Engineering Geological Characterization and Suitability Analysis of Subgrade Materials -A Case Study of Sembo-Shola Gebeya-Gindeber Road, Central Ethiopia

Misgana Oljira

A Thesis Submitted to School of Earth Sciences

Presented in Partial Fulfillment of the Requirements for the Degree of Master of Science (Engineering Geology)

ADDIS ABABA UNIVERSITY

Addis Ababa, Ethiopia

May, 2014

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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. Trufat H/Mariam and Dr. Fekerte Arega, School of Earth sciences, Addis Ababa University during the year 2014 as part of Master of Science Program in Engineering Geology in accordance with the rule and regulation of the institute. I further declare that this work has not been submitted to any other University or institution for the award of any degree or diploma and all sources of materials used for the thesis have duly acknowledged.

Misgana Oljira

Signature_______________________________

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

May 2014

SIGNATURE PAGE

Addis Ababa University
School of Graduate Studies

This is to certify that the thesis prepared by **Misgana Oljira**, entitled: "***Engineering Geological Characterization and Suitability Analysis of Subgrade Materials -A case study of Sembo-Shola Gebeya-Gindeber Road, Central Ethiopia"*** and submitted in partial fulfillment of the requirements for the Degree of Master of Science (Engineering Geology) complies with the regulations of the University and meets the accepted standards with respect to originality and quality.

Signed by the Examining Committee:

Examiner________________________Signature ______________ Date _______________

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Abstract

Characterization of Subgrade soils is one of important parameters in the design and construction of long life pavement structures. Thus the characterization involves identification of problematic soils to design accordingly and to take counter measures. The most common problematic soils in flexible pavement construction are expansive soils that change in volume under moisture content and ground water fluctuation, that cause severe damages to infrastructure unless proper measures are taken in the design and construction phases.

The subgrade soils are characterized in Sembo-Sholagebeya-Gindeber upgrading project located in Northern Shoa connecting Kimbibit and Hagremariam woreda bounded by UTM coordinates of 538556E, 1040338N and 544927E, 1018404 N that ranges from 2600m to 3000m above sea level and geologically covered by volcanic rocks of mostly basaltic. The main objective of this study were: (1) to characterize the engineering properties of the subgrade soils, (2) to identify the problematic soil and analyze its suitability for pavement, (3) to propose a remedial measure and (4) to propose alternative pavement thickness.

To achieve these objectives, the pre-field, field and post field activities including literature survey, primary data, sample collection and laboratory works were conducted during the study. Accordingly 82 samples have been collected at an interval of 500m and tests are conducted at project geotechnical laboratory for the determination of Atterberg limits, grading, MDD and OMC, CBR and CBR swell values. In addition 5 samples are chemically analyzed using XRD, 2 rock samples for thin section and 2 samples for triaxial.

Finally interpretations have been made with the data obtained from field works and laboratory investigations, supported by previous studies, researches and standard manuals of subgrade characterization. From the laboratory investigation, it has been concluded that about 41% on plasticity, 62% on CBR, 64% on CBR swelling, 46% on Group index of the sub grade soils are found to be unsuitable and all are highly compressible. Hence remedial measures such removal, mechanical compaction, proper drainage, chemical treatment, blending with existing soils and proper pavement thickness design are proposed in this paper.

Key words: Subgrade, Suitability, Characterization, Pavement, Engineering properties.

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(Misgana Oljira)

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❖ Acronyms

AASHTO	AASHTO, American Associations state Highways & Transportation Officials
ASTM	American Society of Testing and Materials
ATU	Alberta Transportation and Utilities
CBR	California Bearing Ratio
DCP	Dynamic Cone Penetration
ERA	Ethiopian Roads Authority
FHWA	Federal Highways Administration
GI	Group index
LL	Liquid Limit
LS	Linear Shrinkage
MDD	Maximum Dry Density
NP	Non Plastic materials
OMC	Optimum Moisture Content
PI	Plasticity Index
PL	Plastic Limit
SPT	Standard Penetration Tests
TRL	Transport Research Laboratory
UCS	Unconfined Compressive Strength
USCS	Unified Soils Classification System
UTM	Universal Transverse Mercator
UU	Unconsolidated Undrained
XRD	X-ray diffraction

CHAPTER I INTRODUCTION

1.1 Background of the Problem

Any engineering project that takes place on or in the ground has to be concerned with geotechnical engineering or engineering geology studies. This branch of science is concerned with earth's materials, soil mechanics, its geological formation and response under any stress applied from external structures. A geotechnical engineer should examine subsurface formations and locations to determine safety, stability and environmental concern of engineering structures to be placed on selected geological setup.

The study of the engineering, chemical and physical properties of soil helps in managing resources while working with a particular kind of soil. Since all soils have different engineering properties under different condition, working with them requires understanding soil properties in detail. During the planning of any engineering structures, characterization of its basement, especially subgrade materials of road structures is the first activity.

Engineering geological characterization of subgrade material is the important point on safe and long life pavement design and construction activities, since the stability and serviceability of the structures is a function of sub grade quality.

Accordingly, characterizations of sub grade materials have been done through the field visual investigation, field description and laboratory investigation of different engineering parameters which are closely correlated with its quality and performance. To perform such laboratory tests the samples were collected with standardized interval with the consideration of geological variability.

Depending on certain consideration, different countries used different sampling standard and laboratory values under different moisture content, which is dependent of site condition. To avoid unreality of the result, careful sampling, testing, interpretation and analysis of the required parameters should be adopted according to predefined standard. The commonly used manuals for pavement design and material characterization in our country are AASHTO manuals, ERA manuals, Tanzanian and Kenyan pavement design manuals in different sense. These manuals have their own similarity and differences.

ERA design manual (2002), the soil samples are collected at an interval of 1km to 2km during design and 0.5km to 1km during construction for different parameter tests. In addition it has been recommended that the sub grade should be categorized into homogeneous section of uniform CBR value and the pavement thickness should be designed accordingly. The sub grade materials could be different strength class depending on CBR values, hence different stabilization methods can recommended for different classes.

According to AASHTO (1993), the spacing of sample collection for sub grade material characterization should be at the range of 150m to 450m during construction phase, but this should be depends on geological variability and requirement of detail study. Additionally CBR, OMC, MDD and swelling potential should be determined before designing.

The United Republic of Tanzania pavement design manual, (1999) recommended investigation of soils along the road center line to be carried out in order to establish its strength in terms of CBR. The manual gives a minimum of two CBR strength tests per kilometer (500m interval) for paved trunk roads, minimum of one CBR strength tests per kilometer for other roads and a minimum of a single CBR strength testing per 2 kilometers for gravel roads.

According to Kenyan road design manual (1979), subgrade strength values should specified based on either CBR values measured after 4 days soak or measured at optimum moisture content where it has been established that prolonged soaking may occur.

In the process of characterization, different techniques and procedures are applied for interpretation of sub grade soil condition. These interpretation techniques are often site-specific and are influenced by geological, topographic, and climatic conditions of the area.

Subgrade materials are typically characterized by their resistance to deformation under load, which can be either a measure of their strength (the stress needed to break or rupture a material) or stiffness (the relationship between stress and strain in the elastic range or how well a material is able to return to its original shape and size after being stressed). In general, the more resistant to deformation a subgrade is the more loads it can support before reaching a critical deformation value. Three basic subgrade stiffness/strength characterizations are commonly used are California Bearing Ratio (CBR), Resistance Value (R-value) and elastic (resilient) modulus. Although there are other factors involved when evaluating subgrade materials such as swell (in the case of certain clays) and stiffness are the most commonly used characterization.

The geological setup of the area to be characterized might highly influence the types of engineering parameters' to be considered during the design and construction of engineering materials. Some lithological units are problematic in nature due to their distinctive character up on their exposure to environmental and moisture content change. These subgrade problems are associated with susceptibility to erosion, swelling, collapsibility, bearing capacity and incompetency. Expansive soil is one of the most problematic soils in nature due to their swelling nature on saturation and shrinkage on drying, which develop cracks on overlying structures and results in failure of the engineering structures.

The use of such problematic soil as sub-grade materials requires a detail engineering geological characterization by undertaking different engineering parameters, to reduce its effects on structures by taking some stabilization methods; hence it requires a close field investigation, proper sampling and accuracy of testing machine. The properties of expansive soil that should be determined to evaluate its anticipated effects on engineering structures are: - consistency; (liquid limits (LL), plasticity index (PL)), proctor density (MDD), optimum moisture Content (OMC), California bearing ratio (CBR) and swell values.

The present research is aimed in conducting a detail engineering geological characterization and analyzing the suitability of the subgrade soils along selected section from-Sembo to Sholagebeya of Sembo-Shola gebeya-Gindeber road, which passes through different lithological unit, drainage system and topographic setup that could be affects the performance of the pavement. In addition the research will delineate the problematic area and suggest some remedial measurements depending on the result of tested parameters and visual inspection on field.

1.2 Problem of the statement

Pavement structures represent a significant infrastructure investment that is critical to the wellbeing, growth and expansion of any geographic location. Hence such pavements are expected to be durable and resilient, and to perform satisfactorily throughout its service lives. In designing such pavements, several factors are assessed and one such primary factor is subgrade materials properties. The subgrade materials on which pavement and traffic loads are rest could be source of problem if it is less competent, high plasticity, low strength/ bearing capacity, highly compressible, susceptible to erosion and highly expansive in nature. Expansiveness of subgrade is the most common problem of subgrade soils on stability of pavement.

The road construction reality in Ethiopia context is mostly focused on the quality of upper pavement layer by giving less attention to the lower pavement layers, this is due to different standard manuals which compromise the quality of sub grade and sub base (Beniyam Alemue, 2010). Thus most of the pavement failures resulted due to poor engineering properties of sub grade

Expansive soils are materials that increase and decrease significantly in volume as moisture content changes. The expansive soils swell if their moisture content increases and they shrink if their moisture content decreases.

Expansive soils are distributed all over the world (Chen, 1988; McKeen, 1976). The distribution is generally a result of geologic history, sedimentation and local climatic conditions. The authors indicated that the areas with the most severe problems from expansive soils are those with semiarid and arid climates of the tropical region that have poor drainage system.

Expansive soils, or shrink-swell soils, cause a damage of property and life throughout the world under moisture fluctuation (Krohn and Slosson, 1980 as cited in Habtamu Soloon, 2011). The moisture change can be due to natural effects such as normal seasoned effects or the plant root activity, or manmade effects such as leaking from underground water pipes, or a deficient storm water drainage system. Currently, swelling of clay soil is a very troublesome subject in various fields of engineering.

Expansive soils, which swell on absorption of water and shrink on removal cause severe damage to light engineering structures unless measurement have to be adopted beforehand (**http://wwweng.uwyo.edu/classes/sp2011/ce4630/reports/wes/expansive/expansive.htm**)

The problems that arisen have often been caused by the lack of detail engineering parameter characterization to know swelling mechanism, combined with an underestimating of the high pressure existing from swelling clays and inappropriate treatment depending on the laboratory result. Most of the time, engineering project identify and analysis suitability of expansive soils by physical observation without laboratory tests.

But physically, soils of different engineering properties looks similar and leads to miss designing and inappropriate providing of mitigation measures. Hence consideration of different parameters that affects the swelling of soils should be taken.

However expansiveness of subgrade materials is not only the source of problem on pavement stability, but also the subgrade materials have properties of erodibility, low bearing capacity, compression /settlement/ under stress and sliding.

Most of subgrade material problems are related with fluctuation of water content. The fluctuation of water content may be due to seasonal variations (often related to rainfall and the evapotranspiration of vegetation), or brought about by local site changes such as leakage from water supply pipes, reservoir or drains, changes to surface drainage and landscaping.

The problem of expansive soils, susceptibility o erosion, sliding and bearing capacity on stability and safety of engineering structures can be reduced or mitigated by thoroughly characterizing the soils and treating using different methods depending on the relative economic feasibility of the mitigation methods and types of problems identified.

In present study area swelling, low bearing capacity, compressibility and susceptibility to erosion have been identified as the most common problems associated with subgrade materials, Hence the mitigation or preventive measures have been recommended accordingly.

1.3 Objectives of the Study

1.3.1 General Objective

The main objective of this research is to characterize the index properties, strength and bearing capacity of sub-grade soils for constructing the pavement structure and to evaluate the suitability of selected area for selected pavement type.

1.3.2 Specific Objectives

- To characterize the geotechnical parameters of the sub-grade materials in the selected area.
- To check the suitability of the sub-grade materials for pavement construction depending on the amount of traffic loads and strength of the materials from the result of parameters to be tested.

- To categorize sub-grade soils and materials in to appropriate traffic class depending on result of CBR.
- To evaluate the general suitability of selected area for selected pavement type.
- To confirm the pavement working design with the actual site condition/ recommending pavement thickness, depending on the result.
- To suggest some countermeasures depending on the economic feasibility of stabilization methods for un-suitable parts of the sub-grade soils and materials to reduce their effects on stability of pavement if any un-suitability exist.

1.4 Expected outcome of the Study

After the detailed characterization of sub grade material for pavement construction, the designer or contractor can adopt the results of this research to design safe engineering structures of required service time without a possible threat of deterioration. The expected outcome of this research will be;

- The engineering geological characteristics of subgrade materials.
- The general suitability of the selected site of the project will be assessed.
- The verified pavement thickness of the road structures will be recommended
- Recommendations of stabilization method for the unsuitable portion of the subgrade materials will be forwarded.

1.5 Methodology

Most newly developed pavement and overlay design procedures require engineering-geological characterization of sub-grade materials. This requirement resulted in the development of several field and laboratory tests to simulate actual field conditions in the laboratory. In present study three phases of activities have been performed in order to come up with the goals of the objectives; these phases are;

1.5.1 Desk study

Desk study includes;

- Reviewing and compiling literatures, papers, standard manuals and related books in order to get a conceptual frame work on general methods of subgrade characterization and flexible pavement design was conducted.
- Collecting, reviewing and compiling of available maps, such as topographical, geological and other maps of the study area has been done.
- Secondary data such as geological and meteorological data has been collected from relevant sources.

1.5.2 Field work

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

In this phase the main activities are;

- Field description and soil extension survey have been done
- Collection of representative samples that used in laboratory test was conducted.
- Field test of some engineering parameters to evaluate the supporting characteristics of in situ pavement layers.

During the field work, instruments such as GPS, geological map and topographic map were used in order to locate the exact position of sampling and to get some inputs for the description on the local geology and soils conditions of the area by detail field description and measurements

Some field recorded data such as depth at which subgrade materials has been taken, its physical characteristics and structural interruption (discontinuity) has been recorded and measured for latter description of the geological setup of the area. Additionally some physical properties such as hardness, strength and other have been noticed.

Different literatures indicate that the representative samples should be collected with different interval for different purpose. However many standard of sampling subgrade materials are used by different countries depending on geological (lithological) variability and types of pavement to be designed. For examples According to the Ethiopian Roads Authority (ERA) site investigation manual (2002), soil samples for sub grade material characterization shall be taken at 0.5km interval for identification test and 1km interval for CBR tests.

As per American Association of State Highway and Transportation Officials (AASHTO), Design guide (1993) sample spacing shall be in the range from 150m to 450m interval during construction phase (Nibret Chane, 2011).

Samples in this study were collected depending on geological variability of the area, hence spacing and depth of sample collection has been decided during field visit. So samples are collected at an interval of approximately 0.5km at a depth between 20 and 100cm for all parameters. For the sake of simplicity the samples taken from one station have been tested for all considered parameters.

Along with sample collection, description of the site has been done as soon as the sample is taken, which help for latter confirmation of the site condition with laboratory results and help to have information on parameters that can be changed under variable environmental conditions.

1.5.3 Post field work

After the collection of field data, the parameters that characterize sub-grade material were tested in site laboratory of AKIR contractor's on representative collected samples. The sub-grade properties including CBR, CBR swelling, Atterberg limits, grain size distribution, moisture-density relation and field moisture content were obtained and the interpretation followed by analysis of this data have been done in order to geotechnical characterize the sub-grade materials.

Additionally geochemical tests such as X-ray diffraction, Triaxial test and petrographic analysis have been conducted to confirm the results with the engineering properties. Finally, after all required test and data are collected; the suitability analysis of the materials for pavement construction has been done according to different design parameters.

In general the parameters that will be tested and examined to characterize sub-grade materials depending on existing soil types (AASHTO, 1993; ERA, 2002) are;

- Atterberg limits (liquid limit, plastic limit and plasticity index)
- Grain size analysis (sieve analysis),
- Modified proctor test for compaction.
- California Bearing Ratio (CBR)
- Loaded swelling potential (CBR swelling).

1.6 Scope and Limitation of the Study

The main scope of the present study is limited to providing information on geotechnical characteristics of subgrade materials in selected area along road section. In order to characterize the sub grade materials, depending on geological variability of the study area and standardized sampling manual (ERA, 2002) for different test; the study have been conducted by collecting of soil samples with different intervals along the routine corridor for laboratory testing of various parameters.

In this thesis, an attempt has been made to consider the effects of engineering geological characteristics of subgrade materials on safety of pavements, response of different soils to traffic loads. Finally after characterization of sub grade materials and identification of unsuitable soils/rocks some countermeasures are proposed to reduce the problems of such soils/rocks on pavement stability and serviceability.

However, every effort was made to perform the present study in a scientific principle and logical manner, it has been carried out under limited time and resource by considering limited parameters, hence these limitations affects the completion time for materials testing, reporting the results and accuracy of the result. Therefore, it is strongly recommended that the results and the findings of the present study must be considered as an exhaustive only for roads in the construction phase. And further studies and additional tests are required before implementing these results or finding to other engineering project, hence shall be considered as indicative only.

In addition Limitation on literatures, published report and other secondary data have been faced during the present study, since the area was not well studied in engineering geological sense previously.

1.7 Scheme of presentation

This paper has six chapters with different subtitles, which all focused on characterization and suitability analysis of sub grade materials along selected road section.

Chapter 1; Introduce the present study generally in terms of its objectives, problem statement, methodology followed in the study, application of expected result, scope of the study and limitation faced during the study. The main objective of this chapter is to make the reader clearly understand the general background of the research.

Chapter 2; Give a brief reviews on certain literatures and standard manuals of some countries used in flexible pavement design and materials characterization. The main points of this chapter is to come up with some conceptual framework related to sub grade characterization, parameters to be considered and suitability analysis of a given materials for a given engineering structures. Additionally the chapter gives emphasis to the genesis of the present study.

Chapter 3; Describe briefly the study area in terms of its geological setup, topography, climatic condition and seismicity as the supplementary to general description of the area.

Chapter 4; Presents the actual work of the research that the researcher conducted both in the field and laboratory activities in order to characterize the sub grade materials with different parameters which in turn used in suitability analysis of the site and sub grade material for flexible pavement.

Chapter 5; present the discussion, interpretation and analysis of the result in terms of suitability analysis and characterization of existing materials with respect to standards.

Chapter 6; present the conclusion and recommendation suggested by the researcher depending on the result of field and laboratory investigations.
Finally annexes and reference used during the present study are given.

CHAPTER II LITERATURE REVIEW

2.1 Preamble

To have a conceptual framework and general guideline on the characterization of various types of sub grade materials and methods of engineering geological characterizations, during the present study different published, unpublished, books, magazines, journals and on-line materials available on the internet are revised in details. Previous studies and practical works conducted in Ethiopia that are of similar nature to the present research were assessed and included as part of the literature review.

Additionally some standard manuals such as AASHTO, ERA manuals of subgrade characterization, ERA manual on site investigation, Tanzanian pavement design manual, Kenyan design manuals and some academic research are closely revised to understand the methods of geotechnical characterization of sub grade materials, stabilization methods of problematic subgrade and engineering geological parameters to be considered in pavement design, construction and maintenance.

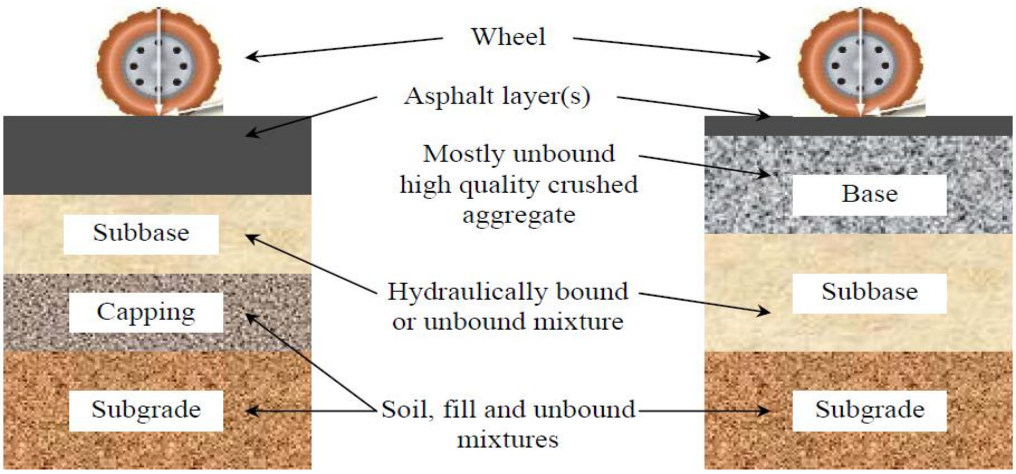
The surveyed materials and papers of different researchers indicate that, engineering geological (geotechnical) characterization of materials in the construction of any engineering structures should conduct prior to any activities, since the safety and stability of the structure highly affected by the construction materials and construction site condition. Hence laboratory and site test of parameters that influence the engineering properties and performance of natural and filling materials are critical thing in safe design of engineering structures on selected site.

In general, subgrade characteristics and performance are influential in pavement structural design. Characteristics such as load bearing capacity, strength, moisture content, index properties and expansiveness will influence not only structural design but also long-term performance and cost of maintenance of pavement. Hence the thoroughly characterization of sub grade materials is important in economic design of safe pavement and reduction of damage on pavements due to the engineering properties of sub grade.

2.2 Sub grade soils

Subgrade soil is the integral part of the road pavement structure which provides support to the pavement sections (ERA, 2002). The subgrade materials and its different properties are very important in the pavement structural design. The major function of the subgrade is to provide the support to the pavement against traffic loading and for this its engineering properties should possess sufficient stability under adverse climate and heavy loading conditions.

When a soil is used in the embankment construction, along with stability, incompressibility and strength are also an important factor as differential settlement may cause failures. The soil or subgrade is therefore considered as one of principal highway material in supporting distributed loads. Depending on economic feasibility and amount of loads to be imposed, materials with different quality can be used in pavement design. The selection of materials quality and thickness more or less depends on economic level of a country. Thus different countries uses different materials for pavement layers.



(a) Industrialized countries **(b) Developing countries**

Figure 2.1 Cross section of flexible pavement structures and materials selection (Alemgena A., 2011)

The main objectives of pavement designing with different layers by incorporating different quality of construction materials is to provide support for dynamic traffic loads by distributing loads in the layers. Depending on the nature of traffic loads (intensity and magnitude), site condition and economic feasibility, the designer can design a pavement with subgrade, sub base, base course and surfacing in which quality of materials increase from bottom to top. In some flexible pavement design, addition of geotextile is provided to stabilize surface of subgrade materials. Most literatures give the problem associated with subgrade soils to be only the expansiveness, but also there are different problems associated with natural subgrade soils.

2.3 Problems associated with subgrade soils

Most of the time pavement distresses are associated with subgrade materials problem that resulted in pavement failure within short period of time. However pavement distress is not only the result of subgrade materials. In general the common pavement distress those are directly or indirectly related with subgrade problems are presented in table 2.1. These problems range from very minor to very serious and to a complex one for maintenance (Zeleke Tadesse, 2013).

Table 2.1 Categories of pavement distress

Category	Distress types
Cracking	Longitudinal, fatigue, transverse, reflective, block, edge
Deformation	rutting, corrugation, shoving, depression and overlay bumps
Deterioration	Delamination, potholes, patching, raveling, stripping, polished
Mat problem	Segregation, checking and bleeding
Seal coats	Rock loss, segregation, bleeding/fat spots

Source; (Maintenance Technical Advisory Guide, 2011)

As aforementioned the failure of flexible pavement is resulted from different factors such as properties of subgrade materials, properties of asphalt materials, environmental effects, overloading of uncontrolled traffic and faulty in structural designing. Since the traffic and upper layers exert loads both directly and indirectly on underlying subgrade, the failures of the pavement generated from subgrade materials. Hence subgrade should be characterized in detail and the problem associated with such soils should be identified to overcome the problems. The most common subgrade associated problems are briefly discussed below. This does not mean that other problems (eg, slope failures, undermined ground, etc) may not occur.

2.3.1 Volume change (expansiveness)

Damages caused by expansive soils are almost entirely restricted to light structures and is a particular problem with transportation facilities (Gourley and Schriener, 1993 as cited in Habtamu Solomo, 2011)

The problem associated with expansive soil is a result of volumetric change due to water absorption and releasing under change in environmental conditions (Seed et al., 1962). The water can percolate into the soils from different sources such as precipitation, ground water seepage, some utilities and reservoirs. Since smectite minerals are bonded by weak hydrogen bonds, it forms some spacing between the tetrahedral and octahedral which allow water to inter and layer interchange to occur that makes the soil to expand upon wetting, while illite have strong bond and water cannot cause expansion, hence illite groups are classified as non-expansive CLAY mineral.

The simplest way of identifying the presence of expansive soils is through field observations where the surface expression of cracking in dark grey, black or sometimes red soils is evident, but it doesn't mean that all cracking is a cause of expansive soils.

2.3.2 Collapsible (Compressible) soils

Collapsible soils result from a unique condition in which "bridges" of fine materials (usually CLAYs or iron oxides) within a framework of coarser and harder particles (usually quartz) become weak when wet and collapse under load. The important condition is that the material must be in a partially saturated condition and then wetted up and loaded simultaneously, which is a common situation beneath road structures (Brink, 1979 as cited in Fasil Abagena, 2003).

As some literatures show, such soils are characterized by low density due to large number of voids separating the quartz framework. Typical collapsible soils have densities of less than about 1.6 g/cm^3 (mostly in the range 1 to 1.585 g/cm^3) (AASHTO, 1987). In addition, usually more than 60% of the mass of the material lies in the 0.075 to 2 mm range and less than 20% is finer than 0.075 mm sieve.

In flexible pavement 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.

The collapsibility of subgrade materials is related with settlement which can be recognized by using oedometer test that used to determine the settlement potential under specific loads (Fasil Abagena., 2003). The main objective of determination of collapsibility is to implement measures to disrupt the collapse structure as far as possible and to produce a subgrade that is relatively uniform, so that differential settlement can be minimized.

2.3.3 Susceptible to erosion (Dispersive soils)

Dispersive soils are those soils that, when placed in water, have repulsive forces between the CLAY particles that exceed the attractive forces. This result in the colloidal fraction going into suspension and in still water staying in suspension, in moving water, the dispersed particles are carried away. This obviously has serious implications in earth dam engineering by forming piping, but is of less consequence in road engineering, since the subgrade is less subjected to moving water. However, in long periods of time the water can reach subgrade along the horizontal drains and results in piping of the materials.

Purely erodible soils will not necessarily disintegrate or go into dispersion in water. They tend to loosen material as a result of the frictional drag of water flowing over the material exceeding the cohesive forces holding the material together. Problems are thus mostly associated with poor culvert and drainage design.

2.3.4 Soft CLAYs

This problem is common in lagoon areas where thick alluvium deposits of CLAY materials are found. Soft CLAYs are generally, but not necessarily saturated and normally consolidated to lightly over-consolidate (as a result of fluctuating water tables). The materials thus have low shear strengths, are highly compressible and their low permeability result in time-related settlement problems.

Moreover there are problems associated with subgrade materials such as karst area of carbonate rocks and salinity of soils due to availability of soluble salts. However these problems are not as such common in present study area. To certain extent, there is a carbonate rock dominated area in Ethiopia that makes the subgrade materials to collapse.

Generally, one aspect common to all of the problems discussed above is the moisture content whether it is steady in-situ moisture or fluctuating moisture conditions as a result of seasonal change. A good understanding of the moisture conditions and environment needs to be defined during any investigation to consider and incorporate these problems in subgrade materials characterization and problem treatment.

2.4 Subgrade characterization parameters

In order to evaluate the above discussed problem associated with subgrade materials, the materials should be characterized and evaluated prior to construction and during construction.

Subgrade materials can be characterized by their engineering properties such as strength, atterberg limits, grain size, moisture content, compact ability under loads, swelling potential and dry density which can be tested in the field or laboratory to classify and analysis soils or rocks in terms of suitability for a given engineering structure construction on, in or with it.

The engineering performance of sub grade materials is a function of its moisture content, load bearing capacity and shrinkage or and swelling potential, which are a function of the nature of soils and environmental condition. Thus subgrade can be characterized in terms of classification tests (Atterberg limit and grain size distribution), strength, swelling potential and degree of compaction.

2.4.1 Soil classification

The most widely used soil classification systems for engineering purposes are American Association of State High way and Transportation Officials (AASHTO) and Unified Soil Classification System (USCS). The AASHTO system of soil classification comprises seven groups of inorganic soils from A- l to A-7 with 12 subgroups in all. The system is based on particle size distribution, liquid limit and plasticity index (AASHTO, 1993)

On the other hand, the Unified Soil Classification System is based on the recognition of the type and predominance of the constituents considering grain-size, gradation, plasticity and compressibility. It divides soil in to three major divisions: coarse-grained soils, fine grained soils and highly organic soils. In this classification paired letter symbols are used to show the grain size and degree of gradation. Both classification based on Atterberg limit and grain size distribution.

Atterberg limit is a measure of water content at which the soils alter their physical state and behavior. Moisture that exists within a pavement structure or within the subgrade soils beneath a pavement may be generated from many sources such as percolation through cracks, permeability of pavement surface, percolation from side slope, lateral movement of water in a shoulder and shallow water table.

Furthermore, soil grain size distribution analysis is a geotechnical process that allows civil engineers and geoscientists to classify soils by determining the different percentages of grain diameters in the sample. The process utilizes two tests; Mechanical Sieve Analysis which is used for larger diameter aggregates (> 0.075 mm) while the Hydrometer is used for aggregates passing the last sieve (diameters less than 0.075 mm).

2.4.2 Subgrade Strength

The characterization of subgrade strength used to design a structure that can be supported by the existing soils in the site. In construction of engineering structures, geotechnical engineer determine the strength of soil foundation using field and laboratory tests. The commonly used strength determination methods are California Bearing Ratio (CBR), Resistance Value (R-value) and elastic (resilient) modulus.

- **Californian bearing ration (CBR) Strength Test**

The strength of a soil or subgrade can be determined by using a test known as California Bearing Ratio Test (CBR) which was developed in California in the year 1930's and it is way to determine the soil properties such as density and strength. Mostly all the design standards are based on the value of CBR for the subgrade (ASTM, 1997)

As aforementioned CBR is the parameter that used to categorize sub grade materials into different strength classes according to design manuals such as ERA (2002), AASHTO (1993), Kenyan (1987) and Tanzanian (1999) pavement design guides.

- **Resilience modulus (M_R) test**

The 1986 AASHTO Guide for Design of Pavement Structures provided substantial motivation to the test by declaring that resilient modulus (M_R) is the definitive property to characterize both subgrade soil and flexible paving materials. M_R is a parameter of subgrade characterization that is sensitive to soil type, its moisture content, level of compaction, density and stress level to which the soil is subjected.

The resilient modulus (MR) has become the standard parameter to characterize unbound pavement materials. It is the ratio of deviator stress (D_s) to the recoverable strain (R_s) (K.P. George, 2006 as cited in Alemgena Alene, 2011)

$$MR = D_s/R_s$$

Since the resilient modulus test equipment is currently not present in many laboratories, researchers have developed correlations to converting CBR values to approximate M_R

values [AASHTO 93]. The correlation considered reasonable for fine grained soils with a soaked CBR of 10 or less is:

$$M_R \text{ (MPa)} = 10.3 * CBR \quad \text{eq. 2.4}$$

2.4.3 Compaction test (modified and standard protocol test)

Compaction is the process by which the bulk density of an aggregate of matter is increased by driving out air. For any soil, for a given amount of compactive effort, the density obtained depends on the moisture content. The relationship between density and moisture content can be determined using proctor test.

Proctor test is a laboratory test method used to measure the relationship between the dry density and moisture content of a soil. These laboratory tests generally consist of compacting soil at known moisture content into a cylindrical mold of standard dimensions using a compactive effort of controlled magnitude. The soil is usually compacted into the mold to a certain amount of equal layers, each receiving a number blows from a standard weighted hammer at a specified height. This process is then repeated for various moisture contents and the dry densities are determined for each. From this relationship (a plotted curve) we can estimate the maximum dry density and optimum moisture content of the soil.

2.4.4 Swelling potential

CLAY soils such as bentonitic shales are excessively expansive under moisture content change due to water interference; hence it requires special attention during designing of structures along such soils. These materials contain minerals which result in volume changes (swelling and shrinking) with changes in moisture content.

Compaction of this soil type at moisture contents slightly in excess of optimum moisture content, using of soil modifier such as lime or Portland cement and controlling the intrusion of moisture into such soils is of major importance in order to mitigate swelling effects.

2.5 Overview of subgrade characterization and flexible pavement design standards

Depending on the traffic loads (intensity, magnitude), sub grade material characteristics, environmental conditions and economic feasibility different countries of the world develop their own standardized manuals with variable materials' parameters consideration. Most of the standardized manuals are up dated from time to time in accordance with technology and increasing in amount of traffic loads. However some countries adopt the same standard from the others. Hereby highlight of some common subgrade characterization and pavement design manuals that are relevant to present study are briefly revised below.

2.5.1 Overview of AASHTO pavement manuals

In 1960, the American Association of State Highways and Transportation Officials (AASHTO) Road Test was the seminar experiment from which AASHTO design guide was developed. The main objective of the AASHO Road Test was to determine the relation between the number of repetitions of specified axle loads (different magnitudes and arrangements) and the performance of different flexible and rigid pavement structures.

In 1972, the AASHTO (American Association of State Highways and Transportation Officials) pavement design guide was first published as an interim guide. Updates to the guide were subsequently published in (1986, 1993, 2000 and 2002).
The 1986 revision of the 1972 Interim Guide added more features to the design procedure. The focus was on four important issues:

- Better characterization of the subgrade and unbound materials before construction
- Incorporation of pavement drainage structures.
- Better consideration of environmental effects on material properties
- Incorporation of reliability as a factor into the design equation.

In this interim guide, the subgrade was for the first time characterized by its resilient modulus (M_R).

AASHTO (1993) suggests the requirements of material characterization for flexible pavement design in addition with parameters to be considered in pavement design that focused by the previous interim. These parameters are CBR, maximum dry density (MDD) and group index (GI) that are important in determination of homogenous soil section.

The AASHTO manual also revised again in 2000. In this manual the following points are clearly indicated as the guidelines for subgrade characterization (AASHTO, 2000):

- The spacing of sample collection for material characterization should be at the range of 150m to 450m during construction phase in consideration of geological variability.
- The strength in terms CBR value and swelling/shrinkage potential of subgrade materials should be determined.

- Optimum moisture content (OMC) and maximum dry density (MDD) of the materials should be determined before design.

According to AASHTO (2004) Soils are classified into seven basic groups and twelve subgroups. From the tested results in AASHTO classifications, it is possible to calculate group index (GI). The bearing ratio of soil under normal condition is inversely proportional to the value of GI.

