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Geotechnical Characterization of Sub grade Materials for
Pavement Construction,

A case Study on Aposto – Wondo – Negele Road Upgrading Project,

Contract 2: IrbaModa ~ Wadera Road Construction



SCHOOL OF GRADUATE STUDIES, ADDIS ABABA UNIVERSITY

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By

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◆ **List of abbreviations & definitions**

AASHTO, American Associations state Highways & Transportation Officials
ARE, Assistant Resident Engineer
ASTM, American Society of Testing and Materials
BKS, A french Engineering Consultant who conducted the design review works
CBR, California Bearing Ratio
DCP, Dynamic Cone Penetration
DRC, Design Review Consultant
ERA, Ethiopian Roads Authority
FHWA NHI, Federal Highways Administration, National Highways institute
GI, Group index
ITCZ, Inter Tropical Convergence Zone
Kpa, Kilo Pascal
LL, Liquid Limit
LS, Linear Shrinkage
MDD, Maximum Dry Density
NP, Non Plastic materials
ODA, Overseas Development Association
OMC, Optimum Moisture Content
PI, Plasticity Index
PL, Plastic Limit
RQD, Rock Quality Designations
RE, Resident Engineer
SPT, Standard Penetration Tests
TRL Transport Research Laboratory
UCS, Unconfined Compressive Strength
USCS, Unified Soils Classification System
UTM, Universal Transverse Mercator

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Abstract

Sub grade soils and materials characterizations are the main parameter for flexible pavement both in the design and construction phases. Expansive soils in construction sites do have significant influence on planning, structural design, construction and maintenance costs, performance and engineering life of roads and highways. They are susceptible to considerable volume changes in response to fluctuations in groundwater table and moisture content following seasonal climatic variations. This property can cause severe damages to infrastructure unless proper measures are taken in the design and construction phases. Identification of expansive soils and characterization of their anticipated behavior is thus important for pavement structure & drainage design, and estimating the cost, mitigating their undesirable properties. Hence to mitigate the possible damages, sub grade soils and materials characterization have been conducted.

The sub grade soils and materials characterization is conducted at Aposto ~ Wondo ~ Negele road upgrading project, contract 2: Irbamoda Wadera Road construction located in Adola Rede woreda, Gujji Administrative zone of Oromia Region, The geographic locations for the study area is bounded by UTM coordinates of 490698 E, 656488 N and 507644 E, 648577 N. The main objectives of the study were: (1) to classify the sub grade soils in to homogenous classes (2) to propose remedial measures for the unsuitable part of the sub grade soils and (3) to propose alternative pavement thickness design.

To achieve these objectives, a total of 112 sub grade soils samples have been taken at 200m regular interval and tests are also conducted at project geotechnical laboratory for the determination of Atterberg limits, grading, MDD and OMC, CBR and swell values. Interpretations have been made with the data obtained from field works and laboratory investigations, supported by previous studies and researches. In addition, geological data have also been incorporated. From the laboratory investigation, it has been concluded that at least 82% of the sub grade soils are found to be suitable for bearing stratum and construction materials. Mechanical stabilization methods have been proposed for the unsuitable section. After evaluating the pavement structure thickness conducted by the design review consultant, alternative pavement design approaches have been forwarded. Based on the results and findings, certain recommendations have also been given. These recommendations are removal and replacement, in situ treatment, rock fill with geo textiles and underground drains.

CHAPTER ONE: INTRODUCTION

1.1 Background of the study

Geotechnical soils and material characterization is important components for the design and construction of road projects. During characterization visual descriptions, sampling and testing of natural sub-grade soil and embankment material along the project route are carried out. 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. The data collected in the field, soil samples tested in the laboratory and results obtained determine the design of pavement structure as well as construction cost of the project. Hence, prior to construction, the sub grade soils of the road have to be sampled and investigated for the suitability of load bearing behavior. This behavior can be expressed as their response towards the application of load without significant failure of its plasticity nature and swelling potential. All these engineering properties of the sub grade soils affect the riding quality and serviceability of the road with in the design life (Atkines, 1983). Therefore, to get reliable test results for evaluation of those engineering properties and minimize errors and deviation with regard to the inherent variability of the sub grade soils, proper sampling and testing procedures are of paramount importance.

According to the Ethiopian Roads Authority (ERA) site investigation manual (2002), soil samples for sub grade material shall be taken at 0.5km interval for identification test and 1km interval for CBR tests during final design phase. 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. Sample spacing for this research is at 200m regular interval by adopting the verbal communication made with the Consultant Pavement/Materials Engineer. After excavation, delineation of the lateral extent is defined with the help of laboratory test results and visual field observation for homogeneity.

Expansive soils in construction sites have significant influence on planning, structural design, construction, maintenance costs, performance and engineering life, especially for shallow foundation structures. Expansive soils are susceptible to considerable volume changes with response to fluctuations in ground water table and moisture content following seasonal

climatic variations. This property can cause severe damages to infrastructure unless proper measures are taken in the design and construction phases (Chen, 1988).

Identification of expansive soils and characterization of their anticipated behavior is thus important for pavement structure, drainage design and estimating the cost of construction. In the present study, expansive soil engineering parameters such as; consistency; liquid limits (LL), plastic limits (PL), proctor density (MDD), optimum moisture Content (OMC), California bearing ratio (CBR) and swell values has been measured at site geotechnical laboratory.

Therefore, this research is aimed at characterizing unsuitable sub-grade soil and materials on a case study basis at Aposto ~ Wondo ~ Negelle Road Upgrading project, Contract 2: Irbamoda ~ Wadera road by taking samples at regular interval in the road segment from km 137+400 to 159+800. In addition, the study discusses the possible problems which are related to unsuitable sub-grade soils and recommendations by considering feasible remedial measures.

1.2 Origin of Expansive Soils

The most significant group of expansive clay minerals are the smectite of which montmorillonite is the most common occurring. These minerals result from the chemical weathering or hydrothermal alteration of basic and intermediate igneous and metamorphic rocks containing feldspar and ferromagnesian minerals. The stability of expansive minerals produced depends on environmental conditions, particularly the drainage, oxidation region and seasonality of the rainfall in the area (ODA, 1993).

According to ODA (1993), the types of clay minerals formed in the soils profiling during chemical weathering of rocks depends on a complex interaction of a number of controlling variables which includes;

- (i) The age of the land surface (i.e. for how long the soils have been forming)
- (ii) Climates during soils development (i.e. temperature, humidity, etc.)
- (iii) Mineralogy of the parent rocks
- (iv) Topography in the area of formation

In humid tropical regions, weathering of rock is more intense and extends to greater depths than in other parts of the world. Residual soils develop in place as a consequence of weathering, primarily chemical weathering, consequently climate (temperature and rainfall), parent rock, water movement (drainage and topography), age and vegetation cover are responsible for the development of the soil profile. Ferruginous and aluminous clay soils are frequent products of weathering in tropical latitudes. These are characterized by the presence of iron and aluminum oxides and hydroxides. These compounds, especially those of iron, are responsible for the red, brown and yellow colors of the soils. The soils may be fine grained, or they may contain nodules or concretions. Concretions occur in the matrix where there are higher concentrations of oxides in the Soil. More extensive accumulations of oxides give rise to laterite. Lateritic is a residual ferruginous clay-like deposit that generally occurs below a hardened ferruginous crust. During drier periods, the water table is lowered. The small amount of iron that has been mobilized in the ferrous state by the groundwater is then oxidized, forming hematite, or goethite if hydrated. The movement of the water table leads to the gradual accumulation of iron oxides at a given horizon in the soil profile. A cemented layer of laterite is formed that may be a continuous or honeycombed mass, or nodules may be formed, as in laterite gravel. Concretionary layers often are developed near the surface in lowland areas because of the high water table. Laterite hardens on exposure to air. Hardening may be due to a change in the hydration of iron and aluminum oxides. Laterite commonly contains all size fractions from clay to gravel and sometimes even larger material. Usually, at or near the surface, the liquid limit of laterite does not exceed 60% and the plasticity index is less than 30%. Consequently, laterite is of low to medium plasticity. Lateritic soils, particularly where they are mature, furnish a good bearing stratum. The hardened crust has a low compressibility and, therefore, settlement is likely to be negligible. In such instances, however, the strength of the soil may decrease with increasing depth. Red earths or latosols are residual ferruginous soils in which oxidation readily occur (Bell, 2007).

1.2.1 Formation of Expansive soils

Soils are formed by the processes of both chemical and physical weathering of rocks. Some minerals are more resistant to weathering than others. The resistant minerals are called the residual minerals.

As shown in Table 1.1, the most susceptible ones will be altered and changed to unstable ions. The process is called leaching. After leaching, the residual minerals may form clays and

it is called laterites soils, leaving soils rich in iron (Fe) and aluminum (Al) oxides (Steven, 2003).

Table. 1.1 Weathering of rocks & the residual Minerals

Rock	Primary minerals	Residual minerals	Leached Ions
Granite	Feldspars	clay minerals	Na ⁺ , K ⁺
	Mica	clay minerals	K ⁺
	Quartz	Quartz	-
	Fe-Mg minerals	clay minerals, Hematite, Goethite	Mg ⁺
Basalt	Feldspars	clay minerals	Na ⁺ , Ca ⁺²
	Fe-Mg minerals	clay minerals,	Mg ⁺²
	Magnetite	Hematite, Goethite	-
lime stone	Calcite	None	Ca ⁺ , CO ₃ ⁻²

The parent rocks and the equivalent metamorphic products are shown in Table 1.2 (Duggal, 2003).

Table 1.2 Parent rocks and their metamorphic products

S/No	Parent Rock	Metamorphic equivalent	Remarks
1	Granite, syenite	Gneiss	Foliated rock
2	Sandstone	Quartzite	Granulose structure
3	Lime stone	Marble, schist	Foliated rock
4	Marl	Marble	Granulose structure
5	Shale	Slate, schist, phyllite	Foliated rock
6	Mudstone	Slate	Foliated rock
7	Dolomite	Marble	Granulose structure
8	Dolerite, basalt	Schist	Foliated rock
9	Felsite, tuff	Schist, slate	Foliated rock
10	Conglomerate	Gneiss, schist	Foliated rock

1.3 Objectives of the Study

The present research is conducted based on literature review, field observations and testing of samples in laboratory for analysis, interpretations and evaluations. The research includes geological assessment, sub grade soils investigations, identifying the physical properties and classifying soils for engineering use. In pavement structure, the main variable in design of the pavement is the thickness (Atkins, 1983). The criteria involved in design of pavement thickness are; (I) the strength of the sub grade soils and (ii) the magnitude of the imposed loads. Accordingly, general and specific objectives have been framed as;

1.3.1 General Objective

The main objective of this research is to characterize the strength of sub-grade soils and materials for constructing the pavement structure.

1.3.2 Specific objectives include:

- To check the suitability and bearing capacity of the sub-grade soils and material
- To categorize sub-grade soils and materials in to appropriate traffic class
- To confirm the pavement working design with the actual site condition
- To find countermeasures for unsuitable parts of the sub-grade soils and material

1.4 Methodology of the research

In order to achieve the above mentioned objectives, the following methodology was adopted;

- Literature review to acquire background information of the regional and local geological soils characteristics, physiography and climatic conditions, that are used as the foundation for this research study.
- Field visits to overview the physiographic and geologic setup of the study area using GPS, aerial photos and topographic map in order to locate the exact position of sampling and to get some inputs for the description on the local geology and soils conditions.
- Investigation of soils at test pits along the road alignment at two hundred meters intervals
- Collection of representative soils samples at each test pits and photographs at representative locations
- Conducting soil tests such as; Atterberg limits and grain size analysis, modified proctor density (MDD), Optimum Moisture Content (OMC), California Bearing Ratio, (3 point

CBR), loaded swelling potential.. In addition, Petrographic examinations and quality tests for rocks samples have been carried out.

- Classification of the sub-grade soil material based on the laboratory test results using AASHTO M 145 (2004) Standard.
- Characterizing the sub-grade soil by comparing and relating different parameters of engineering properties of soil.
- Interpretation of the results of the tests and the classification in parallel to determine, the index properties, bearing capacity and swelling potential of the sub-grade soil along the road alignment.
- Characterization of the sub-grade soil alignment by integrating the results obtained from geotechnical field investigations and laboratory result analysis.
- By comparing different pavement design manuals, it has been attempted to provide the design considerations for minimizing the problem of expansive sub-grade materials.
- Data processing, presenting and preparation of maps are done with the help of ArcGIS-9.2 and MS Excel software.

1.5 Outcomes of the study

- ◆ Categorized the sub grade soils and materials in to homogenous soil strength classes
- ◆ Removal and replacement, and mechanical stabilization are proposed for the unsuitable portion of the sub grade materials
- ◆ An alternative pavement structure thickness is determined

1.6 Scope and Limitation of the Research

The scope of the research is to provide geotechnical characterizations of the sub-grade materials on a case study basis at Irbamoda ~ Wadera Road project, Contract 2. The research is conducted based on data collected from km 137+400 to 159+800, only. The data were collected at 200m interval along the route corridor. In the present research, there were limitations on availability of literatures and previous works regarding the geotechnical/engineering geological aspect.

Further, every effort was made to perform the present study in a scientific and logical manner under the constraints of frequent failure of CBR testing machine, Liquid limits apparatus, limited materials testing equipments and variability of the soils moisture contents. Some of

these limitations have affected the completion time for materials testing and reporting the results. 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. However, further studies and additional tests are required before implementing these results or finding to other engineering project, hence shall be considered indicative only.

1.7 Scheme of Presentations

Chapter 1 Introduces the general statement of the problem, origin, & properties expansive soils. The objectives, methodologies, scope and limitations of this study are also included in this chapter.

Chapter 2 Focus on reviews of previous works on the related topics in general and in the research areas in particular.

Chapter 3 Briefly discusses about the study Area, locations, topography and climate, land use and land cover, regional and local geology.

Chapter 4 Field investigation and laboratory testing of the sub grade soils.

Chapter 5 general characterizations of sub grade soils by the test results obtained from primary and secondary data, the sub grade strength classes determination & Evaluation with the classes of the Design Review consultant, the main differences and new pavement structure thickness will also be incorporated.

Chapter 6 highlights the Conclusions and recommendations which incorporates the researcher's ultimate findings and opinions to the study area in particular and road project in general. Possible measures to be taken for unsuitable sub-grade soils, swampy areas, sub grade on rock strata and sub surface water near the sub grade bottom are also emphasized.

CHAPTER TWO: LITERATURE REVIEW

2.1 Introduction

For the present study a detailed literature review was carried out to acquire the necessary knowledge regarding the research objectives. The literature review was mainly focused on the problems and possible solutions to the expansive nature of the sub-grade materials. The review has also given consideration on the delineation of the design sub grade material in to homogenous sections. Moreover, the literature review enabled to give a general description related to the specific project area such as; the local geology, vegetation, climate, soil and construction techniques, etc. Further, the present literature review also helped to understand, what methodology were adopted by the previous researchers and what the ultimate findings were. A systematic compilation of relevant literature review to this study is presented in the following paragraphs.

2.2 Literature Review

Design guide for flexible pavement AASHTO (1993)

Design guide for flexible pavement as per AASHTO (American Associations of State Highways and Transportation Officials) (1993) suggests determination of Homogenous sections using the CBR at 95% of the MDD (Maximum dry density) and analysis of Unit delineation by cumulative differences. In this method, Group index value and quality of sub-grade materials are correlated.

In AASHTO, (2000) standard, the following points are discussed in detail which refers to high way material characterizations and materials intended to be used as a sub-grade layer.

- Sample spacing for geotechnical site investigations be in the range from 150m to 450m interval during construction phase
- CBR values and swell potential of cohesive soils.
- Density /Moisture content clay soils.

The AASHTO (2004) soils classification includes seven basic groups (A-1 to A-7) and twelve subgroups. Of particular interest is the Group Index, which is used as a general guide to the load bearing ability of a soil. The group index is a function of the liquid limit, the plasticity index and the amount of material passing the 0.075mm sieve. Under average

conditions of good drainage and thorough compaction, the supporting value of a material may be assumed as an inverse ratio to its group index, i.e. a group index of '0' indicates a "good" sub-grade material and a group index of '20' or more indicates a poor sub-grade material.

Using AASHTO classification and test methods M145, Group index is calculated by equation 2.1.

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

Where:

F = the percentage passing sieve size 0.075mm (N0. 200), expressed as a whole number

LL = liquid Limit

PI = Plasticity index of the soil

Arora (1997) suggested the following points with regard to clay soils;

- The relationships between shear strength and liquid limits of clay soils and he explained that at the state of liquid limits, the shear strength of all soils are constant and is equal to 2.7 KN/ m². In addition, he developed an empirical relationship between plasticity index (PI) and linear shrinkage (LS) of clay soils as; $PI = 2.13 * LS$. Moreover, he described the main factors that have significant influences for soil compactions such as; type of soils, moisture content, and methods of compactions.
- Weathering of rocks and formation of lateritic and residual soils in tropical regions.
- Factors that govern the type of soils formations (Bell (2007) are;
 - Climates (temperature, rain fall etc.)
 - parent rocks
 - water movement
 - age and vegetation covers
- Liquid limits and plasticity index range of lateritic soils. He also mentioned regarding the lateritic soil's Atterberg limits, that these soils have liquid limits not exceeding 60% and Plasticity indices not more than 30% while characterizing the lateritic soils.

BKS Group in association with Beza consulting Engineer (2007) were the engineering design review consultant and discussed in detail regarding the geotechnical investigations of sub grade soils, sampling and laboratory testing in the present study area. BKS classified the sub-grade soils into different classes according to AASHTO M145. In addition, BKS determined

the bearing strength of the sub-grade soils. The sub-grade soils sampling was done at 1000m interval for classification, gradation, Atterberg limits and at 2km interval for one point and 3 point CBR and swell testing in the study area. BKS and its associates carried out the geotechnical works for three main objectives:

- (i) to determine the quality and properties of the sub-grade material to be incorporated in to the pavement design
- (ii) to confirm the investigation carried out by the previous design consultant (BCEOM, 2003)
- (iii) To indicate problematic sub-grade soil sections along the stretch and suggest possible remedial measures that would suite to the pavement design.

BKS and its associates carried out the geotechnical investigation by taking samples for the sub-grade soils in the study area and analyzed the results according to principles and practice of flexible pavement design guide as suggested by AASHTO (1993).

BKS determined the homogenous sections using the 95% of CBR with the help of unit delineation by cumulative differences (AASHTO, 1993; Appendix J).

