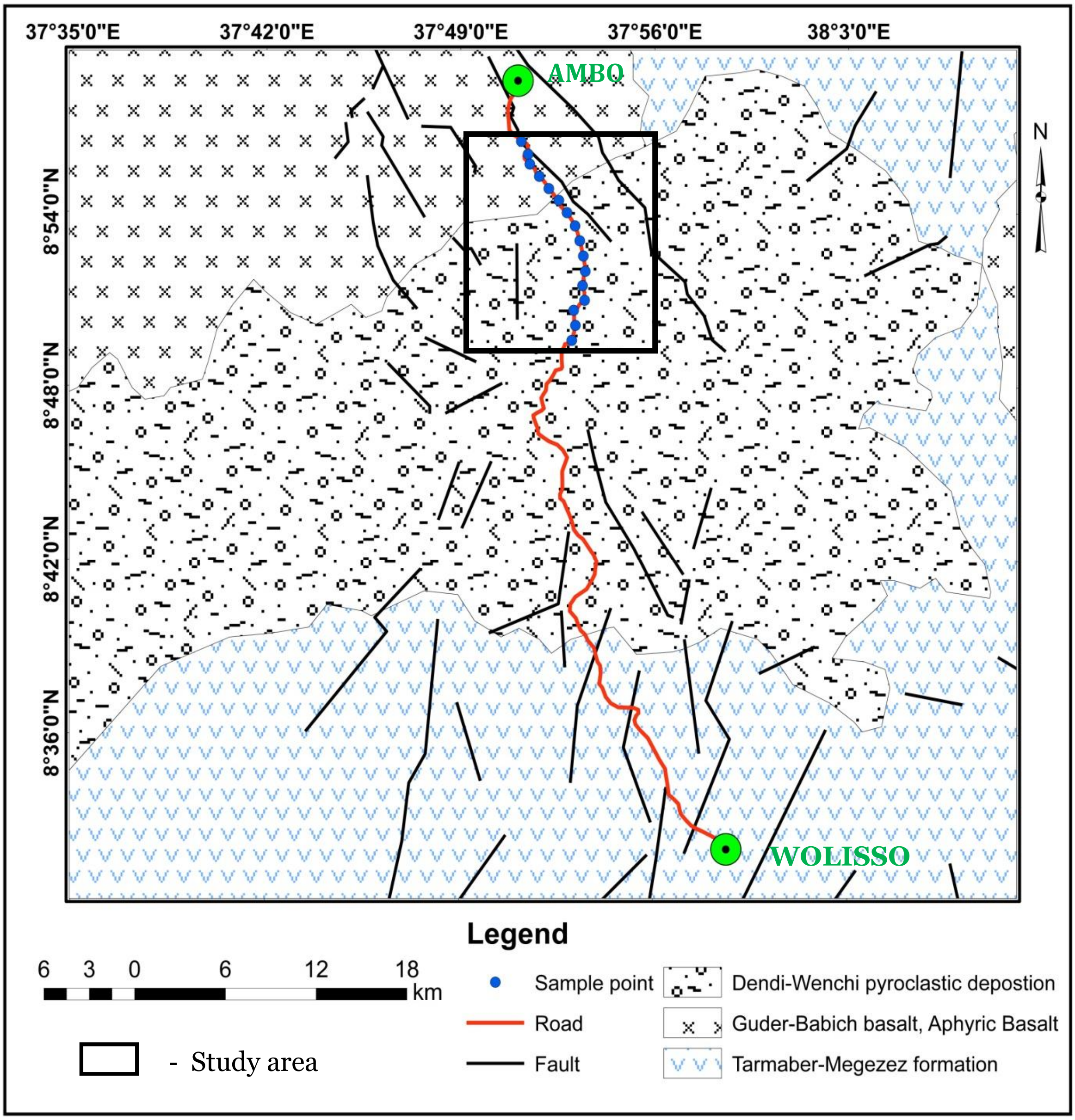

Fig: 3.3 Geological Map of Ambo – Wolliso Road Project area (Source: Ethiopian Geological survey Map sheet/NC 37-14)

Pyroclastic materials is very localized and exposed near and around Wenchi and Dendi lakes, include the products of Mount Dendi, an isolated elliptical cone having a NE-SW direction. At the summit of the cone, there are two crater lakes called Wenchi and Dendi with a diameter of ~ 2 and 1.5 km, respectively. It forms the rolling and mountainous section of the road with sharp pointed hills and steep side slopes. It is noted in the form of ash. In addition, layers of pumice are also noted in the exposed faces and cuts. The pyroclastic material is observed from km 11 to 20 of the study area.

CHAPTER FOUR FIELD INVESTIGATION AND LABORATORY TESTS FOR SUBGRADE SOILS

4.1 Introduction

This chapter presents the major procedures and steps that have been followed during field investigation, laboratory works and the results obtained. The tests include Atterberg limit, Specific gravity, Grain size analysis, Natural moisture content, Moisture-density relation, CBR and CBR swell. The test results are important both interims of characterizing the subgrade soil and evaluating its engineering property.

4.2 Field investigation for subgrade soil

The field soil investigation and sampling have been carried out between January and March 2017. The following activates were performed during the field investigation, along the selected road section alignment.

- Visual subgrade soil survey
- The study of all existing information on soil and ground water condition on proposed study section of the road location.
- The identification of the various soil types from soil profile characteristics occurring on the proposed study section of the road project.
- Collection of representative samples for laboratory testing

4.2.1 Visual subgrade soil survey

Visual subgrade soil survey has been carried out to determine the extent of the subgrade material that has made up the route corridor. Subgrade soils with nearly similar soil type are grouped together and their extent has been determined.

During the field survey, predominantly three types of soil formations cover the selected section of the project route. These are Black cotton soil, Black Brown clayey soil and volcanic ash. These can be grouped into residual (from km 5+000 to 10+000) and transported (from km 11+000 to 20+000) based on their formation. The residual soils are soils formed by in situ weathering and decomposition of the underling basaltic rocks and are characterized by black color and high plasticity. Whereas the volcanic formations are those transported and deposited by the action of volcanic activity and are characterized by Black brown to light grey in color and medium plastic to non plastic. Details of the visual subgrade soil survey along the selected route section corridor between km 5+000 to km 20+000 are summarized in Table 4.1.

Field identification of the subgrade soils has been based on color, texture as well as on the assessment of their plasticity and fine content and course fraction.

According to ERA-b (2013), the average frequency of sampling points for subgrade soil characterization and identification should be one sample per km and similarly the depth of the sampling should be at list 50cm below the expected subgrade level. Accordingly, for the present study Representative samples were collected from 60 cm below the subgrade level for further laboratory test at 1 km interval.

Table: 4.1 visual subgrade soil surveys along the selected section road route corridor

Station(km)		Field visual description	Remark
From	Upto		
5+000	10+000	Black cotton soil	Highly plastic and shrinkage cracks observed
10+000	13+500	Dark Brown clayey soil	Moderately weathered and decomposed ash
13+500	20+000	Volcanic ash	Fresh volcanic ash

4.3 Laboratory investigation for subgrade soil

The aim of this research is to characterize the subgrade soil on selected section of Ambo-Wolliso road project and based on the results to evaluate their suitability as a road foundation.

Subgrade soil investigation was performed mainly:

- To have general information about the site including description of general geology.
- To assess the engineering properties of the subgrade soil based on laboratory tests.

- To define load support characteristics of the study soil and identify unsuitable material (and problematic soil) from the study area.

The subgrade provides a foundation for supporting a pavement structure. As a result, the required pavement thickness and the performance obtained from the pavement during the design life will depend largely upon the strength and uniformity of the subgrade. Therefore, a thorough investigation of the subgrade should be made so that the design and construction will ensure uniformity of support for the pavement structure and realization of the maximum strength potential for the particular subgrade soil (Guyer, 2010).

Investigation should determine the general suitability of the subgrade soil based on classification of the soil, moisture density relationship (degree to which the soil can be compacted), expansion characteristics, susceptibility to pumping (FHWA, 2006).

The subgrade investigation was carried out from km 5+000 to 20+000 of the Ambo – Wolliso road. The selected section is in flat and rolling terrain with localized mountainous terrain. A total of sixteen disturbed subgrade soil samples were collected and subjected to various tests to determine their physical properties. The type of tests carried out includes, Atterberg limit, Grain size distribution, Moisture- density relationship, Specific gravity, California Bearing Ratio, and CBR swell test. The summery of the laboratory test results for various soil properties is tabulated in annex 1.

4.3.1 Specific Gravity Test (ASTM D 854)

The specific gravity of soil is an important weight-volume property that is helpful in classifying soils and in finding other weight-volume properties like void ratio, porosity, and unit weight.

For the present study the specific gravity of soil were determined by using a pycnometer according to ASTM D 854. The test was done at 20°c water temperature.

The specific gravity test results show that, the range of specific gravity of the subgrade soils varies from 2.33 to 2.63. The Black Cotton soils have relatively high specific gravity value than the volcanic ash material.

4.3.2 Natural Moister Content test (ASTM D 2216)

Soils normally contain water in their voids, which can be expressed as the "soil moisture content". Natural moisture content will give an idea of the state of soil in the field. The natural water content is the ratio of the weight of water to the weight of the solids in a given mass of soil. This ratio is usually expressed as percentage.

The natural moisture content, of the subgrade soils in the study area ranges from 20.4 to 32 %. All individual test results for natural moisture content are found to be more than that of optimum moisture content obtained in laboratory test in all samples at the time of sampling.

Table: 4.2 Test Results for Natural moisture content (NMC) and specific gravity (SG) of the study soils

No	Station (km)	NMC (%)	SG	No	Station (km)	NMC (%)	SG
1	5	24	2.63	9	13	30.2	2.38
2	6	24	2.63	10	14	30	2.35
3	7	20.5	2.62	11	15	32.8	2.34
4	8	20.5	2.57	12	16	28.7	2.33
5	9	21	2.57	13	17	28.7	2.34
6	10	29	2.61	14	18	31	2.33
7	11	29	2.51	15	19	31	2.33
8	12	31.3	2.49	16	20	32	2.33

4.3.3 Grain size distribution (AASHTO T 88)

Grain size analysis refers to discerning the percentage of particles (by dry mass) within a specified particle size ranges across all the size represented for the sample. The distribution of the particle size is used to distinguish the particle size and the major portion of the particle size, as well as to characterize the soil. Grain size analysis is useful for characterizing a wide variety of physical properties and affect porosity and permeability, and they are also related to the geotechnical properties of sediment (Boggs, 1995; Fetter, 2001).