Using AASHTO classification and test methods M145, Group index is calculated by the following equation;

$$GI = (F-35)\{0.2+0.005(LL-40)\} + 0.01(F-15)(PI-10) \quad \text{......Eq. 2.1}$$

Where: F = the percentage passing sieve size 0.075mm (N0. 200),

LL = liquid Limit, PI = Plasticity index of the soil

Under normal 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 (AASHTO, 2004)

The liquid limit (LL) and plasticity index (PI) in the above group index (GI) equation are mainly related to expansive (CLAY) soils are discussed in Arora (1997) in relation with other index properties of soils such as shear strength and linear shrinkage which are the main focus in pavement design on such problematic soils.

Accordingly Arora (1997) indicate that;

- At the state of liquid limits, the shear strength of all soils is constant and is equal to 2.7 KN/ m^2.
- An empirical relationship between plasticity index (PI) and linear shrinkage (LS) of CLAY soils is given as; PI = 2.13 * LS, which show direct relationship.
- The compaction of a soils during pavement design or laboratory test can be affected by; type of soils, moisture content and methods of compactions used.

Additionally the factors contributing to the formation of expansive soils from the weathering of rocks are described by Bell (2007). As a general these factors are; climates condition, parent rocks, water movement, age, vegetation cover of the area and other human activities.

2.5.2 Overview of Ethiopian Road Authority (ERA) manuals

Today's Ethiopian Road Authority (ERA) has been established in 1951 under proclamation No. 115/1951 with the name of Imperial Highway Authority (IHA) having an objective of road construction and maintenance activity. The organization was re-established with different names and objectives updated from time to time and develop different standard manuals and policies to make investigation, construction and maintenance of road network within the country. The manuals such as ERA pavement design manual, ERA site investigation manual, ERA geometric design manual and ERA standard specification manuals were developed with some catalogs.

These manuals give specification for the structural design of flexible pavement and gravel roads, site investigation and specification of materials to be used in road construction in Ethiopia (Road note 31, 1993).

According to Ethiopian Road authority pavement design manual (ERA, 2002), Delineation of homogeneous soil section in subgrade characterization is the most important activity of the designer. A road section for which a pavement design is taking place should be sub-divided into homogeneous areas where the sub-grade CBR can be reasonably expected to be uniform. In this delineation the number of subgrade division should not be exceed 4 or 5 uniform sub grade for a given project, without consideration of the length of road project, but this manual should consider the length of the road project and horizontal variability of geological materials along the road.

Depending on CBR value, Ethiopian road authority categorizes the subgrade materials into six different strength classes (ERA, 2002), (table 2.2 below).

Table 2.2 ERA Strength classes of sub grade soils based on CBR value

Strength class of sub grade materials	CBR Values (%)
S1	2
S2	3-4
S3	5-7
S4	8-14
S5	15-29
S6	>=30

Source (ERA, 2002)

Thus, ERA manual structural catalogue had been developed in order to design the flexible pavement thickness design based on the traffic and subgrade strength classes' requirement.

In addition, the Ethiopian road authority site investigation manual (ERA, 2002) gives some consideration to the pavement design on problematic soils such as black cotton soil which covers large areas of the country. In this manual the problem cause by such materials and its solutions are recommended to improve unsuitability of expansive soils. Hence identification of the presence of such materials and stabilization methods should be taken before construction phase. The measures chosen to minimize or eliminate the effect of expansive soils shall be economically realistic and proportionate to the risks of potential pavement damage and decrease required maintenance costs (ERA, 2002).

Thus the manual recommended the following mitigation methods of problem of expansive soils on pavements;

- Excavation of expansive soil and replacing of suitable materials.
- Avoiding the presence of expansive soil along the road path by realignment.
- Treat the expansive soil with physical compaction or chemical stabilization.
- Minimizing the moisture content and swelling potential of the soil by avoiding water.

Generally the manual indicates the structural criteria to be fulfilled by any pavement to give satisfactory service without deterioration before a given design time; these criteria are;

1) The subgrade should be able to sustain traffic loading without excessive deformation.
2) The load spreading ability of granular sub base and capping layers must be adequate.
3) In pavements containing a considerable thickness of bituminous materials, the internal deformation of these materials must be limited; not be beyond its permissible.
4) All pavement sections must be able to sustain the stress and strain generated within it.
5) Asphalt materials used in road design should not crack under the influence of traffic; this is controlled by the horizontal tensile stress or strain at the base of the road.

In addition the manuals clearly indicated the required parameters that should be taken into consideration during design stages; these are;

- Estimation of the amount of traffic loads that the road will be going to support and cumulative number of equivalent standard axles over the design life of the road should be done. Estimated single axle loads (ESAL) used to know the traffic classes.
- The subgrade strength classes based on CBR value should be conducted carefully.
- Finally selection of the combination of pavement material and thickness from the structural catalogue that will meet the satisfactory of pavement service and design life based on traffic classes (T) and strength classes (S) values should be done.

Traffic classes	ESAL (10^6)		Sub grade Strength classes	CBR (%)
T1	<0.3		S1	2
T2	0.3-0.7		S2	3-4
T3	0.7-1.5		S3	5-7
T4	1.5-3.0		S4	8-14
T5	3.0-6.0		S5	15-29
T6	6.0-10		S6	>=30
T4	10-17			
T5	17-30			
	**ESAL is the acronym for equivalent single axle load.			

Table 2.3 ERA pavement design standard on the basis of traffic classes and sub grade strength

2.5.3 Overview of Tanzanian Pavement design Manuals

According to Tanzanian ministry of work manuals (1999), soil surveys shall be planned and conducted in a manner that classifies all materials according to their suitability in load bearing layers with zone of design depth. Based on soil survey result, detail investigations shall be extended to below design depth as required to detect problems that need special consideration. These include;

- Presence of problematic soils under the foundation of project to be constructed.
- Unfavorable sub grade conditions and
- Feature associated with slope and embankment stability.

Sampling the subgrade along the road line and materials testing carried out at a minimum average frequency. The sample collection points (pits) may be distributed un-evenly along the road line to capture changes in soil conditions and required for optimum use of resources allocated for investigation. The minimum frequency of sampling is given in table 2.4 below.

Table 2.4 Tanzanian standard of minimum material testing frequency

Road types	Indicator Test (/km)	CBR Test	Minimum number of CBR test for any homogeneous section	
			Minimum for statistical analysis	Absolute minimum
Paved truck roads	Min 4/km	Min 2/km	5	3
Other paved roads	Min 2/km	Min 1/km		
Gravel roads	Min 2/km	Min 1/2km		

(Source; Tanzanian design pavement manual, 1999)

The indicator test in above table represents the Atterberg limit and gradation test of grain size greater than 75 µm.

Table 2.5 Subgrade strength classes by Tanzanian Design manual

Subgrade class	CBR Design %			Density for Determinations of CBR Design (% of MDD)
	Wet or moderate climatic zone	Dry climatic zone (Both requirements shall be met)		
	4 days soaked Value	Tested at OMC	4 days soaked value	
S15	Minimum 15	Minimum15	Minimum 7	95 BS heavy
S7	7-14	7-14	3-14	93 BS heavy
S3	3-6	3-6	2-6	100 light
BS-Light compaction effort is used on poor in-situ soils and deep in-situ soils rather than BS-Heavy due to its better correspondence with the actual effect from compaction equipment under conditions with poor support for compaction.				

As per this standard, subgrade have to meet general depth requirement which is 0.8 m and 1.2 m for heavy load classes and for other loads the general requirement is 0.6 m and 1.0m.

The United Republic of Tanzanian ministry of work laboratory testing manual (2000), indicated that, the geotechnical parameters of subgrade materials such as atterberg limit, gradation, bulk density, CBR for strength determination and other parameters should be undertaken for any infrastructures design.

The manuals describe the procedure to be followed during laboratory characterization of subgrade materials in accordance with central material laboratory (CML) which is British standard (BS) that depends on American Society for Materials and Testing (ASMT) manual.

According to the Tanzania, Pavement and Materials Design manual (1999), the subgrade materials are classified into three strength categories depending on CBR value under different condition (presented in table 2.5)

Additionally the manual states that the problematic subgrade materials or if there are unfavorable materials along the designed road alignment, it should undergo some improvement as; all subgrades shall be brought to CBR value of greater or equal to 15% by constructing one or more improved layers where necessary. After improvement the subgrade materials should be re-categories into one of the three strength groups (S15, S7 and S3).

This method of subgrade improvement has many advantages such as; economical use of local materials, the improved layers used as filter layers to reduce pumping effects, to provide deeper pavement structures to support heavy axel loads of traffic, provide homogeneous sub grade strength and improved compaction of layer above it and etc.

2.5.4 Overview of Kenyan Road design Manuals

Road Design Manual of Ministry of Transport and Communication Roads Department, Republic of Kenya (1987), has raised some properties of expansive soils and problem faced when structures are constructed on such problematic soils. These properties are;

- Volume change due to variation in moisture content that develop cracking upon drying and swell on wetting condition and results in reduction of stability. Furthermore, these volume changes may produce lateral displacements of the expansive CLAY, if the side slopes are not gentle enough or retaining wall are not provided.
- The bearing capacity of the materials can be reduced due to increase in water content. The manual classifies sub grade materials into six categories depending on the CBR that help to determine bearing capacity. (See table 2.6.)
- Susceptibility to erosion: when dry black cotton soils present with sand like texture; which formed by the agglomeration of CLAY, silt and sometimes sand particles, they are prone to erosion to a greater extent than their plasticity and CLAY content normally anticipated. The continuous erosion of finer particles can cause severe problems to the structures.

The standard manual also gives specification to be fulfilled by a material to be used as sub-grade material. Accordingly the subgrade materials should have 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%.

These show that, no pavement directly placed on such soils (S1class), it should need placement of improved subgrade materials.

Table 2.6 Sub grade strength classifications by Kenyan design manual

Subgrade	CBR Range (%)	Stiffness Modulus (Mpa)
S1	**2-5**	**20**
S2	**5-10**	**50**
S3	**10-15**	**80**
S4	**15-20**	**100**
S5	**20-30**	**120**
S6	**>=30**	**>150**

(Source; Kenyan pavement design manual, 1987)

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

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

2.6 Previous work

Many researchers conduct their research on pavement design, subgrade materials characterization, factors affecting stability of pavement, properties of subgrade materials to be considered during pavement design, properties and effects of expansive soils and improvement methods; some are discussed below briefly;

Atkins (1983), discuss the component of pavement structures that include from top to bottom as; surface, base, sub base, Compacted sub grade and Natural sub grade, (collectively called the sub grade) and have been grouped subgrade materials as very good, good and poor in pavement design that depends on the strength of the sub-grade soils and magnitudes of the imposed loads. In his report he rated silts and CLAYs poor materials only under the following conditions:

- When they occur in low lying areas where the natural drainage is very poor
- Where the condition such that severe frost heave can be expected.
- Where high percentage of mica-like fragments or diatomaceous particles produce a highly elastic condition.
- Where it is desired to be highly expansive soils usually A-7-6 (CH) deeper in the section to limit the effect of seasonal variations in moisture.

So, from his scenario it is possible to understand that materials cannot be grouped as poor or good for subgrade use without considering condition under which they exist and the amount loads to be imposed on it.

Some researchers describe about expansive soils in terms of its formation process, spatial distribution in the world and its negative effects on stability of engineering structures, especially roads.

Stephen. N., (2003), explain briefly the way how expansive soil could be formed from weathering of parent rocks. Accordingly the formation of CLAY minerals (soils) in tropical zone involves chemical weathering of rocks and leaching of minerals. He also describe about the formation of primary minerals, residual minerals and ultimately the leached ions in chemical weathering processes. After leaching the residual minerals may become CLAY depending on the stability of the primary minerals in the parent rocks.

Chen (1988) give emphasis to the spatial distribution of expansive problematic soils in the world and the problem by expansive soil is one of the six major natural hazards (Earthquakes, landslides, volcanic eruption, expansive soils, hurricane, tornado and flood). The CLAY mineral that is mostly responsible for the expansiveness belongs to the montmorillonite groups. The soils containing a considerable amount of monmorrilonite minerals exhibit high swelling and shrinkage characteristics, hence they are highly hazardous. In most area the depth of such soils are shallow only less than 6m, since they are residual soils formed from weathering of rocks on the surface that subjected to intensive weathering agents.

To quantify the swelling potential of CLAY soils, Seed et al., (1962) proposed an empirical relationship between swelling potentials and plasticity index values of CLAY soils. The swelling potential of expansive soils can be predicted by using the empirical equation

For natural soils,

$$SP = 60K\,(PI)^{2.44}$$

Where: SP = swelling potential, PI = plasticity index and K = $3.6 * 10^{-5}$ a factor for CLAY content between 8 & 65%

In our country some related studies to the present work were conducted in the past mainly for academic purposes. The work include the characterization of subgrade soils, geotechnical investigation of subgrade soils, identification of causative factors of pavement distress, stabilization methods of expansive subgrade soils and suitability analysis of sub grade soils for a given pavement design have been conducted.

The works of Daniel Nebro (2002), Nibret Chane (2011), Habtamu Solomon (2011), and Zeleke Taddese (2013) are among the literatures referred during the present study.

Daniel Nebro (2002), "stabilization of expansive soils using lime and Con-Aid along Addis Ababa Jimma road". He characterizes the natural subgrade materials and lime mixed subgrade sample using different laboratory tests. Finally he compare the pure natural subgrade properties with subgrade-lime mixture and he obtain an improvement on subgrade materials as decreasing in plasticity, increasing its CBR strength and significantly decreasing in CBR swelling.

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

Nibret Chane (2011) “Geotechnical characterization of sub grade materials on Aposto – Wondo –Negele road” by identification of the presence of expansive soil and characterization of its geotechnical properties (Atterberg limits, grading, MDD and OMC, CBR and swell values) under adverse condition, the researcher has suggested some mitigation method for un suitable part such as removal and replacement, in situ treatment, rock fill with geo textiles and underground drains. After the laboratory and field test the researcher conclude that at least 82% of the sub-grade soils in the study area are found to be suitable for bearing stratum and construction materials.

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

CHAPTER III THE STUDY AREA

3.1 Preamble

The project area on which this research has been conducted is under the construction by local contractor AKIR construction plc, which have an experience of about 20 years on this sectors. The road project was signed between AKIR construction plc, and Ethiopian road authority (ERA). CORE engineering plc were supervising the activity and closely control the characterization and designing. Because the design and construction of any engineering structures could be affected by fault of designing, using un appropriate materials, inaccurately characterization of materials, location with respect to suitable raw materials, climatic condition, topography and geological set up of selected site.

Thus, this chapter deals with the description of the project area in terms of location, accessibility, climatic condition, physiography, geological setup and hydro-geologic condition which influence the engineering properties of subgrade soils in the project site and the surrounding area. This section also includes description of the seismic condition and soil type distribution of the study area and its surrounding. Finally a brief description of the project site condition is included.

3.2 Description of the Study Area

3.2.1 Location and Accessibility of the area

Sembo-Shola gebeya road project starts at Sembo village (538556E, and 1040338N), which is about 13.1km away from Sheno town in Northern Shewa of Oromia Region on Addis Ababa – Dessie Road and branches to the right side to connect Northern Shewa Zone of Oromiya with Northern Shewa zone of Amhara Regional State about 36.2km length.

Table 3.1 Minor towns along the road path.

No	Station	Minor towns	Geographic position		Elevation (m)
			Latitude (N)	Longitude (E)	
1.	0+000	Sembo Junction	1040338	538556	2943
2.	5+300- 6+900	Kotu Gebeya	1037281	542980	2836
3.	20+600-21+400	Sekorru	1028509	550235	2960
4.	26+800-27+600	Nefasamba	1024217	549555	2886
5.	36+320-37+420	Hageremariam	1018404	544927	2692

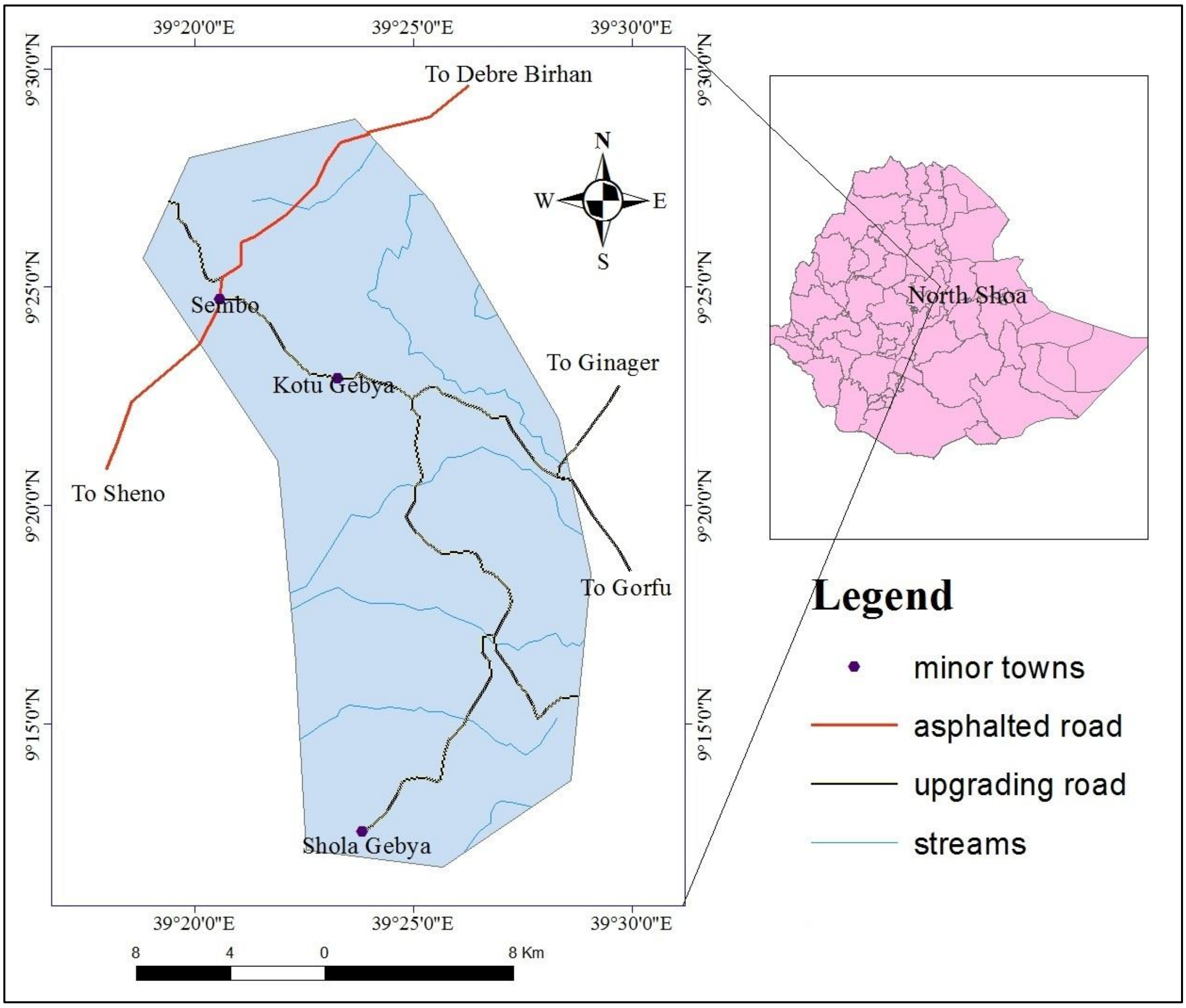


Fig: 3.1 Location map

The area is accessed through different roads of asphalt and gravel. The junction of the study area (Sembo) is found at about 10km from Sheno towards North along Addis Ababa-Mekelle road.

3.2.2 Terrain classification and physiography of the study area

Terrain classification along a road path is important because the landform is the main contributing factor in the soil formation, and each landform in a particular area has its own geology (Robinson, R. and Thagesen, B., 2004). Terrain classification and physiographic setup of an area depends on the geological, geomorphological and tectonic activities takes place in the geological history.

The project area is located in the western highlands of main Ethiopia rift system with elevation range between 2600m and 3000 m above sea level.

According to ERA geometric Design Manual (2002) transverse terrain properties are categorized in to four classes depending on slope geometry, namely;

- Escarpment terrain, transverse terrain slope in excess of 50%
- Mountainous terrain, transverse terrain slope from 25% to 50%
- Rolling terrain, transverse terrain slope from 5% to 25%
- Flat terrain, transverse terrain slope up to 5%

Accordingly the terrain of the area across which the proposed route alignment passes, has been classified during the site investigation. This was done referring to the transverse slope and also the longitudinal profile of the area along which the road traverses will be designed.

Topographically the area under the present study is classified into 4 types of topographic features during the field survey. These features are rolling, escarpment, flat and mountainous. The area is dominated by flat terrain (35%), which covers about 13.7km of the total (36.5km) length of the road project. Most of the problematic expansive black cotton soils are found on flat terrain, because the weathered rocks on the mountainous and escarpment terrains have been eroded and formed thick soil deposits along the flat terrain.

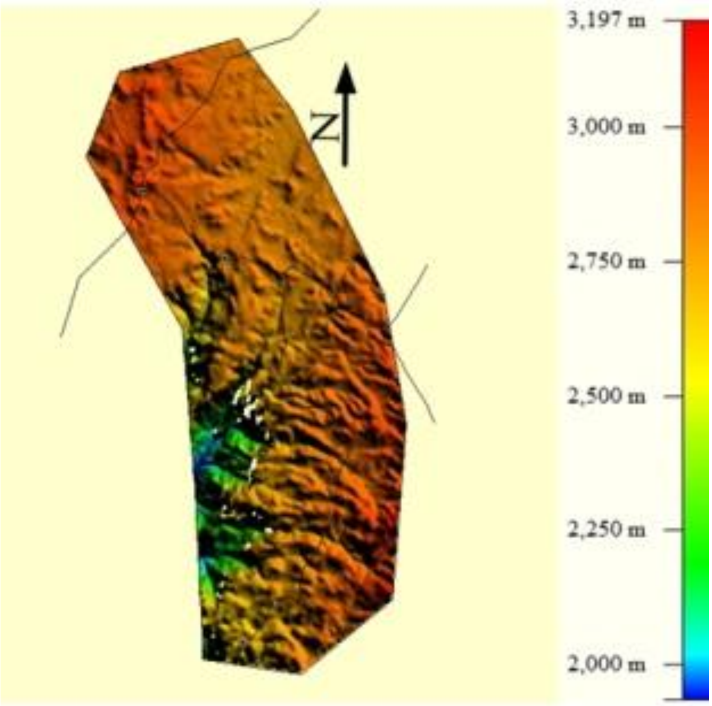


Fig: 3.2 Topography of the study area

Terrain	**Length (km)**	**% coverage**
Flat	13.68	35
Rolling	10.44	28.6
Mountainous	10.78	32
Escarpment	1.6	4.4
Total	36.5	100

Table 3. 2 Terrain classifications

3.2.3 Land use and land cover

As aforementioned, the road from Sembo to Shola gebeya (Hageremariam) is mainly traverses through Mountainous, Flat, Rolling and little Escarpment terrain. The Flat and rolling section is intensively cultivated. Since, the elluvial surficial deposits that are developed from the weathering of volcanic rocks at the plateau and rift escarpment is fertile, it is intensively used by the local people for farming.

There are few places of pastoral land. The area has high potential for cereals and livestock production. Wheat, Bean, Barely, Sorghum, zengada, peas and lentil are the main agricultural products in the project area. The agricultural activities in some place forces to realign the road and to find right of way to design.
However there are few locations with Eucalyptus tree adjacent to the road. The realignment section of this road is also cultivated land.

3.2.4 Climate and rainfall

Ethiopia is classified into five climatic zones (National Atlas of Ethiopia, 1981). These climatic zone of Ethiopia with respective of elevation and mean annual temperature is given in the following table.

Table 3.3 climatic zonation of Ethiopia with respective elevation and mean temp.

No	Climate	Elevation (m)	Annual mean temp (^{0}c)
1.	Kur (Alpine)	3000 and above	Less than 10
2.	Dega (Temperate)	2300-3000	10-15
3.	Weina Dega (Sub tropical)	1500-23000	15-20
4.	Kola (Tropical),	500-1500	21-30
5.	Berha (Desert),	Below 500	30-45

Depending on its elevation range and mean annual temperature, the project route corridor falls within “Dega” climatic zone because the altitude of the routine alignment lies between 2600 and 3000 meters above sea level with annual mean temperature of 6 to 15^{o}C.

In Ethiopia, a variation in geographical location and altitude has produced a variety of rainfall regions and associated problems on road stability. In 2002 the Drainage Design Manual produced by Ethiopian Road authority (ERA), categories the country into seven regions of rainfall on the basis of increments of 400mm variation.

Table 3.4 Metrological records from 2006 to 2013 around the study area.

Months	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**July**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**	**Annual**
Rainfall (mm)	25	50	50	50	50	100	250	300-400	100-150	25	10-25	1-10	**1,105**
Max. Temp(^{0}C)	10-20	15-20	10-15	15-20	20-25	10-15	10-20	10-20	10-15	10-15	10-15	10-20	**15**
Min.temp (^{0}C)	5	5	5	5-10	5-10	5-10	5-10	5-10	5-10	0-5	5	5	**6**

Generally, annual precipitation in the country varies from 800 to greater than 2400 mm in areas where the elevation is greater than 1500 meters in the highland regions in which rains often occur from June to September (TRL, 2009). However the study area gains a rain fall throughout the year with a range of 5mm to 350mm monthly with an average of 13000mm per year, in high land areas.

3.3 Geological setup

3.3.1 Regional Geology

The study of the Geology of Ethiopia goes back to 1860 by Blanford, and then, major advances in the understanding of the geology of Ethiopia have been made from the works of various researchers, such as Danieli (1943), Mohr (1971) and Kazmin (1972, 1974, 1978).

These study shows that Ethiopia has a complex geological history represented in three major geological terrains. Late Paleozoic, Mesozoic and Cenozoic continental and marine sediments occur mainly in the eastern part of Ethiopia. Also Cenozoic volcanic and sedimentary rocks occur, including those of the East African Rift Valley transecting the country from Southwest to Northeast that separate country into East and West region. The Precambrian complexes of the country are exposed in low land areas of Nortern, Western and Southwestern of the country with high, medium and low grade metamorphic rocks.

In addition, extensive areas of the highlands of Ethiopia on both sides of the rift valley are covered by Tertiary (Trap series) volcanic rocks which are mainly basalts with subordinate acidic rocks. In the rift valley, subsequent to the formation of the rift valley, the Trap series were overlain by a variety of younger volcanic rocks of basalts, ignimbrites and rhyolites (Tesfaye Chernet, 1993; Habtamu Solomon., 2011).

The Ethiopia plateau, including part of the study area is composed of thick Tertiary volcanic successions (Zanetiine et al., 1974) and the volcanism is classified into three stages separated by a long period of volcanic inactivity; the pre-Oligocene (alkaline to tholeitic basalts), the Oligocene-Miocene and the Miocene-Pliocene which later reclassified the volcanism as Oligocene-Late Miocene Alaji basalts, rhyolites and the Tarmaber basalts.

In addition, the present day physiography of Ethiopia is determined by two tectonic phenomenons: The uplift of the Arebo-Ethiopian swell and its subsequent dissection by the rift system. The geology of central Ethiopian highlands are characterized by tertiary volcanic rocks (originated from MER tectonics) overlying the Mesozoic sediment (that only exposed in deep and river cuts such as Abay gorge and Mugher gorge (Leta Alemayehu., 2007).

The MER system in Ethiopia has divided the uplifted swell into two separates units: The Ethiopian plateau in the west and Somali plateau to the east.

Structurally the central MER is characterized by a set of NNE-SSW normal faults. "En echelon" arrangement, rift in rift structures, asymmetry and open tensional fissures are its most important features. The down throw of a single step fault can easily exceed 300m.

Table 3.5 Classification of lithologies and stratigraphy of Debre birhan area

Age		**Litho-stratigraphic units**	**Lithologies**
Cenozoic	**Quaternary**	Superficial deposits	o Alluvium o Eluvium
		Volcanic rock	o Dofan basalt -vesicular basalt, aphanitic basalt, olivine phyric basalt and recent scoria cones o Fentale-Alay Dege ignimbrite- ignimbrite, tuffs, ash flows, agglomerate and minor obsidians.
	Tertiary	Volcanic rock	o Tarmaber-Megezez basalt- plagioclase phyric and olivine-plagioclase phyric basalts with minor olivine phyric, pyroxene phyric, plagioclase-pyroxene-olivine phyric and aphanitic basalts. o Trachyte (Tt) o Kesem basalt (Tkb) aphanitic basalt intercalated with plagioclase phyric basalts and thin beds of ignimbrite. o Sela Dengay-Debre Birhan-Gorgo ignimbrite (Tdig) ignimbrite, tuff, rhyolite, aphanitic basalt, tuffaceous sediments, ash, agglomerate
Mesozoic	Sediments		o Sandstone (Msst) o Mudstone (Mmst)

Generally, as mapped by Geological survey of Ethiopia (2009), Debre brihan area consists of two litho-stratigraphic units. These are: - Mesozoic sediments exposure and Cenozoic volcanic rocks and Quaternary superficial deposits.

The Mesozoic sediments are exposed in the highly dissected plateau area at the northwestern part of Debre-birhan. The typical succession in this area is stratified sandstone (top) and mudstone (bottom) of Mesozoic age, situated within the gorge and canyon of Jema drainage basin.

The Cenozoic volcanic rocks cover larger area and grouped into tertiary and quaternary volcanic rocks based on their age and mode of occurrence. The litho-stratigraphic units that mapped by Geological survey of Ethiopia is presented in above table.

3.3.2 Local geology

As aforementioned the present study area is situated at the western margin of main Ethiopian rift along the western plateaus. Geologically the study area is covered by ignimbrite, basalts and Miocene product of rhyolite of about 30m thickness (Dereje Ayelew et al., 2002). The geologic nature of the project area was studied referring to the Geological Map of Ethiopia 1996 edition. According to the map, the area is covered with two main geologic formations of volcanic rocks and two surficial deposits;

1. Volcanic rocks
   - The Tarmaber – Megezez Formation (Ttb) and
   - Sela Dengay-Debre Birhan-Gorgo ignimbrite (Tdig).
2. Surficial deposits
   - Alluvial deposits (Qal)
   - Elluvial deposits (Qel)

i) Ttb (Tarmaber-Megezez basalt)

Tarmaber-Megezez basalt is dominantly exposed on the western (Wenoda, Sendafa and Megezez, north central (Debre Sina, Termaber and Ankober) and south western (around Sembo, Sekorru, sholagebeya and Gorfo) part of the plateau. It forms a series of linear ridges and high mountains (Mt. Megezez, 3595m a.s.l.) and Mt. Yekur (3400m a.s.l.). It has sharp contact with the underlying Sela Dingay-Debre Birhan-Gorgo ignimbrite (Tdig). Termaber-Megezez basalt includes fine, medium to coarse grained, dark gray (fresh color) to light/reddish/dark/yellowish brown (weathering color) and aphanitic to porphyritic basalts. Physically the rock units looks an andesitic (intermediate rocks) with relatively low density. The basaltic unit in present study area covers a larger area and characterized by a columnar joints. The porphyritic basalt is highly weathered. The petrographic description of this unit is presented as sample No.M26, below.

ii) Tdig (Sela Dengay-Debre Birhan-Gorgo ignimbrite).

The ignimbrite forms gentle to steep cliffs, elongated ridges and distributed along isolated hills (e.g., Kotu Gebya). It is medium to coarse grained, light/bluish/brownish gray to gray (fresh color) to dull/dark gray (weathering color), highly consolidated to welded tuff and bedded with columnar joint. It contains rock fragments of rhyolite and basalt ranging up to 2cm in diameter and elongated fibrous glass shards (fiamme), whereas the amount of rock fragments significantly varies from place to place. According to Geological survey of Ethiopia (2009) report, the rhyolite is found around Deneba, northeast of Debre Birhan (Beriyo Baleweld), Shola Gebya and Ginager. It is fine to medium grained, bluish/light/greenish gray (fresh color) to dull gray, light/dark brown (weathering color).

iii) Qal (Alluvium soil)

Is a surficial deposit consists of dominantly fine to medium sand, gravel deposits. Mostly it covers relatively the lowlands of the area. It is derived from the weathering, transportation and reworking of different rocks from the plateau, escarpment and also from the rift area by stream erosion

iv) Qel (Eluvium Soil)

The eluvium soil is mostly found on the plateau and escarpment, occupying flat lying and gentle topography. It is formed by the gradual weathering of the basalt, ignimbrite and rhyolite. There are rock fragments of basalt, ignimbrite and rhyolite within the eluvium soil. It is silt to clay sized, light/dark gray to reddish brown fertile soil. It is highly ploughed by the local people.

Petrological Description of the area

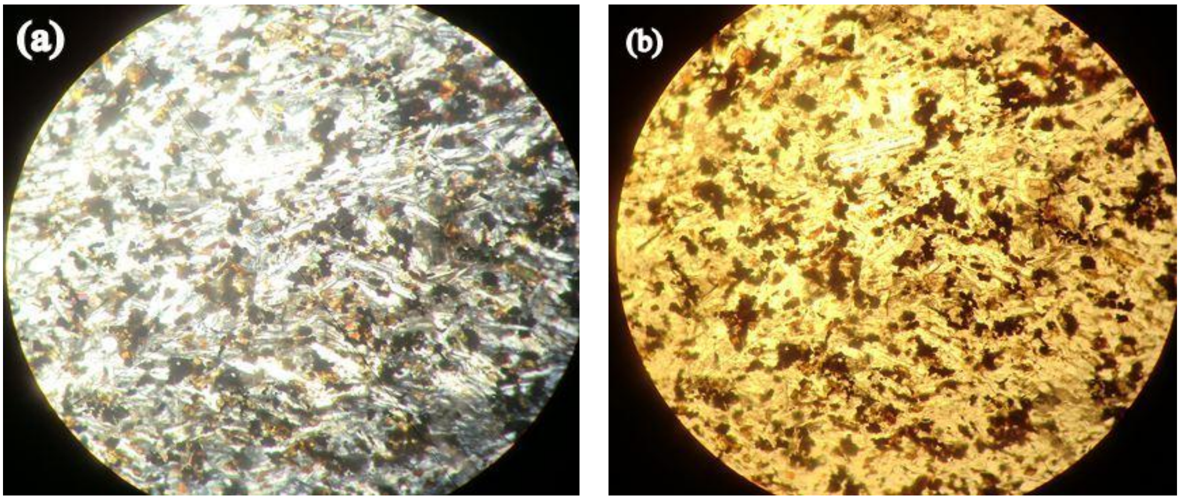
Two Rock samples at two different locations (6+500 and 24+100) were collected and studied for petrographic analysis at the Addis Ababa university thin-section laboratory. The samples were taken in the study area along the route corridor in the road project for the conformation of geological setup of the area mapped by geological survey of Ethiopia. Accordingly the samples have been taken from two geological units dominantly found along the study area.

- **Sample No M26 (24+100)**

The rock sample that taken from station 24+100 (1026300^{o}N, 549534^{o}E), physically looks andesitic of intermediate rocks rather than basaltic. In petrographic analysis it have not as such variable minerals, it dominated by aphenetic grains of feldspars that shows an elongation of grains and Hematite (ilminite) of iron oxide that shows dark to reddish color both under plane polarized and cross polarized light. There is also a smaller percentage of altered pyroxene and olivine.

In general this sample shows Feldspar (50%), Hematite (40%), Altered pyroxine and olivene (5%) and Opeque and voids (5%) with rondam distribution.