Bowles (1997) explained about the types of clay soils, such as montmorillonite, illites, kaolinites and hayllosites for the range of their liquid limit (LL) and Plasticity index (PI) values. He further discussed concerning these clay minerals that montmorillonites have plasticity index of 150⁺, illites have PI values in the range from 30 to 50% and the kaolinites (Hayllosites) minerals have values of plasticity index in between 15 % and 20%.

Construction works of Aposto ~ Wondo ~ Negele (2008), Contract 2, Irbamoda ~ Wadera, it was mentioned in the contract that unsuitable materials have the following physical properties:

- (i) Peat materials from swamps, marshes and bogs that contain excessive amount of logs trees, stems and other perishable materials
- (ii) Clay materials having a liquid limit exceeding 60%, or a plasticity index exceeding 30 or CBR value less than 3% or swell value more than 2%.
- (iii) Any material which is sufficiently wet and soft to prevent them from being trafficked or excavated by normal bulk earth working plant.

Further, the document stated the possible quality of materials to be used for replacement must have the following quality parameters:

- (i) its CBR value shall be greater than 5% at 95% of AASHTO (T180)
- (ii) the plasticity index shall be not exceeding 20%
- (iii) the maximum swell value of 1.5%

Construction works of Woreta ~ Woldiya Road upgrading project, Contract 3, Gashena Woldiya, (2004), stated that excavation of unsuitable materials should consists of:

- (i) Peat materials from swamps, marshes and bogs that contain excessive amount of logs trees, stems and other perishable materials.
- (ii) Clay materials having a liquid limits exceeding 60%, or a plasticity index exceeding 30 or CBR value less than 3% or swell value more than 2%
- (iii) any material which is sufficiently wet and soft to prevent them from being trafficked or excavated by normal bulk earth working plant

Construction works of Zarima Adi Arkay shire roads, contract 2, Mytsebri shire road, (2009) it explained in the document that: materials unsuitable for use in the work may include:

- (i) peat materials and organic decomposition with more than 3% organic matter by weight
- (ii) Clay materials having a liquid limit exceeding 60%, or a plasticity index exceeding 30 or CBR value less than 5% or swell value more than 3%

The documents also include the quality of materials that is intended for the use of the replacement qualifies:

- (i) clay materials having a liquid limits not exceeding 60%, or a plasticity index not exceeding 30 or CBR value more than 5% or swell value less than 3%

Duggal S.K (2003) mentioned the transformation of parent rocks and its metamorphic equivalents. These rocks are formed from igneous or sedimentary or preexisting metamorphic rocks as a result of the actions of the earth movement, temperature changes liquid pressures etc. and the rock cycle in general.

Ethiopian Roads Authority Pavement Design manual (ERA, 2002), mentioned the following points regarding delineations of homogenous sections in the sub-grade areas. A road section for which a pavement design is undertaken should be sub-divided into Sub-grade areas where the sub-grade CBR can be reasonably expected to be uniform, i.e. without significant variations. Significant variations in this respect mean variations that would yield different sub-grade classes as defined herein further below. However, it is not practical to create delineation between sub-grade areas that would be too precise, and indeed this could be the source of confusion during construction. The soils investigations should delineate sub-grade design units on the basis of geology, pedology, drainage conditions and topography, and consider soil categories which have fairly consistent geotechnical characteristics (e.g. grading, plasticity, CBR). Usually, the number of soil categories and the number of uniform sub-grade areas will not exceed 4 or 5 for a given road project. Generally, it is advisable to avoid short design sections along the alignment. Where the sub-grade 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.

The manual (ERA, 2002) also includes strength categories of sub grade materials as shown in table 2.1.

Table 2.1 Sub grade strength categories by ERA

Sub grade strength Classes	Range (CBR %)
S1	2
S2	3 – 4
S3	5 – 7
S4	8 – 14
S5	15 – 29
S6	30+

(Source: ERA manual, 2002)

It is important to differentiate between localized poor (or good) soils and general sub-grade areas. Normally, localized poor soils will be removed and replaced with suitable materials.

Lateritic gravels can generally be assigned a sub-grade classification S5. It must be emphasized that too many variables influence the sub-grade strength for the above to be anything more than a general indication, and detailed investigations, as outlined in the ERA

Site Investigation Manual (2002) is required for final design.

ERA Site Investigation Manual (2002) also gives emphasis to design and construction considerations specific to expansive soils in the following paragraphs, to the extent that they may influence the scope of the investigations undertaken in the field and the laboratory. 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 increased maintenance costs. The advices to the design engineer are also included in the following four main approaches to mitigate or overcome the problem of expansive clays:

- Avoid expansive clays areas by realignment.
- Excavate the expansive clays and replace them with suitable material of S3 type and better quality.
- Treat the expansive clays with chemicals such as lime and other chemicals.
- Minimize moisture changes and potential swelling in sub-grade layer of the expansive clays.

In addition, some considerations are mentioned further to, embankment side slopes and drainage structures.

- (i) After the expansive sub-grade material has been replaced with non-expansive material with maximum 200mm lifts thickness to 95% modified AASHTO density, bring the road to finished level in approved materials, with a side slope of 1:2, and ensure that pavement criteria are complied with; the previously stockpiled expansive soil excavated as directed under (i) should then be spread over the slope.
- (ii) Do not construct side drains unless they are absolutely essential to stop ponding; where side drains are necessary, they should be as shallow as possible and located as far from the toe of the fill (embankment) as possible.
- (iii) Ideally, construction over expansive soil should be done when the in-situ moisture content is at its highest, i.e. at the end of rainy season.

In addition to this, the site investigations section, the supplementary section of ERA manual (2002), has also incorporated stability of materials on cut sections. According to the manual, the maximum possible stable angles for different materials in cut sections are summarized as;

Cohesionless sands	2:1
Silty sand and silt	1:1
Eluvial soils (red friable Clay)	1:1.5 for cut section less than 4m height 1:1 for cut sections more than 4m height
Weathered rocks	1:2 to 1:4
Sound Rocks	1:5 to 1:10

Ministry of Mines and energy, Geologic map for Adola Gold exploration project, (1988) discussed about the mapable lithologic units, structures like fault lines, the dip and strike of the beds and foliation planes found in the area. In this map, the thickness of each formation with their scale of geologic time, geologic time gaps (unconformities), inferred and defined geologic boundaries are also described in detail at 1:100,000 scale.

Atkins (1983) disclosed the main variables in the design of pavement structure which is its thickness; the criteria being the strength of the sub-grade soils and magnitudes of the imposed loads. He also discussed regarding highway materials and soils in brief. In his statement, he rated silts and clays poor materials only under the following conditions:

- i) When they occur in low lying areas where the natural drainage is very poor and will not be improved.
- ii) Where the condition of water table and climate are such that severe frost heave can be expected.
- iii) Where high percentage of mica-like fragments or diatomaceous particles produce a highly elastic condition.
- iv) Where it is desired to “bury” highly expansive soils usually A-7-6 (CH) deeper in the section to limit the effect of seasonal variations in moisture.

He further mentioned regarding the main functions of pavement structures, the load imposed by vehicles and the distributions of loads down the sub-grade layer that are low enough to be carried without failure due to rutting, excessive settlement, or other types of stresses. The various methods for measuring the imposed loads sub-grade strength values, the required pavement structures, have been suggested and used. In addition, Atkins (1983) gave emphasis on the main components of a pavement structures, such as:

- (i) Surface
- (ii) Base

- (iii) Sub base, collectively known as the pavement
- (iv) Compacted sub grade
- (v) Natural sub grade, collectively called the sub grade.

Finally Atkins (1983) suggested that additional thickness of asphalt pavement, above the minimum required level can be placed by base and sub-base materials using the following equivalencies:

$$\begin{aligned} 2 \text{ inch (mm) base} &= 1 \text{ inch (mm) asphalt} \\ 2.7 \text{ inch (mm) sub base} &= 1 \text{ inch (mm) asphalt} \end{aligned}$$

Chen (1988) discussed in detail concerning the world wide coverage of the problems of expansive soils. He also added that this problem is one of the six major hazards in the world. In Chen (1988) view, the major hazards are Earthquakes, landslides, volcanic reuption, expansive soils, hurricane, tornado, and flood.

Chen (1988) defines, swelling soils, which are clayey soils, are also called expansive soils. When these soils are partially saturated, they increase in volume with the addition of water. They shrink greatly on drying and develop cracks on the surface. These soils possess a high plasticity index. The clay mineral that is mostly responsible for the expansiveness belongs to the montmorillonite groups. The soils containing a considerable amount of monmorillonite minerals exhibit high swelling and shrinkage characteristics. Chen (1988) added that expansive soils are residual soils which are the result of weathering of parent rocks. The depth of these soils in some region may be up to 6m or more. Swelling and expansion pressures of clayey soils also increase with an increase in dry density, up to the volumetric shrinkage limits. However, beyond this shrinkage limit, the swelling pressure becomes constant while the dry densities further increases.

Komornik et al., (1969) explained about the effects of initial moisture content and dry density on swelling pressure. Swelling pressure increases with the increases of the dry density. Komornik et al., (1969) proposed the prediction method of swelling potential of soils with empirical equation 2.2;

$$\text{Log } P_s = 2.132 + 0.0208wl + 0.00065DD - 0.0269Wn \quad \dots \text{eq.2.2}$$

Where,

Ps	=	swelling pressure in kg/cm ²
WL	=	liquid Limit, (%)
Wn	=	natural moisture content (%)
DD	=	dry Density of soils kg/cm ³

Dry density and swelling pressure proportionally increases but inversely with natural moisture contents as can be observed from empirical equation 2.2.

United Republic of Tanzania, Pavement and Materials Design manual (1999), describes about problematic soils. The manuals present descriptions of the problem related with expansive soils investigation procedure.

- Routine investigations carried out during surveys of the projects includes:
 - (i) Field reconnaissance
 - (ii) Atterberg limits and grading tests
 - (iii) Analysis and evaluation of those test data
- Extended investigations including simple additional indicator testing in the laboratory when expansive soils are suspected.
- In-depth studies including specialized laboratory testing and is employed where the extended investigations have shown occurrence of expansive soils, and the required in-Depth studies to quantify swell potential and expansiveness of these soils.

The manual also recommended remedial measures commonly employed to minimize the damages on pavements by expansive soils.

Low strength soils with soaked CBR values less than 3% (< 2% in dry climatic Zones) occurring within the design depth is described as low strength soils. The foundations of the pavement within these soils require special treatment that may include one or more of the following measurements:

- Mechanical stabilization
- Removal and replacement of soils

- Rising of the vertical alignment to increase soil cover and thereby redefine the design depth below the pavement structure. The sub-grade soils strength classes are also included in the manual as shown in the Table 2.2.

Table, 2.2 Sub grade bearing strength classes by Tanzanian Design manual

Sub grade 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	Minimum 15	Minimum 7	95 BS heavy
S7	7-14	7-14	3-14	93 BS heavy
S3	3-6	3-6	2-6	100 light

(Source: Tanzania Design Manual, 1999)

Road Design Manual of Ministry of Transport and Communication Roads Department, Republic of Kenya, Part III (1987) has discussed regarding problems associated with expansive clay.

These problems are;

- Volume change due to moisture variation: black cotton soil shrink and crack when they dry out and swell when they get excess moisture. The cracks allow water to penetrate deep in to the soil, hence causing considerable expansion. This results in deformation of the road surface, since the expansion and the subsequent heave are never uniform. Furthermore, these volume changes may produce lateral displacements (“creep”) of the expansive clay, if the side slopes are not gentle enough.
- Bearing capacity reduction: when the moisture content increases, expansion occurs and the bearing capacity of the black cotton soil decreases and so is the CBR if the soil becomes completely saturated.
- Susceptibility to erosion: when dry, black cotton soils present sand like texture (polyhedral segments formed by the agglomeration of clay, silt and sometimes sand particles). In this state, they are prone to erosion to a greater extent than their plasticity and clay content normally anticipated.

This manual has also defined the solutions in the same way as the Tanzanian manual for the above problems. Furthermore, the manual incorporated recommended design and construction procedure.

Table 2.3 Sub grade bearing strength classes

Sub-grade strength class	CBR Range	Median
S1	2 - 5	3.5
S2	5 - 10	7.5
S3	7 - 13	10
S4	10 - 18	14
S5	15 - 30	22.5
S6	>30	

(Sources: Kenyan Road Design Manual, 1987)

Shehedi-Gelego-Guba and Gelego - Theodros Ketema, consultancy service for detailed engineering Design and contract Document preparation (Saba Engineering, 2004) mentioned that:

- Problems associated with unsuitable (expansive) soils are;
 - a) Volume change upon wetting and drying
 - b) Low bearing capacity
 - c) Susceptibility to erosion
- Remedial measures for these unsuitable soils proposed in this Report are :
 - i) Realignment
 - ii) Lime treatment
 - iii) Removal and replacement
- Recommendations given regarding construction on expansive soils are;
 - (i) The road bed on unsuitable soils shall be placed below 600 mm and replacement with better quality materials
 - (ii) For fill sections, the embankment slopes should be flattened to a suitable slope depending on the height of the embankment
 - (iii) Side drains in the unsuitable sections should be avoided or should be shallow and be located far from the toe of the embankment

- (iv) Expansion pressure and potential volume change increase with dry density of the swelling soils, so their density should not exceed 97-98% of MDD
- (v) The road bed of the expansive section should be moist during the road bed construction (preparation) and should be covered with appropriate improved materials without undue delay.

The report further mentioned the delineation of homogenous sections using the CBR values of the sub-grade materials. For design of pavement structure and sub-grade strength category, the road sub-grade is divided in to homogenous sections solely based on CBR. The division of the road in to homogenous sections has been carried out based on the method of cumulative differences as given in AASHTO (1993) Pavement Design Guide (Appendix J). The homogenous sections are obtained by locating the main slope changes in the graph so as to determine the limits of fairly homogenous sections along the road alignment.

TRL (1993) Overseas Road Note 31 mentioned that the strength of road sub-grade materials is assessed in terms of the California Bearing Ratio (CBR) and this is dependent on the type of soil, its density and its moisture content

- a. Soils types
- b. Dry density of the soils
- c. Moisture content of the sub grade materials (soils).

According to TRL (1993), there are three categories of sub-grade conditions under impermeable road pavement,

- Sub-grade where the water table is sufficiently close to ground surface to control the sub-grade moisture content.
- Sub-grade where the water table is deep below the bottom of the sub-grade layer and where rainfall is sufficient to produce significant changes in moisture conditions under the road pavement.
- Sub-grades in areas with no permanent water table near the ground surface and where the climate is dry throughout the year with annual rainfall of 250 mm or less.

Zaruba, (1976) mentioned that during the preliminary engineering geological investigation, the grade of weathering must be determined on the basis of simple macroscopic criteria and Divided in to four different grades;

- (i) zone of complete weathering

- (ii) zone of intensive weathering
- (iii) zone of slight weathering
- (iv) zone of partial weathering

In complete weathering zones, the physical and chemical weathering has advanced so that the rock is completely disintegrated and decomposed. The weathered rock consists of secondary minerals and finely disintegrated resistant rocks.

In intensive weathering zones, the rocks are disintegrated in to loose small fragments and single minerals. Interstices between fragments are commonly filled with loamy materials. Chemical weathering is so strong that all less resistant minerals are decomposed but the original macrostructures of the rocks may be preserved.

In the zone of slight weathering, the rocks are mechanically weathered in to large fragments and blocks in primary positions. The space between fragments and blocks are formed by widening of primary joint or tectonic fractures and are filled with loam or minute rock fragments. Chemical weathering is limited to the surface layers of fragments and blocks.

In the zone of partial weathering, physical weathering is manifested by the widening of primary and tectonic fractures and by loosening of coherent along planes of weakness, which facilitates disintegration of rocks along them. Chemical weathering is effective along the walls of fractures. Steep wide fractures may be filled with loam or small rock fragments.

Seed et al., (1962) proposed an empirical relationship between swelling potentials and plasticity index values of clay soils. Swelling potential increases with the plasticity index of those clay soils. The swelling potential of expansive soils can be predicted by using the empirical equation 2.3:

$$\text{For natural soils, } SP = 60K (PI)^{2.44} \quad \dots\dots\dots eq.2.3$$

Where:

SP = swelling potential

PI = plasticity index

K = $3.6 * 10^{(-5)}$ a factor for clay content between 8 & 65%

Steven (2003) briefly explained about the processes of chemical weathering rocks and formations of clay soils in tropical regions. He also detailed the primary minerals, residual

minerals and ultimately the leached ions in chemical weathering processes. In Steven's view, after leaching the residual minerals may become clay depending the stability of the primary minerals in the parent rocks.

US Department of Transport (2006) FHWA stated regarding Grain size distributions, group index methods of soils classifications, swelling and shrinkage factors, dry density and moisture content, the elastic deformation and Mica minerals in the sub-grade soils. Shear strength and liquid limits relationships. It is also mentioned the importance of soils compaction on engineering properties such as strength, permeability void ratio, stiffness, compressibility and settlement on foundations.

Drainage conditions of working sites. The drainage quality of an area and its rating with time is mentioned in Table 2.4.

Table 2.4 Drainage quality of an area and its rating with time

S/N	Quality of drainage	Water removal time
1	Excellent	2 hrs
2	Good	1 day
3	fair	1 week
4	poor	one month
5	very poor	water will not drain

US Department of Interior Geological Survey (USGS, 1999) mentioned regarding the weathering of Rocks and formations of clay minerals. Factors governing rock weathering and soils formations include the initial type of rocks, the rate of water to rock, the temperature, etc. The types of minerals found in weathering rocks strongly control how the weathered rock behaves under various climatic conditions. Some clay minerals swell when they take up water and swell and shrinkage are reversible process.

Moreover, for the investigation and analysis of soils of the present study area, references are also made to the researcher's personal experiences acquired through his participation in road construction such as the Mekelle city Roads upgrading construction, The Jijiga ~ Togochole asphalt Road Construction, Woreta ~ Wodiya Road upgrading project, Gashena ~ Woldiya (contract 3) Road construction and Aposto ~ Wondo ~ Negele Road upgrading project

,Irbamoda ~ Wadera (Contract 2) road construction and currently Zarima Adi Arkay Shire Road upgrading project, Contract 2: Mytsebri Shire asphalt Road construction. While collecting the data for the present research, the researcher was also an employee to the Turkish contractor, Ahmet Aydeniz KMC Joint venture, at Irbamoda ~ Wadera Road project, Contract 2, at the capacity of Pavement / Materials Engineer.