For the present study, grain size analysis was carried out using the methods of wet sieving and hydrometer method. The soil is granular material if less than 35% of the soil by weight passes the no 200 sieve (75 µm) and soils having more than 35% passing the no 200 sieve are silt- clay (AASHTO, 2000). The test results for grain size analysis are tabulated on table 4.5. Additionally, for selected subgrade (2 from volcanic ash and 1 from Black cotton soil) soils passing sieve size 75µm hydrometer test was done to determine the percent of silt and clay content in the soil according to AASHTO T 88. The test results indicate that the volcanic ash is composed of, 97 to 99% silt and 1 to 3% clay, while the black cotton soil contains 66% silt and 34% clay. The test is done at 20c and with a specific gravity for volcanic ash 2.33 and 2.62 for black cotton soil.

Table: 4.3 Hydrometer test result for volcanic ash and Black cotton soil

Hydrometer test results				
Volcanic ash			Black cotton soil	
Diameter of the soil particle (mm)	Sample 1 % pass	Sample 2 % pass	Diameter of the soil particle (mm	% pass
0.038	42.95	42.95	0.033	49.96
0.025	32.07	32.07	0.024	45.92
0.015	19.03	19.03	0.013	41.89
0.011	14.68	14.68	0.009	39.87
0.008	8.15	8.15	0.006	37.85
0.004	2.72	0.54	0.002	33.81
0.002	2.72	2.72	0.001	31.79
Silt %	97	99	66	
Clay %	3	1	34	

4.3.4 Atterberg limits (AASHTO T 89, T 90)

The consistency of fine grained soil is the physical state in which it exists; it is related to a large extent to water content. Even though it is not possible to interpret the Atterberg limits and plasticity characteristics in fundamental terms (FHWA, 2006), these parameters are of great practical use as index properties of cohesive soils. The engineering properties of fine grained soils are, generally related to those index properties. The more plastic a soil means the more comprisable, higher shrinkage – swell potential and the lower is its permeability (Abramson et al., 1996).

The liquid limit (LL) is the water content, expressed in percent, at which the soil changes from a liquid state to a plastic state. The liquid limit of a soil highly depends upon the amount and type of clay mineral present. A soil containing high water content is in the liquid state and it offers no shearing resistance (Arora, 1997).

The plastic limit (PL) is the water content, expressed in percentage, below which the soil stops behaving as a plastic material and it begin to crumble when rolled into a thread of soil of 3.0mm diameter. The soil in the plastic state can be remolded into different shapes. When the water content is reduced the plasticity of the soil decreases changing into semisolid state and it cracks when remolded.

From the results of plastic limit (PL) and liquid limit (LL) plasticity index of the soil will be calculated. Plasticity index is the range of water content over which a soil behaves plastically and is important in classifying fine grained soils. The larger the plasticity index, the greater will be the Engineering problem associated it, such as foundation support for road subgrade (Bowles, 1996).

Table: 4.4 Atterberg limit test result for the study soils

No	Station (Km)	LL (%)	PL (%)	PI (%)	No	Station (Km)	LL (%)	PL (%)	PI (%)
1	5	80	45	35	9	13	49	32	17
2	6	67	38	29	10	14	NP	NP	NP
3	7	68	39	29	11	15	NP	NP	NP
4	8	67	38	29	12	16	NP	NP	NP
5	9	60	37	23	13	17	NP	NP	NP
6	10	56	36	20	14	18	NP	NP	NP
7	11	52	33	19	15	19	NP	NP	NP
8	12	50	31	19	16	20	NP	NP	NP

The above conventional tests were conducted on the 16 soil samples and a range of test results achieved. From the conventional Atterberg limit tests, a liquid limit value ranging from non plastic up to 80%, plasticity limit value of non plastic up to 45% and a plasticity index value of non plastic up to 35% were obtained (Table 4.4).

4.3.5 Subgrade soil classification (AASHTO M 145)

The main objective of a soil classification is to divide the soil into groups so that all the soils in a particular group have similar engineering characteristics by which they may be identified. Approximate assessment of engineering properties of a soil can be obtained from the index properties after appropriate classification is made. From geotechnical or engineering geological point of view, the classification of a soil may be done with the objective of finding the suitability of the soil for construction of structure or foundation. Such a classification should provide some guide to the engineering performance of the soil type and should provide a means by which soil can be identified quickly (ISRM, 1981 cited on Gebremdhin Brhane, 2010).

Gradation and plasticity are the principal determinants for engineering soil classification using either the AASHTO or Unified Soil Classification System. The ASSHTO classification system (M-145) is commonly used for highway projects, and groups the soil into categories having similar load carrying capacity and service characteristics for pavement subgrade design.

In this research the AASHTO soil classification system was employed and three soil classes were found. A-7-5 and A-7-6 frequently found from station 5+000 to 13+000 and for the rest section A-4 soil class is obtained. The summery of the test results is tabulated in Table 4.5.

Table: 4.5 Grain size analysis and AASHTO classification results for the study soils

Station (km)	% pass sieve size			% Gravel 19mm-2mm	% Sand 2mm-75 µ	% fine < 75 µ	AASHTO Soil Class (GI)
	2mm	425µm	75µm				
5	91.81	88.91	83.94	8.19	7.87	83.94	A-7-6(37)
6	96.43	85.73	79.94	3.57	16.49	79.94	A-7-6(23)
7	93.80	80.21	78.66	6.20	15.14	78.66	A-7-6(23)
8	93.78	87.14	79.78	6.22	14.0	79.78	A-7-6(27)
9	97.84	90.94	78.84	2.16	19.0	78.84	A-7-6(21)
10	94.99	80.42	67.81	5.01	27.18	67.81	A-7-5(14)
11	94.10	80.0	62.26	5.90	31.84	62.26	A-7-5(11)
12	92.10	79.9	58.54	7.90	33.76	58.34	A-7-5(10)
13	95.70	78.68	51.41	4.30	44.29	51.41	A-7-5(7)
14	88.39	75.08	48.54	11.61	39.85	48.54	A-4(0)
15	87.82	73.03	46.20	12.81	41.62	46.20	A-4(0)
16	86.29	71.20	49.38	13.71	36.91	49.35	A-4(0)
17	81.59	63.26	43.92	18.41	37.67	43.92	A-4(0)
18	82.69	63.80	41.74	17.31	40.95	41.74	A-4(0)
19	83.89	65.31	41.36	16.11	42.53	41.36	A-4(0)
20	82.32	59.35	38.11	17.67	44.21	38.11	A-4(0)

4.3.5.1 Group index

To evaluate the quality of a soil as a subgrade material, the Group Index (GI) is also used along with the groups and subgroups of the soil. Group index is a function of the liquid limit, plasticity index, and the amount of material passing the 0.075mm sieve. A group index of 0 indicates a "good" subgrade material and a group index of 20 indicates a poor subgrade material (AASHTO, 2004). Group index is calculated using the following formula.

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

Where F= percent of fine passing sieve size 0.075mm

LL = Liquid Limit

PI = Plastic Index

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)

Consequently, after the group index calculation was done, the first five km (from km 5 to km 9), which has been classified as A-7-6, have found to have group index value above 20, A-7-5 between 7and 14 and all the A-4 have group index value 0.

4.3.6 Density- Moisture Relationships (AASHTO T 180)

The dry density of a soil obtain by a given compactive effort depends on the amount of moisture the soil contend during compaction. For a given soil and a given compaction effort there is one moisture content called "optimum moisture content" that gives a maximum dry density of the soil. Those moisture contents both greater and smaller than the optimum value will result in dry density less than the maximum (Chen, 1999).

Consequently, after the Procter test was done the maximum dry densities ranged from 1.30g/cm^3 to 1.59g/cm^3 and Optimum moisture contents range from 19.0% to 26.60% were obtained for subgrade soils in the study area.

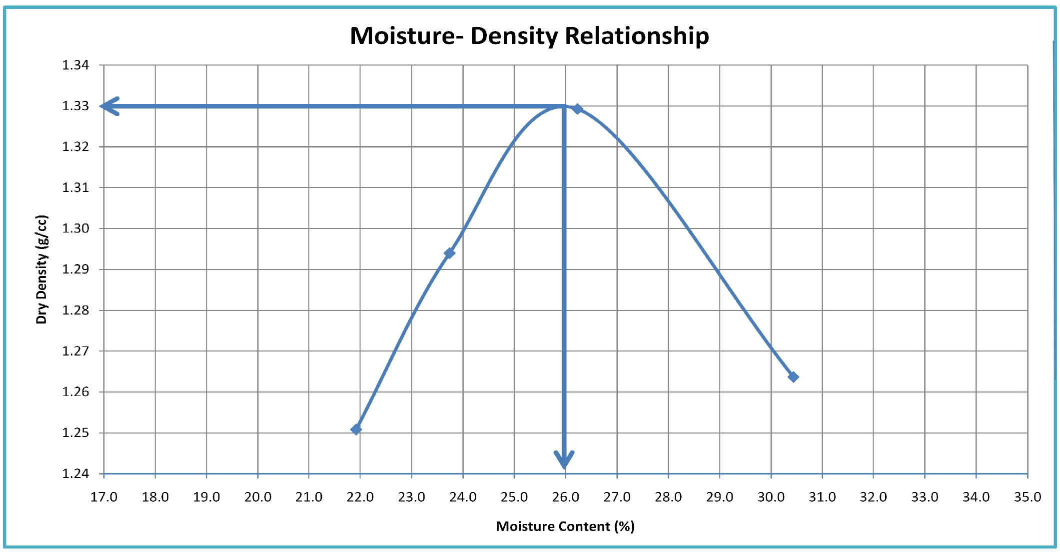

Fig: 4.1 Density moisture relationship graph at km 14 volcanic ash subgrade soil.