In bown's reaction series the minerals that pricipitate at early stage have high susceptability to weathering. Minerals such as feldspars and olivene pricipitate at the beginning of magma differentiation and tends to form mostly mafic rocks.



(a) XPL **(b) PPL**

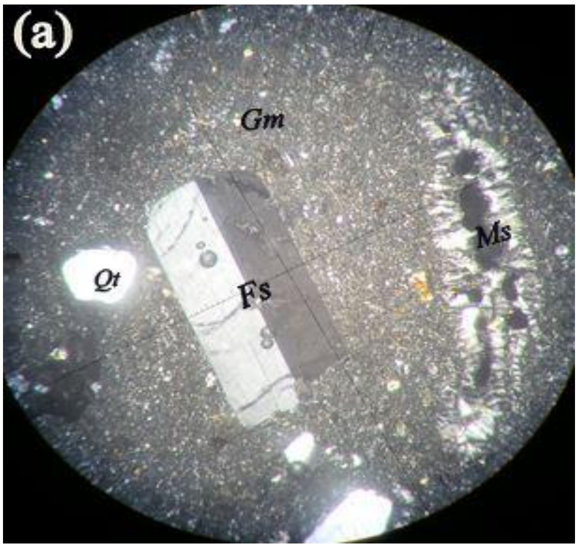
Plate: 3.1. petrographic representation of Termaber basalt

Accordngly the rocks that dominate the study area (sample No26) consists about 50% feldspars, hence the rate of weathering and formation of black cotton soils is high in the study area since black cotton soil is a product of intensive weathering of basic igneous rocks (Seed et al., 1962). In other word some minerals weather more readily than others, and new minerals formed during weathering depend on bedrock mineralogy. Rocks containing abundant feldspars and quartz (felsic rocks) weather to kaolinite-bearing sandy soils, quartz being the sand fraction (Cleaves, 1974)

In addition hematite (ilmanite) also consisted large percentage nexxt to feldspars. In nature the presence of iron oxide between grains of other minerals acts as a cementing materials, but from thin section analysis, the hematite is observed as a grain that develop separetly. In engineering application the iron rich rocks.

- **Sample No M6 (6+500)**

This sample has been taken as a representative for the lithological units that mapped and described as ignimbrite in a local geology (at 1037235^{o} N, 543543^{o} E on UTM reading). In petro-graphical analysis the samples show a propheritic texture in which ground mass dominated.



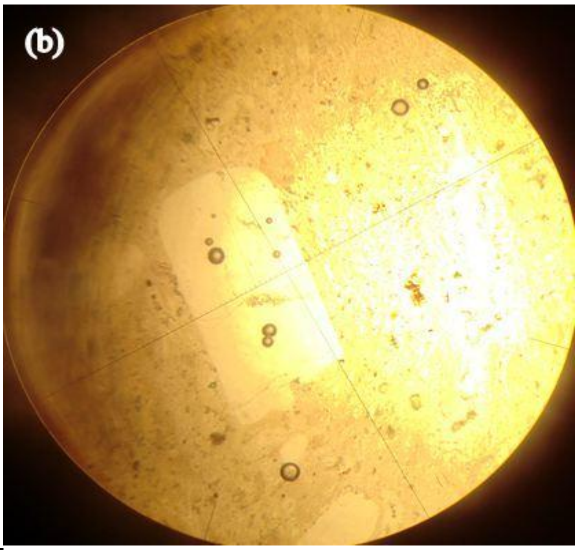


Plate 3.2 petrographic representation of ignimbrite units in the study area (a) XPL, (b) PPL (Pcr-phenocryst, Gm-ground mass, Fs-feldspar, Ms-microcystalline silica and Qt-quartz fragment)

The phenocryst (40%) consists grains of Hornblend (20%) shows elongated grain, feldspar (10%) that have an alternative banding of light and dark with cleavage under cross polarized microscope, microcrystalline of silica (3%) radiating, quartz fragment (5%) and opaque minerals. And the ground mass (60 %) have a grain fragments of various minerals especially fragment of biotite and quartz.

3.3.3 Geological structures of the study area

The major structures around the study area are mostly localized in the rift-margin and floor, having a series of NE-SW normal faults following the trend of main Ethiopian with variable strike length (few meters up to kilometers) and within the rift-floor some of these faults are characterized by chains of volcanic centers that extended to the plateaus as small lineaments and localized shear zones. These structural elements within the study area are identified by the combination of results from google earth observation, visual inspection during field survey and geological map of Ethiopia.

The structures are associated with the extensional tectonic activities. They include faults, local shear zones, folds, fractures and joints, having variable magnitude and orientations.

Differently oriented lineaments with variable strike length are recognized in the study area in which some of the lineaments cross cut the proposed road alignment. As shown on geological map (Fig: 3.3), most of the lineament trends in NW-SE, NE-SW and E-W directions, in which some of the stream flow directions are controlled by the trend of such lineaments in addition with slope gradient.

Additionally differently oriented joint sets and irregular fracture (few mm to cm in width) are observed along the rocks of road colliders. They are penetrative to non-penetrative joints, having significantly variable strike length. These joints and fractures are responsible for the formation of soils due to weathering of jointed rocks.

Mostly in the ignimbrite around Kotugebeya, two sets of joints are encountered; these are horizontal (dipping 10^{0} towards SE) and vertical (trending NS and $N15^{0}E$) set of joints. In addition irregularly oriented columnar joint sets (mostly hexagonal faces) are also observed in the ignimbrites and basalts outcropped along road cut.

Plate: 3.3 a) two joint sets of ignimbrite. b) Horizontal lineaments on basalt.

These geological structures have their own negative effects on performance of pavement materials by initiation of intensive weathering, slope failure (fall) percolation of water through discontinuous structures and closing of surface drains (ditches). Hence the detail structural map of an area is need to identify the presence of geologic structures and to take certain measures such as using culvert boxes across the fracture and sealing of small discontinuities to reduce water percolation.

In addition the geological structure can be deeper that covered by alluvium deposit and difficult to treat, so the realignment of routine will be effective measure.

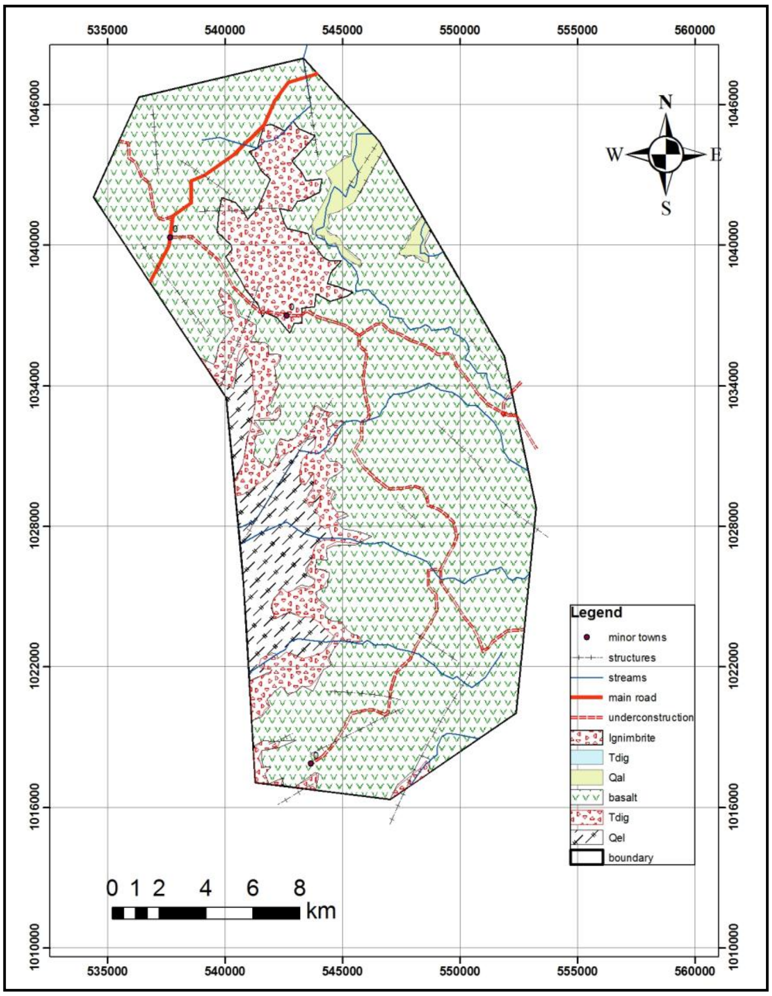


Fig.3.3 Geological map of the study area

3.4 Seismicity of the study area

There are different types of seismic waves that produce the violent ground shaking, or motion, associated with earthquakes. These wave vibrations produce several different effects on the natural environment that also can cause tremendous damage to the built environment (buildings, transportation lines and structures, communications lines, and utilities).

Additionally Strong ground motion during an earthquake can cause water-saturated, unconsolidated soil to act more like a dense fluid than a solid; this process is called liquefaction. Liquefaction occurs when a material of solid consistency is transformed, with increased water pressure, in to a liquefied state and responsible for the horizontal movement of materials in the form of liquid. This phenomenon is common when thick soil or sand is subjected to seismic wave. To study and reduce the effects of such event, all countries in the world divided into different seismic zone.

The seismicity of Ethiopia is controlled and influenced by the active Ethiopian Rift System which divides the country into two along the NE–SW direction (Tilahun Mamo, 2005). Earthquakes are intense and high in magnitude in the Afar triangle and Main Ethiopian Rift becomes sparse and low in magnitude in the rift border faults and seismicity is less common in the south, a reflection of a difference in the stage of rift evolution (Keir et al., 2003).

Ethiopia is divided into five seismic zones depending on intensity and magnitude of seismic activities in the area which is a function of distance from the rift valley and seismic amplification of the formation of the area. Seismic amplification of a given formation depends on the thickness of soil formation, wetness and degree of compaction (Tilahun Mamo., 2008)

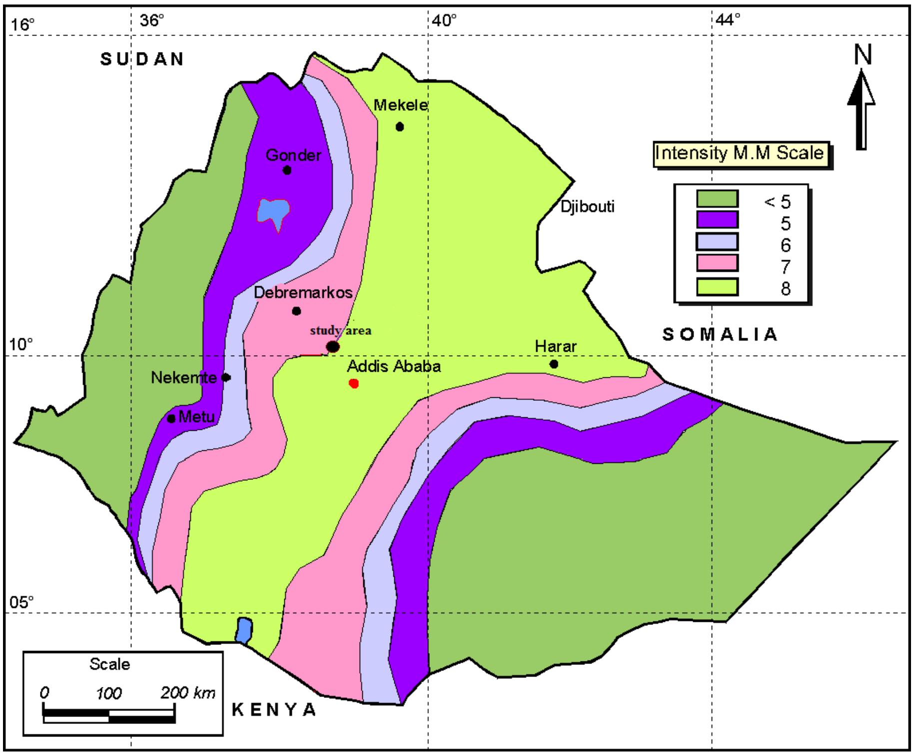


Fig: 3.4 Seismic zone of Ethiopia (Source: Laike Mariam Asfaw, 1986 and Habtamu Solomon, 2011)

The study area is located in a seismically active zone and it will have impact on road stability and sub-grade soils. So, it requires consideration and a systematic study to know exactly the damage an earthquake can cause to these structures and should be included in design parameters. However, such study is beyond the scope of the present research and simply recommended to include in pavement design.

Generally seismicity is mainly related to the area of active in volcanism and rifting activity (i.e. plate margin). The study area is situated at the western margin of main Ethiopian rift; in which sequence of volcanic and earthquake activities are dominant, hence the area is situated under zone 7 to 8 M.M scale of Ethiopian seismic zone. Due to the high seismic activities of the area, the roads and other infrastructures are subjected to seismic effect, so seismic code of the country should be considered during design of any infrastructures in this zone.

3.5 Evolution of the present Study

From literature review and physical observation, it has been learned that many flexible pavements are distressed due to inappropriate design of pavement structures, lack of suitability of sub grade (natural materials) and other construction materials. Thus suitability of natural materials (subgrade) for pavement design and construction is a function of its engineering properties such as strength, bearing capacity, swelling potential, gradation and atteberg limit which can be tested in the field and laboratory. The engineering performances of such materials are dependent of Geology, environmental condition and nature of loads to be imposed on it.

The main objective of present research is to characterize the strength and bearing capacity of sub-grade soils for constructing the pavement structure and to evaluate the suitability of selected area for selected pavement type. In addition to the common characterization parameters, in present study the geochemical composition of selected subgrade soils using XRD and its bearing capacity using triaxial test will be conducted and its effect will be known. Finally the relationship between topography and drainage system with other properties has been evaluated.

An important issue to conduct the present study was that, most of the study area is covered by problematic soils such as expansive, erodible, instable and low bearing capacity. Expansive soils have the behavior of swelling under saturation and cracking when wet, which results in reduction of pavement performance. The presence of paelosoils between rocks and soil stratigraphy (plate 3. 4 (d)) act as lubricant and in-stabilize the cut slopes. Additionally local landslide which manifested by hummocky terrain and lateral spreading (seismic effect) that manifested by tilting of electric poles are found dominantly.

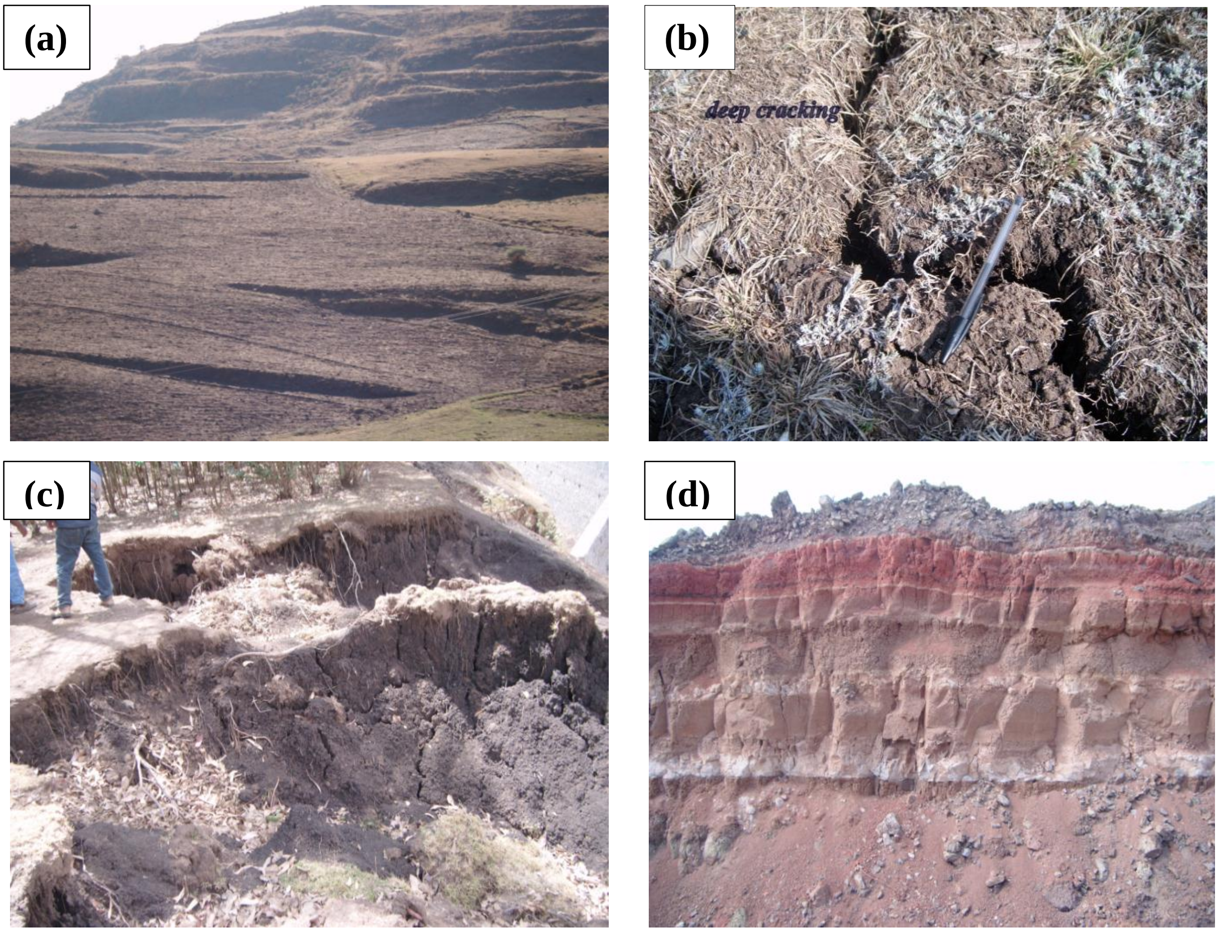


Plate: 3.4 Photo showing different problems in the study area. **(a)** Hummocky terrain showing local landslide (station 22+100), **(b)** Cracking of expansive soils during wet (station 23+000), **(c)** Landslide due to expansion (cracking) (station 30+300) and **(d)** Presence different formation along the road colliders (station 21+000)

The present research focused on characterization of sub grade materials along the selected road colliders in terms of their engineering properties like atterberg limit (Casagrande's plasticity), strength (CBR), gradation (sieve analysis), compaction (modified proctor tests), swelling and linear shrinkage from calculation aspect.

Then from the result of tested, measured and calculated parameters, the suitability of subgrade materials will be analyzed and classified as suitable or unsuitable. In another way the general suitability of the area in terms of stability (slope), topography and seismicity has been analyzed. Thus leads us to suggest some recommendation to stabilize the unsuitable materials economical.

CHAPTER IV

SUBGRADE SOIL CHARACTERIZATION

4.1 Preamble

Subgrade soil investigation is an integral part of location, alignment and designing of economically and structurally feasible and stable pavement for intended service time (Robinson, R. and Thagesen, B., 2004). Thus characterization of subgrade soils mainly gives a concern to the determination and evaluation of its performance under traffic loads.

On the other hand the soil investigation provides important information about soil and rock for decisions making related to selection of road alignment, subgrade treatment, slope stability analysis, location of structure (ditches and culverts), borrow materials selection and designing of foundation for bridges (ERA, 2001).

Since the road crosses through different topographic and geological features, engineering properties of soil may vary from place to place along the path. When the soils within the possible corridor for the road vary in property significantly from place to place, it is desirable to locate the pavement on the stronger and uniform soils, unless it is subjected to other constraints.

The objective of subgrade investigation is to provide a structural and economical combination of materials to carry traffic in a given climate over the existing soil conditions for a specified time interval. Subgrade soil's mechanical properties represent a key factor that affects subgrade's bearing capacity and structural design, since flexible pavements derives their ultimate support from the underlying sub-grade; therefore, knowledge of basic soil mechanics is essential for investigation of road collider (Braja M. D., 2008). Thus, thorough investigation should be done on the characteristics of existing subgrade materials both in field and laboratory.

Accordingly, the investigation and characterization of subgrade materials in present study have been done both on field and laboratory testing of engineering parameters such as Atterberg limits (liquid limit, plastic limit and plasticity index), Grain size analysis (sieve analysis), Modified proctor (compaction test), California Bearing Ratio (CBR) for strength test and Loaded swelling potential (CBR swelling) of subgrade materials that help to characterize the subgrade materials and design the pavement accordingly.

4.2 Field investigation of subgrade materials

The main object of field investigation is to identify the types of materials exists and its actual condition on the field. In present study during the field subgrade investigation the following basic field works have been done;

i) Soil extension survey and visual inspection
ii) Genetically classification of soils
iii) Sampling of the sub grade for laboratory analysis

4.2.1 Soil Extension Survey and Visual inspection

Soil extension survey has been carried out by identifying the subgrade materials along the road. The survey was undertaken based on the materials physical characteristics (colour, texture, and size), state of occurrence (fresh rock, weathered rock, decomposed rock and soil) and observable engineering properties (shrinkage cracks).

Sub-grade soils with nearly similar soil type are grouped together as homogeneous section and their extent has been determined along the roads. These extensions could be used as the bases to determine the required depth of earthworks during construction as well as to design the pavements for the roads.

In these sections there are an intercalation of one soil types with others and it is difficult to get a sharp distinctions between each sections, hence the naming of the section consider the most dominant types of soils along each stretch.

Depending on some visual inspection of physical properties such as colour, degree of weathering and grain size, the subgrade soils along the road path have been classified into four homogeneous sections. These are dark brown silty CLAY with mixed weathered rocks, dark brown CLAY underlined by weathered rocks, light brown silty CLAY and rock sections. The summary of homogeneous sections and its stretch are given in table 4.1.below and the general soils extension survey conducted during the field work are presented in annex-1-

1. **Dark CLAY soils underlain by weathered rocks (HS1).**

These soils are softer with higher moisture content and dark CLAY in which underlying rocks are exposed along the road cut mainly along rolling and escarpment topography. Along such escarpment and rolling terrain there is an exposure of slightly weathered rocks of basaltic composition, especially along the stream cut and road cuts.

The soil is highly plastic and smaller amount of inclusion of finer weathered rock is found in. Genetically the soil is dominated by transported (mainly colluvium) that possess thinner layers. This type of soil in present study is less frequency of occurrence and relatively covers small area along the road path (7.4km). In another way, its extension is not continuous, because it depends on topographic setup. The cracking of ground surface during dry seasons is the most common features indicators of expansive nature of these materials along the foot of hills. Thus from its physical observation, the soils is unsuitable for subgrade materials. Hence certain measurement should be taken prior to the construction of pavement structures.

2. **Dark browns silty CLAY mixed with some weathered rocks (HS 2).**

This is softer and dry in nature with brown colour in which gravel sized weathered rocks of different origin especially basalt is mixed with different proportion from place to place. On physical test, it shows relatively less plasticity due to the inclusion of sand to gravel weathered rocks. In this soil extension an alternating of dark CLAY soils and brown to reddish soils mixed with weathered rock dominated and the soil covers larger stretches of the road path. The soil is thicker and covers most of the flat terrains of the road extension. The alternative of soil types that exposed at road cut sections, shows a paleo-soil nature which formed by eruption gap. This soil can contribute to the instability of a slope facing to the road cut.

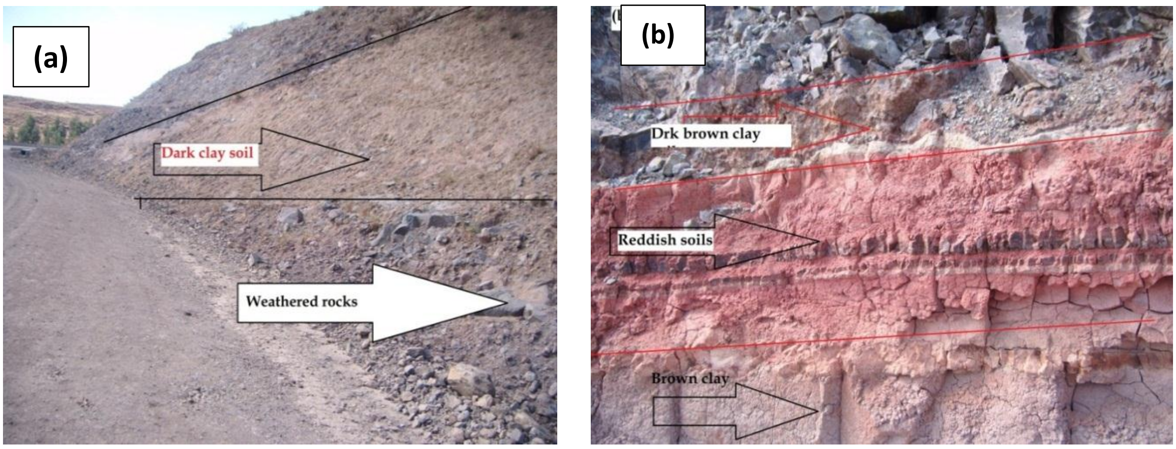


Plate: 4.1 soils of the study area (a) Dark CLAY overlying weathered rocks
(b) An alternative colour of CLAY soils

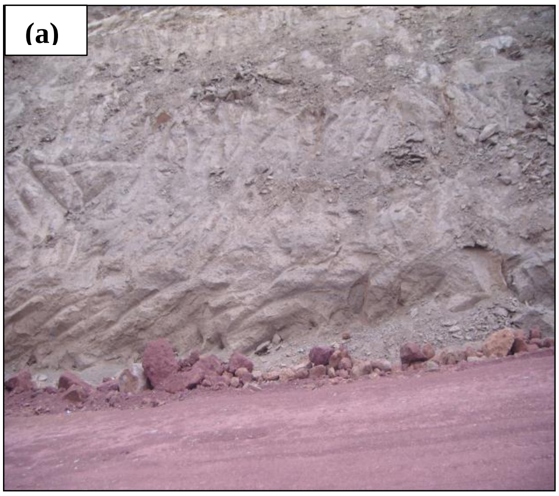
This homogenous section covers approximately about 14.5 km which is about 40% of the total length of road from Sembo to Sholagebeya (36.2 km) and genetically it is both transported and residual. So the soils are less suitable and stable for pavement design.

3. **Light brown silty CLAYs with minor weathered rock (HS 3)**

These soils are comparatively denser and dry which is an ash deposits covers about 15% of the road stretch, which dominated on foot of slope sides facing towards the rift valley. The soil is lighter in colour and less plasticity when rolling on hand. However the soils have a colluvium appearance in which weathered gravelly soils are intercalated with finer soil particles. On its physical property, comparatively the soils are suitable for pavement design because it is thinner and less plasticity in nature.

4. **Rock sections of fresh to highly weathered (HS 4).**

Geologically the area is covered by volcanic rocks of mafic composition basalt and ignimbrite with some felsic rocks of rhyolite. The exposure of basalts along the existing road path has columnar structures that slightly weathered and altered. This exposure used for aggregate as raw materials in the production of concretes and slabs. This section is suitable for subgrade materials on its physical properties; however it possess slope instability problems and form problematic soils as it intensively weathered due to its dominance in feldspar composition. However the rate of weathering and formation of problematic soils are incomparable with the design life time, hence it have not as such problem in this sense.



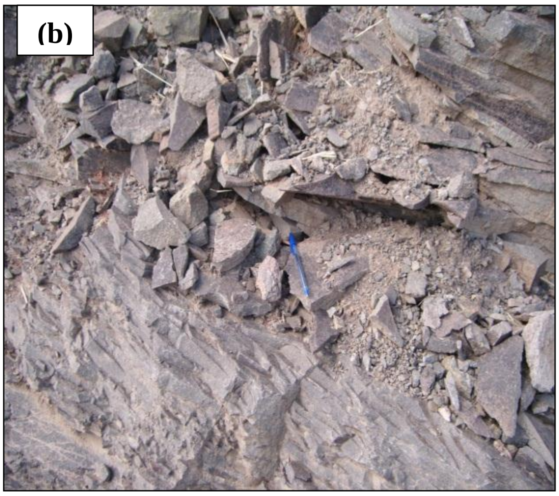


Plate: 4.2. (a) Light brown soils **(b)** Slightly weathered columnar basalt

From the soil extension survey, it is possible to conclude that, the subgrade material along Sembo (0+000)- Nefasamba (26+800) stretch was found to be dominant of dark to dark brown silty CLAY soils underlain by and mixed with moderately to highly weathered basaltic and ignimbrite rock. And the extension from Nefasamba to Sholagebeya dominated with slight to moderately weathered rocks section of basaltic and rhyolite intercalated with other loose soils of light brown to reddish. Hence more consideration should be given to the first stretch.

Table 4.1 Homogeneous sections of the study area on physical properties

HS	Physical identification	Station	km	% cover
HS1	Dark CLAY soils underlain by weathered rocks	0+500-1+500, 2+500-3+000, 4+000-5+500, 6+000-6+500, 7+000-7+500, 8+000-9+000, 10+500-11+500, 15+00-15+500,20+00, 22+500-23+500, 29+00, 31+000-31+500, 33+000-33+500	7.5	21
HS2	Dark browns silty CLAY mixed with some weathered rocks	0+000-0 +500, 3+000-3+500, 07+500-8+000, 9+000-9+500, 10+00, 12+500-13+500, 16+000-17+100, 18+500-19+600, 21+000, 22+000, 22+000, 23+500, 25+000, 26+500-27+000, 32+000-32+500, 36+000-36+500	14.5	40
HS3	Light brown silty CLAYs with minor weathered rock.	14+000-15+500, 19+000, 20+000-21+500, 24+000-24+500, 27+500-28+000, 30+500, 34+000	5.5	15
HS4	Rock section of different weathering	6+500, 12+000-13+500, 18+000-18+500, 21+500-22+500, 24+000-24+500, 25+500-26+200, 28+500, 29+500-30+000, 31+000, 33+00, 34+500-35+500	8.7	24

Therefore based on the available information and field survey, there are some sub-grade materials like that of black cotton (Vertisol), low bearing soils along the project area requiring special treatments especially in HS1 and HS2 of above table.

Topographically the area is dominated by mountainous and rolling physiography, hence small scale landslides caused by human activities and topographic effects are observed during soil extension survey. These landslides are dependent on the slope materials and cut slope geometry (slope height and slope angle), hence it is dominated along steeper road cut on which loose soils are exposed.

Plate 4.3 small scale landslide along road cuts (1022053N, 548200E UTM)

Additionally there are some features such as uniformly tilting of electric and telephone poles along the road routine on a flat topography especially increasing as we go closer to the rift (from station 31+000 to 36+000) and hommoky terrains around station 22+200 that show the presence of a mass movements such as lateral spreading which may be a result of seismic activities or incompetancy of available geological materials along the road collider’s. The tilting of poles shows the presence of lateral spread on a flat surface, and hummocy terrain and tilting of trees on slopy surface are an indication of slow speed of mass wasting.

Plate 4.4 Examples of features indicating mass movements in the study area.

In another way, during the field survey, some features that indicate the expansiveness of the soils along the road colliders are observed especially within the dark CLAY that underlain by weathered rocks (HS1). These features are a desiccated cracks formed during the dry season and closed during the rainy season (plate 4.5). The cracks develop even to centimetres of width and meters of depth that allows the percolation of water and resulted in saturation of materials to great depth in which the treatment of subgrade materials by removal and replacement is being difficult.

From the visual observation, the area has been observed to be dominated by problematic soils of different types, hence samples are collected to examine its engineering properties under anticipated environmental and traffic condition. In addition genetic classification of subgrade materials have been done.

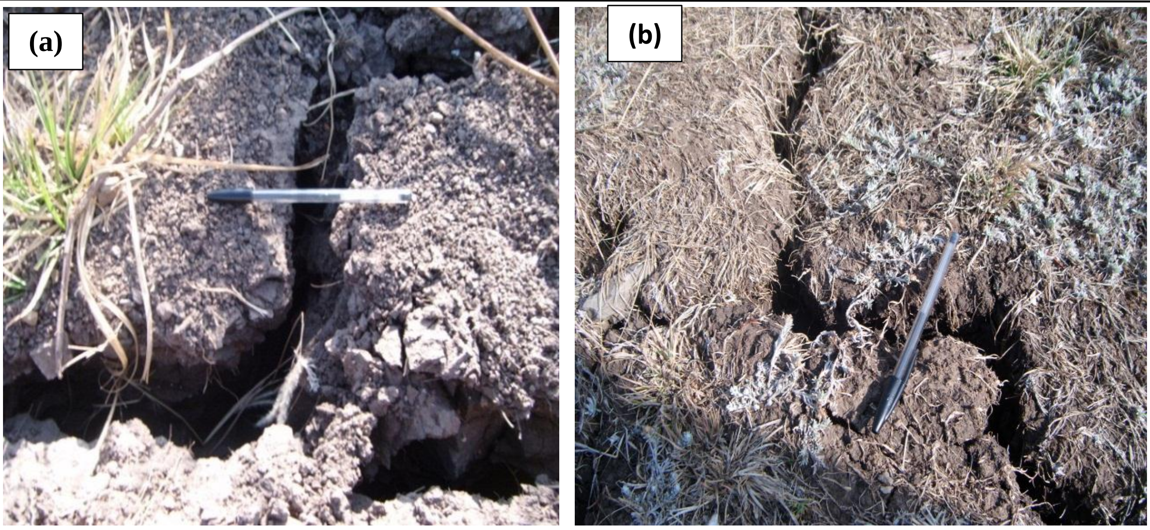


Plate 4.5 Polygonal desiccated crack during dry season (1026769N, 550030E))

4.2.2 Genetic classification of soil in the area

As shown in geological map of the study area (See section 3.5), basalt is the most dominant geological formation with some ignimbrite exposure in the study area. From this map it is possible to know the probable parent materials of the soils being a weathering of volcanic rocks.

Thus, genetically soils are further classified into two types based on place of deposition in relation with its origin. These are residual soils and transported soils and it is adopted in this study too.

(a) Residual soils

Residual soil is a soil that originates from weathered rock and remains at its original site. A profile of the subsurface will consist of a preponderance of soil near the ground surface changing to more rocky with depth until un weathered rock is encountered. So the residual soils are highly weathered at the top and less as the depth increase. In present study area, this soil type is deposited along the flat surface of plateau where the susceptibility to erosion is less. Even it is difficult to determine the accurate thickness of this soil, it is possible to observe along the road cut that the soil is thinner. And also it shows low plasticity as compared to the low laying and transported soils.

This soil is dominated with an intercalation of weathered rock ranging from finer to coarser gravel and classified under HS2 on its physical properties. Accordingly, this soil class in the study area is less problematic.

(b) Transported

The product of weathered rocks can be transported by agents such as stream, wind, glacier or gravitation. The stream and gravitation transportation is the most common agents and form Alluvium and colluvium deposit respectively in the study area.

- **Alluvium soils**

Alluvium soil is a surficial deposit consists of dominantly fine to medium sand, less gravel deposits. In present study area mostly it covers relatively the lowlands areas. It is derived from the weathering, transportation and reworking of different rocks from the plateau, escarpment, mountainous and also from the cliff area by stream erosion. The study area has less stream density, hence the alluvium deposits covers relatively small area. However the low laying deposits are classified under dark CLAY soils (HS1) that possess soft, wet, high plasticity and highly weathered. Hence this soil class is problematic and some measurement should be taken.

- **Colluvium soils**

Colluvium soils are soils that transported from its place of formation to its deposition by gravitational effects. This gravitation effects makes the lose particles to move down the slope of certain degree from horizontal ground surface. As mentioned in chapter 3, about 37% of present study area is covered by mountainous and escarpment. The weathered rocks and soil formed on such area move down the slope due to gravity and form thicker deposit of colluvium soils at the foot of the slope.