Thus, for the present study, the various literatures have been utilized to provide a basis for comparing the physical properties of the soils of the study area with those of other expansive soils. Such comparisons has aided in the evaluation of the soils of the study area and the remedial measures that have been taken.

CHAPTER THREE: THE STUDY AREA

3.1 Introduction

The Ethiopian Roads Authority, the client, and Ahmet Aydeniz- KMC joint venture, the contractor, signed a contract agreement for the construction of IrbaModa ~ Wadera Road upgrading project on 30th of October, 2008 for 1095 days contract completion periods. In the mean time, the Consulting Engineer assigned by ERA to supervise the construction work for this upgrading project was Grontmij/Carl bro in Association with Gondwana Engineering, PLC. The Aposto ~ Wondo ~ Negelle Road Upgrading project, Contract 2: IrbaModa ~wadera, Road segment is under construction since 01 April 2009 in an asphalt concrete standard and the second contract of the road has a 7m carriageway width and a 1.5m shoulder on both sides. In urban and Town sections, the road width is designed to have a carriage way of 7m, 7m parking lane and 5m shoulder width. There is no major structure on the route corridor but 128 minor structures, of which 18 new structures, 91 to be replaced and the rest 19 to be rehabilitated and extended. The pavement has a thickness of 50mm asphalt concrete surfacing (Black Top), 175mm base course, 260mm sub base and capping layer varying in thickness from places to place. In general, the total length of the road project is 108.46km from IrbaModa to Wadera and the research area covers only 22.40km length, i.e. from km 137+400 to km 159+800.

3.2 Location and accessibility of the study area

The relative location of the study area is found in Adolla Rede woreda, Gujji administrative zone of Oromia Region, and an integral part of the Aposto - Wendo -Negelle road upgrading project that links the Town of Aposto and Negele via KibreMengist. Hence, the road project lies on the Hawass Kibre Mengist Negelle Borena main road and starts at IrbaModa Town, roughly 414km from Addis Ababa. On the other hand, the town of Kibre Mengist is found at 56km from IrbaModa where the road branches off to Negelle Borena and Legedembi Gold exploration center and the Town of Wadera is located at 202.56km from Apost on which the contract 2 ends. The main towns along the traverse from the beginning to the end of the road project are Irbamoda, Buanbuawuha, Meleka, Anferara, Kibre mengist, Zenbaba and Wadera of which only the two towns, Anferara and Kibre mengist are found with the limits of the study area. In general, the road project follows a south easterly direction as it progresses to Negelle Borena (Fig. 3.1). The location of the road project forms the principal artery for the development of Sidamo, Borena and Dollo areas in the Oromia, and Somali Regions in south

eastern part of Ethiopia, this is accessible through all seasons of the year. The geographic locations for the study area in particular is bounded by UTM coordinates, 490698 E, 656488 N at the starting point and 507644 E, 648577 N at the end point of the study area within the geologic map index No NB 37 – 6, 37 – 7, 37 – 10 and 37 – 11, in the geologic map of Adola Gold exploration project at the scale of 1:100,000.

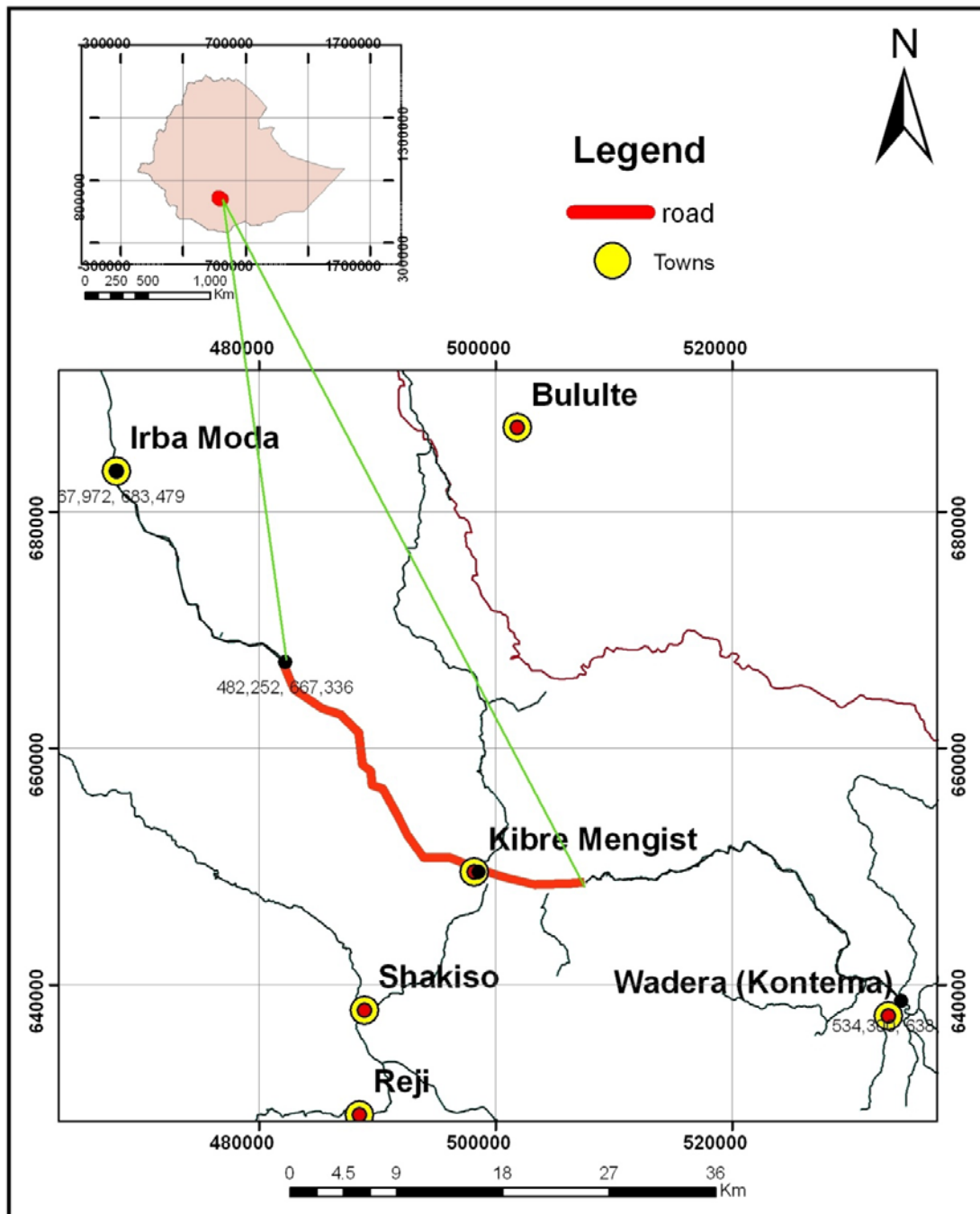


Fig. 3.1 Location map of the study area

Table 3.1 Villages / towns along the road traverse

S/N o	Village and/or Towns name	Chainage (km stations)		Coordinates (UTM)	Remarks
		from	to		
1	Starting point of the study area	137+400		490698E,656488N,1811 m	Anferara hill
2	Anferara Town	137+820	140+220	1730m	Normal Town section
3	Kibre Mengist / Adola	144+160	151+180	498720E,649474N,1689 m	Normal Town section
4	End point of the study area	159+800		507644E,648577N,1850 m	Scattered forest and bare land

ASL = above sea level

3.3 Land use and land cover

During the field observations and investigations, it has been confirmed that in general the watershed that drains across the road from km 137+400 to km 159+800 is covered with scattered artificial and natural forests. The density of natural forest is comparatively less along the route corridor in the stations from 140+000 to 144+000 and from 152+000 to 159+800. From the land use patterns (Plate 3.1), it is clear that the areas are used alternatively for residential, cultivation, and grazing lands. Coffee is the main cash crop along the route corridor of the road project. Corn is also one of the most widely cultivable crops in the woreda.



Plate 3.1 Partial view of the land use patterns (km 156 + 900), Photo September 2010

3.4 Soils of the study area

From the visual inspection of the sub-grade soil during the site visit, in most parts of the study area, it was found that it is reddish brown to dark brown, yellowish, dark gray, silty clay type and in few stretches of the road alignment with reddish brown silty clay/clayey silt soil mixed with weathered lateritic gravel (Plate 3.2) where these are of residual nature. The soil extension survey was carried out in such a way that different soil types along the alignment can be recorded and classified according to color, texture and composition. In rolling and flat terrains such as; in the chainage, km 143+500 to 144+500 and 146+200 to 147+600, the surface drainage is very poor since the clay soil permeability is very low.

In general, soil formation along route corridor of the project mainly consists of the residual soils. The residual soils have resulted from in-situ weathering of parent rocks that are not subjected to transportation and are still in the place of their origin.



Plate 3.2 Main soil types in the study area (km123 + 600), photo August 2010

They often grade, most of the cases, into decomposed or completely weathered rocks down to a significant depth. The residual soils are commonly lateritic in nature and usually described as mostly dark brown, reddish, pinkish, greenish yellow and white colored and in some localities grey to dark clayey silts or silty clays.

However, clear distinction between the individual soil types could not be made due to their similarity in origin and soil properties, and gradual transitions from one soil type to the other, which in general, is common in relation to the topography of that area.

3.5 Climatic Condition of the Study Area

3.5.1 Rainfall

The seasonality of the rainfall in the project area is generally governed by the migration of the Inter Tropical Convergence Zone (ITCZ). The forward and the backward migration of the ITCZ produce substantial rainfall bringing moisture largely from the south Atlantic and Indian Oceans. The project area is located in bimodal rainfall region with annual rainfall varying from 700 mm at Negele to above 1200 mm at Yirga Alem. Figure 3.2 shows the monthly distribution of rainfall along the Aposto~Wondo~Negele road. It is seen that rainfall more than 100 mm is experienced in March and April all along the road. March to April is the main rainy season along the road from Kinebre Mengist to Negele. July to August are

also high rainfall months along the Aposto to Hagera Selam / Kibere Mengist road. November, December and January are months of low rainfall all along the road. (Design Review, 2006).

The observed seasonal variations are best explained by reference to the position of the Inter Tropical Convergence Zone (ITCZ). Seasonal oscillation of the ITCZ causes variation in the wind pattern. Between June and September, the area comes under influence of westerly and southerly winds, whilst from October to May; the Easterly air currents dominate this part of the country (Bekele Y. 2004).

Table 3.2 Rainfall data (Meteorological agency, 2010)

Rainfall Record, November 2009 to October 2010, at Kibre Mengist station													
Month	2009		2010										
	Nov	Dec	Janu	Feb	March	April	May	June	July	Aug	Sept	Oct	Total
Rainfall (mm)	26.80	28.8	5.60	25.10	101.60	269.80	276.5	5.5	9.0	82.3	30.3	118.	979.30

The maximum rainfall is recorded in May, and the minimum is recorded in January and June. From the rainfall distribution chart shown in Fig. 3.2, two rainy seasons are observed. The months, March, April, and May, are the main rainy season of the year. The second rainy season starts in August and ends in October. January, June and July are the driest months in the year but light rain is also recorded in November and February. Based on the data obtained as shown in table 3.2, the rainfall distribution of 12 months are categorized into four: (i) June to July, average rainfall of 7 mm; (ii) August to October, average rainfall 77mm; (iii) November to February, average rainfall 22 mm; (iv) March to May, average rainfall 216 mm. The area has received 980 mm annual average precipitation for the year from November 2009 to October 2010. The highest monthly average precipitation was 270 mm recorded in May 2010.

According to ERA (2002) drainage manual, the project area is in B₂ (Apsoto- Kibere Mengist) and D₂ (Kibremengist to Negelle) types rainfall regime (Design Review, 2006). B₂ is also a rain fall region that receives annual rainfall of 1600 to 1999mm while D₂ is a rain fall region which receives annual rainfall of 1200 to 1599 mm.

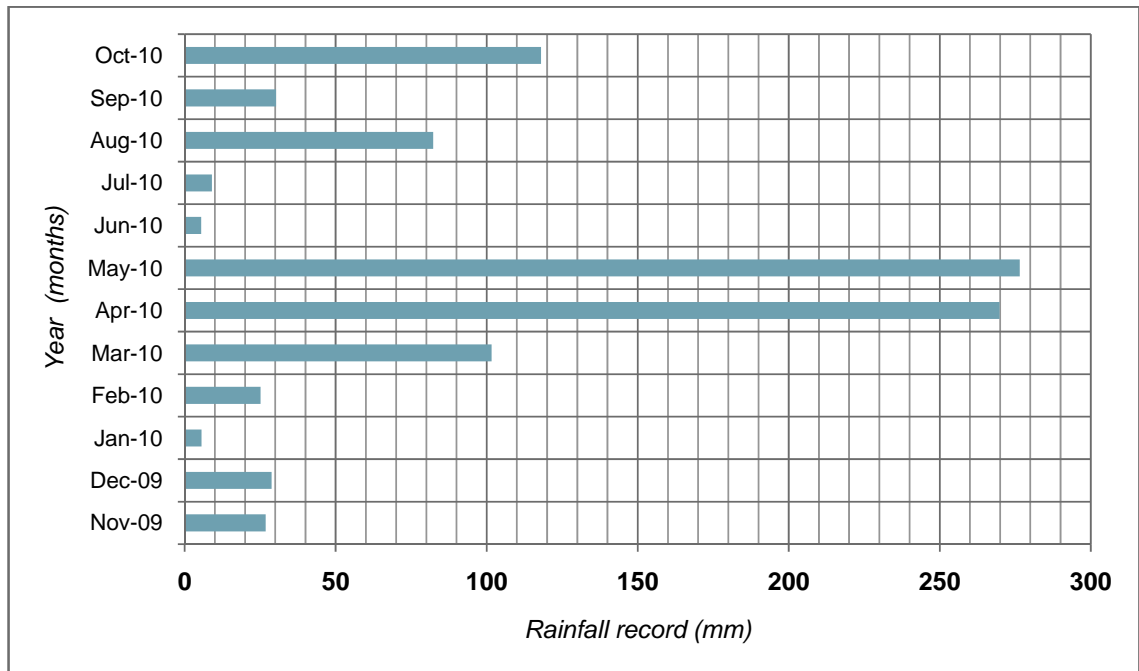


Fig. 3.2 Rainfall in the study area

The amount of annual rainfall decreases from Aposto to Negele. In spite of this, Kibremengist receives annual rainfall of 980mm, which is below the records from both Aposto Kibre mengist and Kibre mengist Negele areas. This may happen in due course of deforestation and environmental degradation which facilitates desertations. As per the annual rainfall records obtained from Kibre Mengist Meteorological stations, the study area is then classified in the rainfall region where its annual Rainfall lies from 800mm to 1199mm.

3.5.2 Temperature

The climate of the area is subtropical with moderate temperatures. The precipitation and temperature data (1974-2003) of the area has been collected from National Meteorological Service Agency. The daily average temperature varies between 11°C-27°C. The average annual temperature ranges between 15°-20°C, with maximum daily temperature variation being during the rainy seasons (Bekele Y. 2004). Based on the data obtained from the National Meteorological agency, the two main rainy seasons are March to May and August to October. Eventually, the study area is categorized in the seasonally wet tropical regions.

3.6 Topography / Terrain characteristics

The topography of the study area is characterized by undulated landscape with an altitude that varies from 1665m to 1860m and found in the Dawa Genele drainage basin and all the waters

drain towards the Awata River catchment area. The geologic formations and related structure play significant role for the formation of the present land forms and drainage characteristics. As it is observed in the local geologic map, the study area is characterized by a number of faults whose orientation of dip angle are variable and be the main factor to the formations the existing undulated land forms. Due to this, most of the topographic features of the study areas are dissected to each other. All but the river at 138+500 are intermittent streams such as at, 141+830, 149+000, 149+500, 151+500, 153 + 360 and 157+000 all of which drain towards Awata River.

The geometric design elements of a road depend on the type of traverse terrain through which the road passes. The terrain type will have a significant influence both on the vertical and horizontal alignments of the route corridor. The speed limit, carriage way and shoulder width, type of drain line, parking lots and visibility distances of traffic movements are also specified based on terrain classes. The general terrain classifications are shown in Table 3.3. According to ERA geometric Design Manual (2002) transverse terrain properties are categorized in to four classes, namely;

- (i) Flat terrain, transverse terrain slope up to 5%
- (ii) Rolling terrain, transverse terrain slope from 5% to 25%
- (iii) Mountainous terrain, transverse terrain slope from 25% to 50%
- (iv) Escarpment terrain, transverse terrain slope in excess of 50%

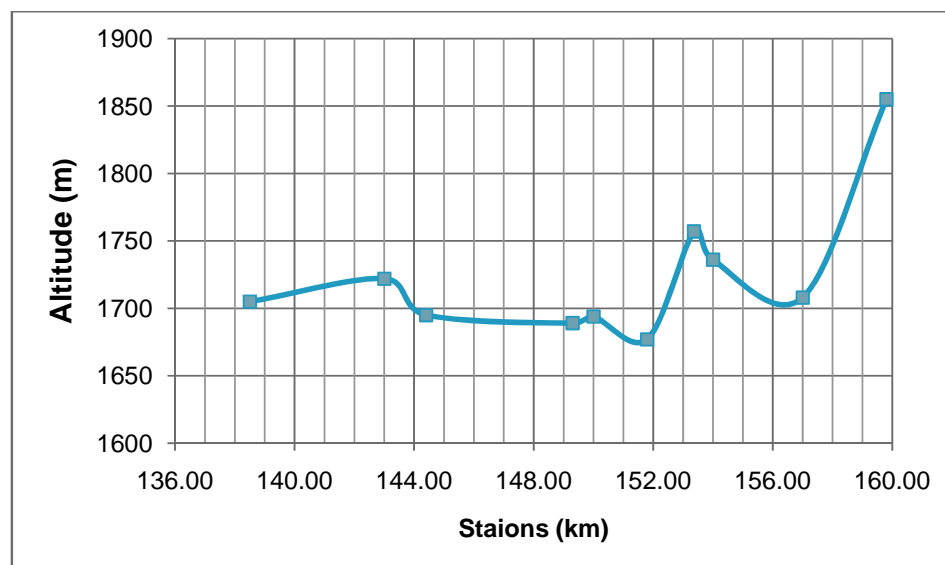


Fig 3.3 Elevation profile of the study area

Hence, the route of the study area passes through rolling to escarpment terrain at an elevation ranging from 1665m to 1855m altitude as can be observed in Fig. 3.3. However, the terrain of the study area is dominated by rolling but localized mountainous and flat in some sections. The terrain classes for the study area are shown in table 3.3.