4.3.7 California Bearing Ratio Test (CBR) (AASHTO T 193)

The CBR is essentially a penetration test that responds to vertical stiffness in granular material and shear strength in clays. It measures that force needed to push 49.6mm diameter plunger very slowly (1.25mm/min) into a compacted cylindrical soil sample. The test is usually carried out on saturated specimen from which above 20mm size stones have been removed, as this impede to run penetration.

For the present study three point CBR test was performed as per AASHTO T 193. The CBR at the 95% maximum dry density is determined from a graph of CBR versus dry unit weight according to the project technical specification.

Consequently, after the penetration test were carried out a CBR value ranging from 0.85% up to 32% is obtained at 95% MDD of modified AASHTO proctor density.

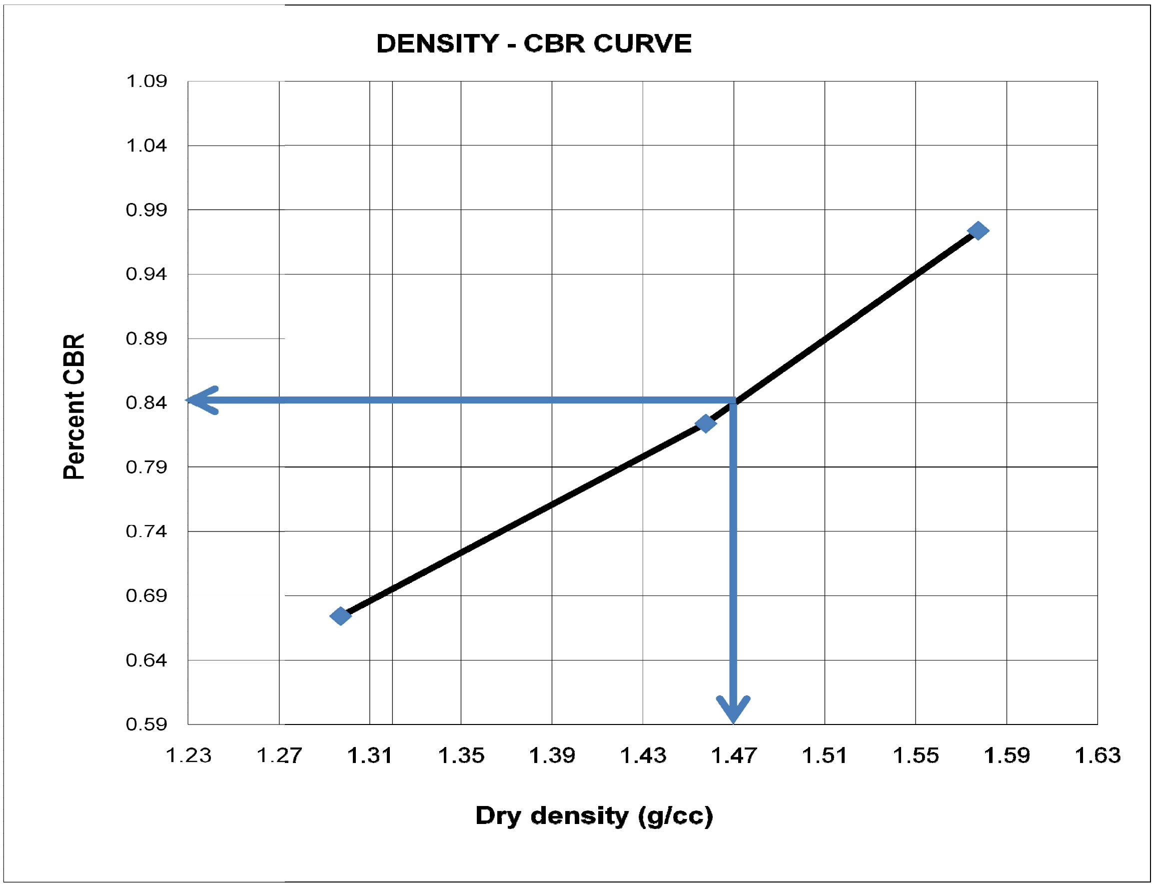

Fig: 4.2 Dry densities Vs CBR curve for black cotton soil at km 5

4.3.8 CBR swell test

The swell potential, in this study, is defined as the percentage swell of laterally confined samples, which has been soaked under a surcharge weight of 4.5kg after being compacted to a maximum density at optimum moisture content according to the compaction test. The swell was expressed as a percentage increase in a sample height.

The results show that the soil samples in the study area have swelling values between 0.16% and 8.2%. The swelling values of the samples classified as A-7-6 and A-7-5 according to AASHTO M-145 ranges from 1.19%-8.2% and the swelling value for A-4 soils is found between 0.16% and 0.80%.

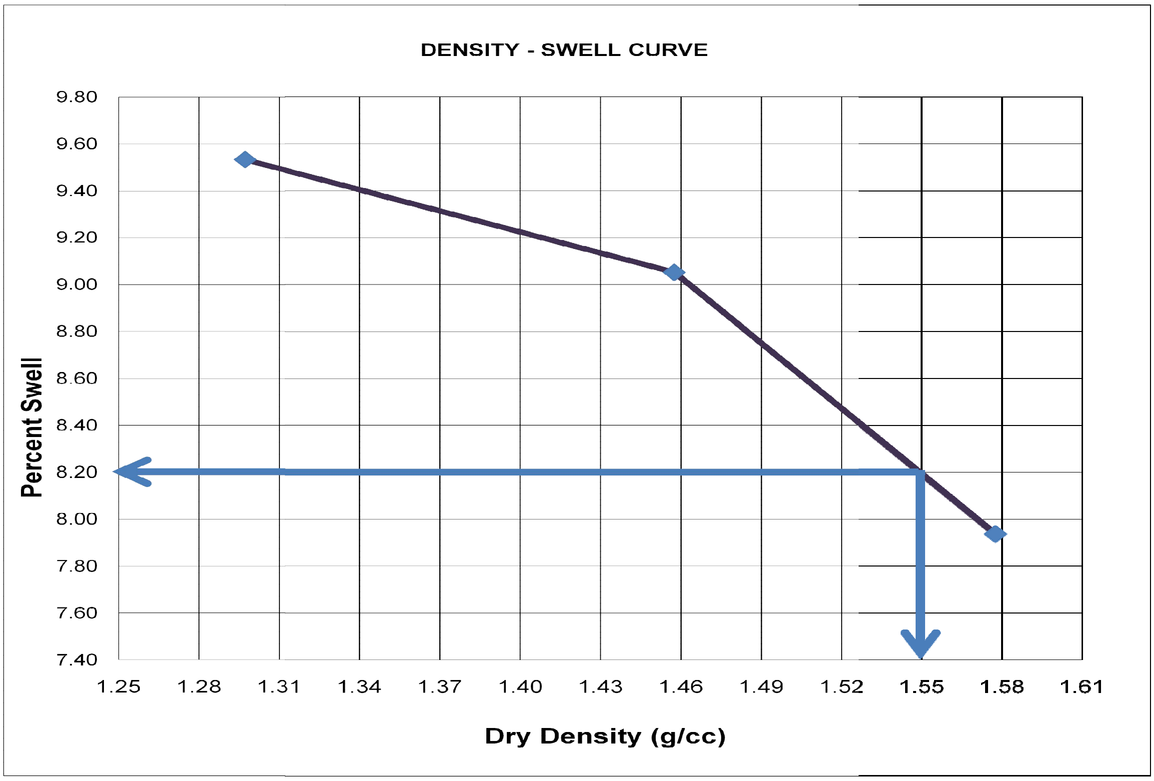

Fig: 4.3 dry density Vs swell curve for Black Cotton soil at km 5

Chapter Five INTERPRETATIONS AND DISCUSSION

5.1 Introduction

Subgrade soil is a part of a pavement support system. Subgrade performance generally depends on the three basic characteristics; Strength, Shrinkage or swelling and Moisture content (Iowa, 2013). Based on data collected in field and laboratory test conducted, for this study, subgrade soils are characterized by evaluating in terms of its engineering and geological properties in comparison with ERA-a (2013) and project standards.

5.2 Grain size distribution

For the present study particle size distribution are determined by means of wet sieving as discussed in chapter 4 section 4.3.3 and for fine friction passing 0.075mm by hydrometer test. From all test results on grain size indicates the presence of silt, sand and to some extent gravel and clay. However, the percent of fine (clay & silt) content shows increasing slightly as the elevation reduced as shown in Fig 5.1. Grain size affects Atterberg limits, bulk density and pore pressure transients in response to cyclic loading (Hein, 1991). Based on wet sieve analysis all the subgrade soils in the study area were found to have fine friction of above 35% by weight and the hydrometer test results indicate the fine of volcanic ash are 97 to 99% silts and the fine of black clay soils are composed of 66% silt and 34% clay.

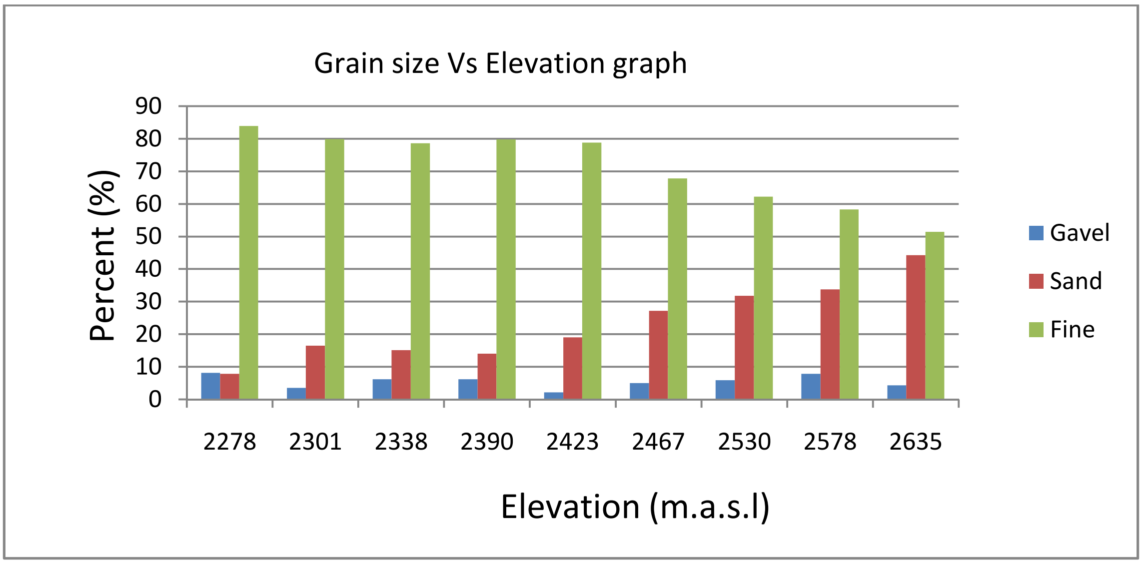

Fig: 5.1 Grain size Vs Elevation for cohesive subgrade soil of the study area

Fig 5.1 shows the % of gravel, sand and fine for cohesive soil in the study area. The % fine along the route in the study area increases while elevation reduces.