During the present study, it has been observed that the thickness of soils increases as the distance closer to the hill foot. This increasing in thickness towards the hill makes some mountains to have a gentle sloping face. Since the soils have the same parent materials, it is difficult to get sharp distinction between each type of soils from field observation, however on the basis of its relative position with topography and drainage, it is possible to identify.

4.2.3 Sample collection

The samples for the required parameters to characterize sub grade materials have been collected at an interval of 500m and in some case three samples per kilometers due to the geological variability and topographic features of the area. In addition, due to the depth of influence of traffic loads on sub grade materials, the sample were collected at a depth ranging from 20 to 100cm, because the amount and intensity of traffic loads expected along the designed pavement are expected to be less effective and the effect of load dissipated at a depth less than 1.5 m below the proposed wearing course.

In general the depth and spacing of sample collection for pavement construction varies depending on standard followed by the designer, the magnitude of loads to be imposed by traffic and geological variability of the route. The magnitude of loads and its intensity is responsible for the variation of depth of influence by the traffic loads.

For pavement design, the depth of influence is usually assumed to be related only to the magnitude and distribution of the traffic loads imposed on the pavement structure under consideration. AASHTO (1993) describes this depth could be 1.5 m (5 ft) below the proposed subgrade elevation; however this depth increased for special circumstances (e.g., if deep deposits of very soft and compressible soils exist).

The zone of load influence under the completed pavement varies with the pavement section, however typically 80 - 90 percent of the applied stress is dissipated and dimensioned within 1 m (3 ft) below the surfacing section of pavement (AASHTO, 1993). Thus standard have been adopted for the present study.

4.3 Laboratory investigation of subgrade

The samples collected from the field have been tested for different parameters that more or less fully characterize and used to analysis the suitability of the site for the proposed project. The result of tested samples has been used for soil classification, engineering geological and mineralogical characterization purpose.

The amount of laboratory testing required for a project would vary depending on availability of preexisting data, the characteristics of the soils, the magnitude of stresses to be imposed by infrastructures and the requirements of the project (Engineering structure to be place).

During the laboratory investigation, prior to laboratory test activities, the samples collected from the sites have been prepared in accordance with the method described in AASHTO T-86-87. This method involves;

i) Air drying of samples and/ or oven drying at 60°C or less and
ii) Breaking up soil aggregation by rubber covered mallet.

In this research, parameters of subgrade materials such as strength, atterberg limit, density-moisture content relation, grain size distribution, compaction, CBR swelling and soil classification to characterize and analysis the suitability of the materials under the existing and adverse condition for the proposed project area were conducted.

Additionally Geochemical and mineralogical tests to know the percentage of CLAY minerals that influence the expansiveness of the soil is conducted on bulk soil samples.

4.4 Soil Classification

Most of the methods for soil identification and classification are based on certain physical (index) properties of the soils which are developed for the purpose of soils application in engineering works. Index properties are indicator in predicting the engineering properties of any soil material and helps in classifying the soil (Hagos G/Tsadik., 2006).

In this study, both American Association of State Highway and Transportation Officials (AASHTO) Classification and United Soil Classification system (USC) of soil classification method for the sub-grade soils have been adopted in which two important index properties tests of soils has been used; (i) Grain size distribution and (ii) Consistency (Atterberg) limits.

Even both classification based on the same parameters, different parameters can be determined from the two. Accordingly from USCS classification, it is possible to determine the presence of silty, clay, the compressibility of soils under loads and plasticity nature. While from AASHTO it is possible to identify the soil content whether it is Gravel (G), sand (S) silty (M), CLAY (C) or fat CLAY. Hence both classifications are adopted and samples are classified under both classifications in this study.

4.4.1 Grain size analysis (AASHTO T27)

Grain size analysis provides the grain size distribution and it is required in classifying the soils into different groups (SATTC, 1988 as cited in Nadew Abdisa., 2010). In present study, the collected samples of soils has been dried by oven and sieved using three sieve of No.10 (2mm), No.40 (0.425mm) and No.200 (0.075mm) opening and the following results are obtained. It shows the soils of the area is dominated by finer (<0.075mm), those contribute to the plasticity of the soils.

Table 4.2 Range of grain passing the three sieves used in present study

Range of grain size passing the sieves (%)	Sieve size used (mm)
32 to 100	2
15 to 96	0.425
7 to 92	0.075

From the field observation and laboratory tested results on grain size analysis, the percentage of grain passing 0.075mm increase slightly as the elevation reduced within one water shade (water dived), this may probably due to the temperature effects that increase with 0.71^{0}F within soil per 31m as elevation decrease that increase weathering rate.
Even the study area categorized under Dega climatic condition according to Ethiopian metrological agency (2000) ranging from 2600 to 3000m above sea level, and it shows slightly coarsening with increasing elevation. Thus the low laying soils tend to be more problematic comparatively in present study area.

On the other hand, it have been observed that, the soils of the same origin found at the hill side is different from the soils on the top of the hill only in grain size, this may be due to the effect of erosion makes the soils below the hill foot to be finer and more plastic than the residual soil.

Thus the performance of subgrade materials depends on physical properties especially grain size which control the expansiveness of materials. Expansive CLAY has finer CLAY minerals of intensive weathering product. In general, the finer the materials are the higher probability of expansive nature and less suitability for subgrade.

4.4.2 Atteberg limit tests (AASHTO T89-91)

As aforementioned the liquid limit is water content where a soil changes from plastic to liquid behavior. At liquid state, a soil contains higher water content. It offers no shearing resistance, and can flow like liquids and has no shear deformations.

The plasticity index (PI) is a measure of the plasticity of a soil. The plasticity index is the size of the range of water contents where the soil exhibits plastic properties. The PI is the difference between the liquid limit and the plastic limit (PI = LL-PL). Using the casagrande,s method, the study area gives the following result.

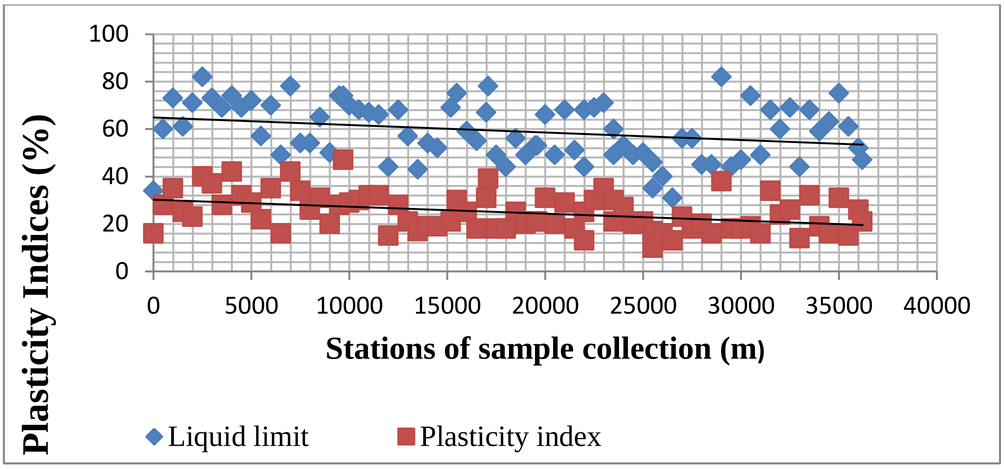


Fig 4.1 Distribution of plasticity indices of the tested samples

Figure 4.1 shows that, when the distance is closer to rift valley, both liquid limit and PI show slightly decreasing trends, this shows that the soil found far away from is more dominated with coarser materials and denser that poses less plasticity natures. This may be related with the degree of weathering of rocks that observed on the field that shows decreasing towards the rift. This may require sound reason; however detail study should be conducted to know the reason. But it might be relate with the age of the soils and deposition of finer materials could be at higher distance as compared with the coarser one.

Generally the plasticity of soils shows certain relation with grain size. Accordingly, from the tested result (figure 4.2), it has been observed that as the percentage passing the 0.075mm sieve increase the plasticity of the soil slightly increase. This shows that the plasticity of soils is governed by the percentage of finer materials (CLAY).

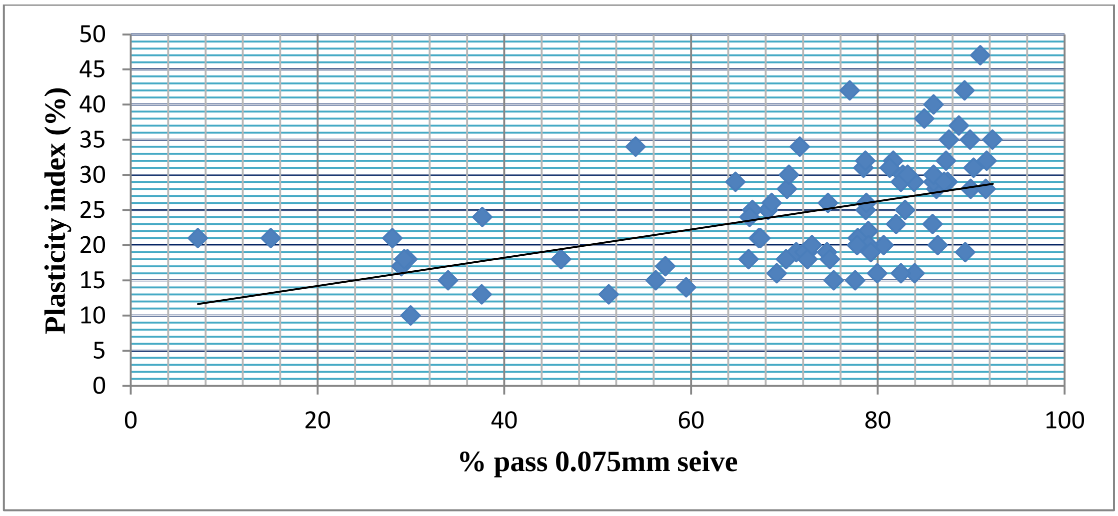


Fig 4.2 Relation between grain size and plasticity index

In addition, there is weighted plasticity index (PIW) parameter that used to determine the requirement of extended investigation on selected area.

Weighted plasticity index is a function of grain size and plasticity index of the soils which can be determined using the empirical methods, and used to indicate whether extended investigation should be performed in detail or not. According to ERA site investigation manual, (2002), the subgrade materials having PIW greater than 20, needs to conduct extended investigation. The PIW of subgrade soils can be calculated using equation 4.1

PIW=PI*(% passing 0.425mm)/100--------------------------------------eq.4.1

Using the above equation, the PIW of the soil samples has been calculated and range from 2 to 43, in which about 34 % of the samples have plasticity index weight of greater than 20, that almost all the samples have PI >25 and LL>60 which dominated along flat surface of poor drainage especially HS1 and HS2 homogeneous section of soil extension survey.
These shows us that the location from which these samples have been collected could be problematic, hence an extended investigation have been conducted to investigate the area in detail. The extended investigations include simple additional laboratory tests to estimate expansiveness and should be employed routinely whenever special measures against damage from expansive soils are expected.

Finally the shrinkage limit (SL) can be determined from Casagrande's plasticity chart by extending U-line and A-line until they intersect (University of Texas, 2002). Then plotting a line from the intersection point to the sample placed on the chart and the intersection of line connecting the U-line-A-line intersection with the sample at X-axis is the shrinkage limit of the sample.

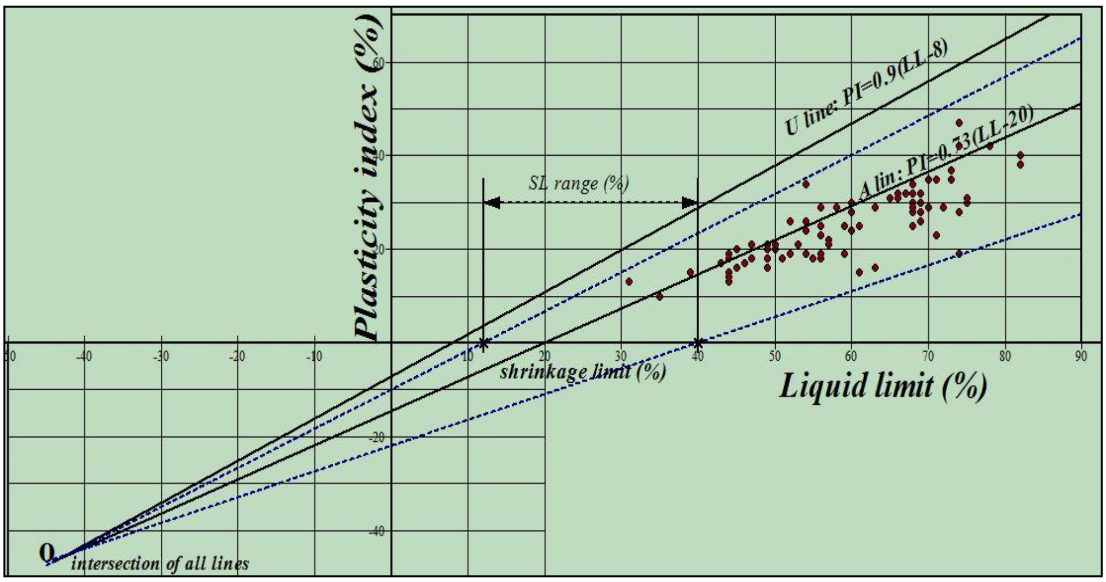


Fig 4.3 Shrinkage limit determiation from plasticity chart

Accordingly for the present study, the maximum and minimum shrinkage limits are determined using the chart. Thus it ranges from 12 to 40%, in which the higher LL and lower PI possesses higher shrinkage limit (HS 1 of soil extension survey) **(fig. 4.3)**

4.4.3 Group index (GI) (AASHTO M145-91)

A group index value (GI) is used to provide a measure of quality of a soil as highway sub grade material. The group index is given as an empirical function that related to grain size and plasticity index result as per AASHTO M145-91; equation 4.2 is used for all soil groups, except A-2-6 and A-2-7.

$$GI = (F-35)(0.2+0.005*(LL-40)) + 0.01(F-15)(PI-10) \text{ ....eq. 4.2}$$

Where, F is % finer than #200 sieve size (0.075mm) and PI is plasticity index

GI is expressed in a nearest whole number. If GI value is less than 0, set it to 0. If any terms in the above equation are less than 0, set them to 0. For them partial group index is used. The higher the group index, the lower the quality of soil as subgrade material.

However group index for A-2-7 and A-2-6 soil groups is given by;

$$GI = 0.01(F - 15)(PI - 10) \text{ ....eq. 4.3}$$

The first term in equation 4.2, became zero because the percentage passing sieve # 200 is less than 35 and LL is less than 40 in A-2-6 and A-2-7 soil classes (AASHTO, 1993)

Table 4.3 Soil classification with its Group Index values of present study

AASHTO Soil classes	Group index (GI)	Number of samples	% occurrence	Remark
A-2-6	GI<20	1	100	Suitable for subgrade
A-2-7	GI<20	7	100	Suitable for subgrade
A-6	GI<20	2	100	Suitable for subgrade
A-7-5	GI<10	3	5.5	Suitable for subgrade
	GI<20	16	29.6	Intermediate, require treatment
	GI >20	35	64.8	Poor for subgrade materials
A-7-6	GI<10	3	16.6	Suitable
	10<GI<20	11	61.1	Intermediate, require treatment
	GI >20	4	22.3	poor for subgrade

According to AASHTO, 1993 the materials should have a group index less than 20 to be used as subgrade. In this study about 48% have GI>20 in which about 43% is A-7-5 and 5 % is A-7-6 subgroups. However, this parameter has not been used by different projects in our country.

4.4.4 AASHTO Classification

AASHTO soil classification is a textural-plasticity classification that uses sieved fractions and Atterberg limits for categorizing of soils to 7 main groups and 12 subgroups (A-1 to A-7). Accordingly A-1 to A-3: is granular or coarse grained soil while from A-4 to A-7 are silt-CLAY or fine grained soil. In this classification the groups and sub groups are differentiated on the basis of their grain size and attererg limits.

The AASHTO soil classification is used to determine the suitability of soils for earthworks, embankments and road bed materials (sub base and sub grade). The AASHTO sub-grade soil classes for various soil samples collected during present study are summarized in the table 4.5 and all the samples classifications are given in annex-2

Table 4.4 AASHTO classifications system of soils

General classification	**Granular materials (35 %or less of total sample pass No #200 sieve)**							**Silt-CLAY materials (more than 35% of total sample pass sieve No. 200)**			
Group classification	A-1			A-2							A-7
	A-1-a	A-1-b	A-3	A-2-4	A-2-5	A-2-6	A-2-7	A-4	A-5	A-6	A-7-5/6
% passing:											
No 10 sieve	50max	-	-	-	-	-	-	-	-	-	-
No 40 sieve	30max	50mx	51mn	-	-	-	-	-	-	-	-
No 200 sieve	15max	25mx	10mx	35mx	35mx	35mx	35mx	36mn	36mn	36mn	36mn
Characteristic of #200 passing											
Liquid limit		-	-	40mx	41mn	40mx	41mn	40mx	41mn	40mx	Mn*
PI		6mx	NP	10mx	10mx	11mn	11mn	10mx	10mx	10mx	10mx
Group index	0	0	0		0		4mx	8mx	12mx	16mx	20mx

- ✓ The maximum value given for GI is the allowable to use the materials for subgrade
- ✓ PI <LL-30 (A-7-5) whereas PI>LL-30 (A-7-6)

Table 4.5 Summary of AASHTO classification of homogeneous section

AASHTO Classification	**Homogeneous section on soil extension survey**					
	HS1	**HS2**	**HS3**	**HS4**	TOTAL	%
A-2-6	0	0	0	1	1	1.2
A-2-7	0	3	1	3	7	8.6
A-6	0	0	0	2	2	2.4
A-7-5	22	18	8	6	54	65.8
A-7-6	2	5	5	6	18	22
Total	**24**	**26**	**14**	**18**	82	

4.4.5 USCS Classification (ASTM D-2487)

In this classification system, paired letter symbols are used to indicate its predominant grain size and plasticity in fine grained soils which are determined from sieve analysis and Atterberg limit tests. In the same manner, in coarse grain the first letter indicate the dominant grain size and the second shows gradation for clean (little or no fine) soils and the presence of silt and CLAY size particles for soils with appreciable amounts of fines. In fine grained the second symbol indicates the plasticity of the soils which obtained from laboratory investigation.

The plasticity index is an important index property of soil for USCS classification. It is one of the important parameters used in the classification of soils, determines its suitability and is also an indicator of whether the soil is expansive or not.

According to USCS soil classification fine grain soils are classified into two depending on their compressiblity as **(i)** low compressible and plasticity (L) if the liquid limit is 50% or less and given with the symbols ML, CL and OL. **(ii)** Soils of high compressibility (H) if the liquid limit is more than 50%. These are given with the symbol MH, CH and OH. The types of soils can be identified on plasticity chart based on their location with respect to A-line (PI=0.73(L-20) that separate CLAY from silt.

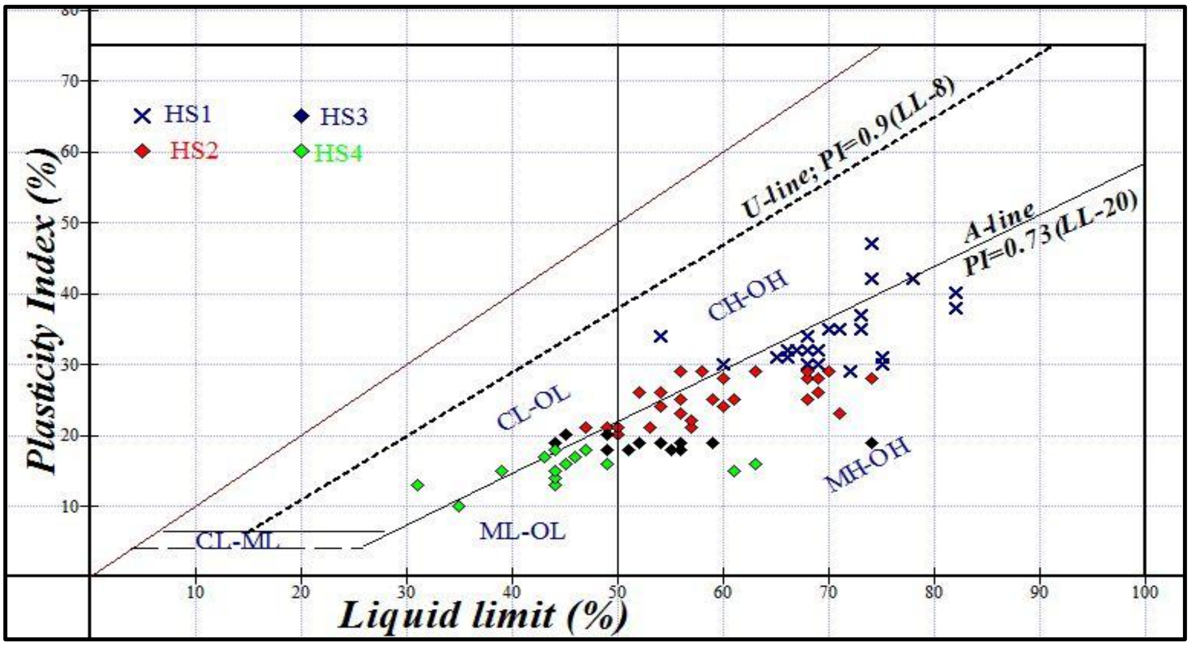


Fig: 4.4 Casagrande LL-PI chart and USCS classification of persent result.

In USCS classification, Casgrande’s chart has been used in which the soils are grouped by using its position relative to U-line, A-line and its plastic limits. Accordingly the classification chart of the collected samples from different area in this study is present on fig 4.4 and summarized in table 4.6 below.

Table 4.6 Summary of USC classification of tested samples

USC classification	No. of samples	Position on casagrande's chart	Plasticity indices
CL or OL	7	Above A-line	LL<50
CH	9	Above A-line	LL>50
MH or OH	50	Below A-line	LL>50
ML or OL	16	Below A-line	LL<50
CL-ML	0	Above A-line	LL<30 and PI between 4 to 7
	82		

From table 4.6, about 72% of the total samples have liquid limit value greater than 50%, that classified as highly compressible soils, hence it indicate the area have compressibility problems. However the homogeneous section grouped under Rock section (HS4) have less liquid limit, thus it show low compressibility.

To differentiate the types of CLAY minerals in the fine grained materials from consistance limit, the chart developed by Holtz and Kovacs in 1981 has been adopted in present soil classification. The plot is shown as Fig. 4.5 and the dominancy of mineral are summarized in table 4.7; which clearly indicates that the subgrade soil of the present study area is dominantly composed of kaolinite (about 60 %).

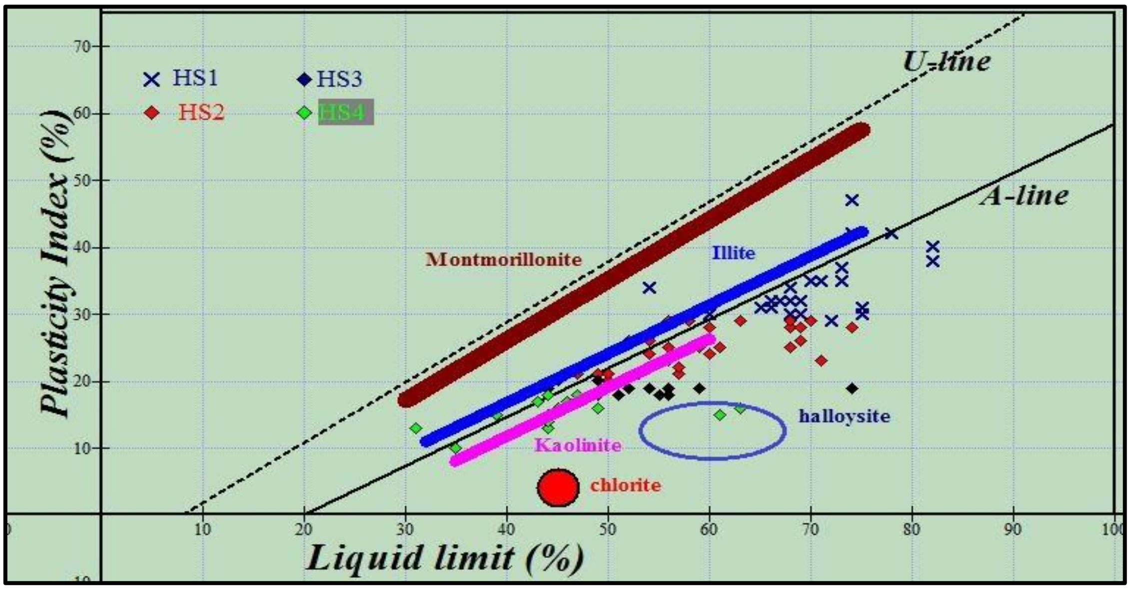


Fig 4.5 Casagrande's LL-PI chart and respective CLAY minerals

Generally the soil index properties could be correlated with some topographic and drainage patterns. Accordingly the grain size passing sieve 0.075mm, plasticity index and plastic limit are relatively high on low-lying flat areas and thicker deposits. Similarly the thickness of soils increase at the foot of the hill and reduced as we go away from the mountain areas.

Table 4.7 summary of mineralogical classification on Casagrande's chart.

Minerals content	**Location on LL-PI chart**	**HS1**	**HS2**	**HS3**	**HS4**	**%**	**Degree of problematic**
Montmorrilonite	Above U-lie	-	-	-	-	-	Very high
Illite	B/n U-line and A-line	9	6	3	4	37.4	High
Kaolinites	Below A-line	21	20	9	8	60	medium
Halloysite	High LL with low PI	-	-	-	2	2.57	low

There are sections of soils that can be classified with casagrande's charts as illite (problematic), but it have low PI (<20) value. All the HS4 samples have PI<20, but 30% are under illite group. Hence the classification may not give accurate result.

4.5 Engineering properties of subgrade soils

4.5.1 Subgrade strength (CBR) test

Subgrade strength and bearing capacity being the main design input for determining the pavement layers thickness and performance. Subgrade strength has been assessed using Californian bearing ration (CBR).

The CBR method is probably the most widely used method for strength test of materials in designing pavement structures. CBR-value is used as an index of soil strength and bearing capacity. This value of sub grade materials is broadly used and applied in design of the base and the sub-base material for pavement (IOWA, 2013). The higher the CBR value of a particular soil, the more strength it has to support the pavement. This means that a thinner pavement structures; base and sub base could be used on a subgrade soil with a higher CBR value than on a subgrade soil with a low CBR value.

As aforementioned, quality of pavements section increase upward from sub grade materials towards base and surfacing materials so CBR value.

The CBR test can be performed in two ways; one point and three point methods. In one point test, the procedure is conducted using one mold in which five layers of relatively equal thickness with 65 blown of standard compaction is prepared, while three point CBR test is conducted using three mold with 5 layers compacted under 56 blown each. This three point CBR test is commonly used, because it is more accurate since the average of the three results are taken. Since CBR is a measure of soil strength and its bearing capacity, different countries develop a certain standards to use the materials as sub base or subgrade that fulfill the required criteria.

ERA standard manuals recommend that, materials to be used for subgrade purpose, it should have a minimum CBR value of 6 at 95% MDD, unless it should be avoided or improved using different methods.

Table 4.8 Relative CBR Values for Sub-base and Subgrade Soils

Material	CBR (%)	Rating	Common materials
Sub base	> 80	Excellent	Mostly high graded metamorphic rocks
	50 to 80	Very Good	
	30 to 50	Good	Fresh sedimentary rocks
Subgrade	20 to 30	Very good	Fresh sand and gravels
	10 to 20	Fair-good	Weathered Sand and gravels
	5 to 10	Poor-fair	Silt and sandy soil
	< 5	Very poor	CLAYs

Source (IOWA, 2013)

In this study CBR-test was conducted to characterize the strength and the bearing capacity of the soils along the road collider's in which the soil samples are collected at an interval of 0.5 km following the soil visual extension. But the interval is varied from station to station depending on the variability of geology along the road collider.

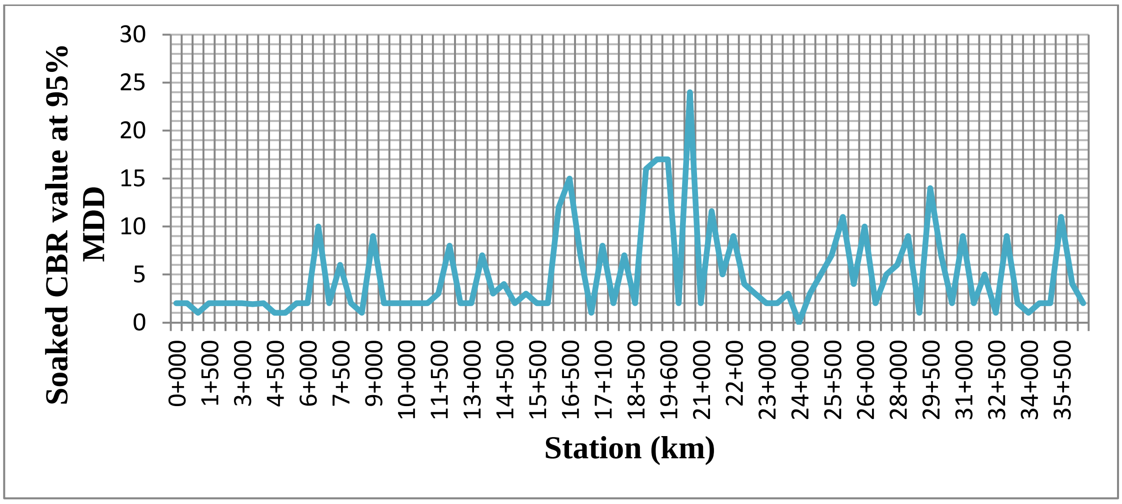


Fig 4.6 Graph showing CBR values of tested samples

From the result given in table 4.9, about 70.5% of subgrade materials along the road colliders have a CBR<6, which are classified under unsuitable according to ERA standard manuals. In addition the correlation between CBR value and compaction test show that for most of the subgrade materials, the materials with increasing moisture content and decreasing dry density, result in reduction of CBR that reduce the resistance of materials to deformation. Thus the CBR value of a soil is sensitive to its moisture content, hence the soaked CBR value shows very low as compared with un-soaked CBR; so the laboratory CBR value doesn't accurately represent the field condition. This show the moistre content negatively affects the strength of subgrade materials.

Table 4.9 ERA, 2002 Strength class on CBR value of the present study

Strength classes	CBR value (%)	No of samples	% occurrence	Strength classes	Remark
S1	>=2	38	46.9	S1,S2 & S3	Unsuitable (CBR<6)
S2	3-4	12	14.8		
S3	5-7	11	13.6	S3,S4,S5 and S6	Suitable (CBR>6)
S4	8-14	15	18.5		
S5	15-29	5	6.17		
S6	>=30	0	0		

4.5.2 Compaction test (modified protocol test) (AASHTO T-180)

Compaction is the process of increasing the bulk density of a soil or aggregate by driving out air and water. For any soil, for a given amount of compactive effort, the density obtained depends on the moisture content. Hence this test is conducted to construct moisture density relationship of soil materials. There are two types of proctor tests. These are standard and modified proctor test.

In the standard proctor test the material is compacted in three equal layers by using a rammer consisting of a 2.5kg mass falling freely through 305mm, each layer receiving 25 blows.

In modified proctor test (used in present study) the oven dried soil sample of known weight is taken and mixed with water of known percentage of the soil sample. In this test 3 to 4 proctors are done with different percentage of water depending on natural moisture content. The sample is compacted in five approximately equal layers that subjected to 56 blown each with a rammer mass of 2.5kg

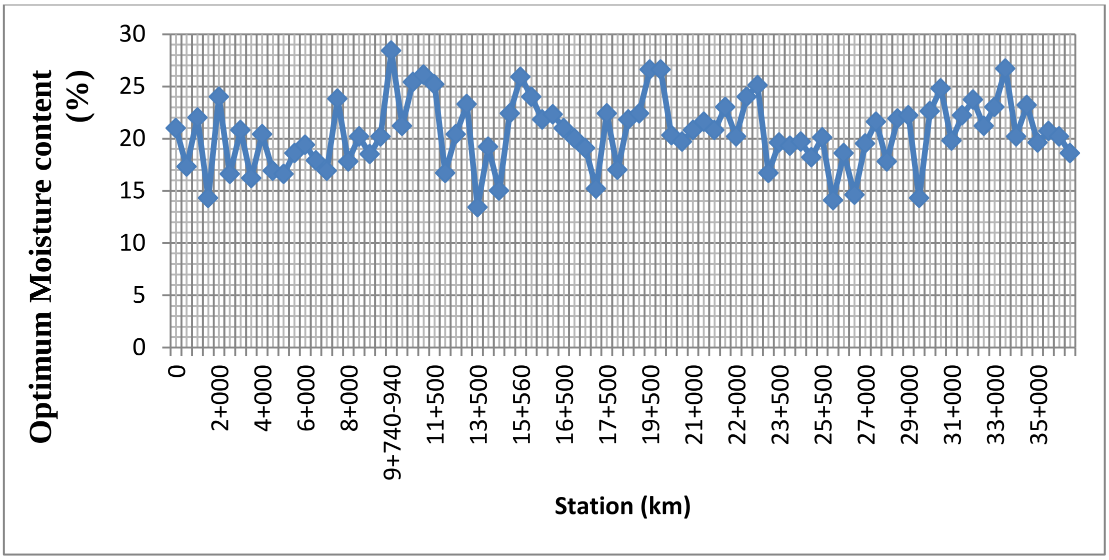


Fig 4.7 Optimum moisture content of soil samples

. Thus the optimum moisture content of present samples are ranging from 14 to 28 %, in which it reduced with the increasing the percentage of coarser grain size in the samples. However the maximum dry density shows inverse relation with optimum moisture content.

4.5.3 Determination of Swelling of soil

Many soils and rocks have the potential to swell under moisture change. That is a function of its mineralogical composition and environmental condition. The actual swelling will be caused by a change in the environment in which the material exists. Different literatures used both swelling and swelling potential (pressure) to show the materials' expansiveness.

When a dry soil becomes wets, during the first stage it undergoes three dimensional volumetric expansions, because its desiccation cracks are still opened. In a second stage, after desiccation cracks were closed, soil volumetric expansion is only one dimension, causing the rising of soil level up. The volumetric change or increasing of level affects the stability of materials overlay such swelling soil.

Swelling potential is the measure of increment in volume of soils due to the addition of excess amount of water, but addition of water to any kind of soils does not resulted in swelling of soils. Hence swelling property depends on the types of soils. Accordingly the soils which have high percentage of CLAY with montimorrionite mineral composition highly swell. Swelling of CLAY soils can be determined directly or indirectly.

i) Direct determination of swelling (CBR swelling)

In the present study, swelling potential of subgrade materials has been done simultaneously with CBR of three point methods of 10, 30 and 65 blown of sample in 3 different molds to take the average of their results. Percentage swelling for each mold of different blown is given by;

$$\%\ \text{CBR Swelling} = \frac{\text{Reading after soaking} - \text{reading before soaking}}{116.34} \text{.............Eq. 4.4}$$

Using the above equation, the CBR swelling percentage of collected samples has been determined from the test results. From the comparison of percentage swelling obtained from the three molds, it is clear that the percentage swelling decrease as the number of blown used increases, because the more the load used the more the soil became compacted and void space escaped out; then small amount of water can be absorbed and swelling will be less.