Table 3.3 Terrain classifications of the study area

No	Terrain class	Chainage (Km Station)		Remarks
		From	To	
1	Escarpement	137+400	137+700	Anferara hill
2	Urban/peri urban	137+700	140+400	Anferara Town
3	Rolling	140+400	144+100	
4	Urban/peri urban	144+100	151+200	Kibre Mengist Town with localized flat terrians
5	Rolling to Mountanous	151+200	159+800	

In general, the study area in the majority of the cases is classified as rolling terrain, with the traverse terrian slope range of 5% to 25%, as per ERA Drainage design manual (2002).

3.7 Regional geology

The study of the Geology of Ethiopia goes back to 1860 by Blanford, and since then, major advances in the understanding of the geology of Ethiopia have been made from the works of various researchers, e.g. Danieli (1943); Mohr (1971); Kazmin (1972, 1974, 1978). The outcome of these studies outlined the lithostratigraphy of Ethiopia into three major categories:

- (i) The Precambrian Basement
- (ii) The Late Palaeozoic to Early Tertiary sediments
- (iii) The Cainozoic volcanic and associated sedimentary rocks.

In the southern part of the country the volcano-sedimentary belt (Adola Group), with its attendant intrusives, occurs enclosed by the gneissic terrains (Mormora and Awata Group).

Earlier works classify the Precambrian rocks of Ethiopia in to two major lithotectonic assemblages (Ayalew et al., 1990; Teklay et al., 1993; Hailu Worku, 1996):

- The Gneissic terrains (pre-Pan-African Crust) of upper to late Proterozoic age.

- Metamorphosed Volcano-Sedimentary Belts (the Pan-African juvenile crust) associated with minor ultramafic bodies and intrusives ranging from mafic to granitic in composition.

The blocks of gneissic terrains are considered to be older than the volcano-sedimentary belts, and the metamorphism ranges from the upper amphibolite to granulites facies. Metamorphic facies in the low-grade volcano-sedimentary succession typically ranges from green schist to lower amphibolite facies.

The general foliation trend is (N-S) with exceptional deviation to the northeast and northwest. The boundaries between the gneissic and volcano-sedimentary sequences are of tectonic origin, represented by sheared mylonitized and tectonically highly deformed ultramafic rocks.

Two major structural zones are identified in the Adola Belt:

- i) A folded and refolded gneisses and schists with complex structural and lithologic relationship in the western and eastern part of the Adola Belt.
- ii) A linear N-S trending belt made up of intercalated metavolcano-sedimentary, mafic-ultramafic and gneissic rocks that form the Megado, Shakisso, Kentcha and Zembaba Terrains.

The lithostratigraphic associations of the Adola Belt are outlined in different works of Gilboy (1970), Charter (1971), Kazmin et al., (1978), Kozyrev et al. (1985), Worku et al., (1992), Wolday Ghebream (1992) and Woldehaimanot (1995). The high-grade rocks comprise the Awata and Mormora Groups.

The Awata Group constitutes gray gneiss, which contains syn-to post-tectonic concordant and discordant granite and pegmatite intrusions. It is considered to represent the pre-Pan-African continental microplate (Kozyrev et al., 1985) and overlain by Mormora Group.

The Mormora Group constitutes the Zembaba, Aflata and Kenticha Formations (Kozyrev et al, 1985):

- The Zembaba Formation Consists of extensively developed and strongly schistose quartzo-feldspathic and leucocratic biotite gneisses of both sedimentary and volcanic origins.

- The Kenticha Formation constitutes amphibolites, graphite-staurolite-kyanite and silimanite bearing mica schists, sulphide-bearing pelitic and psammo-pelitic metasediments and marble.
- The Aflata Formation constitutes interlayered biotite gneiss, biotite-hornblend gneiss, amphibolite, and mica schists. The rocks of this group crop out to the east and west of Megado Graben that is filled with the Adola Group rocks which are the most important ore bearing formations.

The interfaces between the Mormora and Adola Groups are straddled by the occurrence of ultramafic bodies.

Granitoid intrusive rocks occur both in the high-grade gneisses and schists as well as the metavolcano-sedimentary sequences of the Adola area. In general, the intrusive rocks range in composition from mafic to granitic.

3.8 Geology of the study Area

As shown in fig 3.4 local geologic map, there are three main formations in the Adola belt (Ore geology review 35(2009),-68-86)

- (i) The low grade metamorphic psammo pelitic and ultramafic schists consists of
 - The Kenticha tectonic unit
 - The Megado Tectonic Unit
- (ii) The high grade metamorphic schists and gneises comprises of
 - Sodda tectonic unit
 - Shakisso tectonic unit and
 - Zembaba tectonic unit
- (iii) The post tectonic biotite granite units

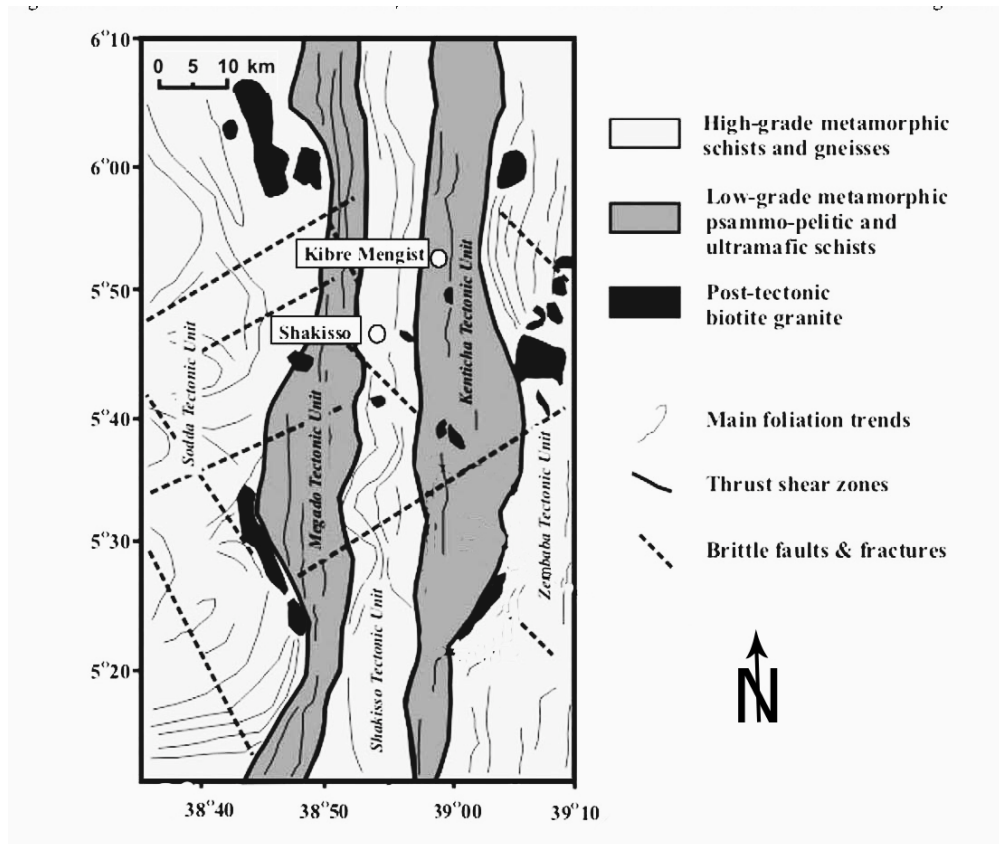


Fig. 3.4 Geology map of Adola area (Extracted from; Ore geology reviews journal, 2009)

3.9 Geology of the route corridor

The road project lies on the metamorphic basement complex of Adola and its surrounding areas. As shown in the geologic map for the study area (Fig.3.5), various types of metamorphic rocks are encountered along the route corridor of the road project, particularly the study area.

As per the field observation and thin section analysis (table 3.3) supported by review of literature, the main lithological units encountered in the study area are as follows (Geological map of Adola Area, 1988):

- (i) Amphibolites
- (ii) Graphite schists
- (iii) Kyanite schists
- (iv) Mica schist
- (v) Mica Schist with frequent Garnet
- (vi) Talc Chlorite tremolites

- (vii) Quartzo Feldspathic Gneisses
- (viii) Biotites and Biotites hornblende

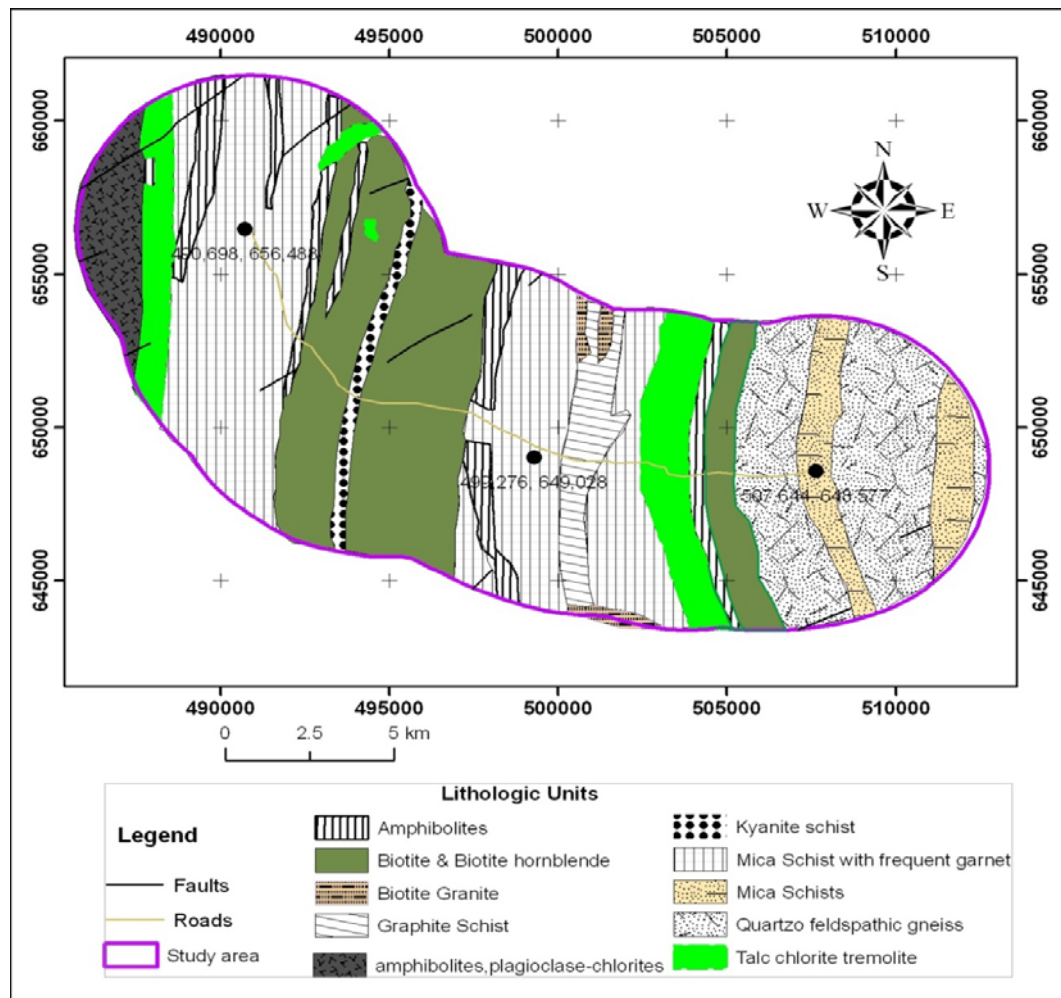


Fig.3.5 Geologic map of the route corridor (extracted from the geologic map of Adola Area, 1988)

4.9.1 Rock sampling and petrographic investigation

Two Rock samples at two different locations were collected and studied for petrographic analysis at the Ethiopian Geological Survey Central laboratory. The samples were taken outside the study area limits but along the route corridor in the road project limits, i.e at 167+200, LHS and 195+500, LHS (plate 3.3). Two pieces for each sample were studied for mineralogical compositions. The laboratory results are presented in Annexure 3 and are summarized in table 3.4.



Plate. 3.3 Biotite Gneiss, Rock Sample km 195+500, LHS, Photo October 2010

Table 3.4 Mineral composition of the rock samples

S/N	Name of Minerals	Mineral Compositions (%)			
		Sample No. 1- 167+200	Sample No 2-167+200	Sample A-195+500	Sample B-195+500
1	K-feldspar (Microcline)	62	53	35	44
2	Quartz	25	20	22	22
3	Calcite	6	1	-	-
4	Muscovite	3	1	-	trace
5	Biotite	-	12	18	24
6	Opaque (Fe-oxides)	4	10	2	2
7	Chlorite	-	1	6	2
8	Apatite	-	1	trace	2
9	Epidote	-	1	3	trace
10	Sphene	-	-	4	4
11	Plagioclase	-	-	10	-
Rock type		Alkali Gneiss	Biotite Gneiss	Biotite Gneiss	Biotite gneiss

3.9.2 Weathering properties of rocks

As indicated above, the most significant group of expansive clay minerals are the smectites, of which montmorillonite is the most commonly occurring mineral. These minerals result from the chemical weathering or hydrothermal alteration of basic and intermediate igneous and metamorphic rocks containing calcic feldspars and ferromagnesian minerals. Sometimes, acidic igneous rocks such as granites are parent materials for the formation of montmorillonites. Igneous, metamorphic and sedimentary rocks can all be parent materials for expansive clays (ODA, 1993). Therefore, to correlate the residual expansive soils with the parent rocks in the study area, mineralogical tests for selected rock samples have been conducted. Besides, it is also explained in the geology part of this paper that the dominant building blocks of rock units in the study area are feldspar, graphite, biotite, Mica schists and gneisses, & quartz feldspars in composition. From laboratory test results obtained for rock samples, four samples from two localities were studied. In these samples, the dominant minerals are the feldspar group (table 3.4). The quartz content is almost uniform in the entire specimen. The amount of the platy minerals such as biotites, muscovites and chlorites are varying from one sample to another sample and so are the feldspar groups. Quartz is the most resistant mineral to weathering among all others. As mentioned in chapter 1, weathering product of some rock forming minerals such as the platy minerals, and feldspars are clay and Na^+ with K^+ ions are leached out.

According to the USGS information handout (1999), it is stated that the factors governing rock weathering and soil formation are the initial type of rocks, the temperature and rainfall in the area. Similarly, the types of minerals found in weathered rocks strongly control how the weathered rocks behave under various climatic conditions.

In metamorphic rocks, the segregation, realignment of minerals into preferred orientation, formation of bands in mineral arrangement such as gneissosity, schistosity, foliation and lineation will enhance entrance of water and the early alteration or decomposition of rocks. The feldspars and mica minerals in the rocks will be removed by leaching and the most resistant minerals such as quartz, Al and Fe will be the residual laterites (Steven, 2003). The leaching process may increase the void ratio in the rock mass so as to increase easily weatherability and erodability of rocks.

3.10 Seismicity of the study Area

Natural hazard analysis is part of engineering geological feasibility study of a project. Its objective is to determine whether the natural risk is at acceptable level or not and to consider the risk in the design parameter of the structure. The determination of the risk involves, measurement of the risk, and judging whether the risk is acceptable or not. The engineering geologist is responsible in defining natural hazard in the area of interest. The definition of natural hazard requires a statement of probability of occurrence. This prediction of future events of a given magnitude usually depends on basic geologic and seismic studies. These studies usually relate present and past conditions in an effort to identify some empirical relationship, which may serve to predict future events. The engineering geologist typically is forced to make the most of the existing information to predict expected risk (Johnson and Degraff, 1991).

When important engineering structures are to be constructed, in seismic risk areas, the peak earthquake ground acceleration will be a variable useful in the evaluation of the response of the structure to earthquake motions. The magnitude of past earthquakes along with generalized curves on attenuation of generated motion for different regions can be combined to provide estimates of expected peak ground acceleration (Hays, 1980; as cited in Johnson and Degraff, 1991).

The Seismic zones of Ethiopia have been delineated by Gouin (1979) and later updated by Asfaw, L.M (1986). In the later work previously omitted earthquake parameters, measurements of Southern Ethiopia were included; strain release and seismic risk maps have been produced for earthquake from year 1900 to 1985; the probable return period of destructive earthquakes has been considered and a discussion of some unique features of earthquake hazard in the Afar Depression have been presented.

The project area is within the influence area of the Southern Ethiopian Rift (SER) earthquake sources hence, the effect of dynamic loading on the stability of the site has been considered in this study. The 'Seismic Risk Map' produced by Asfaw L.M (1986) for a hundred year return period and 0.99 probability shows that the study area falls within 7 to 6 M.M scale.

CHAPTER FOUR: FIELD INVESTIGATION AND LABORATORY TESTS FOR SUB-GRADE SOILS

4.1 Preamble

The supporting ground beneath a pavement structure is called the sub grade. The strength, stiffness, compressibility and moisture characteristic of material underlying the pavement structure can have a significant influence on pavement performance and long term maintenance. The sub-grade and subsequent layers must be strong enough to with stand shear failures and vertical deflections by repeated loadings. Strong and stiffer materials provide more effective foundations for riding surface and can resist the repeated loading. The critical component of any pavement design is the characterizations of the material upon which the pavement structure will be constructed. In cases when the sub-grade is inadequate, methods of improvement for the sub grade material will be provided (ERA, 2002). The intent of the present research is, therefore, to characterize the sub-grade material for the construction of the pavement structure and propose remedial measures for unsuitable portions of the sub-grade material.

The presence of unsuitable soils in construction sites creates significant influence on planning, structural design, construction and maintenance costs, performance and engineering life, especially of shallow depth engineering infrastructures where the moisture fluctuation is significant. Such soils are particularly susceptible to considerable volume changes in response to moisture content fluctuations following seasonal climatic variations. This property can cause severe damage to pavement structures unless proper measures are taken prior to construction phase. Identification of unsuitable soils and characterization of their anticipated behavior is thus an important parameter for site selection, design, and construction projects (AASHTO 1993).