According to IDOT (2006), silty soils that have fewer amounts of clay contents in a given soil have its owen problem. First, such soils are difficult to compact due to excessive resilient (rebound) defalcation (pumping). When loading one spot an adjacent spot will rise. Pumping is the ejection of foundation material, either wet or dry, through joints or cracks, or along edges of rigid slabs, resulting from vertical movements of the soil under traffic, or from cracks in semi-rigid pavements (FHWA, 2006). Second, such soils are sensitive to moisture. Strength and density change rapidly with a slight change in moisture. Field observation shows that sometimes when a volcanic ash soil looks dry and hard on the surface, repeated traffic load can draw moisture from below soil water into the surface and cause excessive routing. Therefore, the volcanic ash in the study area is more susceptible to pumping and compaction problems since the fines in volcanic ash are composed of almost silts only.

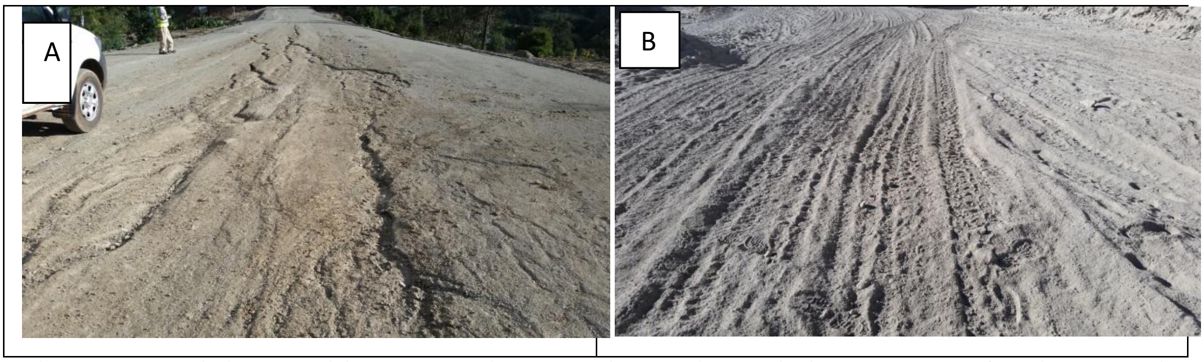

Plate: 5.1 stability problem on volcanic ash a) channeled exposure of volcanic ash subgrade due to erosion by rainfall and b) dust and routing due to traffic

5.3 Atterberg limits

The fine particle interacts with moisture and Atterberg limit are used to characterize the interaction of the fines with moisture. Liquid Limit (LL) defines the transition between the liquid and plastic states while the transition between the plastic and semi-solid state defines the plastic limit (PL). The difference is termed the *plasticity index* (*PI = LL - PL*) (FHWA, 2006).

Liquid and plastic limits tests determine the plasticity of fines particles and thereby allow them to be defined as either silt or clay soils. The word "silty" is applied to a fine material having a Plasticity Index of 10 or less, and the term "clayey" is applied to fine material having a PI of more than 10, AASHTO M 145. Based on this 56.25% of the study subgrade soils are clayey and 43.75% are silty.

Chen (1999) indicates that soils having a plasticity index in excess of 15 should be considered potentially expansive. Soils with high plasticity index and liquid limit have normally high inherent swelling capacity (Guney et al., 2005; Chen 1988; Williams and Donaldson, 1980) and are considered as potentially expansive. Liquid limit less than 35% indicates low plasticity, between 35% and 50% intermediate plasticity, between 50% and 70% high plasticity and between 70% and 90% very high plasticity (Whitlow, 1995). Based on LL, the study area fresh volcanic ash has no Cohesion, Hence it is called non-plastic, weathered volcanic ash has intermediate plasticity and, the Black cotton soils classified as high plasticity to very high plasticity and therefore are potentially expansive.

Further, According to ERA-a (2013) specification clay materials having a Liquid Limit (LL) exceeding 60%; or a Plasticity Index (PI) exceeding 30% are classified as unsuitable material for a road subgrade and 31.25% of the study soils on the bases of LL and 6.25 % (one sample) on the bases of PI are classified as unsuitable as a road subgrade according to ERA-a (2013).

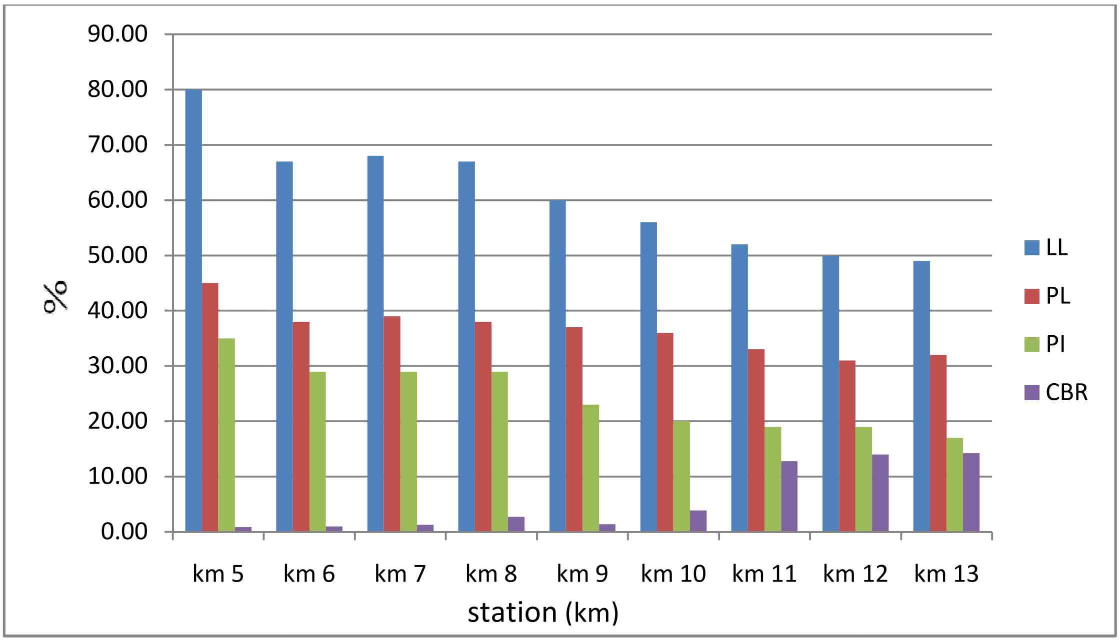

Fig: 5.2 Atterberg limits Vs CBR value for study cohesive subgrade soils.

Fig: 5.2 shows Atterberg limit and CBR values of the cohesive soils in the study road section. The subgrade soil in the road section which have high LL and PI values have low CBR value.

According to the project specification, the top 600mm the subgrade layer should have a Liquid Limit (LL) not exceeding 45% and Plasticity Index (PI) less than of 20%. For remaining layers, the PI should not be more than 25%. Based on this, 56.25% of the study soils on bases of LL and 37.5% on the bases of PI are unsuitable to be used as subgrade soil on top 600mm and 25% on the bases of PI are unsuitable on lower layer of 600mm of the subgrade.

5.4 Soil classification

Based on the test results on Atterberg limit and gradation for the study subgrade soils, AASHTO soil classification is done. AASHTO Soil classification is a systematic method of categorizing soils into various groups and subgroups according to their probable engineering behavior but without detailed description. However, clarity does not exist in the existing soil classification system in classifying inorganic silts, silty sands, silty gravels that are non-plastic (Prakash and Sridharan, 2002). In present study area, 40% of the soils are silty (fresh volcanic ash) and for which neither liquid limit nor plastic limit can be determined in laboratory. However, current ERA-b (2013) and many other standards suggest to use AASHTO soil classification for the purpose of characterization. Hence, for the present study AASHTO classification is adapted with its ambiguity. Accordingly, three soil classes find for the study subgrade soils, namely A-7-6, A-7-5, and A-4.

Table: 5.1 classification and test results of Atterberg limit for the study soil

AASHTO Soil class	No of test results	% Test results	Range of GI	Range of CBR	Range of PI	Range of LL
A-7-6	5	31.25	21-37	0.84-1.7	23-35	60-80
A-7-5	4	25.00	7-14	3.87-14	17-20	49-56
A-4	7	43.75	0	11-32	NP	NP

According to AASHTO M-145, all the soil classes in the study area are generally rate as a poor to fair as a subgrade material. Further, the group index (GI) value is used as a general guide to the load bearing characteristics of a cohesive soil (FHWA, 2006).

Accordingly, about 31.25% subgrade soils of the study area are found to have GI value above 20 which indicate the low load caring capacity of the soil. Correlation between GI and CBR value shows as all subgrade soils which are characterized as low strength material from CBR test results are characterized as a poor subgrade material from GI value except one sample which is characterized as low strength based on CBR and good material based on GI.