About 82 samples are tested for CBR swelling in which 67% of samples resulted in high to very high degree of swelling which need especial treatment.

Table 4.10 Summary of CBR swelling of the samples

CBR Swelling range	No. of occurrence	Percentage	Degree of swelling	Remark
0-2.5	27	33	Low	Suitable for subgrade
2.5-5	22	26.8	High	Unsuitable for subgrade according to project specification.
>5	33	40.2	Very high	

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

$$\text{FSI} = \frac{\text{Final volume (vf)} - \text{initial volume(vo)}}{\text{Initial volume(vo)}} \text{x}100 \text{........................ Eq. 4.5}$$

However this method of swelling estimation has not been conducted in this study.

ii) Indirect methods of swelling determination

- **swelling determination using Seed et al., 1962 model**

According to Seed et al (1962) the swelling potential of a subgrade soils can be determined indirectly from plasticity index of CLAY soils. In this method the swelling potential is directly proportional with plasticity index of the soils.

$SP = 60K (PI)^{2.44}$ ……………………………………………….…………Eq. 4.6

Where: SP = swelling potential, PI = plasticity index and $K = 3.6 * 10^{-5}$ a factor for CLAY content between 8 & 65%

Using the above equation the swelling potential of tested samples have been calculated in this paper and the result indicates that most of the materials in the study area is fall under medium to high swelling potential of Seed et al (1962) classification.

Table 4.11 Estimated swelling potential from (Seed et al., 1962 model)

Range of swelling potential	Degree of swelling	No. of samples	Percentage occurrences
0-1.5	**Low**	4	5
1.5-5	Medium	36	44.4
5 to 25	High	40	49.4
>25	very high	1	1.2

From the above calculation, about 72% of the subgrade materials have swelling potential greater than 2.5, which is the boundary of suitable and unsuitable subgrade soils in pavement design according to the project specification. This shows that the evaluation of CBR swelling using laboratory tests and Seed et al (1962) formula gives almost the same result that categorized under the same degree of swelling to indicate expansiveness. So it is possible to evaluate qualitatively the swelling potential of soils from plasticity index.

- **Potential Swell Based on Plasticity Chart (Hortz and Gibbs, 1956 method)**

Holtz and Gibbs (1956) also demonstrated that the plasticity index and liquid limit are useful indices for determining the swelling characteristics of most of the CLAYs.

Accordingly the following scenarios are made;

- Soils with plasticity index that is above 35 and LL>70 have very high swell potential.
- Soils with PI>25 and LL>50 have high swelling potential
- Soils with PI<25 and LL<50 classified as low to high swelling potential.

The plasticity chart alone, which plots plasticity index against liquid limit, helps to detect the swelling potential of soils by looking the place where the soil samples fall in the chart. The swelling potential for any given plasticity index and liquid limit is indicated. For the present study, the samples tested for plasticity index were plotted on plasticity chart (Fig 4.8)

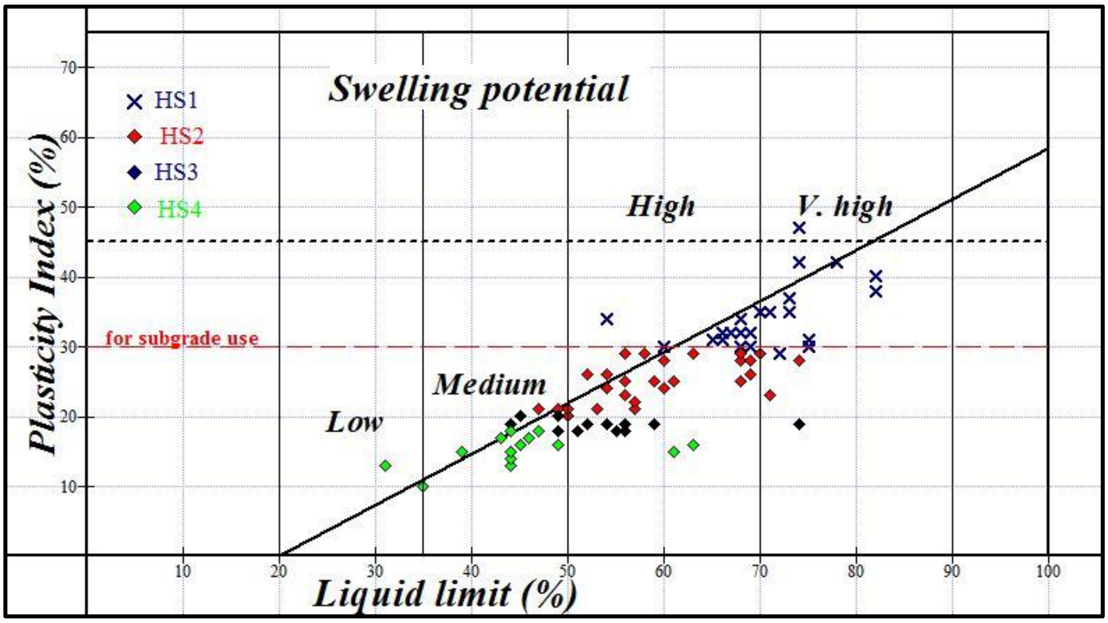


Fig 4.8 Potential Swell of Soil Samples present study based on Plasticity Chart (Daksanamurthy, 1973 as cited in Habtamu Solomon, 2011)

As shown in (Fig. 4.8), most of the samples (60%) fall in the high, about 20% in very high, 17.3 % in medium and 2.7% in low swelling potential region. The dark brown silty clay of underlained and mixed with weathered rocks (HS1 and HS2 of soil visual inspection) fall under high and very high swelling potential. However the scattering of samples on this plasticity chart shows the variability of swelling potential ranging from low to very high within a short distance (36km).

Generally to classify soil into different degree of swelling; different authors use variable parameters as the basis for classification. For instance; Seed et al (1962) use PI, Chen (1988) use LL and Raman (1973) use LL with different values of class limits. The comparisons of their classification are given in table below on present study.

In all of the classification in table 4.12, the high degree of expansive have approximately similar amount of samples, however the result of Chen (1988), classification is very far from the two classifications.

Table 4.12 Classification of present study results for degree of swelling potential

Degree of expansion	**Chen (1988)**		**Seed et al., (1962)**		**Daksanamurthy and Raman (1973)**	
	Criteria	(%)	Criteria	(%)	Criteria	(%)
Very high	LL>60	41.46	PI>35	7.32	LL>70	17
High	LL (40-60)	54.8	PI(20-35)	53.6	LL (50-70)	52.4
Medium	LL (30-40)	3.7	PI 10-20)	39	LL (35-50)	28
Low	LL<30	0	PI<10	0	LL (20-35)	2.44

From this comparison, it is observed that the Chen (1988) classification overestimate the degree of expansion of soils, hence it possible to adopt his classification for worsts condition.

4.5.4 Linear Shrinkage

Linear shrinkage is one of the properties that indicate the expansiveness of soils under moisture change. This parameter can be determined using half mold (AASHTO T-92), in which moist soils at its optimum moisture content is molded and dried, then the soil shrink and the amount of shrinkage can be measured.

Linear shrinkage in present study has been calculated using the Arora (1997) formula that depends on the consistence of the soils, in which linear shrinkage is directly proportional to plasticity index.

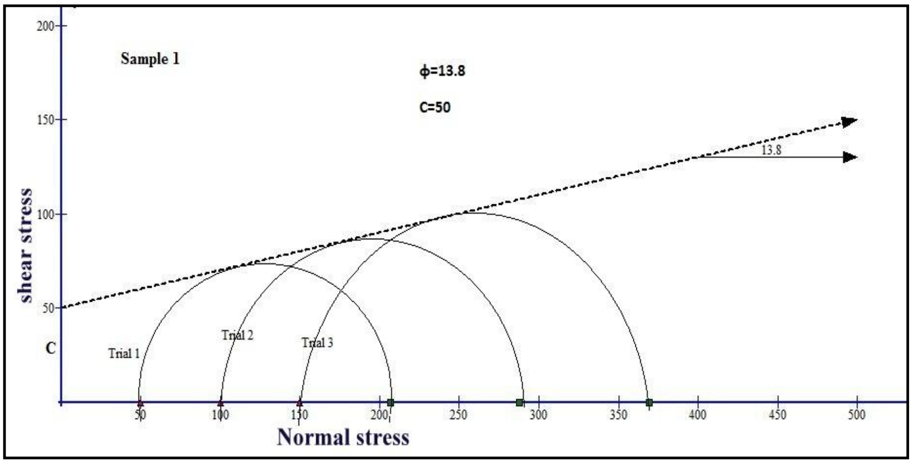
LS=PI/2.13……………………………………………………………Eq. 4.7

Accordingly the present study ranges from 5 to 24, in which about 47% have LS> 12 that possess PI>25%, hence these samples are problematic. Thus the value of LS=12 should be taken as the maximum value for subgrade materials, when PI determination is impossible.

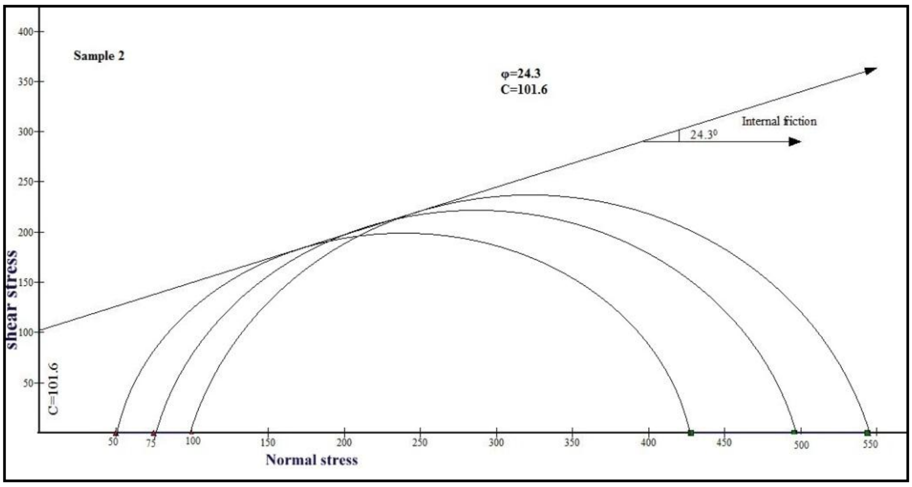
4.5.5 Triaxial Test (UU)

Triaxial test is used to determine the cohesion and internal friction of undisturbed or remolded soils as well as stress-strain relationship (Alemgena Alene., 2011). This test can be consolidated drain test (CD test), consolidated undrained (CU test) and unconsolidated undrained (UU) test (D2850-- www.astm.org). The consolidated undrained method is the most accurate and used for determining the load bearing behavior of soils under adverse conditions.

However the Unconsolidated Undrained (UU) method have been used in present study on two selected undisturbed soil samples to determine its bearing capacity under existing condition by evaluating its internal friction angle and cohesion which are sensitive to moisture content and dry density (Alemgena Alene., 2011). In this method, total stress are applied on a sample without pore water pressure correction, the compressive strength of a soil is determined in terms of the total stress, therefore, the resulting strength depends on the pressure developed in the pore fluid during loading. Accordingly the two samples are subjected to three trials of increasing axial stresses to calculate its maximum principal stress at which it fail. After the determination of the maximum principal stress, the result is plotted using Mohr-Columbs' failure criteria from which the shear parameters can be evaluated.



Sample 1
$\Phi = 13.8^0$
C=50.1Kpa
MC=33.5%
DD=1.24 KN/m^3



Sample 2
$\Phi = 24.3^0$
C=101.6Kpa
MC=32%
DD=1.48 KN/m^3

Fig 4.10 Mohr-Coulomb failure criterions for two selected soil samples at natural moisture content

From the tested result it has been observed that the two shear parameters (cohesion and angle of internal friction) decrease as the dry density decrease. Thus it can be concluded that the cohesion and internal friction of the soils is dependent of soil types, moisture content and compaction (density), so the bearing capacity of soil. The sample that possesses higher internal friction and cohesion (sample 2) is physically granular and higher in CBR strength, so the triaxial test give a correlable and supportive result with the engineering parameters. However the triaxial tests have limitation on uniform distribution of stress in all height of the sample and the speed of application of confining pressure.

On the other hand, the triaxial test is used to calculate the bearing capacity of soils especially foundation of building using Terzaghi (1943) methods. The triaxial is not a common parameter used in subgrade characterization. However using Terzaghi bearing capacity theory, the following results are calculated.

Table 4.13 Triaxial test result of selected samples

Samples	unit weight	Depth (m)	Width (m)	Surch arge (kpa)	ultimate bearing capacity	FS	Allowable load (Kpa) @ FS=3	Allowable load (Kpa) @ FS=1.5
sample 1	12.2	1.5	1	18.3	274	1.5-3	91	182
sample 2	14.8	1.5	1	22.2	784	1.5-3	264	528

Depth is an assumption that the maximum depth of natural subgrade materials (total thickness of overlaying layers) after pavement layer placed would be 1.5m below the wearing course. From this table, the area possesses lower ultimate bearing capacity. Thus FS=3 is the lower limit of soils to be suitable for foundation of building. By taking the factor of safety ranging from 1.5 to 3 that includes sliding to bearing capacity failure, the maximum load allowed to be supported by the soils without failure under the required factor of safety have been obtained. Thus the subgrade materials have lower allowable capacity as compared with traffic load, hence it require some measurement to increase its bearing capacity.

4.6 Geochemical and mineralogical composition of subgrade materials

From the field survey and laboratory result of index properties, the presence of expansive soils has been identified. As aforementioned these soils has a properties of volumetric change which manifested physically by development of cracking, high plasticity indices, low bearing capacity (CBR) and high swelling potential.

The expansiveness of such soils is due to the presence of CLAY minerals, which is fine grained (0.002mm) or less. According to Chen 1988, Murray, 2007 and Nelson, 2010, CLAY minerals are crystalline hydrous alumino-silicates derived from parent rock by chemical weathering. There are two basic building blocks of CLAY minerals; the silica tetrahedron and the alumina octahedron those combine into tetrahedral and octahedral sheets to form the various types of CLAYs.

The three CLAY minerals that commonly important in engineering studies are Illites, Kaolinite and Montmorillonite (smectite). Hyallosyite and chlorite are not as such concern in engineering study since both are not problematic and less abundance in nature (http://www.home-inspections-radon-testing.com/expansive_soils)

Kaolinite is a two layers mineral of octahedral and tetrahedral which bonded by relatively strong hydrogen bond. Due to its strong bond, kaolinite does not absorb water, hence does not expand when it comes in contact with water (Murray, 2007).

Montmorillonite groups have 1-2 layers structures formed by octahedrons between two tetrahedral layers. This group of CLAY minerals has the ability to absorb water molecules between the two layers, causing the volume of the minerals to increase when they come in contact with water (Nelson, 2010).

Expansive soils can be identified using different methods such as indirect index test or direct expansion test (Chen, 1988), grain size using hand specimen or microscope (Nelson, 2010) and geochemical methods (Fell et al., 2005)

Fell et al. (2005) described the identification techniques of CLAY minerals in soils and recommended to apply at least two of them at a time. According to these authors, the mineralogical identification techniques X-ray diffraction, scanning electron microscope and differential thermal analysis and the indirect index property methods Casagrande's plasticity chart.

- **X-Ray Diffraction (XRD)**

XRD is the most widely used method for identification of CLAY minerals and to study their crystal structure. In present study, in addition to Casagrande's plasticity chart methods of identification, Geochemical test using X-ray diffraction methods have been conducted on five selected soil samples to know which CLAY minerals are dominated in the study area and confirm its classifications with previous works.

The XRD test is used for bulk to determine the overall constituent. However XRD test on CLAY fractions is used to quantify the major, minor and trace composition of the CLAY species, but in present study it didn't conducted due to the absence of materials in Geological survey central laboratory. The tested samples shows quartz (0 to 15 %), albite

(8.1 to 82.5), Kaolinite (0 to 33.1), hematite (10.5 to 86.9%), sanidine (0 to 36.5) and laboradorite (0 to 9.1) as presented in table 4.14

Table 4.14 comparisons of XRD and Engineering properties test

Minerals Composition	**Chemical formula**	**Samples**				
		25+500R	**26+500R**	**29+000R**	**30+000L**	**36+200R**
Quartz	SiO_2	**13.7**	**-**	**-**	**-**	**-**
Feldspar (Albite)	$Na(AlSi_3O_8)$	**-**	**8.1**	**56.4**	**-**	**82.5**
Kaolinite	$Al_2Si_2O_5(OH)_4$	**6.7**	**4.9**	**33.1**	**4.9**	**-**
Hematite	Fe_2O_3	**12.9**	**86.9**	**10.5**	**86.1**	**17.5**
Sanidine	$Na(Al_2Si_2O_8)$	**66.5**	**-**	**-**	**-**	**-**
Labradorite	$Ca_{0\cdot 64}Na_{0.35}(Al_{1\cdot 63}Si_{2.37}0_8)$	**-**	**-**	**-**	**9.1**	**-**
Result from Engineering site laboratory						
AASHTO		A-2-6	A-6	A-7-5	A-7-6	A-2-7
USCS		CL-OL	CL-OL	MH-OH	ML-OL	CL-OL
Based on LL-PI chart (Casgrande's)		Kaolinite	Illite	Illite	Kaolinite	Kaolinite
Liquid limit		35	39	82	47	47
Plasticity index		10	15	38	18	21

The lower percentage of quartz in all the samples confirmed with the atterberg result that indicated especially the sample 29+000 to be expansive with higher plasticity. Similarly the samples those shows higher percentage of kaolinite and albite (phylosillicate end- member) with compare to the others can make the soils to be expansive.

However the samples 25+500R, 30+000L and 36+200L which are classified as kaolinite on casagrande's chart shows small amount of CLAY (kaolinite) minerals 6.7%, 4.9% and 0% respectively on bulk XRD test, hence the mineralogical classification base on plasticity chart is not as such accurate or the reason why soils with small CLAY content can be expansive should be studied.

Albite constitutes the sodium end-member of the plagioclase feldspar solid solution series and alkali feldspar series. It has a triclinic framework structure with silicon and aluminum in tetrahedral (fourfold) coordination, which forms relatively large void spaces, that makes the materials to absorb water. In another word feldspars crystalized at the early stage in continuous reaction series, which form rocks of more susceptible to weathering. However the soil containing high percentage of Albite mineral (82.5%) is classified as good soils for engineering purpose based on its engineering and index properties tested result in present study. This soil is classified as A-2-7 (on AASHTO classification) and CL-OL on USCS classification. Hence the rate of weathering of minerals may not have effects on index properties of soils, or it need further study.

Generally in present study, the index properties and chemical analysis of selected samples have been correlated and show the following relation.

- The soils that have higher percent of kaolinite mineral shows very high plasticity indices especially liquid limit is higher comparatively
- The soils that shows higher hematite mineral depicts less plasticity indices
- And the higher sanidine (Na ($Al_2Si_2O_8$)) shows low OMC, higher CBR strength, low group index and lower plasticity indices.

4.7 Problems Associated with Sub grade soils in the Study area

Many literatures show that, there are many problems associated with subgrade materials along road colliders. These problems include expansiveness (volume change), low bearing capacity and non-durability, susceptibility to erosion (dispersion), salinity, karst (caving), hardness, subsidence and moisture absorption (discussed in chapter-2). As observed from field works and laboratory results of engineering properties and chemical composition, the most common problems associated with subgrade materials in the present study area are volumetric change, low bearing capacity, high compressibility and susceptibility to erosion due to variability of soil materials.

4.7.1 Volume change upon wetting and drying

Most soils in nature have the behavior of expansion/swelling and shrinkage under different environmental conditions. This volumetric change is due to the water absorption of CLAY materials. The ability of water absorption is the result of weak bonds between octahedral and tetrahedral units of CLAY mineral building blocks. The presence of expansive soils in the study area has been identified from both laboratory index test and field visual investigation.

Accordingly most of the soils in the study are shows a desiccated cracks of polygonal shapes during the dry season and closure of the cracks during the rainy season. In addition, from the laboratory test the soil show higher plasticity (liquid limit and plasticity index) and higher CBR swelling.

On the other hand the density-moisture relation test (proctor) shows that the materials have low dry density and high optimum moisture content, this is due to the presence of void space between grains. The void space can collapse when loads from upper pavement structures and dynamic traffic loads are exerted which resulted in settlement of the materials differentially or uniformly

In addition the volumetric change problem of soils can be evaluated from its plasticity index (Holtz and Gibbs, 1956). They give the following relations (table 4.14)

Table 4.15 Relation between volume change and PI (Adapted from Holtz and Gibbs 1956)

Plasticity index	shrinkage limit	volume change
0- 15	12 or more	probably low
15 – 30	10 - 12	probably moderate
30 or more	0-10	probably high

Accordingly about 80% of the present study area has volumetric change of moderate to high. Generally the problems associated with the expansive soils such as settlement under loads, development of pore water pressure (swelling), slope instability during wet season and cracking of existing roads are observed in the present study area. This problem can highly affect the stability and long life of pavement; hence some stabilization methods should be used based on economic feasibility.

4.7.2 Low bearing capacity problem

Soils are a product of weathering of preexisting rocks, hence the nature of preexisting rocks and condition under which the soil are formed could govern its engineering properties. These indicate that different soils can be formed under different condition from the same parent materials. The bearing capacity of the soils is one of engineering properties that is sensitive to type of soils.

In present study area, the geological unit is dominated by basalt of basic composition of different structures and textures those results in variable degree of weathering, there are different soil types with different physical and engineering properties that investigated from field work and laboratory test results.

The strength tested using CBR methods show that, most of the soils have less strength value (<5%) under 95% maximum dry density. It is known that the bearing capacity of the soils is directly related with its strength; hence the soils in the study area have been identified to have bearing capacity problems.

4.7.3 Susceptibility to erosion

From certain topographic features along the road path, metrological data and drainage systems some of the soils show continuous erosion. Most of seasonal streams are meandering pattern since the topography are flat surface, hence long time of water contact with low speed are observed. These low speed stream flows during rainy season can erode large amount of loess soils. In addition the water from farm land flow towards the road alignment and can generate flooding of the roads.

The erodible materials are highly slake and dispersive in nature when it susceptible to water flow. These types of problems can be avoided by removing and replacing with non-erodible materials, coating with other materials, avoiding water interference with such soils by redirecting water flow using drains. in addition, it is also important that the material is compacted at 2 to 3% above optimum moisture content to as high density as possible (Donaldson, 1975; Elges, 1985 as cited in Fasil Abagena, 2003).

4.7.4 Highly compressibility

Compressibility is the property of a soil that pertains to its susceptibility to decrease in volume when subjected to load, which can be directly measured by odometer. Other commonly and interchangeably used terms related to compressibility are consolidation, settlement, collapse, and compaction. Even the direct measurement of the settlement and compression using odometer have not been conducted in this study, it is possible to notice from the plasticity chart (Fig: 5.3) that, there are highly compressible materials on the site. Such soils have higher liquid limits with higher porosity; that collapse under traffic loads.

SUMMARY

In this chapter the subgrade materials are characterized in terms of engineering and index properties that influence the performance of the subgrade materials. In addition from the test result and field observation the problem associated with the subgrade materials and the area have been identified as susceptibility to erosion, less bearing capacity and compressibility under load, thus the identified problems are not uniformly distributed along the path. Accordingly the homogeneous sections, (HS1 and HS2) of soil extension survey have been identified as more problematic.

CHAPTER V INTERPRETATION AND DISCUSSION

5.1 Preamble

The design of flexible pavements involves a study of subgrade and paving materials, their behavior under load and the design of the pavement structure to carry that load under all climatic conditions of the site (ATU, 1997). The subgrade materials are characterized in terms of its strength, bearing capacity, susceptibility to deformation under stress (stress response), moisture content, density, response to climatic change (swelling and shrinkage potential), grain size and Atterberg limit.

Based on data collected from the field and various tests conducted over soil samples in the laboratory well supported with secondary data an attempt has been made to analyze the results during the present study. A systematic interpretation and logical discussion has been made in terms of characteristics and suitability.

In general this chapter deals with the general characterization and suitability evaluation of the area and subgrade materials for flexible pavement construction. The sub grade materials could be characterized by evaluating tested value in comparative with standard guide of pavement design and characterization of the country whereas, the suitability of the site will be evaluated from the physical observation and analyzing the effects of parameters obtained from secondary data such as rainfall intensity, temperature, ground water level, traffic density, seismicity of the area, topographic features and drainage pattern which have directly or indirectly reduce the comfort-ability and performance of the road.

5.2 General characterization of subgrade materials

The objective of material characterization in the design of the road pavement is to design appropriate pavement thickness and select competent materials to ensure that the pavement performs adequately and requires minimal maintenance under the anticipated traffic loading for the design period adopted. This characterization process results in selection of material types, select pavement type, decide pavement layer thicknesses to be designed and configurations of the pavement layers to meet the design objectives.

In order to provide the objectives of its construction, the materials of subgrade should be characterized and evaluated in terms of factors that affect the performance of road. To characterize the subgrade materials during the present study, engineering parameters of subgrade are tested in site laboratory of AKIR construction plc, XRD in Geological Survey of Ethiopia and triaxial test in Adds Ababa University engineering geology laboratory.

According to the result of field observation and laboratory investigation conducted on subgrade materials in present study, the materials could be characterized in terms of; its classification under different classification systems, strength (bearing capacity), swelling potential, expansiveness, compaction, consistence and grain size distribution which are interrelated and affects one another.

5.2.1 Field extension survey

From the field soil survey it has been observed that the area is covered by soils of dark brown silty CLAY (7.5km), dark brown CLAY mixed with gravel (14.5km), light brown silty CLAY (5.5km) and Rock section (8.7km) of different engineering and mineralogical properties which were characterized by laboratory investigation.

Accordingly, the subgrade materials along the road collider are classified into four homogeneous sections. In addition based on genetic classification, the soils in the study area have been classified as residual, colluvium and alluvial soils by referring the topographic features and drainage patters along the road path that affects the engineering performance.

5.2.2 Classification

Soil classification systems used in this study are the AASHTO classification System and the Unified Soil Classification System (USCS), in which both based on index tests (gradation and plasticity indices).

According to the result of these tests, the subgrade soils along the road are dominated with finer materials of <0.075, hence the probability of these materials to be CLAY is high. The sample collected from lower topography and at the foot of escarpment have more than 90% pass through 0.075mm, in which 32% to 100%, 15% to 96% and 7% to 92% passes the sieve size 2mm, 0.425mm and 0.075mm respectively. Genetically soils from such area have been classified as transported soils in present study.

From the atterberg limit test result, the subgrade materials have high liquid limit and high plasticity index. The liquid limit and plasticity index ranges from 31% to 82% and 10% to 47% respectively; in which 46.3% of the samples resulted in LL>60 and 44% have PI>25.

The soils having LL>60% and PI>25% shows the presence of expansive CLAY minerals and it classified under problematic soils for engineering purpose (ERA, 2002). These soils are dominated with dark brown silty CLAY (HS1) and dark brown silty CLAY mixed with some weathered rocks (HS2) those classified during soil extension survey.

In addition many soils properties and engineering behaviors have been correlated with the plasticity index including swelling-shrinkage potential using Seed et al., (1962) equation given as

$$SP = 60K\,(PI)^{2.44} \quad \text{……eq.5.1}$$

However, the equation is applicable for a soils having CLAY content ranging from 8% to 65%, but in this study the silty and CLAY content are not identified, hence the CLAY content assumed to be fall in the range because the parentage passing 0.075mm closer to this range.

The swelling potential for the samples in each homogeneous sections of soil extension survey are determined on the basis of Seed et al. (1962) are presented in table below.

Table 5.1 Estimated swelling potential for Homogeneous sections (Seed et al., 1962 model)

Range of swelling	Degree of swelling	HS1	HS2	HS2	HS4	No. of samples	Percentage occurrences
0-1.5	**Low**	-	-		4	4	5
1.5-5	Medium	-	11	14	14	35	44.4
5 to 25	High	22	18	-	-	40	49.4
>25	very high	1	-	-	-	1	1.2

From this table the swelling potential of the soils in present study ranges from 0.4 to 26% in which 5% (low), 44% (medium), 49% (high) and 1% (very high) are categorized under swelling potential classes according to Seed et al., 1962 classification as a general. Thus, the higher swelling potential soils are found only in HS1 (56%) and HS2 (44%). This show the subgrade in such area is unsuitable.

According to AASHTO classification the subgrade materials tends to be classified as A-7-5, A-7-6, A-2-7, A-6 and A-2-6 with 65.8%, 22%, 8.6%, 2.4% and 1.2 % respectively on AASHTO classification (Fig 5.1). The problematic soils (A-7-5) is dominated in dark CLAY underlained by weathered rock (HS1) and dark brown sily CLAY mixed with weathered rocks (HS2) which comprise of about 38% and 36% of the group respectively. The remaining are fall under light brown silty CLAY (HS 3) and Rock section (HS 4) those covers about 14 % and 11 % of the total samples of his group. Hence the first two sections are unsuitable on this sense.

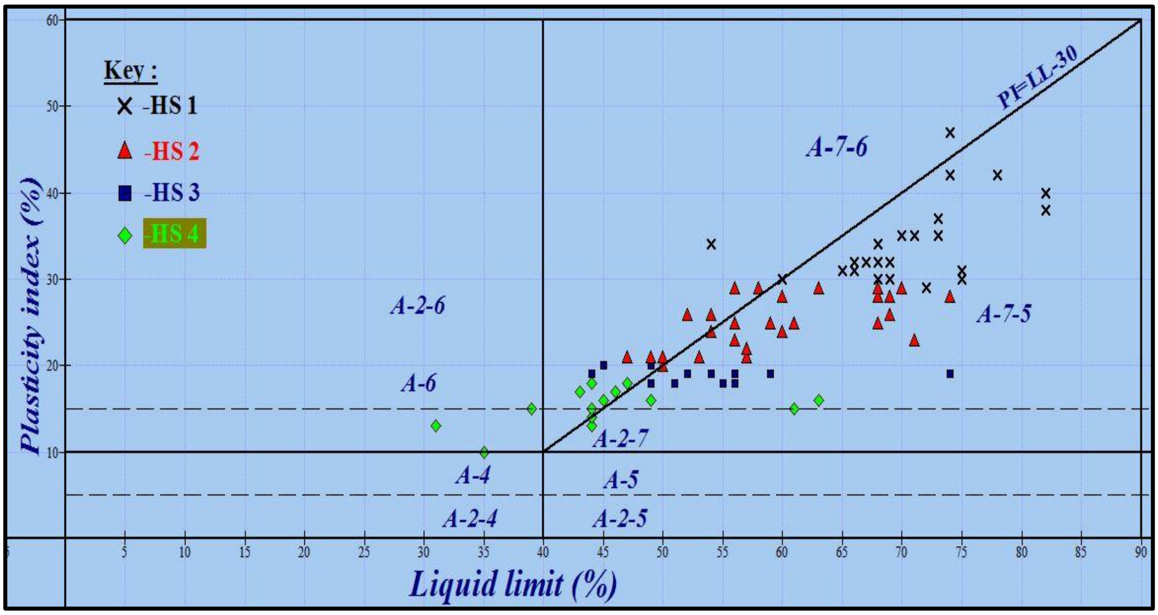


Fig 5.1 AASHTO soil classification of present study on Casagrande's chart

Table 5.2 summary of AASHTO classification for present study

Classes	HS1	HS2	HS3	HS4	TOTAL
A-2-6	0	0	0	1	**1**
A-2-7	0	3	1	3	**7**
A-6	0	0	0	2	**2**
A-7-5	21	19	8	5	**53**
A-7-6	2	6	5	6	**19**
Total	**23**	**28**	**14**	**17**	**82**

Similarly in USCS classification the subgrade materials were grouped into MH-OH, ML-OL, CH-OH and CL-OL depending on its position with respect to A-line and U-line on Casagrande,s LL-PI chart shown in (fig 4.4). In another way the subgrade materials also classified into illite (60.1%), kaolinite (37.2%) and Halloysite (2.7%) using liquid limit and plasticity index chart developed by Holtz and Kovacs (1981) (fig.4.5). Illite shows a medium range of swelling and shrinkage behavior in nature. While kaolinite and halloysite shows a little to no swelling behavior (Seed et al., 1962). Hence the soils in the study area are dominated with Illite group on the basis of Casagrande's chart.

5.2.3 California Bearing Ratio (CBR)

Strength of the sub-grade soils along the road section in present study has also been determined using CBR. A three point CBR test at 10, 30 and 65 blown were conducted according to AASHTO T193 and the CBR values at 95% MDD were determined (presented on Annex 2). The test results shows that the sub-grade soils have very low to high CBR value ranging from 1% to 24% in which most of it falls below 5% that does not fulfill the minimum requirements as sub-grade materials on the basis of ERA (2002) standard. These indicate that the subgrade materials needs to be compacted or other materials should be used by blending with the existing materials.

In addition the correlation between strength and plasticity limits has been conducted and shows that, strength of soils decreases with increasing the plasticity limits; which is a function of increasing CLAY fraction in soils. As shown on Fig 4.2 (pp. 55) of plasticity versus grain size graph, most of the samples comprise of about 75% grain passing sieve No 200 (0.075mm), which is silty and CLAY fraction. Hence the CLAY fraction of the material is higher which is responsible for the reduction of shear strength of the soil.

Also the strength (CBR value) of a soil shows a decreasing pattern as the maximum dry density of a soil reduced. Accordingly the soils with higher maximum dry density have higher CBR value, hence the maximum dry density can be increased by compacting the soils so the CBR.

5.2.4 CBR Swelling

The CBR swelling of subgrade materials has been determined at 95% of maximum dry density of four days soaked samples and the test results depicted that the CBR swelling of subgrade materials ranges from 0.4 to 9.83% in which about 76% have greater than 2% CBR swelling that concentrated in HS1 and HS2 of homogeneous sections of soil extension survey.

The swelling and shrinkage properties of soils are also depends on the mineralogical composition (CLAY minerals) and percentage of fine grained especially CLAY sized which can absorb large amount of water. Thus the soils that dominated with fine grained (<0.075mm) possess high swelling value in present study area. The relation between percentages of silty-CLAY content with CBR swelling values are presented in Fig 5.2

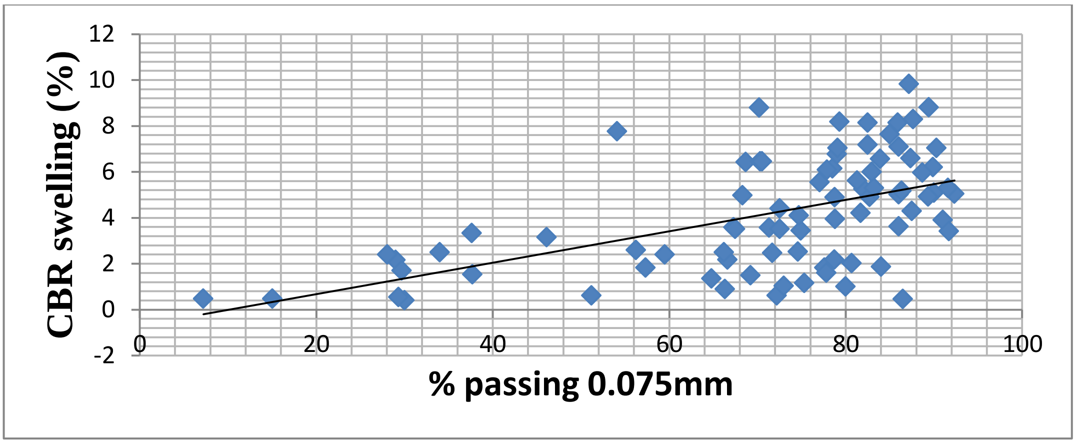


Fig 5.2 CBR swelling Vs grain size

Perusal of Fig. 5.2 indicates that CBR swelling shows slightly increasing trend (CBR swelling =0.0593* % passing 0.075mm) as the silty-CLAY content increase. Usually, development of cracks in subgrade material is directly related to shrinkage and swelling characteristics of the material used. From the tested result it is possible to characterize the subgrade materials in present study in terms of its CBR swelling that it dominated with high swelling materials in which engineering problems are associated with.