The sub-grade soil investigation conducted under present study incorporates field, laboratory investigations and test results evaluation. In the field, sub-grade soil extension survey, investigation and sampling has been carried out from beginning of July 2010 to mid of December, 2010 in connection with the detailed investigation during the construction phase of the road. Representative soil sampling at 200m interval has also been taken in order to conduct gradation tests, liquid limit (LL), plastic limit (PL), California bearing ratio (CBR) and swell potential. This chapter includes the major procedures and steps that have been followed during the field, laboratory, and office works. Moreover, it comprises techniques

that were followed to investigate and characterize the sub-grade soils during the construction phase of the road.

4.2 Sub grade soil investigation

The purposes of the sub-grade material investigation include;

- Assessment of depth and nature of the sub-grade soil characteristics along the project route.
- Assessment of suitability of the sub-grade soils so as to incorporate to the pavement design,
- Classify the sub-grade soils into homogenous sections and to determine the design CBR values
- Identifying the location, depth and nature of unsuitable sub-grade soil sections along the project route and to suggest possible remedial measures that would be suitable for sub-grade construction.

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

4.2.1 Field Investigation for Sub-grade Soil

The sub-grade soil investigation was carried out in accordance with the particular condition of contract for the road project. This task was aimed at assessing the actual condition of the soil along the proposed alignment and includes;

- Visual sub-grade soil extension survey, and
- Sampling and testing representative soil samples at 200m interval to identify the physical properties of soil for engineering uses

4.2.2 Visual Sub-grade Soil Extension

In order to group homogeneous sections, during the field work, the visual sub-grade soil extension survey along the road alignment has been carried out to assess the nature, type and extent of existing sub-grade soil that makes the roadbed. Sub-grade soils with similar soil type were grouped together and their extent was determined. Thus, the type of sub-grade soil encountered along the route corridor is found to be mainly dependent on topography and geology of the project area. The project area generally, lies on rolling terrain with localized

flat and mountainous region in a warm and humid climate region. As a result, erosion effect is minimal along flat terrain sections and development of thick residual soil is facilitated in the chainage from km 143+500 to 144+500 and km146+200 to 147+600. However, erosion is highly aggravated in cut sections along rolling, mountainous and escarpment sections like 155+700, where almost the excavated sections severely eroded by storm water as shown in plate. 4.1.



Plate. 4.1 Severe erosion in the rolling terrain (km 155+850, RHS), Photo, August, 2010

From the visual inspection of the sub-grade soil during the site visit, in most of the cases it was found that it is light brown, dark brown to yellowish silty clay type and in few stretches of the road alignment it was reddish brown silty clay/clayey silt soil mixed with weathered laterite gravel, where these are of residual nature as shown in plate 4.1. The soil extension survey was carried out in such a way that different soil types along the alignment can be recorded and classified according to color, texture and composition. It should be noted that in many cases, clear distinction between the individual soil types could not be made due to their similarity in origin and soil properties, and gradual transitions from one soil type to the other which was common in relation to the topography of the area.

In general, soil formation along route corridor of the project mainly consists of the residual soils. The residual soils probably have resulted from in-situ weathering of parent rocks that were not subjected to transportation and are still in the place of their origin (plate. 4.2).



Plate 4.2 Typical sub-grade soils (Residual) in the road project (km 121+000, RHS)

These rocks often grade into decomposed or completely weathered rocks with depth. The residual soils are commonly lateritic in nature and usually described as mostly dark brown, reddish, pinkish, greenish yellow and white colored and in some localities grey to dark clayey silts or silty clays. Along the flatter section of the road from Km 146+200 to 147+500, dark gray to dark brown thick soils are found as shown in annex 1.

4.2.3 Sub-grade Soil Sampling

Regardless of the type of project, sample spacing should be located to obtain basic knowledge of the engineering properties of the overburden and bed rock formations that will be affected or will have an effect upon the proposed pavement structures (Chen, 1988). There is no rigid rule for the number and spacing of sample locations. The number and sample spacing are dependent on the sub-surface variability of the project site. As per AASHTO, Pavement design guide (1993); the sample spacing varies from 150m to 450m. Based on this, investigation on type and extent of soil type was conducted by taking representative samples from full depth of test pit at intervals of 200m for classification and for strength tests depending on the homogeneity/ variability of the soil types along the route. The interval of the sampling stations was adjusted on field based observations for some sections like at km156+800 and km 157+000; on assessment of fill height and soil conditions. Care was exercised that all soil types along the route must be properly represented. The

depth of sampling varied from place to place and was determined from the grade level of the approved drawing.

4.2.4 Laboratory investigation for sub-grade soils

Soils are used as construction material and also as foundation for engineering structures. However, the wide range of properties under different conditions affects their performance and use. For this reason, soils have to be properly sampled and subjected to various tests so as to understand their properties towards these engineering applications.

For the present study, the sub-grade soil was properly sampled for laboratory testing from each test pits. A total of 112 sub grade soil samples were collected. Later these soil samples were subjected to various tests to determine their physical properties. The types of tests carried out in the laboratory include; Soil Classification tests such as; Grain size distribution and Atterberg Limits, Moisture/ Density Relationship, California Bearing Ratio (CBR), and CBR Swell tests. The summary of the laboratory test results for various soil properties is tabulated as Annexure-1 together with individual test results.

4.3 Sub grade soil classification

An engineering soil classification system indicates engineering soil properties and provides a preliminary understanding of the behavior of soils under various engineering conditions. The AASHTO M 145 soil classification method for the sub-grade soils was adopted for the present research works and it includes the following; (i) Visual description of the soil texture, (ii) Grain size analysis (Gradation) and (iii) Atterberg limits determination (Liquid limit (LL) and Plasticity index (PI)).

These tests are indicators of the physical properties of the sub-grade soils and verify their suitability as road bed material and to incorporate in the pavement construction process (FHWA NHI-05-037, 2006). All sub-grade soil samples collected during the site investigation period were tested for LL and PL values at Kibre Mengist site laboratory for further analysis and evaluation.

4.3.1 Grain Size Distribution

Wet sieve analysis was employed to determine the grain size distribution of sub-grade soils in accordance with AASHTO T-88 Test Method for Particle-Size finer than 0.075mm (No.200)

sieve in mineral aggregate by washing. It is one of the most important soil characterizations as the particle size distribution affects many properties of the soil such as density (compaction), strength, void ratio, and permeability (FHWA NHI -05 -037, 2006). For the present study, wet sieve analysis was carried out only to determine the combined percentage of silt and clay materials passing sieve No 200 (0.075mm) used in the classification of the soil type based on AASHTO M145. Based on wet sieve analysis all the sub-grade soils in the present study area were found to have fine fraction of above 35% by weight.

4.3.2 Atterberg Limits

Expansive clays exhibit higher shrinkage and swelling upon change in moisture content (Chen 1962). These clays are checked for their liquid limits and plasticity Index in accordance with AASHTO T 89 and 90 to determine the nature and response of sub-grade soils upon change to moisture content. Only the materials passing through sieve size 0.425mm (No 40) are considered for Atterberg limit tests. The liquid limit and plastic limit tests collectively are called the tests for Atterberg limits. These Atterberg limit test results are reported to the nearest whole number with the number of blows in the logarithmic scale in the horizontal direction against percent of moisture content in the vertical direction. The moisture content at 25 blows in the graph represents the value of the Liquid limit for that particular sub-grade soil sample (Arora, 1997). As observed from the Table 4.1, more than 94% of the sub-grade soils among the samples are within the specified limits.

Table 4.1 the range of liquid limits values and percentage occurrences

S.No	Liquid Limits	Number of occurrences	(%) occurrences
1	≤ 60 %	106	94.64
2	> 60 %	6	5.36
Total		112	100

The Atterberg Limit depends on the type of predominant mineral in that soil. If Montmorillonite is the predominant mineral the liquid limit can reach or even exceed 100% (ODA, 1993). It is also expected that the Atterberg Limits will be less for soils comprising dominantly illite mineral, whereas it may further be less for soils dominated by Kaolinite mineral (Bell, 2007).

In general, soils that exhibit plastic behavior over wide ranges of moisture content and that have high liquid limit, have greater potential for swelling and shrinking. Besides, the amounts of swell will increase with the amount of clay present in the soils. Plasticity index of soils is the difference between the liquid limits and the plasticity indices of those soils (Chen 1988). The liquid limits and plasticity indices of the sub-grade soils in the study area are shown in Fig. 4.1

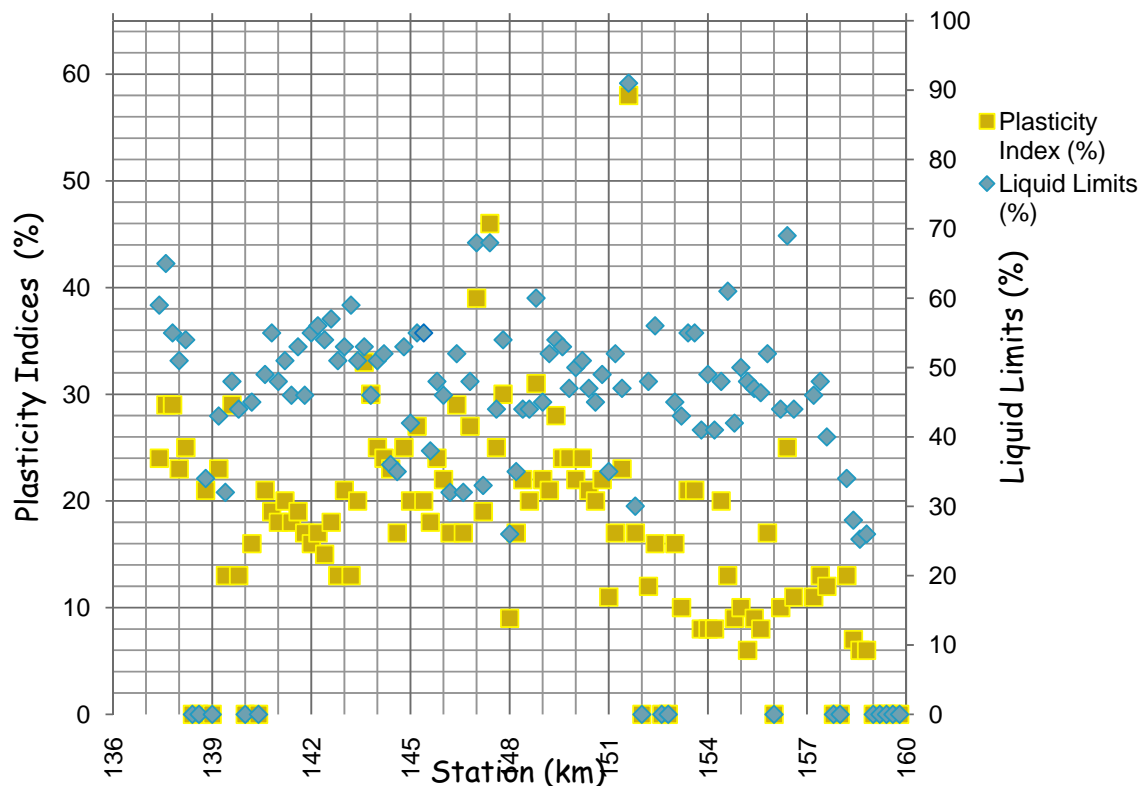


Fig 4.1 Distribution of Plasticity Index & Liquid Limits with stations

The range of plasticity index of soil samples for the study area and the percentage occurrences are also given in Table 4.4 as per AASHTO M 145 – 91 (2000) and Project Contact Document (2008). According to the Project Technical Specification (2008), the test result for plasticity indices as shown in Table 4.2 and Fig. 4.1, 93.75 % of the samples fulfilled the minimum requirements for the use of sub- grade (road bed) material. 94.64% the liquid limits test results summarized in Table 4.1 and Fig. 4.1 fulfilled the minimum requirement as per the project technical specification.

Once the grain size distribution and Atterberg limits are determined, then the classification for the sub grade soils has to be carried out. Based on the results from laboratory tests (Annexure 1), the soils have been classified as per AASHTO M 145 as follows in Table 4.3.

Table 4.2 Range of plasticity index with the percentage occurrences

S.No.	Plasticity Index (PI)	Number of occurrences	(%) occurrences	Remarks
1	$0 \leq PI \leq 10$	31	27.68	Suitable
2	$10 < PI \leq 20$	41	36.61	Suitable
3	$20 < PI \leq 30$	33	29.46	Suitable
4	> 30	7	6.25	Unsuitable
Total		112	100	

Table 4.3 Sub grade soils classification by AASHTO Method

S. No	Soil classes	Number of occurrences	(%) occurrences
1	A-7-6	38	33.93
2	A-7-5	31	27.68
3	A-6	12	10.71
4	A-5	10	8.93
5	A-4	21	18.75
Total		112	100

It can be concluded using soil classes from the Table 4.3 that 61.61% of the sub-grade soil samples are in the soils group A-7-5/6; other soil groups cover only 38.39% of all the sub-grade soil samples along the entire length in the study area.

4.3.3 The group index (GI)

The nature of the sub-grade material can also be characterized by their Group Index Values. The Group Index characterizes the clayey nature of the soil and is calculated by equation. 4.1 as per AASHTO M145-91 (1995);

$$GI = (F - 35) [0.2 + 0.005 (LL - 40)] + 0.01 (F - 15) (PI - 10) \quad \dots eq. 4.1$$

Where; 'F' is the percentage passing 0.075 mm (No. 200) sieve expressed as a whole number. This percentage is based only on the material passing 0.075 mm. 'LL' is the liquid limit and

‘PI’ is the plasticity index. Eq. 4.1 is applicable for soil classes that do not belong to A-2 – 6 and A-2-7. For these soil classes, only partial group index is applied, for this equation.4.2 is used.

$$GI = 0.01(F - 15) (PI - 10) \quad \dots eq. 4.2$$

One of the assumptions in this formula is that, when the value is negative, the group index shall be reported as zero (0). The other assumptions are discussed below;

Under average conditions of good drainage and thorough compaction, the supporting value of a material as sub-grade may be assumed as an inverse ratio to its Group index; that is, a Group index of zero indicates a “good” sub-grade material and a Group index of 20 or greater indicates a “very poor” sub-grade material” (AASHTO, 1993).

Persual of Annexure 1 and Table 4.4 indicates that about 82.14 % of the sections in the study area show GI results less than or equal to 20 while the rest 17.86% of the section show GI greater than 20. From the GI values, it can be concluded that except localized areas, most of the sections are considered suitable to be used as a road bed and embankment material under pavement layers.

In Annexure 1 the Group Index values are shown in parenthesis for each of the samples collected during field investigation in the study area. Classification of sub-grade soils using Group index values (GI) taken from test results in Annexure 1 is also shown in Table 4.4.

Table 4.4 Soils classification with its Group Index Values

Soil Classes	Group Index (GI)	Number of occurrences	(%) occurrences	Remark
A - 7 - 6	$0 \leq GI \leq 10$	5	13.16	Suitable for sub grade
	$10 < GI \leq 20$	20	52.63	Intermediate
	$GI > 20$	13	34.21	Poor for sub grade
A - 7 - 5	$0 \leq GI \leq 10$	7	22.58	Suitable for sub grade
	$10 < GI \leq 20$	17	54.84	Intermediate
	$GI > 20$	7	22.58	Poor for sub grade
A - 6	$0 \leq GI \leq 10$	12	100	Suitable for sub grade
A -5	$0 \leq GI \leq 10$	10	100	Suitable for sub grade
A -4	$0 \leq GI \leq 10$	21	100	Suitable for sub grade
Total		112		

4.4 Density - Moisture Relationship

The most common measure of compaction of soil is its density. Soils density and optimum moisture content should be determined according to AASHTO T180. Optimal engineering properties such as shear strength for a given soil type occur near its Maximum dry density (MDD) and Optimum moisture content (OMC). At this state, soils void ratio, potential to shrink and swell is minimized. Field density is then correlated to moisture density relationship in the laboratory for quality controlling purposes in the field (Arora 1997).

The sub-grade soil samples were subjected to the determination of maximum dry density (MDD) and optimum moisture content (OMC) in the laboratory. The moisture-density curve is different for each soil type.

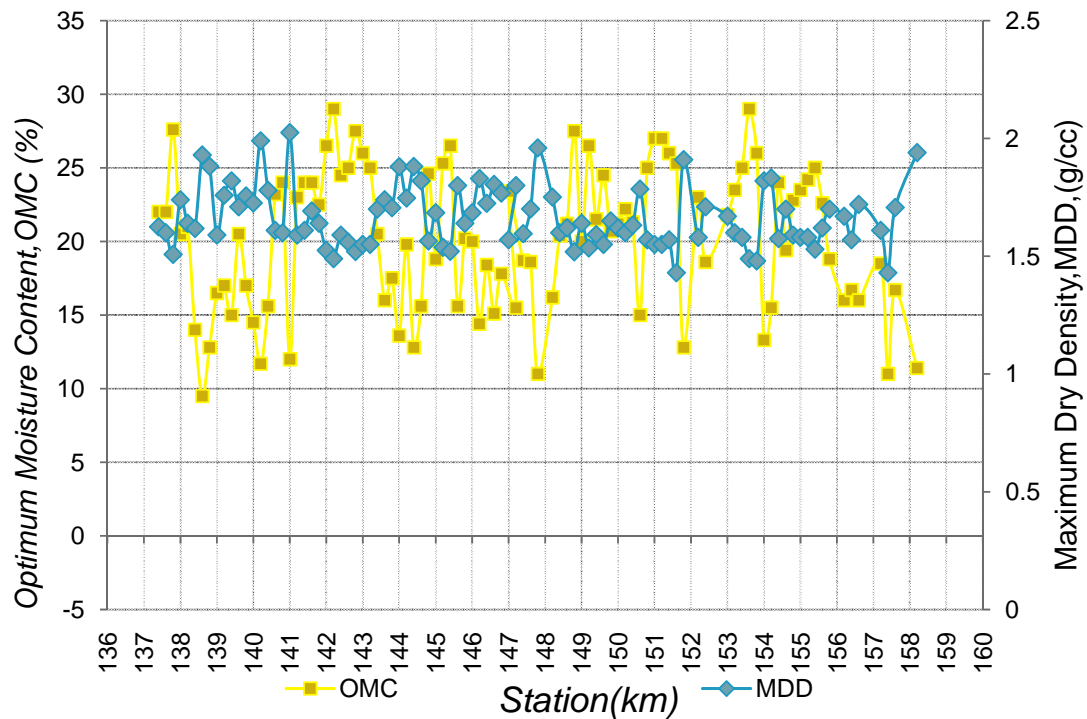


Fig. 4.2 Distribution of OMC and MDD with station

Granular, well-graded soils generally have fairly high maximum densities at low optimum moisture contents, while clay soils have lower densities. The edge-to-side bonds between clay particles resist compaction efforts to force them into a denser structure. Whereas, well-graded

granular soils have spaces between large particles that are filled with smaller particles when compacted that lead to a higher density than with uniform soils (FHWA, 2006).