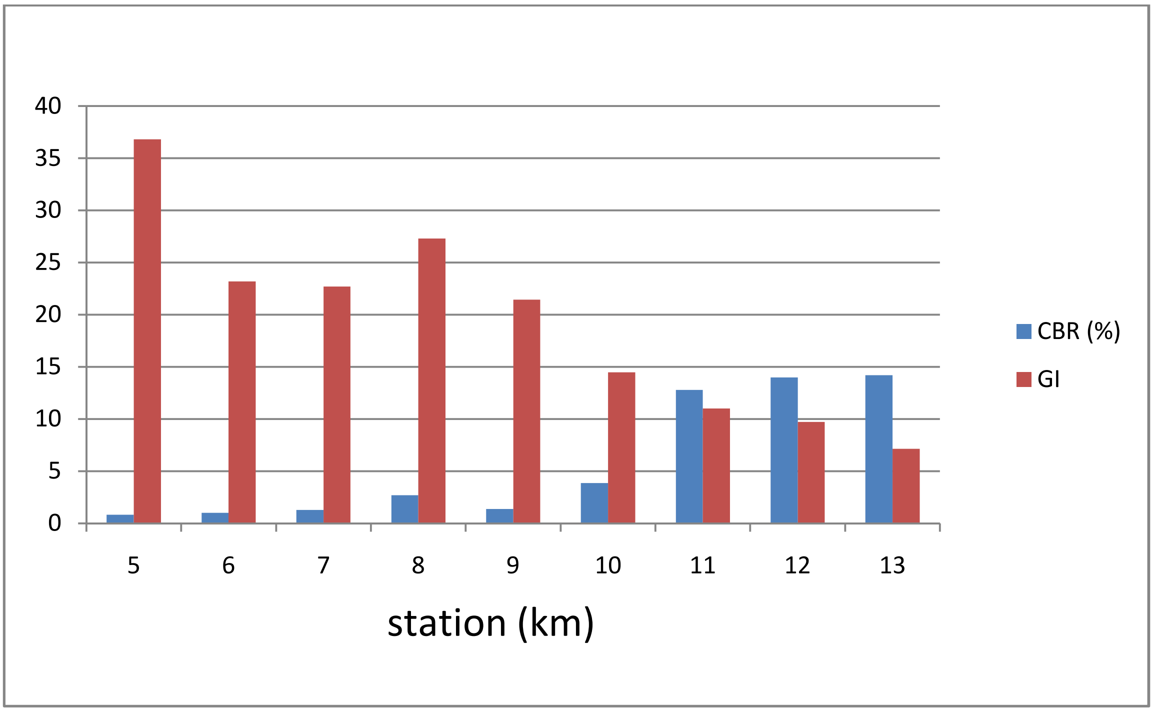

Fig: 5.3 CBR Vs GI results for the study cohesive soils.

5.5 Compaction test

Soil compaction is one of the most important geotechnical concerns during the construction of highway pavements and related fills and embankments (FHWA, 2006). Compaction improves the engineering properties of soils in many ways, including

- increased elastic stiffness, which reduces short-term resilient deformations during cyclic loading.

- decreased compressibility, which reduces the potential for excessive long-term settlement.
- increased strength, which increases bearing capacity and decreases instability potential.
- decreased hydraulic conductivity (permeability), which inhibits flow of water through the soil.
- decreased void ratio, which reduces the amount of water that can be held in the soil and, thus, helps maintain desired strength and stiffness properties and increased erosion resistance.

During construction, the control is only on OMC and dry density that is to be checked on fixed length. To meet with the density criteria is considered as synonymous to meeting strength/CBR. Optimal engineering properties for a given soil type occur near its compaction optimum moisture content, as determined by the laboratory tests.

The most common measure of compaction is density as a measure of the amount of solid materials present in a unit volume. The higher the amount of solid materials, the stronger and more stable the soil will be (FHWA, 2006).

Accordingly, for the present study, compaction test has been conducted as per the AASHTO T 180. The optimum moisture content obtained from the testes range from 18.4 -28.4% where the Black cotton soils optimum moisture content range from 19.0-21.7% average 20.2%, the weathered volcanic ash from 20.3-28.4% average 25.1% and the fresh volcanic ash from 25.5- 26.6% average 26%. Whereas the maximum dry density ranges for black cotton soils from 1.51–1.59g/cm^3 average 1.55g/cm^3, for weathered volcanic ash 1.31-1.47g/cm^3 average 1.40g/cm^3 and for fresh volcanic ash 1.3-1.33g/cm^3 average 1.31g/cm^3.

According to Arora (2004), cohesive soils have high air voids. These soils attain a relatively lower maximum dry density as compared with the cohesion less soils. Such soils require more water than cohesion-less soils and, therefore, the optimum water content is high. Heavy clays of a very high plasticity have very low density and very high optimum water content.

However, Compaction test results of the study soils imply, non cohesive volcanic ash have high OMC, low MDD and good strength. Relatively high OMC may indicate the volcanic ashes are more porous than even the highly plastic soils in the study area and the reason to have low MDD is because volcanic ashes are light weight in nature. Heaver materials are more compact, less pores and have greater specific gravity. Specific gravity is an important index property of soils that is closely linked with mineralogy or chemical composition (Oyediran and Durojaiye, 2011) and measures heaviness of material in relative to water. From the test results on compaction test and specific gravity, maximum dry density obtained for the fresh volcanic ash ranges from 1.30-1.33g/cm^3 and Specific gravity from 2.30-2.39 as shown on Fig 5.4.

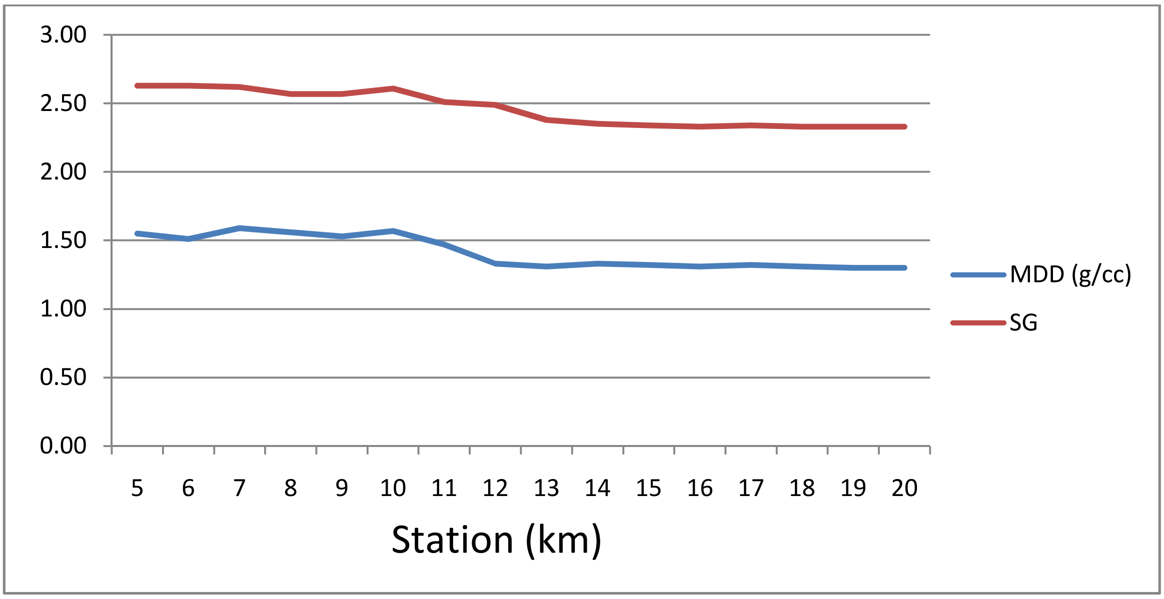

Fig: 5.4 MDD Vs specific gravity of the study soils.

Expansion pressure and potential volume change increase significantly with the dry density of swelling soils. High degrees of compaction may therefore be detrimental and should be avoided. It is recommended that the dry density of expansive soils in no case exceeds 95% (AASHTO T 180) MDD (ERA, 2013).

Based on the results on swelling value, 37.5% of the study soils are potentially expansive and construction on this section (from km 5 to 10) requires not densifying above the limit during road bed preparation.

5.6 California Bearing Ratio (CBR)

The California Bearing Ratio (CBR) test method has been found as the most reliable means for evaluating the strength of the subgrade (bearing capacity of a soil) and construction material, hence, in Ethiopia, California Bearing Ratio (CBR) value of a subgrade is used for design of flexible pavement. The thickness of sub-grade mainly depends on *CBR* value, if the *CBR* value is higher, then the designed thickness of the pavement is thinner and vice versa (ERA, 2013).

The general relationship between CBR values and the quality of the subgrade soils used in pavement application is as follows (Bowels, 1992)

CBR value (%)	Quality of the subgrade
0-3	Very poor
3-7	Poor to fair
7-20	Fair
20-50	Good
>50	Excellent

ERA-b (2013) and the project contract document (2013) stipulated a minimum CBR of 5% for a subgrade soil. Based on this, 37.5% of the subgrade soil studied have CBR value less than 5%, hence are classified as a very poor to poor quality as a subgrade and 62.5% of the studied subgrade soils have CBR value above 12% and are classified as having fair to good quality.

Soils classified as poor quality do not possess enough strength to support the pavement structure, and the wheel loads exerted upon them, therefore, classified as unsuitable as a road subgrade. There is therefore need to improve the quality of the material in order for the road to be able to effectively withstand the heavy traffic that it is usually exposed to. This improvement may take the form of either or combination of techniques namely: removal and replacement (over excavation), mechanical stabilization with a binding material for instance locally available volcanic ash, or chemical stabilization like cement which stabilize all soils except the highly organic clays (Garg, 2009), lime or the introduction of an additional base course layer.

5.7 Swell potential

Swelling soils may impose significant uplift pressure on the pavement. When the volume change (%) increases, the swelling pressure increases, hence, the potential damage to the pavement increases. Similarly, in swelling zone, a reduction on shear strength, elasticity and bearing capacity may occur as a consequence of expansion (ERA, 2013).

Seed et al (1962) defined swell potential as the change in volume of a remolded soil sample. A one dimensional swell potential test is used to estimate the percent swell and swelling pressure developed by swelling soils (FHWA, 2006). For the present study one dimensional swelling value of the subgrade samples was carried out after compacting the soil at optimum moisture content and maximum dry density.

Table: 5.2 Seed et al (1962) Rating of swelling potential

Range of swelling (%)	Swelling potential
0-1.5	Low
1.5-5	Medium
5-25	High
>25	Very High

According to (ERA, 2013), soils with swelling value above 2% are not suitable as a road subgrade. The test results for swelling value show that 62.5% of the subgrade soils in the study area have swelling value less than 2%; therefore, the degree of damage caused by this soils will be low and classified as suitable. Alternatively, 37.5% of the soil in the study area has more than 2% swelling value, which have medium to high degree of damage according to seed et al (1962) therefore are classified as unsuitable.