Chen (1965) develops a correlation between percent finer than the No. 200 sieve size, Liquid limit, and standard penetration blow counts to predict potential expansion, thus the present result of subgrade materials are evaluated accordingly (Table 5.3).

Table 5.3 Chen's (1965) correlation of percent finer than the No. 200 sieve size, and Liquid limit to expansion potential

Laboratory and field data					Probable expansion (% volume change)	Degree of expansion
Evaluation on grain size		Evaluation on LL		Standard penetration resistance (blown/ft) **		
% passing 0.075mm	% occurrence	Liquid limit (%)	% occurrence			
>95	0	>60	41.5	>30	>10	Very high
60-95	80.5	40-60	54.8	20-30	3-10	High
30-60	12	30-40	3.7	10-20	1-5	Medium
<30	7.5	<30	0	<10	<1	Low
After Chen (1965) **- shows the test does not conducted in present study.						

From the above table, the subgrade soils are dominated by high degree of expansion both on grain size (80.5 %) and liquid limit (54.8%) evaluations. However, the evaluation contradict on very high, medium and lower classes (for example about 41.5% are classified as very high degree of expansion on the basis of liquid limit, but no very high expansion on the basis of grain size evaluation). This shows the evaluation on grain size underestimate the degree of expansion. Hence the correlation needs some exceptional.

The soils that falls under very high degree of expansion on its liquid limit (LL>60), also falls under unsuitable for subgrade materials according to ERA (2002) standard. As it compared to the field survey homogenous section soil classification, the unsuitable soils are mostly fall under HS1 and HS2. (See Fig 5.3)

5.3 Suitability analysis of materials and study area

5.3.1 Suitability of sub grade materials

The aim of analysis and interpretation of a result obtained from both field investigation and laboratory testing is to evaluate the suitability of the subgrade materials for pavement construction. In this paper the suitability of subgrade materials have been evaluated in terms of field evaluation and the laboratory results from Atterberg limit tests, Strength test, compaction test, CBR swelling tests and other empirically or analytically obtained parameters such as compressibility and group index by using project and ERA standard; some of the parameters used for suitability evaluation are summarized in table 5.5

I) Suitability on Atterberg Limits

Atterberg limits are one of the parameters used to evaluate the suitability of subgrade materials in present study. According to Bowels, (1992) the materials that have plasticity index greater or equal to twenty (PI ≥ 20), possess a volume change problem which require some kind of precautionary measures. And the materials that have plasticity index greater than 30 (PI≥ 30) is highly plastic and possess swelling problem (ERA, 2002). However, for subgrade materials having plasticity index less or equal to 30 (PI ≤30) and liquid limit (LL<60) be classified as suitable according to ERA, 2002 standard (Fig: 5.3). Hence the evaluation in present study depends on ERA standards.

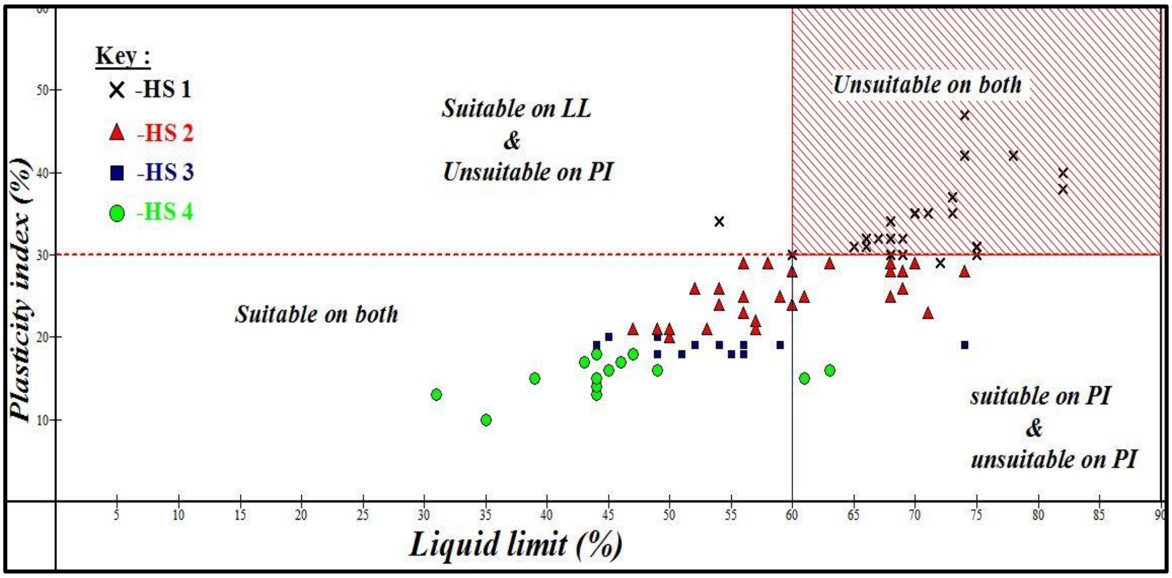


Fig 5.3 Suitability of subgrade materials by Attereberg limits on ERA standard.

From the above Figure, it is possible to conclude that 41.5% of the samples are unsuitable on the basis of their liquid limit according to both project specification and ERA (2002) standard. And about 27.4% according to ERA standard manuals and 40.75% of the tested samples according to project specification are unsuitable on the basis of Plasticity index (PI) evaluation. As general on the basis of atterberg's limit, the study area have plastic soils, hence it requires some measures that reduce the plasticity of the soils such as blending with other non-plastic materials.

II) Suitability analysis on the basis of CBR and CBR swelling

Other parameters used for the suitability analysis of the subgrade materials in present study are its strength and swelling that given as CBR and CBR swelling values. These parameters used to evaluate the response of soils under applied loads and change in environmental conditions. As mentioned earlier the soils that have CBR value greater than 6 % and CBR swelling less than 2% at 95 % of maximum dry density are tends to be classified as suitable for subgrade materials without considering amount and intensity of traffic loads to be imposed latter, according to ERA, 2002 and AASHTO,1993 standards. This evaluation of soils in present study has been adopted and presented in Fig: 5.4.

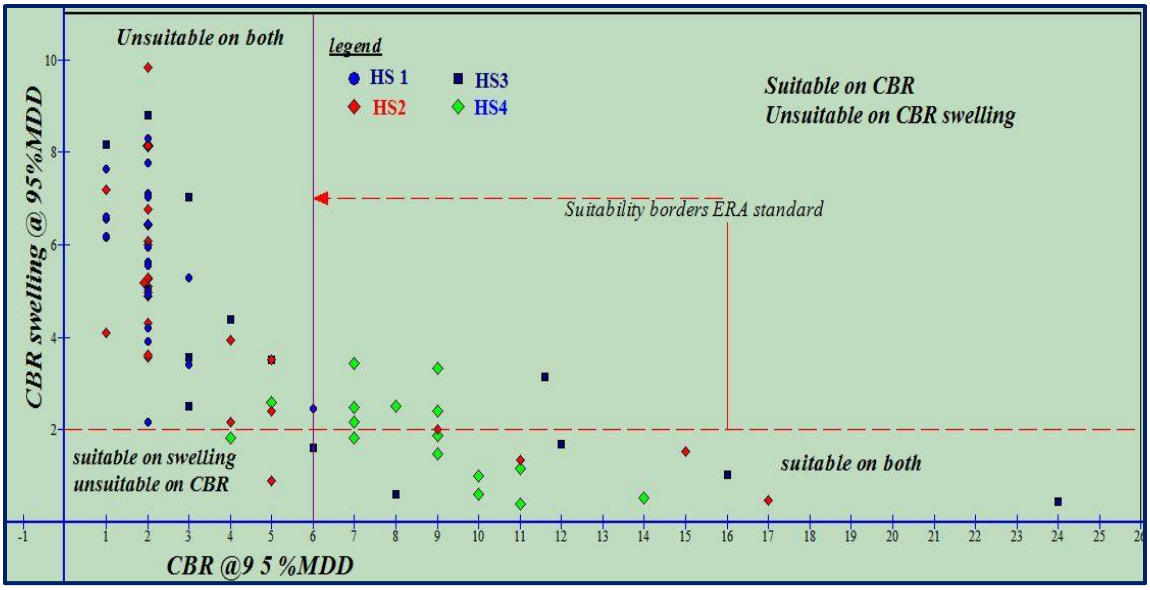


Fig 5.4 CBR and CBR swelling evaluation of subgrade on ERA standard

Table 5.4 summary of CBR value for homogeneous sections

Evaluation	Strength classes	CBR values	Homogeneous sections				Total	%	Total (%)
			HS 1	HS 2	HS 3	HS 4			
Unsuitable	< S1	<2	5	2	1	0	8	9.5	63.5
	S1	2	15	16	2	2	35	43	
	S2	3 – 4	2	2	4	1	9	11	
Both	S3	5 -7	0	3	2	4	10	12	4.5/7.5
Suitable	S4	8 -14	0	2	4	10	16	20	24.5
	S5	15 – 29	0	3	1	0	4	4.5	
	S6	>30	0	0	0	0	0	0	
	Total samples		23	28	14	17	82	100	

From the above figure: 5.4, the CBR and CBR swelling have an inverse relation, in which increasing CBR swelling and decreasing CBR value negatively affect the bearing capacity of the subgrade, so suitability is reduced. Accordingly about 68% and 76% are unsuitable on CBR and CBR swelling values respectively which require other treatments to be suitable under the existing condition.

III) Suitability analysis on compressibility

In present study the suitability of subgrade materials have been also evaluated based on its compressibility under loads and permeability that can be estimated on Casagrande's plasticity chart (Fig: 5.5). Compressibility of CLAY soils increase with increasing liquid limits and decreasing permeability.

CLAY soils are highly porous and less permeable in nature; hence they used as impermeable membrane in embankment dam. The CLAY soils that have less permeability tends to store high amount of water in its pore space under saturation condition; when external load is applied the water spelled out and the void space are tends to collapse that resulted in high compression of soils and settlement.

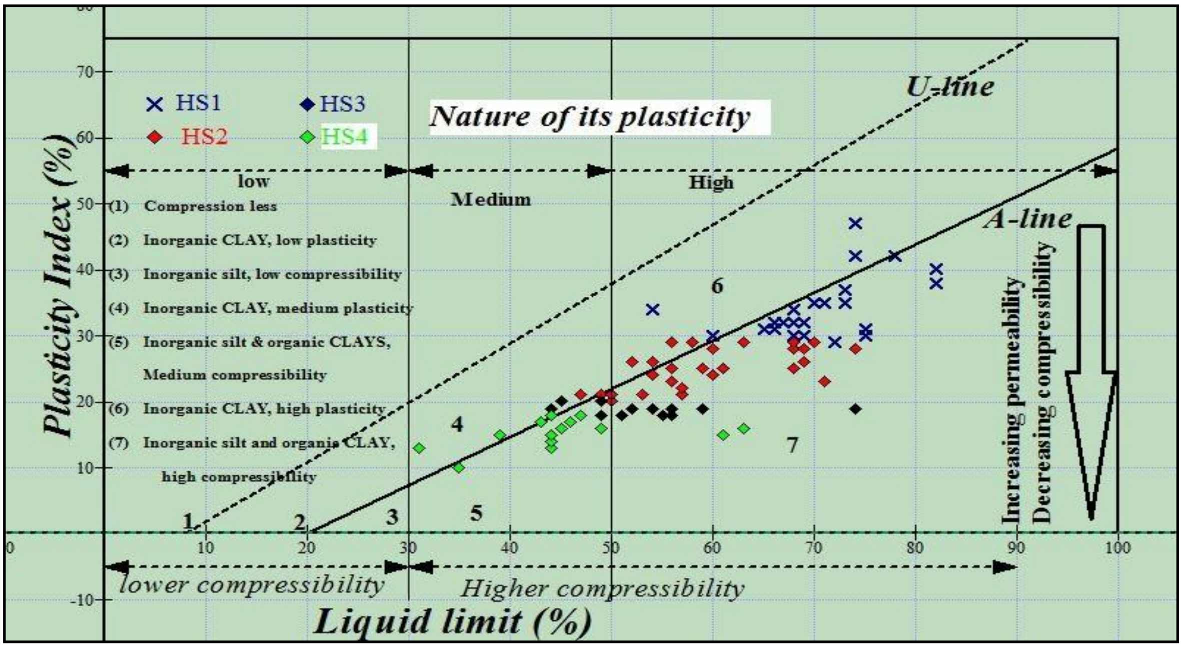


Fig 5.5 Relationship between plasticity and compressibility of soils (AASHTO, 1993)

Figure 5.5 shows that the soils that have liquid limit between 30 to 50% are classified as medium plasticity and higher compressibility, while the soils that have LL>50%, are highly plastic and highly compressible that covers about 70% of the area in present study.

Accordingly, most stretch (almost all of HS1 & HS2) of the present study area is unsuitable on the basis of plasticity and compressibility, even the amount of fat CLAYs (fall above A-line) is less. Also from the XRD and petrographic analysis, some areas are Hematite (iron oxide) rick. Thus literatures state that the iron rich soils have possesses compressibility problems under load application. Hence the area has highly compressible materials that only support light structures.

Generally According to ERA (2002) pavement design manual; soil materials for sub-grade should fulfill a maximum value of less than 30% and 60% for PI and LL values, respectively and a minimum CBR value of 6% and maximum CBR swell value of 2% are the minimum requirement for most of standard manuals. Accordingly the result summarized in table 5.5 depict that most of the soil samples do not fulfill the requirements as a sub-grade, thus classified to be unsuitable for sub-grade in road construction.

Table 5.5 Summary of percentage of suitable/unsuitable samples on the basis of ERA and the Project standard evaluation

Parameters	Evaluation Standards					
	According to ERA Standard			According to Project specification		
	Standard	Suitable	Unsuitable	Standard	Suitable	Unsuitable
PI	<30	75.3%	24.7%	<25	59.25%	40.75%
LL	<60	56.79%	43.21%	<60	56.79%	43.21%
GI	<20	54%	46%	<20	54%	46%
CBR	>6	32%	68%	>5	38%	62%
SWELLING	<2	35.8%	64.2%	<2.5	34.57%	65.43%
Compressibility	LL<30	0%	100%	*not specified on standards.		

All of the above properties and their dominant values in combination explain about the unsuitability of the soil as a subgrade material. The general engineering geological characterization results, which have been presented in chapter-4-, indicate that the subgrade soils need proper improvement before it intended to be used as a subgrade material especially along the sections that categorized under HS1 and HS2 during field extension survey. This can be done by different ground improvement techniques including chemical treatment, pre-wetting, proper compaction, cut and fill with a suitable material, and making the appropriate pavement designs that considers the unsuitability of the subgrade soil.

5.3.2 Suitability of the study area for pavement

Environmental condition has a significance effects on performance and durability of geological formation, since the alteration of rocks and soils are a function of environmental condition and susceptibility of materials to environmental change. These environmental conditions in this sense can be described in terms topography, drainage (moisture), soil types, geological setup and seismicity of the area.

- **Suitability on geology**

Geologically the area is underlined by basic rocks of basalt which contains minerals those crystallized at the earlier stage in Bown's reaction series; hence it is highly susceptible to weathering and disintegration. Since basalt under intensive weathering forms CLAY soils of expansive nature, the area have a probability of having expansive soils that affect the stability of lighter structures especially road. However the intensive weathering and formation of expansive soils requires long time, which is incomparable with the design time of the project. Hence the effect is not as such problematic.

- **Suitability on seismicity**

Ethiopia is categorized under five seismic hazard zones ranging from 0-8 seismic intensity. It is reduced as distance from the rift floor increase, in which the present study area is situated in the highly seismic zone of 7-8 MM scale. Hence different geological structures such as faults, joint and weak zones are associated with seismic activity that can be found in subsurface and cause damage to road.

In addition the soils deposit especially the low laying and flat surface (HS 2) of the area is thicker and loess. These properties of soils are the contributing factors to seismic site amplification. However (HS 2) are dry in nature, hence the amplification is comparatively low. Generally, even there is no seismic record the study area is seismically active and has a probability of high site amplification, hence it is unsuitable. Thus the selection of road collider should consider the presence of thicker deposit and seismic related structures.

- **Suitability on geological structures**

Geological structures play significant role in any developmental activities. For safe design, construction and performance of the structures in its engineering life time, geological structures may pose problems. The presence of discontinuous (brittle) geological structures may propagate and amplify earth quake occurrences and therefore, it will be very important to identify such structures during the investigation so that proper remedial measures can be worked out.

- **Suitability on topographic setup**

During the field survey, the area has been classified into four terrain classification on the basis of slope geometry as flat (35%), rolling (28.6%), escarpment (4.4%) and mountainous (32%) (See table. 3.2). These topographic features have their own effects on properties of soil and performance of subgrade materials placed on it. Accordingly the escarpment and mountainous area are associated with slope instability problem, whereas the flat area for the deposition of thicker soils of expansive and low bearing capacity nature. Hence the study area is somewhat unsuitable in this sense.

- **Suitability on temperature**

In flexible pavement, the seasonal variation of temperature within subgrade materials can cause the expansion and contraction of materials that responsible for the alteration of properties of the soils that affects its performance. However in present study area, the temperature fluctuation is insignificant as observed from 20 years metrological data of the area. On this parameter, the area is suitable because the temperature fluctuation is not as such significant.

- **Suitability on precipitation**

In another way, the area have an annual rain fall about 1200mm, in which heavy rain have been recorded from May to September throughout the year. The quantity and intensity of precipitation, in the form of rain, affects the quantity of surface water infiltrating into the subgrade and the depth of ground water table. Poor drainage may reduce shear strength, or cause pumping or loss of support. Due to the high precipitation, the sub grade has high probability to interfere with water. From the test result and field investigation, the study area is covered by expansive soils; hence the interference of water with such soils could result in swelling and erosion. Hence the area mostly with low land is unsuitable for flexible pavement on the basis of rain fall intensity.

- **Suitability on drainage condition**

Drainage characteristics have a significant effect on pavement performance. Most water in pavements is due to rainfall infiltration into unsaturated pavement layers, through joints, cracks, shoulder edges, and various other defects, especially along deteriorated pavements. Water also seep upward from a high groundwater table due to capillary suction or vapor movements, or it may flow laterally from the pavement edges and side ditches. The present project crosses some streams that flow to the lowland area. Since most of the study area especially along 0+000 to 6+000, 17+000 to 22+500, 24+000 to 28+000 and 31+000 to 34+100 (52%) pass through higher elevation, the subsurface drainage have less effect on pavement. However the study area fall under dega climatic zone, hence it gets rainfall throughout the year, thus the surface water has significant effect on subgrade materials. Hence appropriate surface drainage design should be taken into account in such area.

On the other hand the remaining pass through low laying topography along which water table may be shallow and fluctuation of water table can affect the subgrade materials. In addition these areas possess higher liquid limit and lower CBR value (See fig 5.8). Therefore, providing adequate subsurface and surface drainage to a pavement system should be considered as an important design consideration to ensure satisfactory performance of the pavement.

5.4 Pavement design consideration

5.4.1 Homogeneous section delineation on cumulative difference method

According to AASHTO Guide for design of pavement structures (1993), the statistical method is used to delineate homogeneous sections of more or less similar CBR values. The division of the road in to homogenous sections in present study have been carried out based on the method of cumulative differences as given in AASHTO (1993) Pavement Design Guide (Appendix J) presented as annex-5-

According to AASHTO (1986), a section border is indicated whenever the "trend" in the series of cumulative differences changes from positive to negative or vice versa. The section borders indicated in Figure 5.6 were obtained by finding "substantial" local peaks in the series of cumulative differences, where a peak is identified as "substantial" if its value is the most extreme within a window of minimum of seven neighbors to the left and seven neighbors to the right of it.

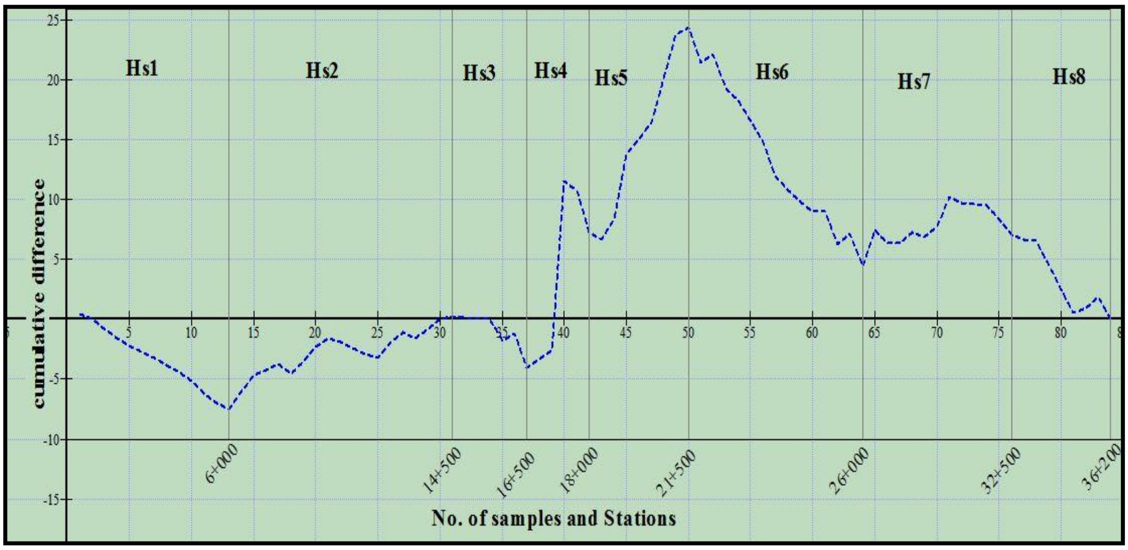


Fig 5.6 Homogeneous section delineation on cumulative difference method

In this method after the delineation of approximately homogeneous sections based on cumulative difference methods, the 90%-le of the tested CBR in one homogeneous section can be evaluated and design CBR can be determined using equation 5.1 and the strength classes are determined by correlating with design standard (in this case ERA pavement design manual (2002), that classified soils into six CBR strength classes.

Design CBR = 1 + (n - 1)0.1 ---eq.5.1

Where: 'n' is the number of tests in one homogenous section.

The 90%-le shows that the CBR values in the homogeneous sections are 90% greater than the obtained values.

In present study, based on cumulative difference methods, the subgrade have been classified into 8 homogeneous sections by locating the main slope changes in the graph so as to determine the limits of fairly homogenous sections along the road alignment as shown in fig. 5.6 for the road length from km 0+000 to km 36+200. The result of delineation of homogeneous section on cumulative difference are presented in Annex-5 and summarized on table 5.6 below.

The unit delineation by cumulative differences involves computing parameters such as; CBR values from tested result, the number of stations in increasing order (n), the distance between two successive CBR measurements (Δxi), Cumulative interval distance (Σxi), the average Actual interval area (the product of average of two successive CBR values and interval distance). The detail calculation for the analysis unit delineation on cumulative differences values are attached in annex-5-

Table 5.6 Summary of homogeneous section by cumulative difference (from annex-5)

HS	**Station, KM**		**NO. samples**	**CBR Values**	**90% le value**	**Strength classes**
	From	**To**				
Hs1	0+000	6+000	13	2,2,1,2,2,2,2,1.9,2,1,1,2,10	1.1	<S1
HS2	6+000	14+500	16	2,6,2,1,9,2, 2,2,2,2,3,8,2,2,7,3,4	2	S1
HS3	14+500	16+500	4	2,3,2,2	2.1	S1
HS4	16+500	18+000	7	12,15,11,1,8,2,7	1.6	S1
HS5	18+000	21+500	9	2,16,17,17,2,24,2,11.6,5	2	S1
HS6	21+500	26+000	11	9,4,3,2,2,3,4,3,5,7,11	3	S2
HS7	26+000	32+500	14	4,10,2,5,6,9,1,14,7,2,9,2,5,1,	2	S1
HS8	32+500	36+200	8	9,2,1,2,2,11,4,5	2	S1

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

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

Design CBR = Average CBR x 2/3……………………………………..eq. 5.2

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

Table 5.7 Design CBR using Virginia Department of Transportation Pavement Design (2000)

HS	Station (km)		NO. samples	CBR Values	Design CBR	
	From	To			Average CBR x 2/3	Strength classes
HS1	0+000	6+000	13	2,2,1,2,2,2,2,1.9,2,1,1,2,10	1.23	<S1
HS2	6+000	14+500	16	2,6,2,1,9,2, 2,2,2,2,3,8,2,2,7,3,4	2.2	S1
HS3	14+500	16+500	4	2,3,2,2	2.1	S1
HS4	16+500	18+000	7	12,15,11,1,8,2,7	5.3	S3
HS5	18+000	21+500	9	2,16,17,17,2,24,2,11.6,5	6.8	S3
HS6	21+500	26+000	11	9,4,3,2,2,3,4,3,5,7,11	3	S2
HS7	26+000	32+500	14	4,10,2,5,6,9,1,14,7,2,9,2,5,1,	3.4	S2
HS8	32+500	36+200	8	9,2,1,2,2,11,4,5	2.6	S1

5.4.2 Homogeneous section delineation on CBR uniformity

Even though the homogeneous section delineation using cumulative difference method is commonly used, but sometimes it categorize the soils of extremely higher and lower CBR values under one homogeneous section, since it depend on slope change on cumulative difference graph. This leads to the inappropriate designing of pavement thickness which results in uneconomical wastage of resources; hence in present study, in addition to the cumulative difference method of delineation, direct delineation based on CBR values have been made.

This method categorizes soils of uniform CBR values in the same homogeneous sections and the design CBR values can be taken as the averages of CBR values in the same homogeneous sections by eliminating the extremely maximum and minimum value if any. Accordingly, using the graph of CBR versus station of sample collection, about 11 homogeneous sections are made in present study from Km 0+000 to km 36+200 and design CBR values are determined being the average CBR value.

According to ERA, 2002 pavement design manual, it is advisable to avoid short design sections along the alignment and larger number of homogeneous sections greater than 5 and where the subgrade CBR values are very variable, the design should consider the respective benefits and costs of short sections and of a conservative approach based on the worst conditions over longer sections. But such consideration may result in short period distress of pavement.

Table 5.8 summary of homogenous sections on the basis of uniformity of CBR values

HS	Station (KM)		Tested CBR values	Av. CBR	Strength classes	Av. Atterberg limit	
	From	To				LL	PI
HS1	0+000	6+000	2,2,1,2,2,2,2.1.9,2,1,1,2,2	1.8	<S1	69	31
HS2	6+000	9+500	10,2,6,2,1,9,2	4.6	S3	62	30.5
HS3	9+500	11+500	2,2,2,2,3	2.2	S1	68	31
HS4	11+500	14+000	2,2,7,3	2.5	S1	53	20
HS5	14+000	16+000	4,2,3,2,2	2.6	S1	65	26
HS6	16+000	17+000	12,15,11	12.6	S4	53	24
HS7	17+000	18+500	1, 8,2,7,2	4	S2	50	20
HS8	18+500	22+000	16,17,2,24,11.6,5,8,9,4	10.8	S4	55	21
HS9	22+000	24+500	4,3,2,2,3,4,3,5,	4.4	S2	60	27
HS10	24+500	35+000	7,11,4,10,2,5,6,9,1,14,7,2 ,9,2,5,1,9,2,1,1,2,2	7	S3	55	21
HS11	35+000	36+200	11,4,5,	6.6	S3	50	24

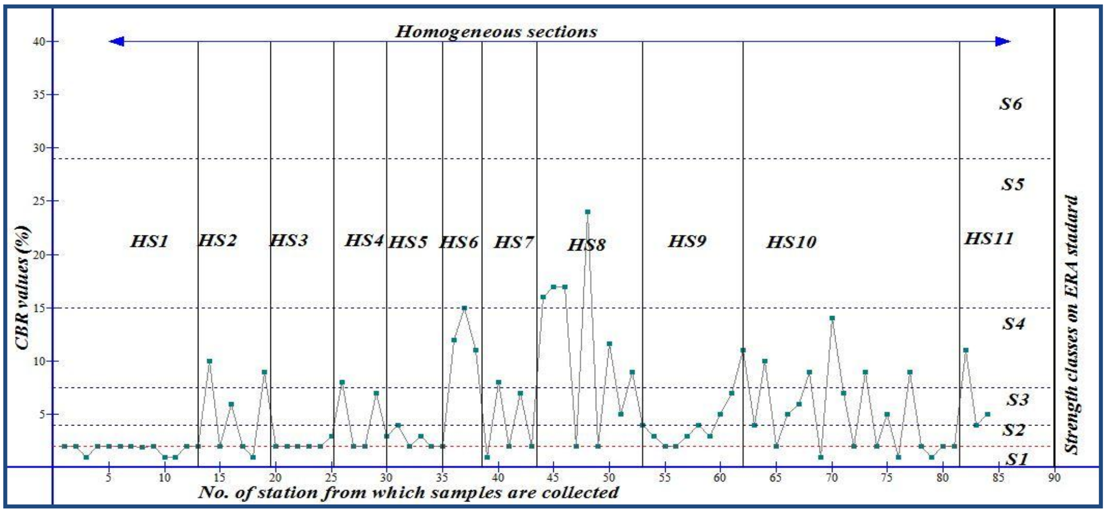


Fig 5.7 Homogeneous subgade sections of the study area on CBR value

In addition the topographic set up and soil types of the area should be taken into considerations which have some relation the formation variable soil type. The following figure shows the relationship between topographic setup and engineering properties of materials.

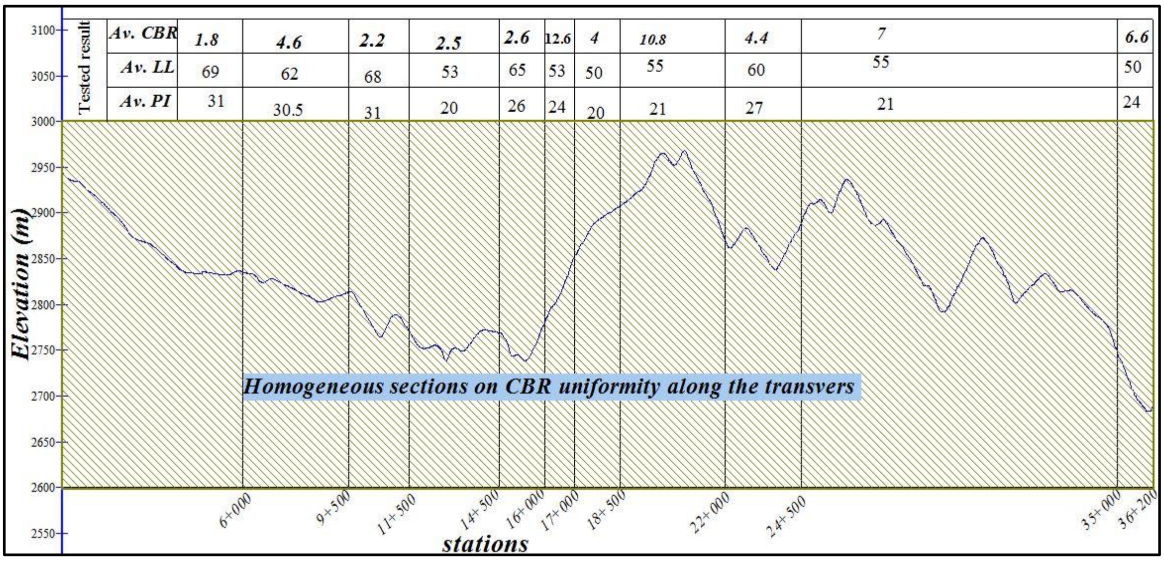


Fig 5.8 Engineering proporties of subgrade with Topographic feature

From figure 5.8, the average of CBR, Liquid limit and Plasticity index have certain relation with topographic setup of the area. Accordingly the escarpments (cliff) have higher CBR, lower LL and PI, this may be because the highly weathered soil is eroded and left with the resistant one that possesses higher strength on such area. In another word the soil samples collected from the flat surface and low laying (foot of hill) possess less CBR, higher LL and higher PI due to the soils genetically transported and poor drainage is found. In such are the soils have a higher probability to be expansive and problematic.

In addition by comparing the CBR of homogeneous sections with field observation, the sections that have CBR class of S4 are those soils with some gravel mixed and genetically describe as residual soils, whereas the S1 is highly weathered and alluvial deposits.

Finally considering such condition and depending on the CBR values of homogeneous sections, the thickness of pavement layers are recommended in comparison with design review according to ERA standard (presented in table 5.10 and 5.11).

Advantage of homogeneous section delineation on CBR uniformity

Even the subgrade in present study area possesses variable CBR values, the delineation of homogeneous sections on CBR uniformity method tried to categorize uniform CBR values as one section because one homogeneous section covers relatively smaller area as compared with cumulative difference methods. However as the number of homogeneous section increase the uniformity of pavement thickness reduced and economically unrealistic, however designing of stable pavement layer can be achieved. But in this paper the pavement is proposed for homogeneous section made using cumulative difference method, also it is possible to recommend for these homogeneous sections.

5.4.3 Proposed pavement thickness

Most of the homogeneous sections in present study area fall under <S1 and S1 strength class which have a CBR values about ≤ 2%, in which 88% covered by HS1 (46%) and HS2 (42%) of soil extension survey, hence the soils being unsuitable falling below the minimum CBR requirement (6 %) used in ERA standard. Because of the variability of CBR values along the stretch, the pavement layers should be designed with variable thickness to reduce the problem and cost of construction as well as maintenance.

In designing road pavement, safety, the future performance, optimizing the construction and maintenance costs shall be taken in to account. However, no traffic count and analysis is made for the purpose of present research, all data related to traffic flow are taken from the Contract Document and the traffic classes along the selected stretch (0+000 to 36+200) being the same and fall under T5 ($3*10^6$ to $6*10^6$ ESA).

In addition during the pavement design, the design time (life time) should be taken into account, thus the present study project area is proposed to design for 15 years under T5 traffic classes. In the design review the thickness of base course and wearing course remain constant for all design CBR values, however from ERA structural catalog, there is an approximate substitution ratio between sub-bases to improved subgrade (capping) as 25:32.

In addition Atkins (1983) suggested that additional thickness of asphalt pavement, above the minimum required can be placed by base & sub base materials using the following equivalencies:

2 mm base = 1 mm asphalt and 2.7 mm sub base = 1 mm asphalt

Therefore, by combining the two scenario, base to sub base substitution ratio will be, 1 inch (mm) = 1.35 inch (mm) and subgrade to asphalt substitution will be 12.3= 1mm. from these ratios the thickness of each pavement layers can be determined. The design review classifies the study area into four homogeneous ad design the thickness accordingly.

In addition, ERA (2002) is pointed out that the minimum thickness of work for a layer in pavement structure is 100mm. From structural catalog of Ethiopian road authority (2002), up to 100mm of sub base may be substituted with selected fill materials provided the sub base is not reduced to less than the road base thickness or 200mm.