Table 4.5 Range of MDD & OMC of the sub grade soils

Soils Descriptitons	AASHTO classes	range of % pass , Sieve # 200	MDD,	OMC,
Non plastic silty soils	A - 4	35 – 50.40	1.59 - 2.15	7.2 – 18.50
Highly elastic with high LL,silty clay	A - 5	37.90 - 66.39	1.48 – 1.832	13.30 – 26.00
Plastic clay soils	A - 6	47.78 - 64.10	1.55 – 1.94	11.40 – 27.00
Materials with moderate plasticity index	A - 7 - 5	42.75 – 89.87	1.43 – 2.025	11.00- 29.00
Materials with high plasticity Index	A - 7 - 6	52.48 – 94.20	1.508 – 1.99	11.7 - 27.6

Table 4.5 shows the soil class, range of MDD and OMC of sub-grade soils in the study area. The MDD and OMC of the sub grade soils along the route in the study area are also shown in Fig. 4.2.

4.5 California Bearing Ratio (CBR) test

The CBR is a comparative measure of the shear resistance of the soil. The test consists of measuring the load required to cause a piston of standard area, 19.35cm² (3 inch²) to penetrate a soil specimen at a specified rate, 1.25mm/min. This load is divided by the load required to force the piston to the same depth in a standard sample of crushed stone (a high quality crushed stone material with a CBR value of 100%) (AASHTO, 1993).

The CBR is the most widely used method for designing pavement structures. The method is primarily intended but not limited to evaluate the strength of cohesive materials. The test procedure is based on, American society for testing and materials, AASHTO T 193. The CBR value for a soil depends upon its density, moulding moisture content and moisture content after soaking. For the present study, km137+400 to 159+800 lengths, 112 three points CBR tests were carried out with 4 days soaking which helped to anticipate the sub-grade soils at the worst moisture conditions.

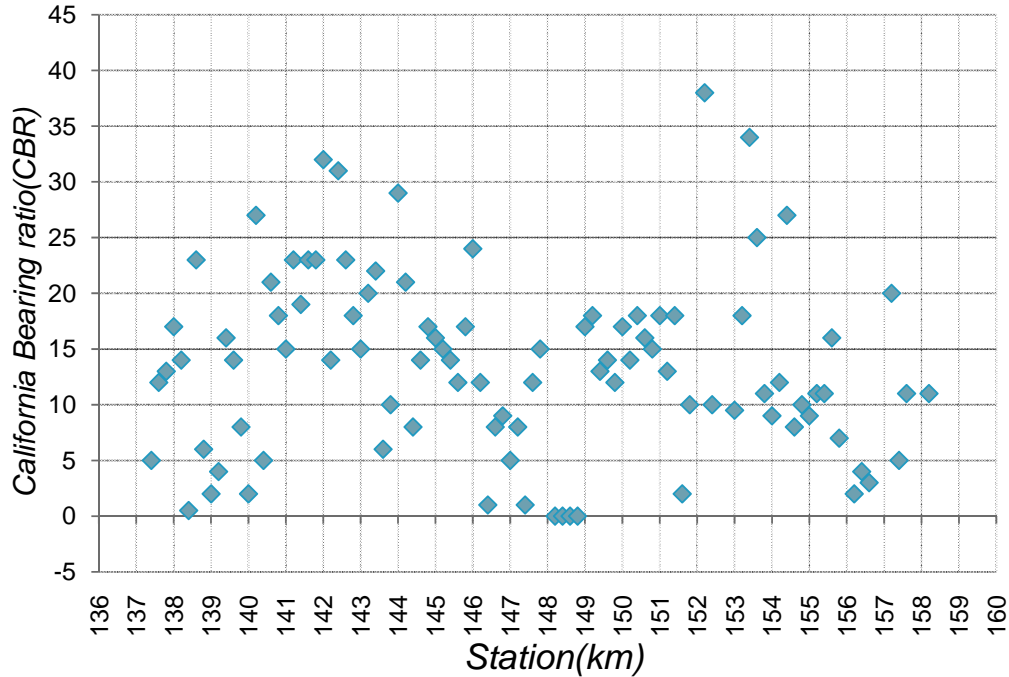


Fig. 4.3 Chart of California Bearing Ratio (CBR) with Staataion

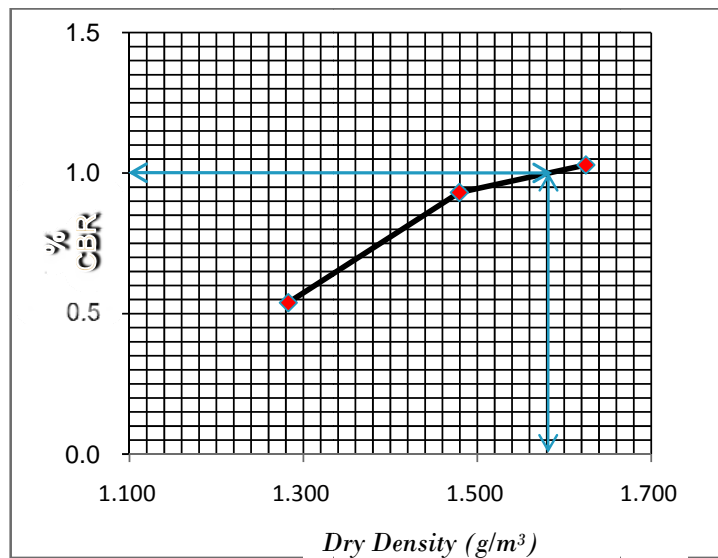
The range of CBR values of the soils for the study area is shown in Table 4.6. To determine the strength and swelling potential of the sub-grade soil samples, test has been carried out by 4-days soaking-3-point CBR and loaded Swell testing procedure. The sub-grade soil strength has been used for design purpose by interpolating the CBR values at different compaction levels, with 10, 30 and 65 blows. The values of the three points CBR is taken from test result in Annexure 1.

Table 4.6 CBR Values range for the sub grade soils (as Per the Specification)

S.No.	CBR values	Number of occurences	(%) Ocurrences	Remarks
1	$CBR < 5$	11	9.82	Unsuitable
2	$5 \leq CBR < 15$	50	44.64	Suitable
4	$15 \leq CBR < 30$	44	39.29	Suitable
5	$CBR \geq 30$	8	7.14	Suitable
Total				

Table 4.7 Data for determination of CBR values at 95% MDD (km 146 + 200)

MDD (gm/m ³)	Number of blows	Moisture before soaking	Dry density (gm/m ³)	% CBR	% moistur e after 96 soaking hrs	% Swell	% Swell	Standard load (KN)	
								2.54m m	5.08 mm
1.621	10	15.5	1.281	0.50	40.2	6.07	11	13.33	20.0 0
OMC	30	15.5	1.481	0.90	36.1	12.01		13.33	20.0 0
15.90	65	15.6	1.626	1.00	32.3	11.00		13.33	20.0 0



MDD = 1.621 g/m³
 95 % of MDD = 1.54 g/m³
 The CBR value at 95% of the
 MDD = 1%
 : The Design CBR is then 1%

Fig 4.4 CBR value determinations (at 95% MDD)

CBR tests can be performed either through one point or with three points for pavement material. For one point system, the CBR value at 100% MDD is considered for design, but for three points, usually the CBR value at 95% of MDD is the design CBR as shown in Fig. 4.4. When CBR tests are conducted for cohesive soils by three point methods at 4 days soaking conditions, the minimum density obtained at 65 blows of CBR value shall be the Maximum Dry Density, MDD, obtained at 56 blows and the value of penetration at 2.54mm is higher than at 5.08mm, which will be considered for design purpose. When the reverse happens, the penetration value at 5.08mm is considered after second trial (Arora, 1997).

As it is observed in Fig. 4.3 and Table 4.6 that only 9.82% of the CBR values of the samples for the sub-grade material are below the minimum requirement.

4.6 The CBR Swell Values

The swelling potential tests conducted during CBR soaking can also be used to estimate the expansiveness nature of the sub-grade soils. The swelling potential is defined as the percentage swell of a laterally confined sample which has been compacted to MDD at OMC and allowed to swell under a surcharge load of 4.54kg conducted on CBR specimen (AASHTO T 193).

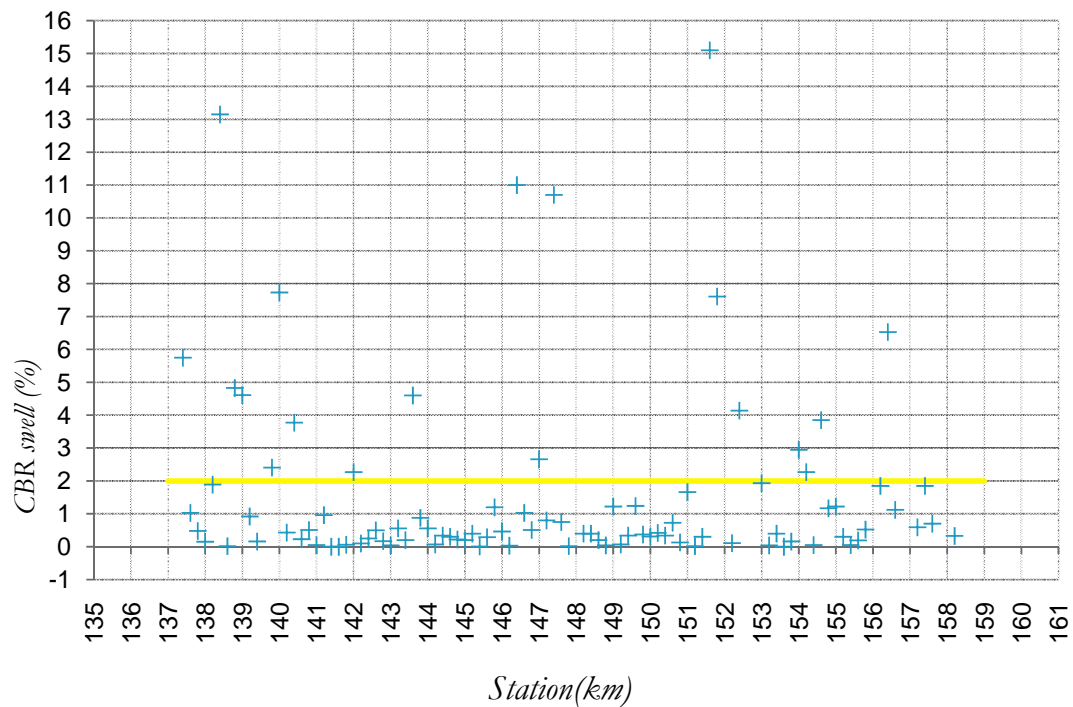


Fig 4.5 Distribution of CBR swell with Staataion

Fig. 4.5 and Table 4.8 showed that 83.06% of the swell values for the sub-grade soil samples fall below the specified maximum.

Table 4.8 CBR swell ranges of the sub grade soils (as per the Specification)

S.No	Swell range	Description	Number of occurrences	% occurrences	Remarks
1	≤ 1.5	low	89	79.46	suitable
2	1.5 to 2.00	intermediate	4	3.57	suitable
3	> 2	high	19	16.94	unsuitable
Total					

4.7 Overall characterizations of sub-grade materials

The characterization or classification of the sub-grade soils of the present study area, for its suitability to be used for pavement, was evaluated based on the properties, such as; (i) CBR swell values, (ii) Plasticity index, (iii) Liquid limits, (iv) California Bearing Ratio (CBR) and (v) Maximum dry Density (MDD). Thus, based on the results following inferences were made;

- The CBR swell values are considered as the first parameter for classifying the sub-grade material, because the moisture fluctuation in the bottom of the sub-grade will have a significant influence for swell and shrinkage of these soils. Therefore, taking the upper limits of the swell value as 2%, only 16.96 % of the soils are classified as unsuitable.
- Plasticity indices are also taken as the second main factor for classifying the sub-grade soils in to suitable/unsuitable ones. Taking the maximum PI value above 30 %, only seven (7) ,or 6.25 % of the samples are unsuitable,
- The third parameter is the Liquid limit value, according to the project technical specification; sub-grade material with LL value more than 60% are unsuitable. Only seven (6) samples or 5.36% of the samples are unsuitable.
- The fourth criteria for classifying the sub-grade soils in to suitable/unsuitable are the value of CBR at 95% of the MDD. The minimum value of CBR for sub-grade soils in this study is defined as 5%. Taking this in to account, only eleven (11) samples or 9.82 % of the samples are considered to be unsuitable.

- The last criterion used for the classification of the sub-grade soils is the Maximum Dry Density. As mentioned above in section, the maximum dry density is obtained by AASHTO T 180 (D) with the modified proctor methods of 56 blows. The minimum values of MDD for sub-grade material are supposed to be 1.5gm/cc (Mytsebri Shire, 2009). Hence, taking this in to account, only five (5) samples or 4.46 % of the samples are below the minimum requirement.

In general more than 80% of the sub-grade soils are suitable for sub-grade layer as well as ordinary fill (cut to fill) material.

Chapter Five: GENERAL CHARACTERIZATION OF SUB-GRADE SOILS AND PAVEMENT STRUCTURE DESIGN REVIEW

5.0 Characterization of Sub-grade Soils

Based on data collected in the field, index tests conducted for sub-grade soil samples and rock samples in the laboratory, analyses has been made by integrating the primary results with secondary data obtained. The interpretation and discussion to characterize the sub-grade have been made in the following paragraphs.

5.1 Properties of sub-grade soils

The laboratory test carried out has been focused to investigate the grading, that is percent pass by weight of material at Sieve No. 200, Liquid Limits (LL), Plasticity Index (PI), maximum dry density (MDD), optimum moisture content (OMC), Soil Strength (CBR), and the potential to swell of the sub-grade soils. Annexure 1 presents the test results for sub-grade soils of the study area. Using the laboratory test results, analysis has been made for Unified Soils classification System (USCS), AASHTO (M145) system of classifications and Casagrande plasticity charts.

5.1.1 Grain size distributions (Gradations)

Gradation, or the distribution of particle size within a soil, is an essential descriptive feature of soils. Soil textural (e.g., gravel, sand, silty clay, etc.) and engineering classifications are based in large part on gradation, and many engineering properties like relative compaction, permeability, strength, swelling potential, and susceptibility to frost action are closely correlated with gradation parameters. Gradation is measured in the laboratory using two tests: a mechanical sieve analysis for the sand and coarser fraction, and a hydrometer test for the silt and finer clay material (FHWA -05-037, 2006). Grain size distribution is done by mechanical sieve to this study in order to determine the percent pass of soils through No 200 sieve for AASHTO soils classification purposes and group index determination.

5.1.2 Atterberg Limits

Correspond to values of moisture content where the consistency of the soils change as it is progressively dried from slurry. Plasticity is the response of a soil to changes in moisture content. When adding water to a soil changes its consistency from hard and rigid to soft and pliable, the soil is said to exhibit plasticity. Clays can be very plastic, silts are only slightly

plastic, and sands and Gravels are non-plastic. For fine-grained soils, engineering behavior is often more closely correlated with plasticity than gradation. Plasticity is a key component of AASHTO and USC soils classification. Soil plasticity is quantified in terms of Atterberg limits. It is important to recognize that Atterberg limits are not fundamental material properties. Rather; they should be interpreted as index values determined from standardized test methods (FHWA NHI-05-037, 2006). It is also mentioned in the same code that: The liquid limit (LL), defines the transition between the liquid and plastic states while the transition between the plastic and semi-solid states defines the plastic limit (PL) and the difference between the two values is termed as the plasticity Index (PI). The shear strength of clay soils at its liquid limit is constant but variable at the plastic limits. Arora (1997) on the other hand, stated that the shear strength of all soils at the liquid limits is constant and equals to 2.7kN/m^2 .

The Project Contract Document (2008) states that clay material having liquid limits exceeding 60%, plasticity index exceeding 30% is considered unsuitable as sub-grade material. Among the index tests, the most important consistency index is the plasticity index; PI. Clay soils have higher value of plasticity than the granular soils. The expansive nature of the soils also arises from the clay content in the soils. As mentioned in chapter four of this research, 112 sub-grade samples have been taken for liquid and plastic limit tests.

Table 5.1 Statistical occurrences of the sub grade soils

Engineering properties	Percent occurrences	Ranges of values for the sub-grade soil
LL	94.64%	$\leq 60\%$
	5.36%	$> 60\%$
PI	35.71 %	$> 30\%$
	49.11 %	$\leq 30\%$
	15.18%	NP

NP = Non plastic soils

Chen (1988) demonstrated that the plasticity indices and the liquid limits are useful indices for determining the swelling characteristics of clays. In addition, the liquid limits and the swelling of clays both depend on the amount of water clay tries to absorb as shown in table 5.2.

In general, the larger the plasticity index implies more problems associated with the use of the soil as an engineering material, such as road sub-grade. Many soil properties and engineering behaviors have been correlated with the plasticity index including swelling-shrinkage potential (USGS 1999).

Table 5.2 Relationship between swelling potential and plasticity index, PI (Chen, 1988)

Plasticity index, PI (%)	Swelling potential
0 - 15	Low
10 - 35	Medium
20 - 55	High
≥ 35	Very high

As indicated in Table 5.1, 94.64% of the soil samples have LL value less than 60% and the rest 5.36% of the sections have the LL value of more than 60%. In the same manner, only 35.71% of the samples have PI values of more than 20% while 64.29% of the samples have PI values less than 20%. As shown in Plate 5.1 some clay minerals swell when they take up water, this swelling and cracking is a reversible process. (USGS, 1999)



Plate 5.1 Cracks in clay due to drying (km156 + 900),

It was observed from the chart in Fig. 5.1 that the plasticity index increases with the amount of fine fraction in the soils.

The plasticity index value of a particular soil can also be correlated with the linear shrinkage by the following empirical formula (Arora, 1997):

$$PI = 2.13 * LS \quad \text{-----eq 5.1}$$

Where: PI, is the plasticity index of the soil and LS, is the linear shrinkage of the soil.