5.8 Climate

Climate is also pertinent since precipitation and evapotranspiration affect soil moisture and, thus, potential swell (Corley, 1980). The excessive wetness of the foundation leads to a weakening of the road foundation and eventually, failure of the surface (ERA, 2013). Hydrological report of the study area (Bereket Fentaw and Mihret Manaye, 2011) point as that in the study area the water table is deep; therefore ground water has no significant problem on the road foundation for the present study area. As a result the water content of the subgrade soil in the study area is controlled by precipitation and evapotranspiration.

During field investigation wide and deep shrinkage cracks (1m deep and about 2-5cm wide) has been observed from km 5 to 9 of the study area. Due to high precipitation and relatively low temperature in the study area, the subgrade soils have high probability to be affected by rain water.

The moisture content expresses the amount of water present in a quantity of soil at a moment (season) when the test was conducted. The natural moisture content test results are all found above the optimum moisture content found in laboratory compaction test as shown on Fig 5.5; thus may indicate high probability of water infiltration. Volcanic ash has relatively high water content; this may be due to high porosity or voids in the material. It has been mentioned that 37.5% of the study area soils have medium to high swelling potential, hence the infiltration of rain water with such soils could result in swell and erosion as shown on plate 5.2. Therefore it is required protection during design for section which is potentially expansive. For non expansive soils presence of more water than optimum moisture require drying until it reaches the optimum moisture content before using this material for construction. This can simply be achieved by spreading the soil in dry sunny climate.

Plate:5.2 Damage on subgrade soil caused by rainfall which indicate importance of subsurface drainage in the area

Hence, due to high rainfall and low annual mean temperature in combination with presence of highly porous material in the study area (pavement edges provide ample path ways for water to infiltrate the pavement structure) address the need for the subsurface drainage for the road project. In areas with deep water tables and proper design and construction it is less likely that the subgrade will get wetter after construction. ERA, 2013 drainage design manual, recommend three effective approaches to controlling or reducing effect of moisture on pavements. These are;

- To provide adequate cross slopes and longitudinal slopes to quickly drain moisture from pavement surface, thereby minimizing infiltrations into the pavement structure.
- To use material and design features, such as stabilized cement (CTB) or Lean concrete bases (LCB) in Portland cement concrete, also known as PCC pavement , that are not sensitive to the effects of moisture; and
- To remove moisture that enters the pavement system promptly.

For effective control of moisture related problems in pavement over the life of the pavement, it is often necessary to apply this approach or in combination with other suitable measures.

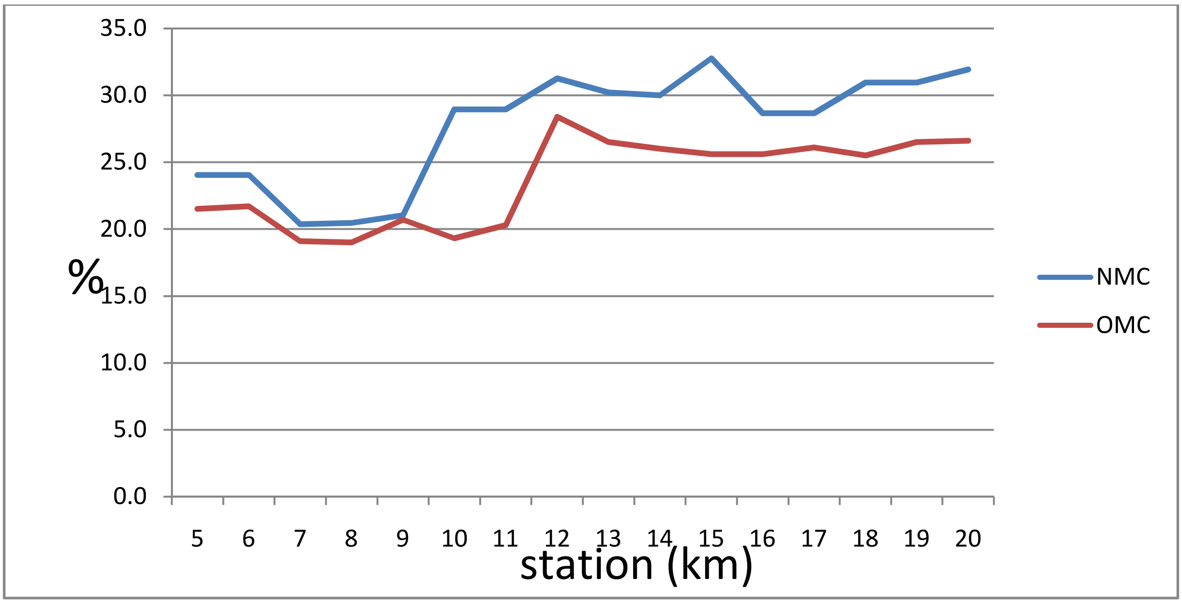

Fig: 5.5 OMC Vs NMC of the study subgrade soil

5.9 Geology

The conditions or processes, which determine the clay mineralogy, include composition of the parent material and degree of physical and chemical weathering to which the materials are subjected.

Basically two lithological units cover the study area; Babich Guder Basalt and Pyroclastic material (refer Fig 3.4).

Babich Guder Basalt covers 37.5% (from km 5 to km10) of the study area.

The fresh pyroclasts are grey to light grey in color and brown to black brown when weathered and are loosely cemented (can be easily excavated by hand). Plate 5.3 shows the pyroclastic material in the study area around km 20. For the purpose of classification, pyroclastic fragments are distinguished by their size and shape in the following way, (Gillespie and Styles, 1999):

Bombs—are pyroclastic fragments whose mean diameter exceeds 64 mm and which have a shape (generally rounded) or texture (e.g. 'bread-crust' surface) which indicates that they were in a wholly or partly molten state during their formation and subsequent transport.

Blocks—are pyroclastic fragments whose mean diameter exceeds 64 mm and which have an angular or sub angular shape indicating they were solid during transport.

Lapilli—are pyroclastic fragments of any shape with a mean diameter of 2 mm to 64 mm.

Ash—grains are pyroclastic fragments with a mean diameter of less than 2 mm. These are subdivided into coarse ash grains (0.032 mm to 2 mm) and fine ash grains (less than 0.032 mm).

In addition in the mentioned reference it is discussed that, Pyroclastic rocks and sediments contain more than 75% by volume of pyroclastic fragments, the remaining materials being generally of epiclastic, organic, chemical sedimentary or authigenic origin. If predominantly consolidated they should be classified as pyroclastic rocks, and if predominantly unconsolidated they should be classified as tephra. The term 'tephra' is synonymous with 'pyroclastic sediment'.

The names lapilli-tephra or lapilli-ash or ash should be used where the average size of more than 75% of the pyroclastic fragments is less than 64 mm. The relative proportions of lapilli- and ash-grade fragments is indicated in the following way: in a lapilli-tephra more than 75% of the pyroclastic fragments are lapilli (i.e. in the range 64 to 2 mm), in a lapilli-ash between 25 and 75% of the fragments are lapilli, and in an ash less than 25% of the fragments are lapilli (i.e. > 75% are ash-grade), (Gillespie, and Styles, 1999).

Plate: 5.3 slope cut and benches in volcanic ash road section around km 20

Accordingly, the pyroclastic fragments in the study area are classified as tephra ash since they are composed of 10 to 20% by lappilli size (2-64mm) and 80-90% ash size (<2mm) material and are predominantly unconsolidated. Locally, there are some cut slopes where thick pumice deposit is observed, rich in lapilli size 70-90% and 10-30% ash and was not considered in the classification.

From the study soils, about 37.5% are resulted from weathering of basalt parent material, 18.75% are from weathered tephra ash and the rest 43.75% are fresh tephra ash. The test results on swelling value confirm all soils overlying the basalt have high degree of swelling (average swelling value about 7.03%) and the weathered ash low swelling value (average swelling value about 1.19%) and the fresh tephra ash are almost none swelling (average swelling value about 0.38%).

However, as mentioned before, degree of weathering in the study area is found to be increased when the elevation is reduced.

According to Chen (1988), the most important weathering process responsible for the formation of montmorillonite is the chemical weathering of parent rock mineral and the most favorable climatic condition is in semi-arid regions with relatively low rain fall or seasonal moderate rainfall particularly where evaporation exceeds precipitation. Therefore, swelling value of the weathered volcanic ash is low may be due to the degree of weathering is moderate and not yet completely weathered to give the clay soil containing Montmorillonite (smectite) mineral or may be due to unfavorable climatic condition of the area, hence in the study area precipitation exceed evaporation.

Generally, the present area covers highly weathered basaltic rocks and fresh to moderately weathered tephra ash. Based on geology and climate condition the weathered area is most sympathetic to have clay soil containing Montmorillonite (smectite) mineral, hence is potentially expansive and unsuitable to be foundation for light structure like road. In other way, for fresh tephra ash even if the material is appropriate for formation of expansive soil, however, since formation of expansive soil requires long time, which is incomparable with the design life time of the project, the effect is not such problematic.

5.10 General characterization of the subgrade material

The study subgrade soils are grouped into 3 on the bases of the geotechnical characteristics and in this section, based on these groups general suitability characterization is presented based on the ERA (2013) and project evaluation standards (table, 5.3).

Table 5.3 Summery of the ERA (2013) and project Standard requirement for suitable subgrade soils

Parameters	ERA Requirement	Project standard Requirement	
		Upper 600mm of the subgrade soil	Lower 600mm Of the subgrade
CBR (%)	>5	>5	>5
CBR Swell (%)	< 2	< 1.5	<1.5
PI (%)	< 30	<20	<25
LL (%)	< 60	<45	<45

1. Black cotton soils (from km 5 to 10)

These soils are soft when wet and hard when dry with high moisture content and mainly exposed along a flat and rolling topography. The soil is composed of very fine material and is highly plastic, covers 5km of the study area (from km5 to 10). The cracking of the ground during dry season is the most common features indication of the expansive nature of this material. Genetically they are residual soils which are weathering products of the underlying basalt rock. The CBR and CBR swell test results indicate the low bearing capacity and high swelling

From the test results, all of the soils in this section do not fulfill all the project standard and ERA requirements for subgrade soils. Therefore, the subgrade soil in this section is characterized as unsuitable to be a foundation for the road construction. The main identified problems for this section are

- Low bearing capacity
- Volume changes in wetting and drying

2. Black brown moderately weathered volcanic ash (from km 11 to 13)

This is soft, medium plastic and black brown in color. The soil covers 3km (about from km 11-13) of the study area. The soil is thick and covers the flat terrain of the road extension. The MDD and SG test results show certain similarity with the fresh volcanic ash and the plasticity test results show more similarity with the black cotton soils. Genetically they are transported soil and the CBR and CBR swell test results indicate good bearing capacity and low swelling potential of the soil in this section.