Table 5.9 Pavement design for 15 years of the project area (from design review)

Station, km		CBR (class)	ESA(traffic class	Capping layer (mm)		Sub-base (mm)	Base course (mm)	Wearing courses (mm)
From	To			Class	t (mm)			
0+000	14+000	S1	T5	S4	600	275	200	TDS
14+000	22+000	S2		S5	200	275		
22+000	25+200	< S1		S4	600	275		
25+200	36+200	S2/S3		S4	600	275		

The catalog on chart 3, indicate that the soils with subgrade strength S1 and T5 traffic class could be designed with 400mm capping, 200mm sub base, 175 base and 50mm asphalt layers. Accordingly the following pavement thicknesses are proposed (table 5.10 and 5.11)

Table 5.10 Comparison of design review and proposed pavement thickness on homogeneous section of cumulative difference

	Present study for 15 years design time under T4 traffic classes (ESA between 1.5 and $3x10^6$)								Design review of the project for 15 years design time under T4 traffic classes (ESA between1.5 and $3x10^6$)						
HS	Station (km)	90% value	classes	CL		SB (mm)	BC (mm)	Asphalt (mm)	CBR classes	Station (Km)	CL (mm)		SB (mm)	BC (mm)	WC (mm)
				Class	t(mm)						Class	t(mm)			
Hs1	0- 6	1.1	<S1	**S4**	400	325	200	50	S1	0-14	S4	600	275	200	TSD
				S3	600	225	150	50							
HS2	6-14.5	2	S1	**S4**	400	150	150	50							
				S3	400	175	200								
HS3	14.5-16	2.1	S1	**S4**	300	250	150	50	S2	14-22	S5	200	275		
HS4	16.5-18	1.6	<S1	**S4**	400	200	175	50							
				S5	600	300	175	50							
HS5	18-21.5	2	S1	**S4**	300	300	175	50							
				S3	400	350	175	50							
HS6	21.5-26	3	S2	**S4**	300	175	175	50							
				S3	300	350	175	50							
HS7	26-32.5	2	S1	**S4**	400	200	175	50	<S1	22-25.2	S4	600	275		
HS8	32.5-36.2	2	S1	**S3**	300	250	175	50	S2	25.2-36.2	S4	600	275		

The selection of capping strength class in present design depends on the availability of competent materials and economic consideration.

In addition the thickness of pavement layers are proposed on the homogeneous soil sections that categorized on the basis of CBR uniformity. Accordingly eleven (11) homogeneous sections are identified and the first two sections are approximately under T6 traffic as compared with other because the road is a combination of two roads in which one extends from Sembo to Ginager and Asagirt while the other from Sembo to Sholagebeya, Gorfo and Gindeber.

Table 5.11 Comparison of design review and proposed pavement thickness on CBR uniformity

HS	**Station (KM)**		**Average CBR**	**Strength classes**	**Traffic ***	**Proposed pavement thickness by present finding**				**From Design review**				
	From	To				**CL (mm**	**SB (mm**	**BC (mm**	**WC (mm**	**Station (km)**	**S. class**	**CL (mm**	**SB (mm**	**BC (mm**
HS1	0	6	1.8	<S1	T6	300	325	200	50	0 -14	S1	600	275	200
HS2	6	9.5	4.6	S3	T6	-	350	200	50					
HS3	9.5	11.5	2.2	S1	T5	400	200	175	50					
HS4	11.5	14	2.5	S1	T5	400	200	175	50					
HS5	14	16	2.6	S1	T5	400	200	175	50	14- 22	S2	200	275	200
HS6	16	17	12.6	S4	T5	-	250	175	50					
HS7	17	18.5	4	S2	T5	300	175	175	50					
HS8	18.5	22	10.8	S4	T5	-	250	175	50					
HS9	22	24.5	4.4	S2	T5	300	175	175	50	22- 25	<S1	600	275	200
HS10	24.5	35	7	S3	T5	-	325	175	50	25.2-36.2	S2	600	275	200
HS11	35	36.2	6.6	S3	T5	-	325	175	50					

In the above table the wearing layer designed to be TSD by the designer and the same for all homogeneous sections, and designed to be flexible bituminous of 50mm for all sections on present finding depending ERA structural catalog (chart-3)

Difference of research finding and design review on proposed pavement thickness

In this research finding the homogeneous soil sections is eight on cumulative difference method and eleven (11) on CBR uniformity, which increase the accuracy of the CBR design value, but to design the thickness of subgrade materials accordingly is not economically feasible.

However, most of the pavement thicknesses are reduced in present finding as compared with design review; hence the cost of construction could be less. For example the capping layers in some section reduced from 600mm to 400mm, sub base layers from 275mm to 200mm and soon. These reductions of thickness involve compaction of the materials to its 100% MDD and reduction in water absorption nature.

For the safety factors, the reductions of thickness consider using the competent materials (higher CBR, lower plasticity and lower CBR swelling) at the same time. In another way the reduction in thickness of pavement layers reduce the susceptibility of the materials to settlement under load.

In addition the present study recommends an alternative use of capping materials depending on it availability at economic distance from the site.

Advantage of proposed pavement thickness

- The sub base and base course of the pavement are reduced comparatively, so the cost.
- The capping layers are remain the same in some case and reduced in another, which in turn result in requirement of thinner base and sub base layers.
- Since the minimum requirement of base course material is economically not feasible and not found in economic distance, the reduction of thickness reduces the amount of supply of qualified materials (higher CBR value and lower CBR swelling)
- In the section of higher CBR (≥ S3) value, the capping layers are not recommended in proposed design, hence it reduce the cost of construction.
- In some case there is a reduction of capping layer and increasing the thickness of the other layers, even it is economically not advisable, it increase the stability and performance of pavement.
- The use of availability of capping material is better in the route corridor, transportation and processing costs for base course & sub base will proportionally be minimized, hence an alternative capping material is proposed.

5.5 Proposed remedial measure for the problematic materials

One of the main objectives of present study is to find a counter measures that reduce the problem associated with subgrade materials and increase its suitability after identification of existing problems from field observation and laboratory results. Thus the station 0+000-16+000, 21+500-26+000 and 33+000-36+000 are identified to be unsuitable and required especial treatment.

As aforementioned in chapter 4, the problems associated with subgrade materials in present study area are bearing capacity problems, expansiveness, susceptible to erosion and compressibility problems. Depending on these problems the following remedial measures are forwarded in this study.

- The subgrade materials of the project roads are mostly categorized on the basis of strength under S1 and S2 which have CBR values of less than 4%, hence preventive measurement such as removal of such soils are recommended. However due to the thickness of the unsuitable materials along the road path, it is recommended to use existing materials by blending it with non-plastic materials with appropriate ratio.
- The CBR swelling result shows that the area have high swelling potential especially the dark brown silty CLAY (HS1 and HS2 of extension survey or HS1, HS3, HS4 and HS5 of CBR uniformity delineation) that affect mostly lighter structures such as road, hence to reduce the effects of swelling the materials should be compacted or filled with other materials that increase the overburden loads.
- From the tested results the subgrade materials show high plasticity index and liquid limits, which are an indication of expansiveness under alternative environmental condition, hence the water should be drained using surface and subsurface drainage such as ditches, horizontal drains and culverts.
- The materials in the study area show high compressibility nature (especially the HS1 and HS2 of soil extension survey, those poses higher Liquid limit) (See fig 5.5) and resulted in settlement of structures under applied loads, hence the compaction of subgrade materials up to its maximum dry density should be done before any construction activities.
- Susceptibility to erosion is one of the problem identified during the present study, the drainage must be well controlled. Covering of the soils with non-erodible materials and vegetation is usually effective. The XRD test result shows presence of Na ion in soils. Thus the erodaility of the soils due to the presence of sodium as an exchange cation in the CLAYs is the major problem. In addition the sodium rich soils shows higher plasticity index. Hence to reduce PI and sodium content 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 to reduce water percolation and piping effect.
- From the grain size analysis the subgrade materials in the study area shows a dominancy of fine grains (<0.075mm) which possesses low bearing capacity (CBR), hence blending of the existing materials with other coarser, competent and non-plastic materials could increase the percentage of coarser, reduce the finer one and increase bearing capacity.

- In flexible pavement the performance of structure is the cumulative contribution of each layer and the effects of traffic loads on subgrade materials is a function of load distribution capacity of over laying layers, hence using materials with higher distributing capacity without deformation can reduce the traffic load on subgrade and increase the performance of pavement.

SUMMARY

In this chapter over all characterization of subgrade materials and its suitability in terms of instability factors have been discussed. The field extension survey and laboratory results are compared to analysis the suitability of subgrade materials. Accordingly the dark brown silty CLAYs (HS1 & HS2) are classified under unsuitable for subgrade materials because these sections poses lower CBR value, higher plasticity, higher compressibility and higher CBR swelling. Hence preventive measurements such as removal, compaction to its maximum dry density, stabilization methods and proper structural design are proposed on the basis of instability factors.

In addition the engineering properties of subgrade materials are compared with topography and the topographic effect have been analyzed in relation with erosion and thickness of soil deposit along each topographic feature.

Finally different homogeneous sections are delineated using both cumulative difference and CBR uniformity methods to design appropriate pavement thickness that support dynamic load without deterioration for design life time.

Accordingly pavement thicknesses are proposed for different section with different layer thickness on the basis of subgrade CBR value. Thus most of the pavement thicknesses are reduced as compared with the design review, hence it is more advantageous.

CHAPTER VI

CONCLUSION AND RECOMMENDATION

As discussed in the previous chapters, the design of pavement depends on the geotechnical characteristics of subgrade materials and traffic volume. Hence, it is important to study the engineering properties of the native subgrade materials of the road alignments. The visual soil survey conducted along the project road alignments revealed that the subgrade formation is dominated by light to dark brown silty clay materials, which are rated as fair to poor for subgrade materials. To confirm this, different laboratory investigations are conducted and suitability of the materials has been evaluated based on the result. Finally some remedial measures for identified problem are forwarded and pavement thicknesses are proposed.

6.1 Conclusion

The present study was carried out on Sembo-Sholagebeya upgrading road project, on subgrade material characterization and analyzing the suitability. The study area is found in Northern Showa connecting Oromia regional state with Amhara regional state covering about 36.2 km. The road project lies on the volcanic rocks of basaltic and ignimbrite with fresh to highly weathered, that resulted in different soils formation with variable properties.

The main objective of the present study was to characterize the sub-grade soil for pavement construction and to analysis its general suitability for pavement design. Geotechnical characterizations of subgrade materials in the present study have been conducted to classify the subgrade soils into suitable and unsuitable for the selected pavement type under existing and adverse condition based on its field and laboratory results.

To come up with the goals of the objectives, during the present study field work and laboratory studies are carried out in a sequential manner. Firstly the geology of the roadway has been identified using Geological map of Ethiopia at scale of (1:2000,000) and geological map of Debre birhan area generated by Geological survey of Ethiopia in 2009 in order to know the various rock types that underlie the terrain and the soil extension survey was conducted.

Then after, disturbed soil samples at 500m and depth of 0.2 to 1m were collected for laboratory analysis. The laboratory tests carried out on the disturbed soil samples include; grain size analysis, moisture-density relationship (compaction), Atterberg (Consistency) limits, CBR swelling determination and California bearing ratio (CBR) for strength and bearing capacity evaluation. These laboratory tests were conducted at the AKIR construction plc. Sekorru site laboratory and were all performed in accordance with the specified standard procedures of (BSI, 1975; ASTM, 1979).

Additionally the bulk X-Ray Diffraction analysis on selected soil samples, petrographic identification on rock samples and triaxial test on undisturbed soil samples have been carried out in the Geological survey of Ethiopia and Addis Ababa University engineering geology and AAU petrology laboratories respectively.

The results of the present study shows that, the area have covered by problematic soils of less bearing capacity, highly plastic and expansive, less dry density and high optimum moisture content. Moreover, from the laboratory and field investigation of subgrade soils along the selected road section during present study, the following conclusions are made;

From USCS soil classification system; the soils in present study were classified as CL-OL, MH-OH, CH-OH and ML-CL group. Thus MH, CH and OH groups' covers about 72% that possesses high plasticity consisting of silt, clay and organic soils respectively. On the other hand ML, CL, OL groups covers 28% of the area that are low plasticity and low compressibility. Thus, the soils of high plasticity are the most dominant in the study area (HS1 & HS2 of extension survey), which are the most problematic for subgrade due to its compressibility and swelling nature.

In another way, using the AASHTO (1993) classification, the soil in the study area have been grouped under A-2-7, A-6, A-7-5 and A-7-6, in which A-7-5 is the most dominant (about 65%) that characterized by dark brown silty CLAY and it is the most problematic soils due to its high plasticity and swelling potential; thus the study area is largely covered by expansive soils, which requires some treatment or removal.

The group index which is a function of liquid limit, plasticity index and percentage passing 0.075mm have been calculated for the subgrade materials and about 46% of total samples shows high group index (GI>20%), which depict the subgrade materials to be unsuitable for subgrade materials according to AASHTO, 1993. Hence the GI can be reduced using compaction, blending with other granular materials to reduce percentage of grain size passing 0.075mm.

The moisture - density relationship, that is the compaction test (annex-2) indicates a maximum dry density value ranging from 1.33 g/cc to 1.92 g/cc while the optimum moisture content range from 12.4 % (HS4) to 33.7% (HS1). This shows the soils have higher porosity, hence it can absorb much amount of water and able to saturate completely and tend to flow and swell. Even the optimum moisture content and maximum dry density have no recommended permissible limits for subgrade materials, the materials should have higher density that have less susceptible to settlement and compressibility.

The linear shrinkage calculated using (Arora, 1997) equation, that is a function of its plasticity index, depict that most of the value is greater than 7, which indicate the soils in the study area is active and highly expansive (ASTM, 1979).

The California bearing ratio at 95% MDD were found to be ranging from 1 to 24% in which most of the soils (60 %) fall under S1 and S2 strength classes of subgrade strength according to (ERA, 2002) classification. This result indicates that the area is covered by less strength soils and it tends to fail if traffic and pavement loads area applied directly without treatment or improvement of the natural materials.

About 64.2% of the present study area possesses CBR swelling greater than 2.5% thus indicate the soils are highly swelling potential. Since the pavement is a light engineering structures, it is susceptible to swelling effects. Thus the study area has been evaluated in terms of its swelling being unsuitable for pavement, especially for flexible pavement.

The strength parameters (internal friction (φ) and cohesion (c) of two selected subgrade materials that have been conducted during the present study using triaxial test show that the parameters are sensitive to compaction condition (moisture content and dry density). Even though the tests have been conducted on different samples, both parameters are slightly decreasing as moisture content increase and dry density decrease. Moreover the failure type also affect the result of strength parameters, thus the soils in present study are cohesive, hence shear failure have been observed rather than plastic and compressional failure.

From the present study it is observed that, there are segments of the roads passing through agriculture lands. The bearing capacity of the subgrade materials in such areas is relatively lower and they do not sustain loads. As the excavation, removal and replacement of such materials with selected materials involve higher cost, to minimize the depth of excavation and increase the thickness of overlying pavement section in order to minimize the intensity of loads applied on the formation is the most important measures.

Generally three distinct categories of sub grade material were observed along the entire road when the sub grade soil properties of Sembo –shola gebeya Road were investigated. The first 12 km of the road section is characterized by highly plastic dark brown silty clay soils, sometimes small stretches of weathered rock underlying the top soils. The laboratory test results indicate that this stretch has lower CBR value (varying from 2.6 to 5.5%) and higher swell potential >2.5 %. The physical properties of these soils associated with their laboratory test results give them the nature of expansive clay soils.

These soils exhibit large volumetric changes when soaked with water and also shrink and crack when they become dry. The cracks allow water to penetrate deep in to the soil causing considerable expansion. These results in deformation of the road surface since the expansion and the subsequent heave are never uniform. Further these volumes changes may produce lateral displacements of the expansive clay if the side slopes are not gentle enough.

In addition to this, expansive clay soils have bearing capacity problems when the moisture content increases. Their CBR value will be reduced to less than 2% if they are completely saturated during rainy seasons. Erosion is also problem to expansive clay soils as when dry they present sand like texture and in these states they are prone to erosion to much greater extent than that normally anticipated from their plasticity and clay content.

In the next stretch, from 12+000 to 28+000, the natural sub grade material has mixed properties of soil and rock. The soil is dark brown silty clay soil mixed with rock fragments and boulders while the rock is weathered rock. But the occurrence of clayey soils is observed repeatedly as compared to the weathered rock road bed. The laboratory test result indicates that the sampled soils have less plasticity index, CBR values varying from 1 to 24 and swell values of greater than 2%. These soils can be used as a direct support to pavement layers without improvement.

The last portion of the road from 28 +000 to 36+200 constitutes rocky sub grade material for majority of its portion. It has a CBR value varying from 5.1 to 21.2% however around 25+100 the CBR value is as low as 1.1 while in other portions the CBR value as high as 46.5% have been encountered. Since these values are not consistent and their degree of occurrence is limited, special treatment is required at the stretches where the sub grade material has poor strength. The sub grade soil has lower swell potential which indicates the soil has no problem of swelling rather it has low bearing capacity.

6.2 Recommendation

From the field investigation and laboratory results in present study, the area is evaluated to be problematic on the basis of its different parameters. Hence the following recommendations are made to reduce unsuitability and make the study more detail;

- The site is located near the active Ethiopian rift. Therefore, detailed seismic study must be performed and appropriate seismic parameters must be incorporated in the design.

- The road alignment pass mostly in agricultural lands that possess lower bearing capacity and higher plasticity, hence realignment of some part of the road is required.

- The present study is conducted by taking limited parameters (atterberg limit, strength, compaction, CBR swelling and grain size analysis) on subgrade soils along the road path which may not be fully characterize the materials. So it is recommended that additional parameters (hydrometers for clay content determination, XRD on clay fraction, consolidation (settlement) using odometers, DCP) should be tested to reinforce the results obtained from this research.

- The comparison between direct mineralogical (XRD) test and determination of minerals using indirect methods of Casagrande's plasticity chart shows a contradiction of the results on the same samples, thus the Casagrande's plasticity chart may not be accurate as XRD clay mineral identification method; hence the plasticity chart method should be modified or be used with exceptional.

- Subgrade materials engineering behaviors are sensitive to the uncontrolled moisture content it will be subjected to during the life of pavement; hence the parameters should be tested at worst, wet and dry condition to know its effect.

- The engineering performance of subgrade materials are a function of its moisture content, hence moisture content and plasticity of subgrade should be incorporated in ERA structural catalog in addition to traffic class and CBR value.

- The suitability of the subgrade materials in this study have been evaluated on the basis of ERA and project standard, but the ERA standard classifies the unsuitable soils as suitable for subgrade especially on the basis of plasticity (let say for example, soils that have LL between 50 to 60%, that AASHTO, (1993) classified as highly plastic and compressible but ERA manual classify as suitable for subgrade) (Figure: 5.5), hence the ERA standard should be reasonable or be exceptional for.

- The homogeneous section delineation for pavement thickness designing using cumulative difference methods categorize materials of extremely higher and lower CBR value under one section, hence other method (CBR uniformity) should be used.

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

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Annex -1-

Terrain classifications and summary of soil extension survey

No.	Station		Terrain Type/Town	Material description on field survey.
	From	To		
1	0+000	5+300	Flat	Dark brown clay about 50m weathered rock outcrops around Kottu gebeya town
2	5+300	6+900	Minor town	
3	6+900	7+900	Rolling	Light brown silty clay alternating with dark clay mixed with weathered rock fragments and 100m fractured weathered rock
4	7+900	10+415	Flat	
5	10+415	11+300	Rolling	Dark clay and dark clay of about 50cm thickness underlain by fractured and weathered trachytic basalt
6	11+300	12+400	Flat	
7	12+400	13+100	Rolling	Dark brown silty clay mixed with rock & underlain by weathered rocks
8	13+100	14+400	Flat	About 100m weathered rock exposed and closely to widely jointed and moderately weathered rock that disintegrated into boulders, cobbles, gravels and finer fraction grain sized.
9	14+400	15+000	Rolling	
10	15+000	15+300	Mountainous	
11	15+300	15+600	Flat	
12	15+600	16+700	Mountainous	
13	16+700	16+900	Rolling	
14	16+900	17+100	Mountainous	
15	17+100	17+500	Rolling	
16	17+500	18+900	Flat	Light to dark brown silty clay mixed with rock fragments and weathered rocks exposed
17	18+900	19+200	Rolling	
18	19+200	20+000	Rolling	Fractured and weathered rock disintegrated into boulders, cobbles and gravels and finer fractions, 300m thick layer of dark clay with rock fragments in between. And some reddish soils of no plastic area exposed.
19	20+000	20+600	Mountainous	
20	20+600	21+400	Minor town	Fractured and weathered rock disintegrated into boulders, cobbles and gravels and finer fractions, 300m thick layer of dark clay with rock fragments in between. And some reddish soils of no plastic area exposed.
21	21+400	21+500	Mountainous	
22	21+500	22+600	Flat	
23	22+600	23+200	Rolling	Dark clay alternating with dark clay mixed with rock fragments at some places weathered rock outcrops. The area is highly cracked.
24	23+200	23+500	Mountainous	
26	23+600	24+000	Flat	
27	24+000	24+300	Mountainous	Closely jointed and moderately weathered basaltic rocks
28	24+300	24+400	Flat	
29	24+400	25+960	Mountainous	Dark brown silty clay mixed with basalt rock fragments underlain by widely jointed and moderately weathered basalt disintegrated into boulders, cobbles and gravels. About 700m extension of Dark clay soil also found.
30	25+960	26+560	Escarpment	Dark clay soils with some yellowish color soils
31	26+560	26+800	Mountainous	Slightly weathered rocks of volcanics
32	26+800	27+600	Minor town	Dark clay soils with some yellowish color soils
33	27+600	28+000	Rolling	
34	28+000	29+300	Mountainous	
35	29+300	29+600	Escarpment	
36	29+600	30+980	Mountainous	Slightly weathered rocks and reddish soil and Brown to dark clay soil
37	30+980	31+300	Escarpment	Moderate to highly weathered rocks, and small scale landslide.
38	31+300	34+800	Rolling	Dark clay soil underlain by moderate to highly weathered rocks and closely jointed and slight to moderately weathered rocks of basalt,
39	34+800	35+300	Mountainous	Weathered and decomposed rock overlain by brown silty clay and Dark clay soil underlain by highly weathered rock
40	35+300	35+600	Escarpment	Dark clay soil underlain by highly weathered rock and Basaltic boulders outcrop
41	35+600	36+320	Rolling	Dark clay soil underlain by slight to moderately weathered rock. Brown silty clay soil underlain by fractured and highly decomposed rocks

Annex-2-

Test results of subgrade materials in sekoru site laboratory of AKIR construction

Client — Ethiopian Road Authority

Contractor — Akir Construction Plc.

Consultant — COREconsultancy

SEMBO - SHOLAGEBEYA - GORFO - GINDEBER ROAD UPGRADING PROJECT
From station 0+000 to 3+500km

No.	station	Depth (cm)	Field description	Sieve analysis % pass (mm)			Atterberg limits			AASHTO Classification	Proctor test		GI	CBR at 95% MDD	ERA Strength Classes	CBR Swelling At 95%
				2	0.425	0.075	LL	PL	PI		MDD	OMC				
1	0+000	20-60	dark brown to gray soil	96	92	87.5	63	34	29	A-7-5 (30)	1.63	18.9	30	2	S1	4.3
2	0+500	20-60	dark brown silty clay	98	93.3	90	60	32	28	A-7-5 (30)	1.57	17.3	30	2	S1	5.09
3	1+000	20-60	dark brown silty clay	98.3	94.6	89.9	73	38	35	A-7-5(39)	1.51	22	39	1	S1	6.2
4	1+500	20-60	Light brown silty clay with weathered gravel	79	73.6	68.3	61	36	25	A-7-5 (18)	1.74	14.3	18	2	S1	4.98
5	2+000	20-60	dark brown silty clay	96	90	82	71	48	23	A-7-5(29)	1.5	24	25	2	S1	5.26
6	2+500	20-60	dark brown silty clay	98	93.3	86	82	42	40	A-7-5(42)	1.59	16.6	42	2	S1	5.02
7	3+000	20-60	dark brown silty clay	96.3	92.8	88.7	73	36	37	A-7-5(40)	1.53	20.8	40	2	S1	5.96
8	3+500	20-60	dark brown silty clay	97	92.3	86.3 3	69	41	28	A-7-5(31)	1.57	16.2	31	1.9	S1	5.17

Tested by

Contractor ____________

Cheked 10/4/14

Client — Ethiopian Road Authority

Contractor — Akir Construction Plc.

Consultant — CORE consultancy

SEMBO - SHOLAGEBEYA - GORFO - GINDEBER ROAD UPGRADING PROJECT
From station 4+000 to 7+500

No.	station	Depth (cm)	Field description	Sieve analysis % pass (mm)			Atterberg limits			AASHTO Classificatio	Proctor test		GI	CBR at 95% MDD	ERA Strength Classes	CBR Swelling At 95%
				2	0.425	0.075	LL	PL	PI		MDD	OMC				
9	4+000	20-60	dark brown silty clay	100	96	89.33	74	32	42	A-7-5 (44)	1.5	20.4	44	2	S1	4.92
10	4+500	20-60	dark brown silty clay	99.33	96	87.33	69	37	32	A-7-5 (34)	1.55	16.9	34	1	S1	6.6
11	5+000	20-60	dark brown silty clay	93.33	90.63	83.9	72	43	29	A-7-5(31)	1.47	16.6	31	1	S1	6.56
12	5+500	20-60	dark brown silty clay	98.6	90	79	57	35	22	A-7-5(20)	1.6	18.6	20	2	S1	6.77
13	6+000	20-60	dark brown silty clay	99	96.33	92.33	70	35	35	A-7-5(39)	1.57	19.4	39	2	S1	5.05
14	6+500	20-60	light brown silty clay	98.3	90.66	79.99	49	33	16	A-7-5(15)	1.64	17.9	15	10	S4	1
15	7+000	20-60	dark brown silty clay	89.7	85.03	77.03	78	36	42	A-7-5(36)	1.65	16.9	36	2	S1	5.55
16	7+500	20-60	dark brown silty clay so	94.8	84.24	71.68	54	20	34	A-7-5 (24)	1.625	23.8	24	6	S3	2.47

Tested by

Contractor ____________

Akir Construction PLC
Sembo-Gindeber A/R/Project

Client — Ethiopian Road Authority

Contractor — Akir Construction Plc.

Consultant — CORE consultancy

SEMBO - SHOLAGEBEYA - GORFO - GINDEBER ROAD UPGRADING PROJECT

From station 8+000 to 12+000

No.	station	Depth (cm)	Field description	Sieve analysis % pass (mm)			Atterberg limits			AASHTO Classification	Proctor test		GI	CBR a 95% MDD	ERA Strength Classes	CBR Swelling at 95%
				2	0.425	0.075	LL	PL	PI		MDD	OMC				
17	8+000	20-60	dark brown sity clay	94	81.33	68.66	54	28	26	A-7-6(18)	1.67	17.8	18	2	S1	6.43
18	8+500	20-60	dark brown silty clay	95	84	78.5	65	34	31	A-7-5 (27)	1.55	20.2	27	1	S1	6.15
19	9+000	20-60	Light brown silty clay	98.3	91	80.7	50	30	20	A-7-6(22)	1.59	18.5	22	9	S4	2.02
20	9+500	20-60	light brown silty clay	99	95.3	91.6	74	46	28	A-7-5 (35)	1.5	20.2	35	2	S1	5.3
21	9+740-940	20-60	light brown silty clay	98.2	94.5	91	74	27	47	A-7-6 (49)	1.33	28.4	49	2	S1	3.9
22	10+000	20-60	Dark brown silty clay	97.3	92.7	86	70	41	29	A-7-5(31)	1.51	21.2	31	2	S1	3.62
23	10+500	20-60	Dark brown silty clay	99	91.3	82.7	68	38	30	A-7-5(30)	1.55	25.4	30	2	S1	4.91
24	11+000	20-60	Dark brown silty clay	99.3	88	81.7	67	35	32	A-7-5 (30)	1.47	26.1	30	2	S1	4.21
25	11+500	20-60	dark brown silty clay	97.7	91.7	91.7	66	34	32	A-7-5(35)	1.5	25.2	35	3	S2	3.42

Tested byContractor

Client — *Ethiopian Road Authority*

Contractor — *Akir Construction Plc.*

Consultant — *CORE consultancy*

SEMBO - SHOLAGEBEYA - GORFO - GINDEBER ROAD UPGRADING PROJECT

No.	station	Depth (cm)	Field description	Sieve analysis % pass (mm)			Atterberg limits			AASHTO Classification	Proctor test		GI	CBR at 95% MDD	ERA Strength Classes	CBR Swelling At 95%
				2	0.425	0.075	LL	PL	PI		MDD	OMC				
26	12+000	20-60	light brown silty clay mix with some gravel	48.6	40.8	34	**44**	29	**15**	A-2-7 (1)	1.77	**16.7**	0.73	**8**	S4	**2.5**
27	12+500	20-60	dark brown silty clay	95.7	81	70.3	**68**	40	**28**	A-7-5 (22)	1.63	**20.4**	22	**2**	S1	**6.46**
28	13+000	20-60	light brown silty clay	96	82	67.3	**57**	36	**21**	A-7-5 (15)	1.48	**23.3**	15	**2**	S1	**3.58**
29	13+500	20-60	light brown silty clay mix with some gravel	38.4	32.2	29	**43**	26	**17**	A-2-7 (0)	1.86	**13.4**	0	**7**	S3	**2.16**
30	14+000	20-60	dark brown silty clay	93.7	82	71.3	**54**	35	**19**	A-7-5 (15)	1.58	**19.2**	15	**3**	S2	**3.57**
31	14+500	20-100	dark brown silty clay	89.5	79.5	72.5	**52**	33	**19**	A-7-2(15)	1.7	**15**	15	**4**	S2	**4.4**
32	15+180	60-100	dark brown silty clay	97	90.5	70.5	**69**	39	**30**	A-7-5(23)	1.5	**22.4**	23.3	**2**	S1	**6.45**
33	15+560	20-60	dark brown silty clay	94.8	84.7	74.59	**56**	37	**19**	A-7-6 (16)	1.5	**25.9**	16	**3**	S2	**2.52**
34	15+500	20-60	Llight brown silty clay	95.6	91.67	86	**75**	45	**30**	A-7-5 (33)	1.55	**2.4**	33	**2**	S1	**7.1**

Contractor ____________

Client

Ethiopian Road Authority

Contractor

Akir Construction Plc.

Consultant

CORE consultancy

SEMBO - SHOLAGEBEYA - GORFO - GINDEBER ROAD UPGRADING PROJECT

No.	station	Depth (cm)	Field description	Sieve analysis % pass (mm)			Atterberg limits			AASHTO Classification	Proctor test		GI	CBR at 95% MDD	ERA Strength Classes	CBR Swelling At 95%
				2	0.425	0.075	LL	PL	PI		MDD	OMC				
35	16+000	20-60	Light brown silty clay	89.8	86.49	82.94	**59**	34	**25**	A-7-6(25)	1.56	**21.8**	24	**2**	S1	**5.99**
36	16+500	20-60	Light brown gravel sandy	57.9	40.01	29.67	**55**	37	**18**	A-2-7(0)	1.59	**22.3**	-0.29	**12**	S4	**1.7**
37	16+500	60-100	sandy silty clay with few gravel	61.45	45.8	37.68	**54**	30	**24**	A-7-5 (4)	1.62	**21**	3.89	**15**	S5	**1.54**
38	17+000	20-60	light brown silty clay	74.6	70.2	64.8	**56**	27	**29**	A-7-5 (18)	1.64		17.8	**11**	S4	**1.36**
39	17+100	20-60	dark brown silty clay	89.25	86	82.5	**58**	29	**29**	A-7-5 (27)	1.62	**19.1**	26.6	**1**	S1	**7.18**
40	17+100	60-100	light weathered gravel	81.15	77.34	72.19	**44**	25	**19**	A-7-6 (13)	1.84	**15.2**	13.3	**8**	S4	**0.62**
41	17+500	60-100	highly waethered gravel	90.81	83.01	70.2	**49**	31	**18**	A-7-6(13)	1.62	**22.4**	13	**2**	S1	**8.8**
42	18+000	20-60	light brown silty clay	93.33	84.31	74.9	**44**	26	**18**	A-7-5(13)	1.71	**17**	13	**7**	S3	**3.45**
43	18+500	20-60	dark brown silty clay	93.2	85.75	78.75	**56**	31	**25**	A-7-5 (22)	1.683	**21.8**	22	**2**	S1	**4.89**
44	19+000	20-60	Light brown silty clay gravel	97.1	85.67	72.97	**49**	29	**20**	A-7-6 (0)	1.534	**22.4**	15	**16**	S5	**1.03**

Tested byContractor ____________________

13456
Tel 011-442-0401

Client — Ethiopian Road Authority

Contractor — Akir Construction Plc.

Consultant — CORE consultancy

SEMBO - SHOLAGEBEYA - GORFO - GINDEBER ROAD UPGRADING PROJECT

No.	station	Depth (cm)	Field description	Sieve analysis % pass (mm)			Atterberg limits			AASHTO Classification	Proctor test		GI	CBR at 95% MDD	ERA Strength Classes	CBR Swelling At 95%
				2	0.425	0.075	LL	PL	PI		MDD %	OMC %				
45	19+500	20-60	Weathered gravely sand	32.4	15.03	7.2	**53**	32	**21**	A-2-7 (0)	1.424	**26.6**	-8.2	**17**	S5	**0.48**
46	19+600	20-60	weathered gravel	51.4	32.32	15.03	**53**	32	**21**	A-2-7 (0)	1.424	**26.6**	-5.28	**17**	S5	**0.48**
47	20+000	20-60	light brown silty clay	96.7	93.52	90.28	**66**	35	**31**	A-7-5 (34)	1.655	**20.3**	34.05	**2**	S2	**7.04**
48	20+500	20-60	waethered gravel	94.3	90.07	86.46	**49**	29	**20**	A-7-6(20)	1.554	**19.7**	19.75	**24**	S5	**0.46**
49	21+000	20-60	dark brown silty clay soil	96.1	91.28	87.15	**68**	39	**29**	A-7-5 (31)	1.58	**20.8**	31.4	**2**	S1	**9.83**
50	21+500	20-60	Light brown silty clay with few weathered gravel	61.6	52.33	46.12	**51**	33	**18**	A-7-5 (5)	1.626	**21.6**	5.3	**11.6**	S4	**3.15**
51	21+500	60-100	Weathered gravel	77.6	63.45	56.24	**44**	29	**15**	A-7-6 (7)	1.625	**20.8**	6.74	**5**	S3	**3.4**
52	22+00	20-60	Yellowish gravely sand	97.9	90.58	37.62	**44**	31	**13**	A-7-5 (18)	1.412	**23**	1.25	**9**	S4	**3.33**
53	22+000	20-100	light brown silty clay with grave;	80.95	76.8	66.6	**68**	43	**25**	A-7-5 (18)	1.532	**20.2**	18.5	**4**	S2	**2.18**

Tested by: contractor

Tel 011-442-0401

13456

Akir Construction PLC

Sembo-Gindeber A/R/Project

Client

Ethiopian Road Authority

Contractor

Akir Construction Plc.