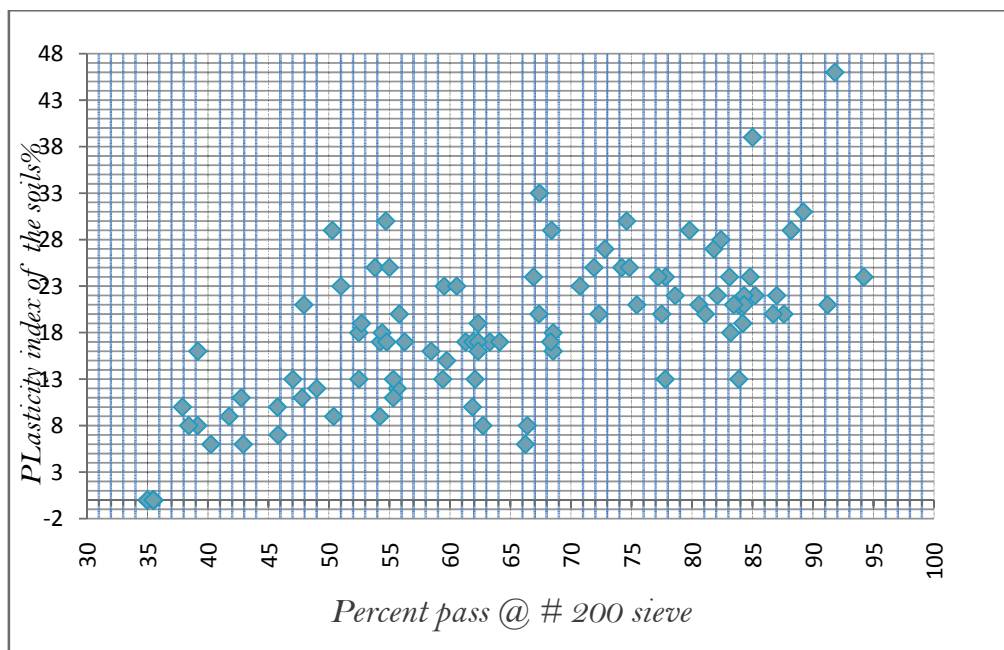


Fig 5.1 Plasticity index with percent pass @ # 200 Sieve

Seed et al (1962) proposed an empirical formula to predict the swell potential for clay soils using plasticity index value as given by equation.5.2,

$$\text{For Natural soils; } SP = 60 K [(PI)^{2.44}] \quad \text{.... eq. 5.2}$$

Where, SP is swelling potential in percent, $K = 3.6 \times 10^{-5}$, a factor for clay content between 8 and 65%, PI is plasticity index.

Thus, using eq. 5.2 the swelling potential for the tested locations ranges between 0.171% and 43.37 %. Based on the data obtained using eq. 5.2, low to medium swelling potential is

obtained for 77.68% of the sub-grade soils samples and high to very high swelling potential for 22.32% of the soils in the study area, as shown in table 5.3.

Table 5.3 Swelling potential as estimated according to Seed et al., (1962) (eq. 5.2)

Swelling potential (%)	Percent of total Samples (%)	classification of potential	Remarks
0 – 1.5	38.39	low	43 samples
1.5 - 5.00	39.29	medium	44 samples
5.00 - 25	21.43	high	24 samples
25+	0.89	very high	1 samples

Swelling potential for soaked sample described in table 4.8 (Chapter 4), 83.06% of the soils samples are categorized by low to medium swelling potential and only 16.94% of the soils have high swelling potential.

For pavement design, taking the higher value of the swelling potential may be advantageous in order to make the safest estimation. However, other parameters such as; climate, drainage and economy, should also be considered in conjunction with safety (FHWA NHI-05-037, 2006).

The climatic conditions in the study area are generally characterized by humid and warm temperature which facilitates intensive chemical weathering to a greater depth and consequently the formations of residual soils. These residual soils found in heavy rainfall tropical area are called laterites. The residual soils are typically bright red, to reddish brown in color. These are formed initially by weathering of igneous rocks with the subsequent leaching and chemical erosion due to high temperature and rainfall.

On the one hand, Bowles (1997) mentioned that well developed lateritic soils are generally porous and relatively incompressible. Bell F.G. (2007), on the other hand, mentioned that, near the surface, the LL of lateritic soils does not exceed 60% and PI value below 30%.

Drainage conditions can also be directly related with the terrain characteristics and rainfall intensity for an area. In chapter three of this thesis, it is already mentioned that most parts of the study area are characterized by rolling with localized flat terrains. In flat terrain like km 143+500 to 144+500, drainage will be the main factor for moisture fluctuations on sub grade material. According to AASHTO, Guide for design of pavement structures, (1993), quality of drainage is characterized as shown in table 5.4.

Table 5.4 Drainage conditions

S.No	Quality of draiange	Water removal time
1	Excellent	2 hrs
2	Good	1 day
3	Fair	1 week
4	Poor	One month
5	Very poor	Water will not drain

In the study area, km 143+500 to 144+500 & 146+200 to 147+600, are categorized as fair to poor drainage conditions while km 151+480 to 151+800 as poor drainage area, swampy area. However, the drainage conditions in all other sections of the study area are in excellent quality.

For these poor drainage sections, the swampy area in particular, possible remedial measure has been suggested, later in the recommendations section.

Among the collected samples, only sample taken at Km 151+600 on swampy area with LL value of 91% and PI value of 58% is visually found to be organic and later confirmed by test results. Higher value of liquid limits and lower value of Plasticity index of a soil is an indication of organic contents (Arora, 1997).

5.1.3 The Group index (GI) values

The nature of the sub-grade material can also be characterized by their Group Index Values. The Group Index characterizes the clayey nature of the soil and is calculated by equation. 4.1, already discussed in Chapter 4 of this thesis.

Eq. 4.2 (Chapter 4) is applicable for soil classes that does not belong to A-2-6 and A-2-7. For these soil classes, only partial group index is applied i.e $GI = 0.01(F - 15)(PI - 10)$.

One of the assumptions in this formula is that, when the value is negative, the group index shall be reported as zero (0). The other assumptions are already discussed in Chapter 4. Annexure-1 shows the Group Index value of each of the samples collected during field investigation in the study area.

Perusal of Annexure-1 and table 4.4 indicate that about 82.14 % of the sections in the study area show GI results less than or equal to 20 while the rest 17.86% of the section show GI

greater than 20. From the GI values, it can be concluded that except localized areas, most of the sections are considered suitable soils to be used as a road bed and embankment material under pavement layers.

Tests results obtained for the study area in Annexure-1 and table 5.1 showed that 15.18% of the sections are found to be non plastic (NP) material. Similarly, 49.11 % of the section have PI value less than or equal to 20% while the rest 35.71 % of the sections are found to have PI value of above 20%. Higher PI and GI values point out that the soil material may be too susceptible to moisture fluctuation effects (AASHTO, 2000). This is an indication that the sections of the alignment with high PI and GI values will be very poor to support the traffic load and are considered here after to be unsuitable for road bed material.

5.1.4 Density- Moisture Relationships of sub grade soils

The dry density achieved depends upon the type of soils. Cohesive soils have high air voids. These soils attained a relatively lower MDD as compared with the cohesionless soils. Such soils require more water than cohesionless soils and therefore, OMC is higher. Heavy clay of very high plasticity has very low dry density and a very high OMC (Arora, 1997).

The moisture content and plasticity index of the sub grade soils of the present study area are shown in Fig. 5.3. From this figure, it can be concluded that highly plastic material have high moisture content.

In general, coarse grained soils can be compacted to higher dry density than the fine grained soils. According to FHWA NHI -05 -037 (2006), soil compaction is one of the most important geotechnical concerns during the construction of highway pavements and related fills and embankments. Compaction improves the engineering properties of soils in many ways, including,

- Decreased compressibility, which reduces the potential for excessive long-term settlement.
- Increases strength which increases bearing capacity and decreases instability potential for slopes.
- Decreased hydraulic conductivity (permeability), which inhibits flow of water through the soil.

- Decreases void ratio, which reduces the amount of water that can be held in the soil and, thus, helps maintain desired strength and stiffness properties and increases erosion resistance.

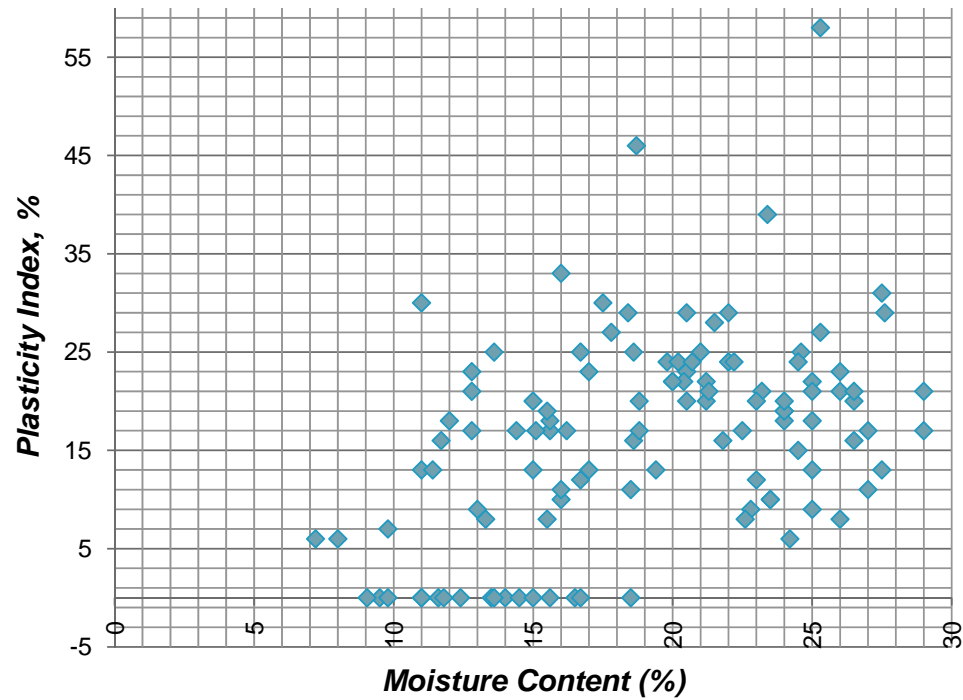


Fig 5. 2 Moisture Content with Plasticity Index Chart

Komornik et al (1969) explained about the effects of initial moisture content and dry density on swelling pressure. Swelling pressure increases with the increases of the dry density. Komornik et al., (1969) proposed the prediction method of swelling potential of soils by using empirical relation given as equation 5.4.

$$\text{Log } P_s = 2.132 + 0.0208wl + 0.00065DD - 0.0269Wn \quad \dots\dots\dots eq.5.4$$

Where; 'Ps' is the swelling pressure in kg/cm², 'Wl' is liquid Limit, (%), 'Wn' is natural moisture content (%) and 'DD' is the dry Density of soils kg/cm³.

Dry density and swelling pressure proportionally increase but inversely with natural moisture contents as can be observed from the empirical equation 5.4.

Density is usually quantified in terms of the equivalent dry unit weight of the soil as a measure of the amount of solid materials present in a unit volume. The higher the amount of

Solid material, the stronger and more stable the soil will be. Standard laboratory testing involves compacting several specimens at different water contents. The total unit weight and water content are measured for each compacted specimen, FHWA NHI -05 -037 (2006).

The maximum dry density and the plasticity index of the sub-grade soils are correlated as shown in Fig. 5.4. From the chart, it can be concluded that highly plastic materials have lower dry density and vice-versa.

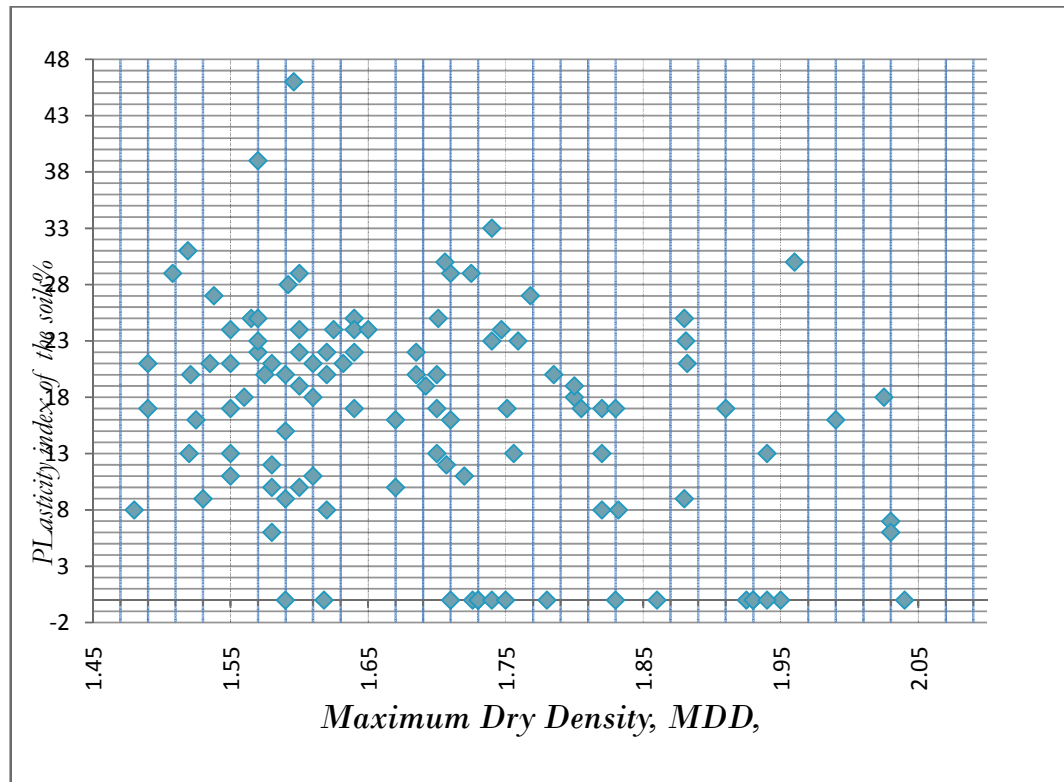


Fig 5.3 MDD VS Plasticity Index values of the sub grade soils

Perusal of Annexure-1 and Table 4.5 (Chapter four) shows the optimum moisture content and maximum dry density of the soil samples for the study area in the road project. The optimum moisture content obtained from the tests range from 7.20% to 29% where 19.01% on average. Whereas the maximum dry density ranges from 1.43 g/cc to 2.025 g/cc and an average value of 1.70g/cc. higher value of dry density are obtained in coarser materials than fine ones. Dry density is related with the voids ratio and plasticity nature of the material. Experience has also showed that sub-grade material with minimum dry density value of 1.5gm/cm² can be considered as suitable for sub-grade (road bed) material. In relation with this view, 95.54% of the sub grade soils samples do have dry density values above 1.5gm/cc.

5.1.5 California Bearing Ratio (CBR) of the sub-grade soils

Strength of the sub-grade soil material along the project road has also been determined.

The CBR values at 95% of the Modified AASHTO (T180, method D) Density have been interpolated from the CBR at densities obtained from different compaction level and by interpolating single compaction level. The test results are shown in Annexure-1. More than 90% of the CBR values of sections for the sub-grade soil in the study area are found to be above 5%, which indicate good bearing capacity as a foundation. However, in some localized area, CBR values are less than 5%. The Project Contract Document (2008) states that clay material having CBR value less than 5% is considered unsuitable as sub-grade material. Only 9.83% of the natural sub-grade soils are with CBR value less than 5% in the study area, which have poor engineering property as sub-grade material. During excavation, the sub-grade soils are again checked for vertical and lateral continuity of the soil texture in order to classify as suitable or unsuitable for the road foundation. The CBR values of coarser materials are higher than the fine material. Similarly, well graded soils attain higher values of CBR than the poorly or gap graded ones (AASHTO, 1993)

The values of CBR and plasticity nature of the sub-grade soils are displayed in Fig. 5.5. In the same figure, as the plasticity nature increases, the value of the CBR is decreasing, but near its optimum, the sub-grade soils attain better CBR values.

Since the numbers of blows proportionally increase the density of the sub grade materials, the density obtained at 65 blows should have exceeded the density at 56 blows. However, for some subgrade materials, the MDD obtained at 56 blows becomes more than the density of same materials at 65 blows in three point CBR value determinations.

Most of the above properties and their combined dominant values explain about the suitability of the soil as a sub-grade material except localized sections. The characterization results, which have been presented in previous chapter and the empirical relations in this chapter indicates that localized poor sub-grade soil needs stabilization before its intended use as a sub-grade material. This can be done by mechanical stabilization such as compaction control, cut and fill with a suitable material, and reconsidering the appropriate pavement designs that considers the unsuitability of the sub-grade soil during construction phase. However, the MDD values for sub-grade soil samples at few stations enumerated in Annexure-1 are higher than the CBR density at 65 blows.

Detail Engineering Design and contract Document preparation for Shehedi – Gelego- Guba and Gelego-Tewodros Ketema final Soils and material Report, Saba Engineering Plc (August, 2004), mentioned that as expansion pressure and potential volume change increase with the dry density of swelling soils, so it should not be attempted to densify expansive soils. Their density should not exceed 97-98% MDD (standard compaction) during road bed preparation.