Based on the laboratory test results, this soil fulfills all the ERA (2013) requirements and is characterized as suitable subgrade material. However, according to the project specification, this soil fulfill the CBR, CBR swell and plasticity index requirement while, do not fulfill the liquid limit requirement. Hence, the soil in this section is unsuitable on the bases of the project standard.

3. Fresh volcanic ash (from km 14 to 20)

These soils are light weight, non plastic and light grey in color, have high OMC and are exposed in rolling and mountainous terrain. Genetically the soils are transported and are very thick. They cover 7km (from km 14 to 20) of the study area. All the laboratory results show that the volcanic ash is suitable subgrade material except the high OMC which indicates high porosity of the material. Its light weight nature make more susceptible to erosion and when they dry easily can eroded by wind which indicates stability problems, therefore requires certain measures prior to use as a subgrade.

In addition, the test results on hydrometer indicate the volcanic ash has almost no clays (only few 1-3%). This has workability problems as discussed before in this chapter

Chapter six Conclusions and Recommendations

Engineering geological properties of the subgrade soil is important for design of flexible pavements and affects the pavement performance. From various test conducted on the laboratory and field observation the following conclusion and recommendation is proposed for the study soils.

6.1 Conclusions

The present study was carried out on Ambo-Wolliso road project. The study area is located in the Oromia Regional state. The project starts at Ambo Town, which is 125km from Addis Ababa along Addis – Nekemte trunk Road and traverses in the south direction to Wolliso town. The project road connects the two trunk roads, namely Addis- Jimma and Adiss-Nekemte. The total length of the route is 62.7Km and from this, the present study is conducted on 15km of selected section. The study area passes on elevations ranging from 2278m to 3132m above mean sea level. The geographic positions of the project falls between 8° 56' N 37° 52' E and 8° 49' N 37° 53' E.

During the field survey, it was observed that predominantly three types of soil formations cover the study area. These are Black cotton soil, Black Brown clayey soil and volcanic ash. Further these can be grouped into residual (from km 5+000 to 10+000) and transported soils (from km 11+000 to 20+000) based on their formation.

The general objective of this research was to characterize the subgrade soil in selected section of the Ambo-Wolliso road and to evaluate its suitability for use as a road foundation.

In order to achieve the objectives of the present study a systematic methodology was followed. A total of 16 sub-grade soil samples were taken at 1km interval and tests were conducted at project geotechnical field laboratory for the determination of Atterberg limits, grading, and MDD, OMC, specific gravity, NMC, CBR and swell values. Thus, based on the test results interpretations were made to meet the general objectives of the present study.

The various results of the analysis show that the samples have liquid limit values of between 0 and 80%, Plastic limits of 0 to 45% with corresponding plasticity index values of between 0 and 35%; whereas 31.25% soils possess plasticity index values higher than 60% and 6.25% plasticity index above 30%. About 43.75% of soils are non plastic. Likewise, the optimum moisture content (OMC) ranged between 19 and 28.4% with maximum dry densities (MDD) of between 1.30 and 1.59g/cm^3. The soaked CBR values of the samples on the other hand were between 0.84 and 32%. About 37.5% of the subgrade soils show CBR value less than 5%. The swelling potential for the sub-grade soils ranges between 0.16% and 8.2 %. About 62.5% of sub grade soils posses low swelling potential and 37.5% of the soils show high to very high swelling potential. Further, the specific gravity value of the soil ranges between 2.63 and 2.33.

Based on the analysis and interpretation of the test results, the subgrade soils from km 5 to 9 are classified as A-7-6 on the AASHTO classification system and the soils in this class generally posses low bearing capacity and high to very high swelling potential hence, are characterized as unsuitable as a road foundation.

The soils from km 10 to 13 under the present study are classified as A-7-5 and the soil in these class posses fair to good bearing capacity and high to low swelling potential. Accordingly, the subgrade soil at km 10 is characterized as unsuitable and the rest of the road section in these class are characterized as suitable subgrade soil in the basis of bearing capacity and swelling potential. However, the soil in this section is characterized as unsuitable on the bases of LL according to the project standard.

The soil from km 14 to 20, are classified as A-4 on the AASHTO classification system and the soil in this class posses good bearing capacity and low swelling potential hence are characterized as a suitable subgrade soil for use as a road foundation.

6.2 Recommendations

Based on the findings of this research, ease of construction, time and economy, the following remedial measure is recommended for the study road section.

For the study unsuitable sub-grade soils, subgrade soils with low bearing strength and high swelling characteristics, it is recommended to excavate down upto right depth depending on the thickness of the swelling soils. (According to ERA the depth of the replacement depends on the thickness of the swelling soils). The material that to be used to replace the excavated soil should have minimum CBR of 5%, maximum 2% swell and 20 % PI which is equivalent to an S3 type material indicated as per ERA, 2013. The replacement depth can be reduced for high fill sections (fills greater than 2.0m).

It is observed from different excavation in black cotton clay section, the thickness of the swelling soil is above 2m. According to ERA standard specification for such thick swelling soil the replacement should be minimum 1m and the work shall be executed in five layers of equal thickness and must be compacted to the required minimum density.

- Side drains in expansive soils should be avoided or if this is not possible, they should be as shallow as possible and located as far away as practicable from the toe of the embankments. Additionally, Provide paved shoulder and vertical moisture barrier on pavement edges helps to avoid infiltration of surface waters down the sub-grade layer. Where possible, the Original Ground Level shall be graded to an outward crass fall of certain inclination from the toe to the ditch.
- During construction, the roadbed of expansive soil should be kept moist and cover with earthworks prior to any drying. Attempts to process and compact the soil beyond normal density requirements is inappropriate. Fill material over the expansive soils shall be impermeable soils with a plasticity index of between 15% and 20%.
- Trees require water and so can results localized moisture extraction by their roots with the development of cracking. Therefore trees should not be planted and allowed to grow near the road.
- Proper compaction of the subgrade at the appropriate moisture content is typically the most economical means of mitigating problems associated with expansive soils. The potential volume change decreases with increase in initial moisture content. Hence, it is critical to compact them at 1 to 3% above optimum moisture content (AASHTO T99).
- Differential movement of the road surface under culvert structures could arise, as surcharge load on the sub-grade is lighter at culvert sites. Hence, the expansive clay beneath them must be replaced with an inert material, all joints

must be carefully sealed and inlets and outlets well graded to avoid ponding of water.

ii) For non plastic volcanic ashes, with zero (0) value of liquid limit and plasticity index. Their light weight and low cohesion forces among the grains of the soil increase their susceptibility to erosion. Further the high OMC and low MDD values indicate high porosity of the volcanic ashes. Therefore, these materials must be blended with fine plastic material (clay) to bind and fill those voids between aggregates. Care should be given during blending in order to not exceed from the plasticity index value of the material intended for subgrade.

- During construction, when the volcanic ash reach its finished subgrade level it is recommended to observe the subgrade performance by conducting proof rolling test. The length of the subgrade, prepared for proof rolling test should be 150m to 300m at a time (ERA site investigation manual, 2013). Accordingly the number of truck pass in proof rolling can be 2 to 6 or it can be dictated by field conditions. Test area showing more than 12mm of rutting and area of high rebound deflection (pumping) should be determine and reworked. If this is not possible, performing DCP test can indicate the stability of the material. Therefore, DCP can be driven to the depth of subgrade layer at each test location without excavating any soil layers.
- High silt and fine sand content and little (no) clay in volcanic ash may make difficult to obtain the strength/density requirement on the field, as obtained in laboratory investigation. These are because silty soils strength and density changes rapidly with slight change in moisture. Therefore, greater care is necessary to keep the moisture content of the volcanic ash as close as possible to the OMC for compaction.
- Due to high amount of cuts in volcanic ash section (about 35m cut at km 20) and relatively good laboratory test results (high strength and low swelling property) fresh volcanic ashes, can be used as source of ordinary fill material (or other uses) elsewhere in the project road. Therefore, before constructing high fill embankments using non cohesive volcanic ash require detail slope stability analysis such as information of soil strength parameters including cohesion, friction angle, undrained shear strength, unit Weight and long term moisture changes. However, it may lay to significant stability problems. If volcanic ash is

used to construct embankments, then the final surface of the embankment slopes should be covered with imported less permeable soil (clays) and a grass should be plant over the slope in order to minimize erosion and permeability of the side fill.

- Clays that have developed from volcanic ash may also have a fragile structure prone to collapse under embankment loads. Such clays exist from km 11 to 13 in present study area. Therefore, detail investigation should be carry out to analyze the collapse potential in this section prior to any construction.