Consultant

CORE consultancy

SEMBO - SHOLAGEBEYA - GORFO - GINDEBER ROAD UPGRADING PROJECT

From station 22+500 to 25+500

No.	station	Depth (cm)	Field description	Sieve analysis % pass (mm)			Atterberg limits			AASHTO Classification	Proctor test		GI	CBR at 95% MDD	ERA Strength Classes	CBR Swelling At 95%
				2	0.425	0.075	LL	PL	PI		MDD	OMC				
54	22+500	20-60							0				1.5			
55	23+000	20-60	dark brown silty clay	95.6	91.77	87.65	71	36	35	A-7-5 (37)	1.522	25.1	36.8	2	S1	8.3
56	23+500	20-60	Dark brown silty clay	91.9	85.35	77.88	49	28	21	A-7-6 (17)	1.725	16.7	17.42	2	S1	6.09
57	23+500	20-100	dark brown silt clay	95.5	90.31	83.22	60	30	30	A-7-5 (28)	1.672	19.6	28.11	3	S2	5.3
58	24+000	20-60							0				1.5			
59	24+500	20-60	Dark brownsilty clay with few gravel	94.4	86.32	79.06	49	29	20	A-7-6 (17)	1.633	19.7	17.20	3	S2	7.04
60	25+000	20-60	Light brown silty clay	77.3	73.64	67.47	50	29	21	A-2-6 (14)	1.625	18.2	13.88	5	S3	3.51
61	25+500	20-60	dark brown silty clay	86.8	71.8	57.3	46	29	17	A-7-6 (8)	1.69	20.1	8.09	7	S3	1.83
62	25+500	20-60	Weathered gravely sand	48.1	38	30	35	25	10	A-2-6 (0)	1.92	14.1	-0.8	11	S4	0.4

Tested by

Contractor ____________

Akir Construction PLC

Tel 011-442-0401

13456

Client — Ethiopian Road Authority

Contractor — Akir Construction Plc.

Consultant — CORE

SEMBO - SHOLAGEBEYA - GORFO - GINDEBER ROAD UPGRADING PROJECT

From station 26+000 to 30+000

No.	station	Depth (cm)	Field description	Sieve analysis % pass (mm) 2	0.425	0.075	Atterberg limits LL	PL	PI	AASHTO Classification	Proctor test MDD %	OMC	GI	CBR at 95% MDD	ERA Strength Classes	CBR Swelling At 95%
63	26+000	20-60	light brown silty clay	97.9	88.3	77.6	40	24	16	A-7-6 (12)	1.69	18.6	12.27	4	S2	1.83
64	26+000	20-60	light brown silty clay mix with gravel	67.9	60	51.2	31	18	13	A-6 (4)	1.88	14.6	3.59	10	S4	0.61
65	27+000	20-60	light brown silty clay	98.5	94.5	85.9	56	33	23	A-7-5 (23)	1.67	19.5	23.46	2	S1	8.14
66	27+500	20-60	dark brown silty clay	94.8	83.5	72.5	56	38	18	A-7-5 (15)	1.61	21.6	15.1	5	S3	3.51
67	28+000	20-60	light brown silty-clay	99.5	89.1	77.8	45	25	20	A-7-6 (16)	1.68	17.8	15.91	6	S3	1.61
68	28+500	20-60	brown silty clay	96.3	87.6	69.2	45	29	16	A-7-6 (11)	1.63	21.9	11	9	S4	1.49
69	29+000	20-60	light brown sandy clay with s gravel	94.5	89	85	82	44	38	A-7-5 (40)	1.49	22.2	40.1	1	S1	7.64
70	29+500	20-60	light brown sandy clay mix with some gravel	40.1	33.5	29.3	44	26	18	A-2-7 (0)	1.92	14.3	-0.1	14	S4	0.54
71	30+000	20-60	dark brown silty clay	98.6	84.3	66.2	47	29	18	A-7-6 (11)	1.61	22.6	11.4	7	S3	2.49

Tested by

Contractor ____________

Akir Construction PLC Sembo Gindeber A/R/Project 13456 Tel 011 442-0401

Client — Ethiopian Road Authority

Contractor — Akir Construction Plc.

Consultant — consultancy CORE

SEMBO - SHOLAGEBEYA - GORFO - GINDEBER ROAD UPGRADING PROJECT
From station 30+500 to 34+000

No.	station	Depth (cm)	Field description	Sieve analysis % pass (mm)			Atterberg limits			AASHTO Classification	Proctor test		GI	CBR at 95% MDD	ERA Strength Classes	CBR Swelling At 95%
				2	0.425	0.075	LL	PL	PI		MDD	OMC				
72	30+500	20-60	dark brown silty clay	99	93.7	89.4	74	55	19	A-7-5 (27)	1.54	24.8	26.84	2	S1	8.81
73	31+000	20-60	light brown sandy clay mix with some gravel	98.3	93.1	84	49	33	16	A-7-5 (16)	1.71	19.8	16	9	S4	1.87
74	31+500	20-60	light brown silty clay soil	63.7	60.4	54.1	68	34	34	A-7-5 (16)	1.6	22.2	15.87	2	S1	7.77
75	32+000	20-60	light brown silty clay soil	98	88.3	66.3	60	36	24	A-7-5 (17)	1.41	33.7	16.57	5	S3	0.89
76	32+500	20-60	dark brown silty clay	98.3	87	74.7	69	43	26	A-7-5 (23)	1.51	21.2	23.2	1	S1	4.1
77	33+000	20-60	Red to brown silty clay mix with some gravel	91.8	75	59.5	44	30	14	A-7-6 (7)	1.49	23	7.17	9	S4	2.4
78	33+500	20-60	brown silty clay	97.7	84.3	78.7	68	36	32	A-7-5 (29)	1.56	26.7	28.87	2	S1	2.18
79	34+000	20-60	dark brown silty clay	96.7	89	79.3	59	40	19	A-7-5 (19)	1.55	20.2	19	1	S1	8.18

Tested by

Contractor ____________

13456
Tel 011-442-0401
Akir Construction PLC

Consultant

Contractor

Client

Ethiopian Road Authority — *Akir Construction Plc.* — *CORE consultancy*

SEMBO - SHOLAGEBEYA - GORFO - GINDEBER ROAD UPGRADING PROJECT

From station 34+500 to 36+200

No.	station	Depth (cm)	Field description	Sieve analysis % pass (mm)			Atterberg limits			AASHTO Classification	Proctor test		GI	CBR at 95% MDD	ERA Strength Classes	CBR Swelling At 95%
				2	0.425	0.075	LL	PL	PI		MDD	OMC				
80	34+500	20-60	dark brown silty clay	97	87.5	82.5	**63**	47	**16**	A-7-5 (19)	1.52	**23.2**	19	**2**	S1	8.14
81	35+000	20-60	dark silty clay soil	99.3	92	81.3	**75**	44	**31**	A-7-5 (31)	1.5	**19.6**	31	**2**	S1	5.62
82	35+500	20-60	light brown silty clay soil	95.5	86.3	75.3	**61**	46	**15**	A-7-5(15)	1.75	**20.7**	15.3 0	**11**	S4	1.16
83	36+000	20-60	Red to brown silty clay	98.5	92.3	78.8	**52**	26	**26**	A-7-6(22)	1.62	**20.2**	22	**4**	S2	**3.94**
84	36+200	20-60	brown silty clay with some grave	42.9	35.7	28.03	**47**	26	**21**	A-2-7 (0)	1.63	**18.6**	-0.2	**5**	S3	2.4

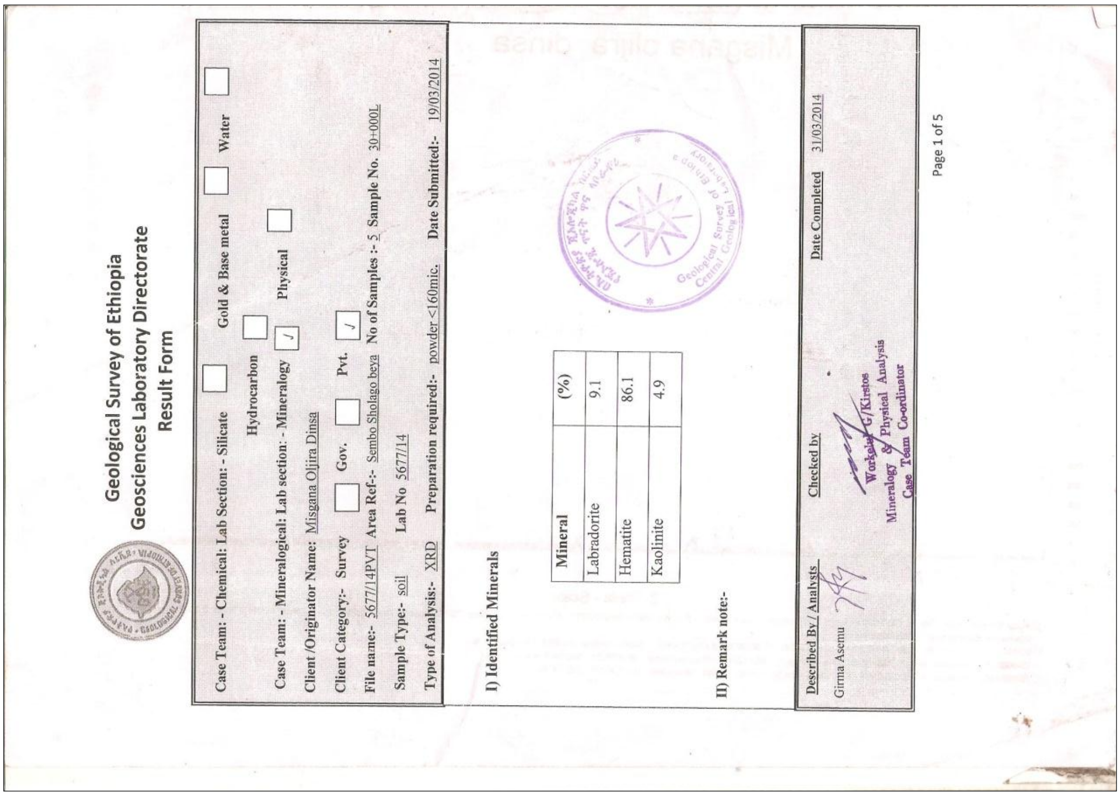
Tested by

Contractor ____________

13456
Tel 011-442-0401
Akir Construction PLC
Sembo-Gindeber A/R/Project

Annex- 3-

X-ray diffraction results of selected soil samples



Geological Survey of Ethiopia
Geosciences Laboratory Directorate
Result Form

Case Team: - Chemical: Lab Section: - Silicate [ ] Gold & Base metal [ ] Water [ ]
Hydrocarbon [ ]
Case Team: - Mineralogical: Lab section: - Mineralogy [√] Physical [ ]
Client /Originator Name: Misgana Oljira Dinsa
Client Category:- Survey [ ] Gov. [ ] Pvt. [√]
File name:- 5677/14PVT Area Ref-:- Sembo Sholago beya No of Samples :- 5 Sample No. 30+000L
Sample Type:- soil Lab No 5677/14
Type of Analysis:- XRD Preparation required:- powder <160mic. Date Submitted:- 19/03/2014

I) Identified Minerals

Mineral	(%)
Labradorite	9.1
Hematite	86.1
Kaolinite	4.9

II) Remark note:-

Described By/ Analysts	Checked by	Date Completed
Girma Asemu	Worke G/Kirstos Mineralogy & Physical Analysis Case Team Co-ordinator	31/03/2014

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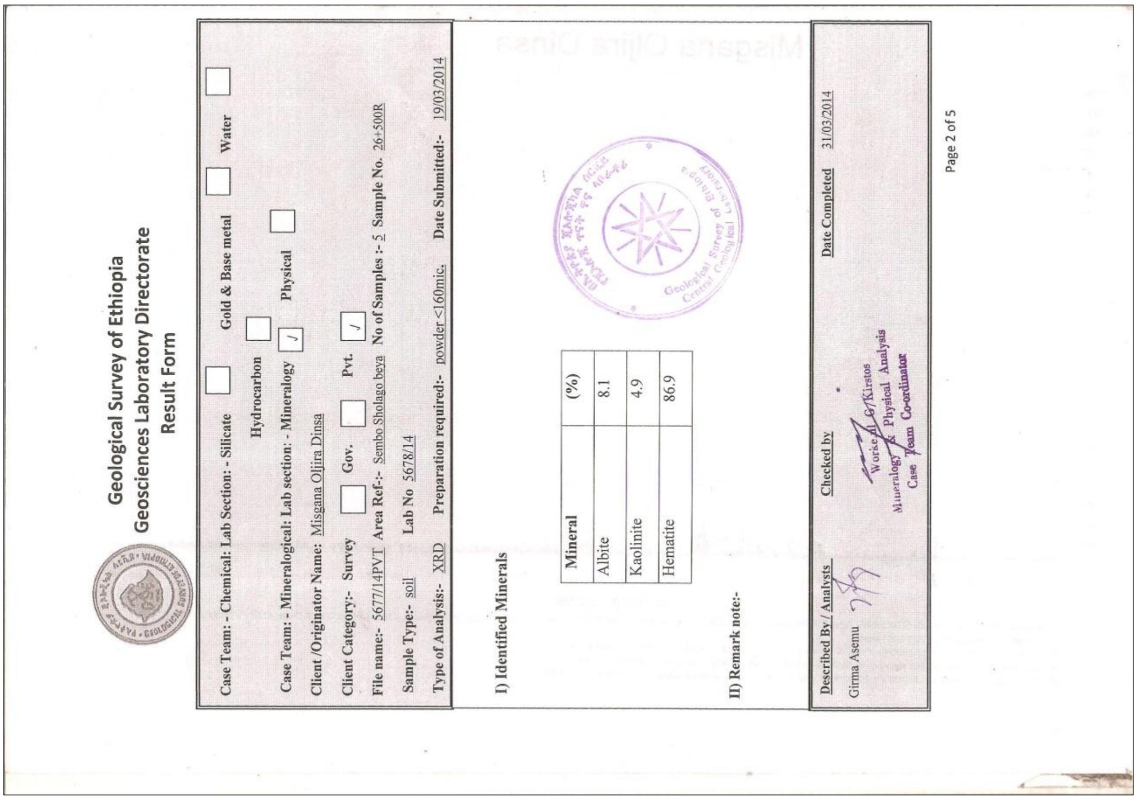

Geological Survey of Ethiopia
Geosciences Laboratory Directorate
Result Form

Case Team: - Chemical: Lab Section: - Silicate [ ] Gold & Base metal [ ] Water [ ]
Hydrocarbon [ ]
Case Team: - Mineralogical: Lab section: - Mineralogy [√] Physical [ ]
Client /Originator Name: Misgana Oljira Dinsa
Client Category:- Survey [ ] Gov. [ ] Pvt. [√]
File name:- 5677/14PVT Area Ref-:- Sembo Sholago beya No of Samples :- 5 Sample No. 26+500R
Sample Type:- soil Lab No 5678/14
Type of Analysis:- XRD Preparation required:- powder <160mic. Date Submitted:- 19/03/2014

I) Identified Minerals

Mineral	(%)
Albite	8.1
Kaolinite	4.9
Hematite	86.9

II) Remark note:-

Described By/ Analysts	Checked by	Date Completed
Girma Asemu	Worke G/Kirstos Mineralogy & Physical Analysis Case Team Co-ordinator	31/03/2014

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Geological Survey of Ethiopia
Geosciences Laboratory Directorate
Result Form

Case Team: - Chemical: Lab Section: - Silicate ☐ Gold & Base metal ☐ Water ☐

Hydrocarbon ☐

Case Team: - Mineralogical: Lab section: - Mineralogy ☑ Physical ☐

Client /Originator Name: Misgana Oljira Dinsa

Client Category:- Survey ☐ Gov. ☐ Pvt. ☑

File name:- 5677/14PVT Area Ref-:- Sembo Sholago beya No of Samples :- 5 Sample No. 36+200R

Sample Type:- soil Lab No 5679/14

Type of Analysis:- XRD Preparation required:- powder <160mic. Date Submitted:- 19/03/2014

I) Identified Minerals

Mineral	(%)
Albite	82.5
Hematite	17.5

II) Remark note:-

Described By / Analysts	Checked by	Date Completed	31/03/2014
Girma Asemu	Workeaul G/Kirstos Mineralogy & Physical Analysis Case Team Co-ordinator		

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Geological Survey of Ethiopia
Geosciences Laboratory Directorate
Result Form

Case Team: - Chemical: Lab Section: - Silicate ☐ Gold & Base metal ☐ Water ☐

Hydrocarbon ☐

Case Team: - Mineralogical: Lab section: - Mineralogy ☑ Physical ☐

Client /Originator Name: Misgana Oljira Dinsa

Client Category:- Survey ☐ Gov. ☐ Pvt. ☑

File name:- 5677/14PVT Area Ref-:- Sembo Sholago beya No of Samples :- 5 Sample No. 25+500R

Sample Type:- soil Lab No 5680/14

Type of Analysis:- XRD Preparation required:- powder <160mic. Date Submitted:- 19/03/2014

I) Identified Minerals

Mineral	(%)
Sanidine	66.5
Kaolinite	6.7
Quartz	13.7
Hematite	12.9

II) Remark note:-

Described By / Analysts	Checked by	Date Completed	31/03/2014
Girma Asemu	Workeaul G/Kirstos Mineralogy & Physical Analysis Case Team Co-ordinator		

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Geological Survey of Ethiopia
Geosciences Laboratory Directorate
Result Form

Case Team: - Chemical: Lab Section: - Silicate ☐ **Gold & Base metal** ☐ **Water** ☐

Hydrocarbon ☐

Case Team: - Mineralogical: Lab section: - Mineralogy ☑ **Physical** ☐

Client /Originator Name: Misgana Oljira Dinsa

Client Category:- Survey ☐ **Gov.** ☐ **Pvt.** ☑

File name:- 5677/14PVT **Area Ref-:-** Sembo Sholago beya **No of Samples :-** 5 **Sample No.** 29+000R

Sample Type:- soil **Lab No** 5677/14

Type of Analysis:- XRD **Preparation required:-** powder <160mic. **Date Submitted:-** 19/03/2014

I) Identified Minerals

Mineral	(%)
Albite	56.4
Hematite	10.5
Kaolinite	33.1

II) Remark note:-

Geological Survey of Ethiopia Central Geological Laboratory

Described By / Analysts Girma Asemu

Checked by Workneh G/Kirstos Mineralogy & Physical Analysis Case Team Co-ordinator

Date Completed 31/03/2014

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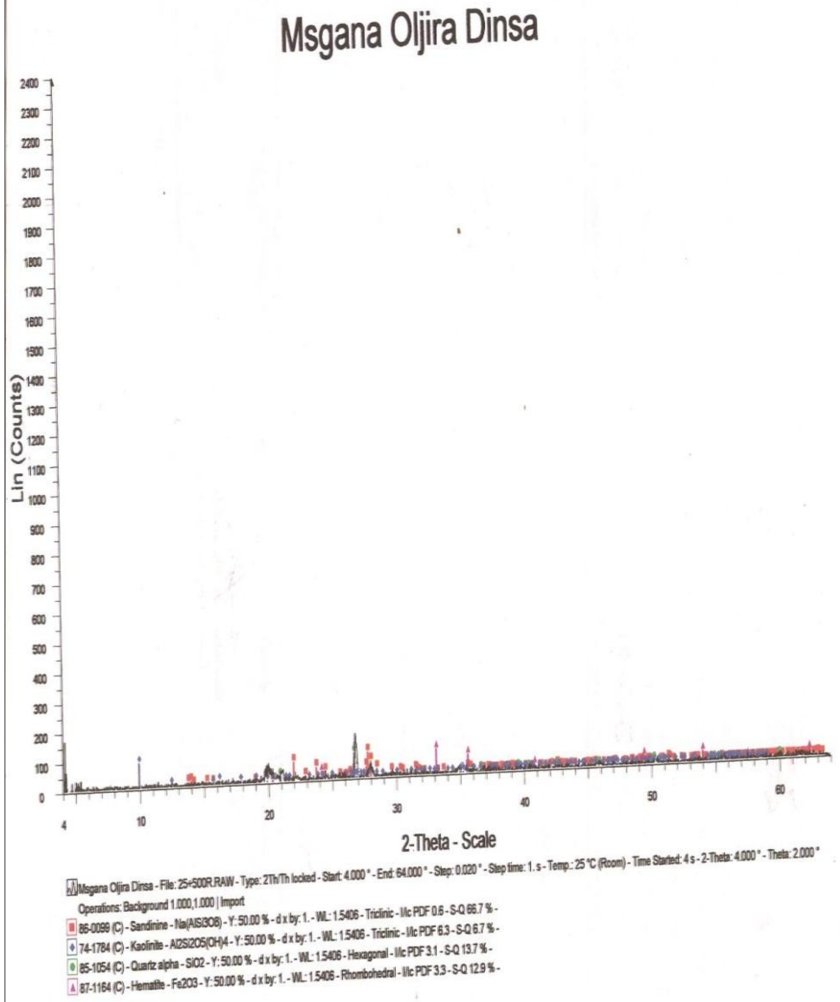

Annex-4-

Comparison of AASHTO and USCS soil classification system of present study

No.	Station	Atterberg limit			CBR at 95 %MDD	Classification (AASHTO)	USCS based on LL and PI of Casagrande's chart
		LL	PL	PI			
1	00+000	**63**	34	29	2	A-7-5 (30)	MH or OH
2	00+500	**60**	32	**28**	2	A-7-5 (30)	MH or OH
3	01+000	**73**	38	**35**	1	A-7-5 (27)	MH or OH
4	01+500	**61**	36	**25**	2	A-7-5 (32)	MH or OH
5	02+000	**71**	48	**23**	2	A-7-5(29)	MH or OH
6	02+500	**82**	42	**40**	2	A-7-5(42)	MH or OH
7	03+000	**73**	36	**37**	2	A-7-5(40)	MH or OH
8	03+500	**69**	41	**28**	1.9	A-7-5(31)	MH or OH
9	04+000	**74**	32	**42**	2	A-7-5 (44)	CH or OH
10	04+500	**69**	37	**32**	1	A-7-5 (34)	MH or OH
11	05+000	**72**	43	**29**	1	A-7-5(31)	MH or OH
12	05+500	**57**	35	**22**	2	A-7-5(20)	MH or OH
13	06+000	**70**	35	**35**	2	A-7-5(39)	MH or OH
14	06+500	**49**	33	**16**	10	A-7-5(15)	ML or OL
15	07+000	**78**	36	**42**	2	A-7-5(36)	MH or OH
16	07+500	**54**	20	**34**	6	A-7-5 (24)	CH or OH
17	08+000	**54**	28	**26**	2	A-7-6(18)	CH or OH
18	08+500	**65**	34	**31**	1	A-7-5 (27)	MH or OH
19	09+000	**50**	30	**20**	9	A-7-6(22)	ML or OL
20	09+500	**74**	46	**28**	6	A-7-5 (35)	MH or OH
21	10+000	**70**	41	**29**	2	A-7-5(31)	MH or OH
22	10+500	**68**	38	**30**	2	A-7-5(30)	MH or OH
23	11+000	**67**	35	**32**	2	A-7-5 (30)	MH or OH
24	11+500	**66**	34	**32**	3	A-7-5(35)	MH or OH
26	12+000	**44**	29	**15**	8	A-2-7 (1)	ML or OL
27	12+500	**68**	40	**28**	2	A-7-5 (22)	MH or OH
28	13+000	**57**	36	**21**	2	A-7-5 (15)	MH or OH
29	13+500	**43**	26	**17**	7	A-2-7 (0)	CL or OL
30	14+000	**54**	35	**19**	3	A-7-5 (15)	MH or OH
31	14+500	**52**	33	**19**	4	A-7-2(15)	MH or OH
33	15+500	**75**	45	**30**	3	A-7-6 (16)	MH or OH
34	15+560	**56**	37	**19**	2	A-7-5 (33)	MH or OH
35	16+000	**59**	34	**25**	2	A-7-6(25)	MH or OH
36	16+500	**54**	30	**24**	12	A-2-7 (0)	MH or OH
37	16+500	**55**	37	**18**	15	A-7-5 (4)	MH or OH
38	17+000	**56**	27	**29**	14	A-7-5 (18)	CH or OH
39	17+100	**44**	25	**19**	1	A-7-5 (27)	CL or OL
40	17+100	**58**	29	**29**	8	A-7-6 (13)	CH or OH
41	17+500	**49**	31	**18**	2	A-7-6 (13)	ML or OL
42	18+000	**44**	26	**18**	7	A-7-5(13)	CL or OL
43	18+500	**56**	31	**25**	2	A-7-5 (22)	MH or OH

44	19+000	**49**	29	**20**	16	A-7-6 (0)	ML or OL
45	19+500	**53**	32	**21**	17	A-2-7 (0)	MH or OH
46	19+600	**53**	32	**21**	17	A-2-7 (0)	MH or OH
47	20+000	**66**	35	**31**	2	A-7-5 (34)	MH or OH
48	20+500	**49**	29	**20**	24	A-7-6(20)	ML or OL
49	21+000	**68**	39	**29**	2	A-7-5 (31)	MH or OH
50	21+500	**44**	29	**15**	11.6	A-7-5 (5)	ML or OL
51	21+500	**51**	33	**18**	5	A-7-6 (7)	MH or OH
52	22+000	**44**	31	**13**	9	A-7-5 (18)	ML or OL
53	22+000	**68**	43	**25**	4	A-7-5 (1)	MH or OH
55	23+000	**71**	36	**35**	2	A-7-5 (17)	MH or OH
56	23+500	**49**	28	**21**	2	A-7-6 (17)	ML or OL
57	24+500	**49**	29	**20**	3	A-7-6 (17)	ML or OL
58	25+000	**50**	29	**21**	5	A-2-6 (14)	ML or OL
59	25+500	**35**	25	**10**	7	A-7-6 (8)	CL or OL
60	25+500	**46**	29	**17**	11	A-2-6 (0)	ML or OL
61	26+000	**31**	18	**13**	4	A-6 (0)	CL or OL
62	26+000	**40**	24	**16**	10	A-7-5 (4)	CL or OL
63	27+000	**56**	33	**23**	2	A-7-5 (23)	MH or OH
64	27+500	**56**	38	**18**	5	A-7-5 (15)	MH or OH
65	28+000	**45**	25	**20**	6	A-7-6 (16)	CL or OL
66	28+500	**45**	29	**16**	9	A-7-6 (11)	ML or OL
67	29+000	**82**	44	**38**	1	A-7-5 (40)	MH or OH
68	29+500	**44**	26	**18**	14	A-2-7 (0)	CL or OL
69	30+000	**47**	29	**18**	7	A-7-6 (11)	ML or OL
70	30+500	**74**	55	**19**	2	A-7-5 (27)	MH or OH
71	31+000	**49**	33	**16**	9	A-7-5 (16)	ML or OL
72	31+500	**68**	34	**34**	2	A-7-5 (16)	MH or OH
73	32+000	**60**	36	**24**	5	A-7-5 (17)	MH or OH
74	32+500	**69**	43	**26**	1	A-7-5 (23)	MH or OH
75	33+000	**44**	30	**14**	9	A-7-5 (7)	ML or OL
76	33+500	**68**	36	**32**	2	A-7-5 (29)	MH or OH
77	34+000	**59**	40	**19**	1	A-7-5 (19)	MH or OH
78	34+500	**63**	47	**16**	2	A-7-5 (19)	MH or OH
79	35+000	**75**	44	**31**	2	A-7-5 (31)	MH or OH
80	35+500	**61**	46	**15**	11	A-7-5(15)	MH or OH
81	36+000	**52**	26	**26**	4	A-7-6(22)	MH or OH
82	36+200	**47**	26	**21**	4	A-2-7 (0)	CL or OL

Annex-5

Analysis Unit Delineation By cumulative Differences (Using AASHTO 1993, Appendix J)

No.	Station	depth (cm)	CBR	N	(Δxi)	Σxi	Air	(ai)	(Σai)	(Zx)
1	0+000	20-60	**2**	1	0.5	0	1	0.5	0.5	0.5
2	0+500	20-60	**2**	2	0.5	0.5	2	1	1.5	0
3	1+000	20-60	**1**	3	0.5	1	1.5	0.75	2.25	-0.8
4	1+500	20-60	**2**	4	0.5	1.5	1.5	0.75	3	-1.6
5	2+000	20-60	**2**	5	0.5	2	2	1	4	-2.2
6	2+500	20-60	**2**	6	0.5	2.5	2	1	5	-2.7
7	3+000	20-60	**2**	7	0.5	3	2	1	6	-3.2
8	3+500	20-60	**1.9**	8	0.5	3.5	1.95	0.975	6.97	-3.8
9	4+000	20-60	**2**	9	0.5	4	1.95	0.975	7.95	-4.4
10	4+500	20-60	**1**	10	0.5	4.5	1.5	0.75	8.7	-5.2
11	5+000	20-60	**1**	11	0.5	5	1	0.5	9.2	-6.2
12	5+500	20-60	**2**	12	0.5	5.5	1.5	0.75	9.95	-7
13	6+000	20-60	**2**	13	0.5	6	2	1	10.95	-7.5
14	6+500	20-60	**10**	14	0.5	6.5	6	3	13.95	-6.1
15	7+000	20-60	**2**	15	0.5	7	6	3	16.95	-4.6
16	7+500	20-60	**6**	16	0.5	7.5	4	2	18.95	-4.2
17	8+000	20-60	**2**	17	0.5	8	4	2	20.95	-3.7
18	8+500	20-60	**1**	18	0.5	8.5	1.5	0.75	21.7	-4.5
19	9+000	20-60	**9**	19	0.5	9	5	2.5	24.2	-3.5
20	9+500	20-60	**2**	20	0.5	9.5	5.5	2.75	26.95	-2.3
21	9+740	20-6	**2**	21	0.5	9.6	2	1	27.95	-1.6
22	10+000	20-60	**2**	22	0.5	10	2	1	28.95	-1.9
23	10+500	20-60	**2**	23	0.5	10.5	2	1	29.95	-2.4
24	11+000	20-60	**2**	24	0.5	11	2	1	30.95	-2.9
25	11+500	20-60	**3**	25	0.5	11.5	2.5	1.25	32.2	-3.2
26	12+000	20-60	8	26	0.5	12	5.5	2.75	34.95	-2
27	12+500	20-60	**2**	27	0.5	12.5	5	2.5	37.45	-1.1
28	13+000	20-60	**2**	28	0.5	13	2	1	38.45	-1.6
29	13+500	20-60	**7**	29	0.5	13.5	4.5	2.25	40.7	-0.9
30	14+000	20-60	**3**	30	0.5	14	5	2.5	43.2	0.1
31	14+500	20-100	**4**	31	0.5	14.5	3.5	1.75	44.95	0.3
32	15+560	20-60	**3**	33	0.38	15.56	2.5	0.95	47.94	0
33	15+500	20-60	**2**	33	0	0	2.5	0	0	0
34	16+000	20-60	**2**	34	0.5	0.5	2	1	1	-1.8
35	16+500	20-60	**12**	35	0.5	1	7	3.5	4.5	-1.2
36	16+500	60-100	**15**	35	0	1.5	13.5	0	4.5	-4
37	17+000	20-60	**11**	36	0.5	2	13	6.5	11	-0.4
38	17+100	20-60	**1**	37	0.1	2.5	6	0.6	11.6	-2.6
39	17+500	60-100	**2**	38	0.4	0.5	5	2	13.6	10.8
40	18+000	20-60	**7**	39	0.5	1.5	4.5	2.25	15.85	7.3
41	18+500	20-60	**2**	40	0.5	2	4.5	2.25	18.1	6.7
42	19+000	20-60	**16**	41	0.5	2.5	9	4.5	22.6	8.4
43	19+500	20-60	**17**	42	0.5	3	16.5	8.25	30.85	13.8
44	19+600	20-60	**17**	43	0.1	3.1	17	1.7	32.55	15
45	20+000	20-60	**2**	44	0.4	3.5	9.5	3.8	36.35	16.5
46	20+500	20-60	**24**	45	0.5	4	13	6.5	42.85	20.1
47	21+000	20-60	**2**	46	0.5	4.5	13	6.5	49.35	23.8
48	21+500	20-60	**11.6**	47	0.5	5	6.8	3.4	52.75	24.4

49	21+500	60-100	**5**	47	0	5.5	8.3	0	52.75	21.5
50	22+000	20-60	**9**	48	0.5	6	7	3.5	56.25	22.2
51	22+000	20-100	**4**	48	0	6.5	6.5	0	56.25	19.3
52	22+500	20-60	**3**	49	0.5	7	3.5	1.75	58	18.3
53	23+000	20-60	**2**	50	0.5	7.5	2.5	1.25	59.25	16.7
54	23+500	20-60	**2**	51	0.5	8	2	1	60.25	14.8
55	23+500	20-100	**3**	51	0	8.5	2.5	0	60.25	12
56	24+000	20-60	**4**	52	0.5	9	3.5	1.75	62	10.9
57	24+500	20-60	**3**	53	0.5	9.5	3.5	1.75	63.75	9.8
58	25+000	20-60	**5**	54	0.5	10	4	2	65.75	9
59	25+500	20-60	**7**	55	0.5	10.5	6	3	68.75	9.1
60	25+500	60-100	**11**	55	0	11	9	0	68.75	6.3
61	26+000	20-60	**4**	56	0.5	11.5	7.5	3.75	72.5	7.2
62	26+000	60-100	**10**	56	0	12	7	0	72.5	4.4
63	27+000	20-60	**2**	57	1	12.5	6	6	78.5	7.5
64	27+500	20-60	**5**	58	0.5	13	3.5	1.75	80.25	6.4
65	28+000	20-60	**6**	59	0.5	13.5	5.5	2.75	83	6.4
66	28+500	20-60	**9**	60	0.5	14	7.5	3.75	86.75	7.3
67	29+000	20-60	**1**	61	0.5	14.5	5	2.5	89.25	6.9
68	29+500	20-60	**14**	62	0.5	15	7.5	3.75	93	7.8
69	30+000	20-60	**7**	63	0.5	15.5	10.5	5.25	98.25	10.3
70	30+500	20-60	**2**	64	0.5	16	4.5	2.25	100.5	9.7
71	31+000	20-60	**9**	65	0.5	16.5	5.5	2.75	103.25	9.6
72	31+500	20-60	**2**	66	0.5	17	5.5	2.75	106	9.5
73	32+000	20-60	**5**	67	0.5	17.5	3.5	1.75	107.75	8.4
74	32+500	20-60	**1**	68	0.5	18	3	1.5	109.25	7.1
75	33+000	20-60	**9**	69	0.5	18.5	5	2.5	111.75	6.7
76	33+500	20-60	**2**	70	0.5	19	5.5	2.75	114.5	6.6
77	34+000	20-60	**1**	71	0.5	19.5	1.5	0.75	115.25	4.5
78	34+500	20-60	**2**	72	0.5	20	1.5	0.75	116	2.5
79	35+000	20-60	**2**	73	0.5	20.5	2	1	117	0.6
80	35+500	20-60	**11**	74	0.5	21	6.5	3.25	120.25	1
81	36+000	20-60	**4**	75	0.5	21.5	7.5	3.75	124	1.9
82	36+200	20-60	**5**	76	0.2	22	1	0.5	0.5	0

***Abbreviation used in annex -5-**

CBR-Percentage response value

N- interval number

(Δxi)- interval distance

(Σxi)- cumulative interval

Air- Average interval response

(ai)- Actual interval area

(Σai)- Cumulative area

(Zx)- Cumulative difference