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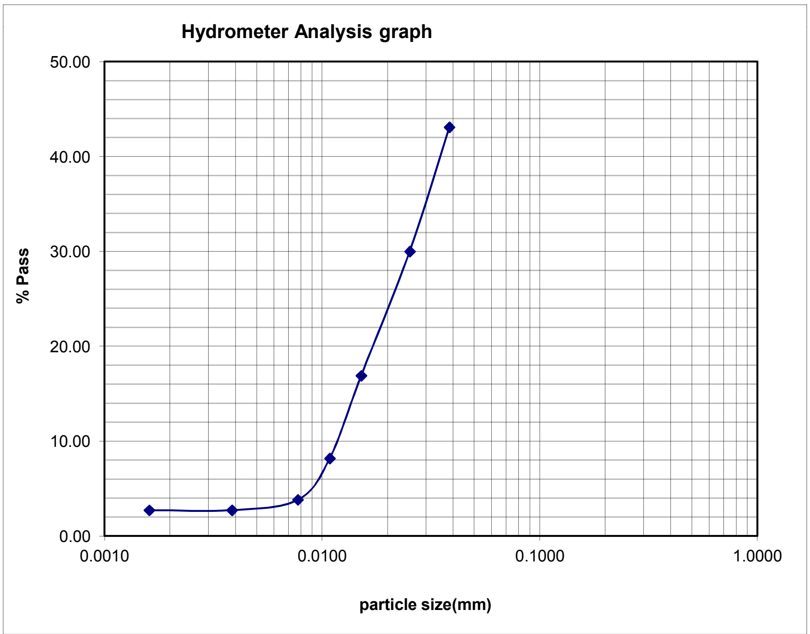
PROJECT: AMBO - WOLISSO ROAD
SUBMITTED BY:
SAMPLE OF: Sub grade
STATION: 16+000
DATE SAMPLED: 15-May-17 **DATE TESTED:** 16-May-17
TEST REQUESTED: Hydrometer analysis & Sieve Analysis (AASHTO T-88)

Hydrometer analysis Data Specific Gravity= 2.33

Time (minutes)	Hydrometer Reading	Temp.	Corrected H.Reading		Correction factor(a)	Effe. Depth of Hydrometer(L)	Values of K	Diameter of soil Particle(mm)	% Finer
			R'	R"					
2	23	20	24.25	19.75	1.0907	12.50	0.01539	0.0385	43.08
5	17	20	18.25	13.75	1.0907	13.50	0.01539	0.0253	29.99
15	11	20	12.25	7.75	1.0907	14.50	0.01539	0.0151	16.91
30	7	20	8.25	3.75	1.0907	15.20	0.01523	0.0108	8.18
60	5	20	6.25	1.75	1.0907	15.50	0.01523	0.0077	3.82
250	2	20	3.25	-1.25	1.0907	16.00	0.01523	0.0039	2.73
1440	2	20	3.25	-1.25	1.0907	16.00	0.01523	0.0016	2.73

Total oven Dry mass= 50



	Diameter of soil Particle (mm)	% Pass
Hydrometer Analysi	0.038	43.08
	0.025	29.99
	0.015	16.91
	0.011	8.18
	0.008	3.82
	0.004	2.73
	0.002	2.73

1.Particles larger than 2mm = 0%
2.Coarse Sand 2mm - 0.425mm = 0%
3.Fine Sand 0.425mm - 0.075mm = 0%
4.Silt 0.075-0.002mm = 97%
5.Clay smaller than 0.002mm = 3%

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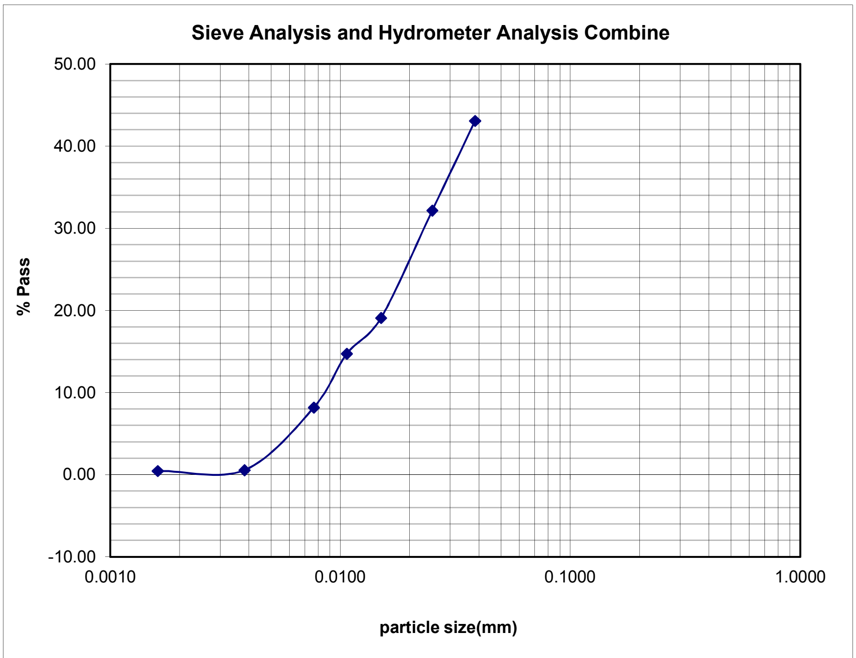
PROJECT: AMBO - WOLISSO ROAD
SUBMITTED BY:
SAMPLE OF: Sub grade
STATION: 20+000
DATE SAMPLED: 15-May-17 **DATE TESTED:** 16-May-17
TEST REQUESTED: Hydrometer analysis & Sieve Analysis (AASHTO T-88)

Hydrometer analysis Data Specific Gravity= 2.33

Time (minutes)	Hydrometer Reading	Temp.	Corrected H.Reading		Correction factor(a)	Effe. Depth of Hydrometer(L)	Values of K	Diameter of soil Particle(mm)	% Finer
			R'	R"					
2	23	20	24.25	19.75	1.0907	12.50	0.01539	0.0385	43.08
5	18	20	19.25	14.75	1.0907	13.30	0.01539	0.0251	32.18
15	12	20	13.25	8.75	1.0907	14.30	0.01539	0.0150	19.09
30	10	20	11.25	6.75	1.0907	14.70	0.01523	0.0107	14.72
60	7	20	8.25	3.75	1.0907	15.20	0.01523	0.0077	8.18
250	3	20	4.25	-0.25	1.0907	15.80	0.01523	0.0038	0.55
1440	2	20	3.25	-1.25	1.0907	16.00	0.01523	0.0016	0.45

Total oven Dry mass= 50



	Diameter of soil Particle (mm)	% Pass
Hydrometer Analysi	0.038	43.08
	0.025	32.18
	0.015	19.09
	0.011	14.72
	0.008	8.18
	0.004	0.55
	0.002	0.45

1.Particles larger than 2mm = 0%
2.Coarse Sand 2mm - 0.425mm = 0%
3.Fine Sand 0.425mm - 0.075mm = 0%
4.Silt 0.075-0.002mm = 99%
5.Clay smaller than 0.002mm = 1%

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PROJECT: AMBO - WOLISSO ROAD

SAMPLE OF: Sub grade

STATION: 7+000

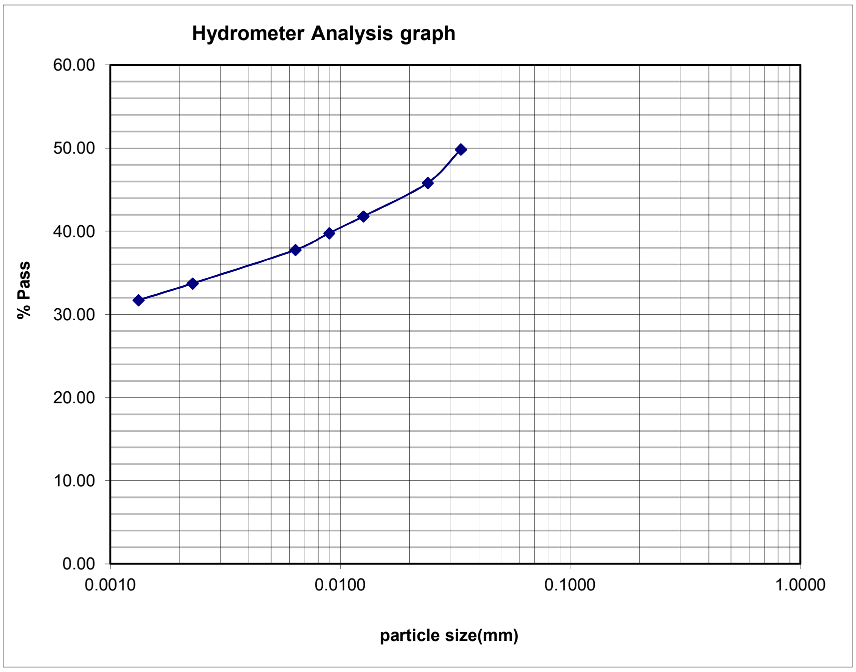
DATE SAMPLED: 15-May-17 **DATE TESTED:** 16-May-17

TEST REQUESTED: Hydrometer analysis & Sieve Analysis (AASHTO T-88)

Hydrometer analysis Data Specific Gravity= 2.62

Time (minutes)	Hydrometer Reading	Temp.	Corrected H.Reading		Correction factor(a)	Effe. Depth of Hydrometer(L)	Values of K	Diameter of soil Particle(mm)	% Finer
			R'	R"					
2	28	20	29.25	24.75	1.0069	11.70	0.01380	0.0334	49.84
4	26	20	27.25	22.75	1.0069	12.00	0.01384	0.0240	45.81
15	24	20	25.25	20.75	1.0069	12.40	0.01384	0.0126	41.79
30	23	20	24.25	19.75	1.0069	12.50	0.01384	0.0089	39.77
60	22	20	23.25	18.75	1.0069	12.70	0.01384	0.0064	37.76
480	20	20	21.25	16.75	1.0069	13.00	0.01384	0.0023	33.73
1440	19	20	20.25	15.75	1.0069	13.20	0.01384	0.0013	31.72

Total oven Dry mass= 50



	Diameter of soil Particle (mm)	% Pass
Hydrometer Analys	0.033	49.84
	0.024	45.81
	0.013	41.79
	0.009	39.77
	0.006	37.76
	0.002	33.73
	0.001	31.72

1.Particles larger than 2mm = 0%
2.Coarse Sand 2mm - 0.425mm = 0%
3.Fine Sand 0.425mm - 0.075mm = 0%
4.Silt 0.075-0.002mm = 66%
5.Clay smaller than 0.002mm = 34%

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ENGINEERING GEOLOGICAL CHARACTERIZATION AND SUITABILITY ANALYSIS OF SUBGRADE SOIL – A CASE STUDY ON SELECTED SECTION OF AMBO –WOLISSO ROAD PROJECT, SOUTH WESTERN ETHIOPIA

A THESIS SUBMITTED TO THE SCHOOL OF GRADUATE STUDIES OFADDIS ABABA UNIVERSITY IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTERS OF SCIENCE IN ENGINEERING GEOLOGY

BY

BIRUK MITIKU

(GSR/0459/08)

MAY, 2017