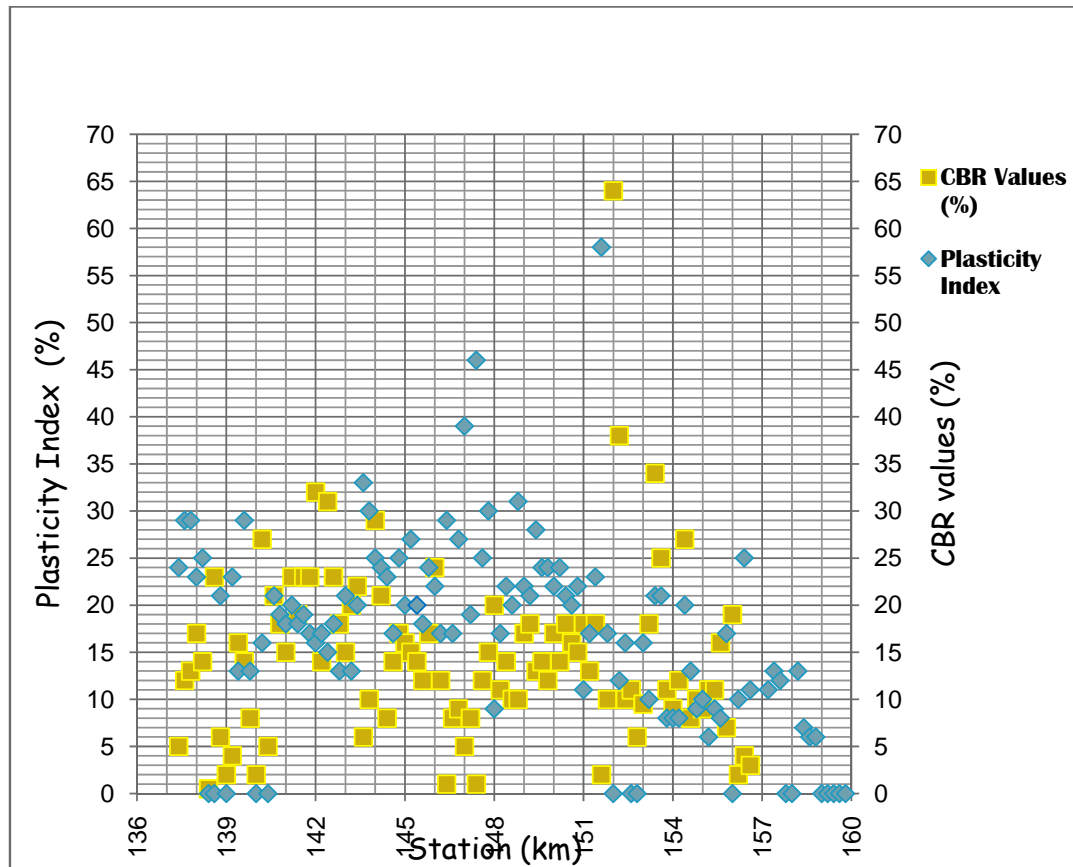


Fig 5.4 Distributions of Plasticity Index and CBR values with stations

Further, AASHTO (2003) mentioned that the sub-grade soils shall be compacted based on standard proctor (AASHTO T 99) and for expansive soils of sub-grade materials like; A-2-6, A-2-7, A-4, A-5, A-6 and A-7, the minimum compaction with standard proctor (T99) is mentioned to be 95% with $\pm 2\%$ of optimum moisture content.

Similarly, the FHWA NHI -05-037, (2006) stated that an elastic deformation and rebound of soil particles under load may contribute to shrinkage and swelling behavior, particularly in soils with flat platy particles. Test done using Mica and dune sand showed that the

consolidation and rebound of compacted mixtures are proportional to the Mica content and the contribution of elastic bending depends on particle structure and properties as shown in the Table 5.5.

Table 5.5 Mica proportion – effect on volume / void ratio.

Mica,%	Volume decrease under 10 kg/m², (142PSI),%	Increase in voids ratio upon removal of load,%
10	36	26
20	47	31
40	51	42

As mentioned under geology (Chapter three) and petrographic test results, in annexure 3, the rock units in the study area are dominated by feldspar and Mica minerals. The composition of Mica (platy) minerals in the study area for instance are; 3%, 14%, 24% & 26% for sample number 1-167+200, 2-167+200, 195+500-A, and 195+500-B, respectively. The composition of Mica minerals and the voids ratio are directly proportional after removal of the applied load. Therefore, it can be concluded from the statements mentioned above that soil rich in platy minerals such as Mica should be compacted by the standard proctor method, AASHTO T 99.

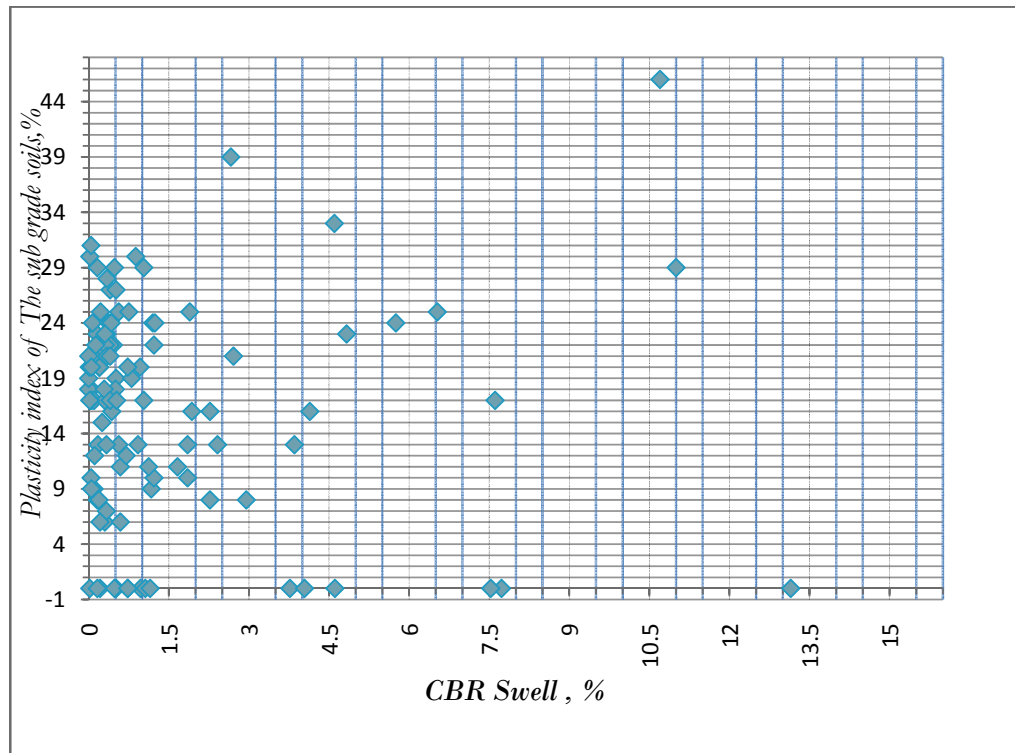


Fig 5.5 Chart for CBR Swell vs Plasticity Index

Technical specification for the contract document of Irbamoda ~ Wadera contract 2, road project (December, 2008) states that soils with swell results greater than 2 % are considered to be unsuitable as a road bed material. Accordingly, in the present study different test result analysis were made in connection to the determination of the expansiveness nature of the soil samples collected. Based on these analysis and interpretations, the degree of expansiveness of the sub-grade soils are estimated according to AASHTO T 258-81(2000), from the LL and PI test results. Even though, no test is carried out to determine the soils suction property, the degrees of expansion for sub-grade soils in the study area are classified with the help of LL and PI test results as shown in table 5.6

Table 5.6 Degree of expansions for sub grade soils (AASHTO, 2000)

S.No.	Degree of Expansions	LL (%)	PI (%)	Percent of the total samples	Soils Suctions
1	High	> 60	>35	5.36	383 kPa
2	Marginal	50 -60	25 - 35	31.25	144 to 383 kpa
3	Low	<50	<25	63.39	<144 kpa

According to AASHTO T 258-81(2000), low to marginal expansion is obtained for 94.64% of the sub-grade soils samples tested, while only 5.36% of the sample results have high degree of expansion as shown in Table 5.6. Materials with higher value of plasticity index will eventually have higher swell potential than materials with lower value plasticity indexes, seed et al (1962)

5.2 Unified Soils Classification System (USCS)

Even though, the unified soil classification system is not directly applicable to roads & highways projects, the chart is usually prepared with the help of liquid limits and plasticity index data obtained from laboratory test results. The liquid limits are plotted in the horizontal direction and plasticity index is plotted in the vertical direction. This chart has two linear lines, the “A” and the “U” lines representing the boundary conditions at which a specific soil sample can be categorized as silty or clayey.

Using a chart, test results are distributed both under and above “A” line but the entire test results are below the “U” line. Thus, based on USC, the following may be concluded;

- Silt soils are found below A line , classified and covers as follows;
 - ML or OL, 18.75 % of the total sampes
 - MH or OH, 15.18 % of the total samples
- Clays soils are found above A line, classified and covers as follows;
 - CL or OL, 50.89% of the total samples
 - CH or OH, 12.5%, of the total samples
 - CL or ML, 2.68%, of the total samples

AASHTO (M145) System of Soils Classification

It can easily be concluded from results in Annexure-1 that 18.75% of the soils are in A-4 class, 8.93% in A-5 class, 10.71% of the soil samples are in A-6 class and the rest 61.61% of the sub-grade soils along the route of the study area are mainly characterized as A-7-5 and A-7-6 soils. Thus, the general suitability of soils to be used as sub-grade based on AASHTO classification can be summarized as; A-4 and A-5 soils are generally silty soils, non-plastic or moderately plastic and highly elastic or compressible soils, respectively. A-6 and A-7 group

of soils are plastic clays that result in low and high values of liquid limit, respectively. They have high volume change properties with variation in moisture content.

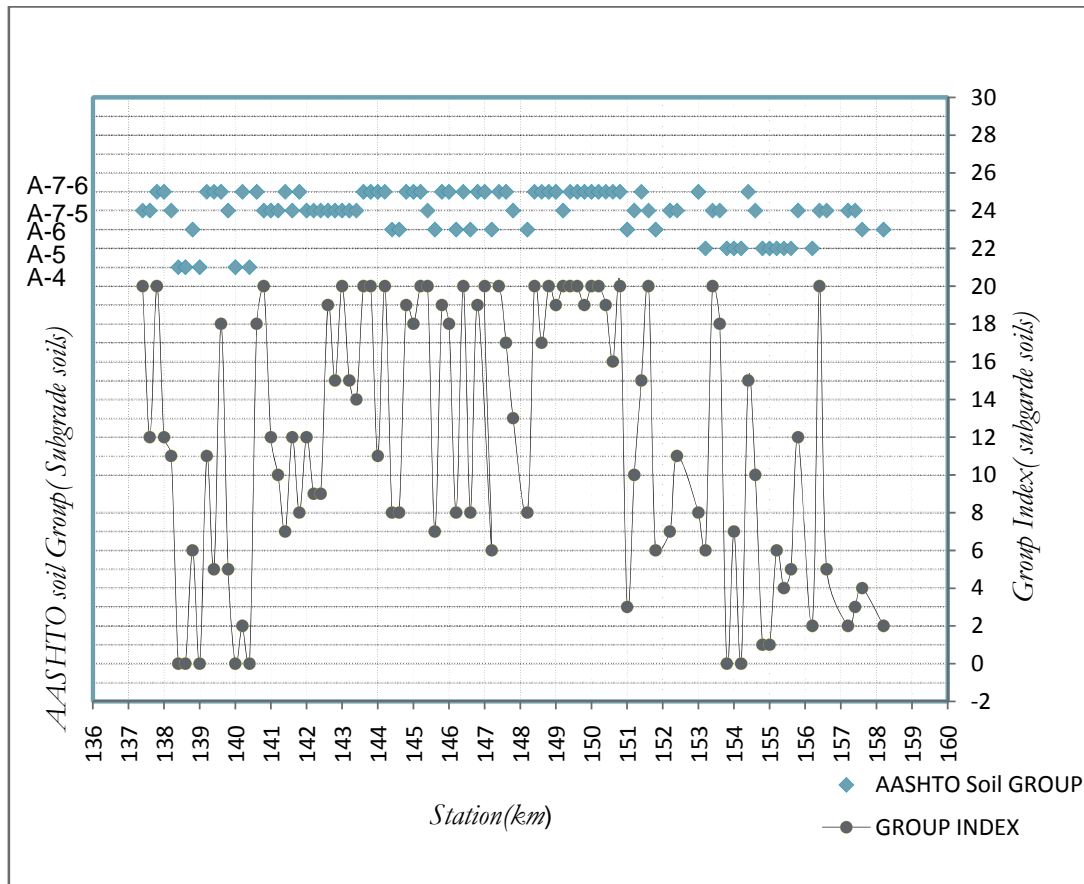


Fig 5.6 Distribution of sub-grade Soil Group (AASHTO Classification)

5.4 Casagrande plasticity chart

By adopting the chart in Fig. 5.7, it can be logically differentiated which clay mineral type is dominant in the soils along the road alignment. According to this chart, the sub-grade soils are dominantly composed of illite, kaolinite and hayllosites in deceeding order. Similarly, Bowles (1997), discussed about the clay mineral illites have PI value in the range of 30 to 50%, Kaolinites have 15 to 20% and montmorillonites have values of PI 150⁺.

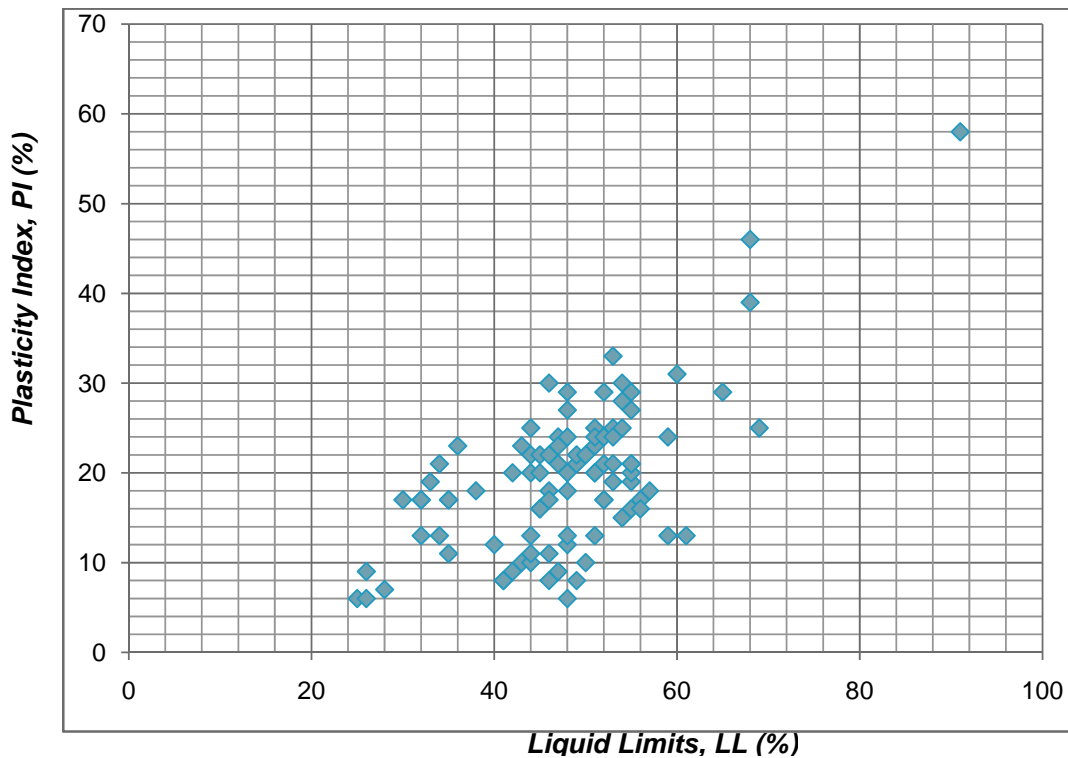


Fig. 5.7 Casagrande's LL vs PI chart for subgrade soils

5.5 General assessment of sub-grade soils of the study area based on classification Systems

- ◆ In AASHTO soils classification, we use group index values to determine the clayey nature of the sub grade soils. Group index is a combined result of gradation, liquid limits & plasticity indices of the sub grade soils. Liquid limit & plastic limit tests are carried out for fine sand, silt & clay soils, i.e the tests are conducted for materials passing (-) sieve size 0.425mm (#40 sieve). However, CBR values are determined for all materials passing sieve size of 19mm (-). These values can be overestimated depending on the type and size of materials between sieve size 19mm & 0.425mm (#40). Due to these reasons, soils classifications using the CBR values are not always reliable. The CBR swell value can also be a parameter to classify sub grade soils but the swell potential of soils are directly related with plasticity index of the same, Seed et al (1962). Among which, the group index values have been chosen most appropriate for the sub grade soils classification in AASHTO system.
- ◆ In Unified Soils Classification System, USCS, the sub grade soils are classified in to silt & clay From this graph, 2.68% sand, 33.93% silt & 63.39% clayey sub grade soils

are found. As sandy & most silty soils are non swelling, the part of soils that need precaution is the clayey soils. In the clayey soils some are swelling but others are non swelling. Hence, the sub grade soils with high liquid limits & plasticity indices are the swelling clays. They are designated by the group symbol, CH or OH covering only 12.5% of the sub grade soils in the study area.

- ◆ The casagrande plasticity chart, on the other hand, is an indirect method of identification of clay minerals from graph of the liquid limits vs plasticity indices. The minerals found in the chart from the fig. 5.7, are illites, kaolinites, and hyallosites in descending order.

Thus, based on the above mentioned three classification system the sub-grade soils of the study area may be classified as;

- ◆ The sub grade soils in the study area are classified with group index (GI) values and rated 82.14 % suitable for both the bearing stratum & construction materials.
- ◆ In the unified classification system, 87.50% of the sub grade soils are rated as suitable for both the bearing stratum and construction materials.
- ◆ Since this method is an indirect one, we can only infer from fig. 5.7 in the graph with respect to LL & PI, that 93.75% of the sub grade soil samples are suitable as supporting ground or construction materials.

Finally, when we compare the three methods of soils classifications, both AASHTO and USCS considered soils based on their texture, i.e. grain size as a parameter, but in Casagrande plasticity chart, only the mineral type are used for classifying sub-grade soils. Conversely, Casagrande plasticity chart is an indirect method to classify clays soils and will indicate the parent rock where the soils were formed, but AASHTO and USCS do not indicate about the genetic classes. Therefore, for highway projects, the combined analysis of AASHTO and Casagrande plasticity charts will provide sufficient information for textural and genetic classifications. However, the USCS classification system is intended to classify soils for general use (Arora, 1997).and may be supplemental to the other two classification methods.

5.6 Pavement structure – design consideration and review

For road design purpose, CBR is the most frequently used parameter to assess the strength of the sub-grade material. CBR is a dynamic penetration test performed on a laboratory

compacted sample and often performed after 4 days soaking to take in to account possible worst field moisture conditions and effect of shallow ground water table fluctuations (ODA, 1993). In arid (dry climate) areas, Road design manuals such as; ERA (2002) also allow consideration of CBR at the optimum moisture content.

5.6.1 Homogeneous section

For the present study purpose, the road is divided into homogenous sections and the alignment soils have been sub-divided into different sub-groups, largely based on AASHTO soils classification, and their general behavior, composition, range of laboratory test results, and extent of occurrences. For design of pavement structure and sub-grade strength category, the road sub-grade is divided in to homogenous sections solely based on CBR. The division of the road in to homogenous sections has been carried out based on the method of cumulative differences as given in AASHTO (1993) Pavement Design Guide (Appendix J).

As shown in table 5.7, after delineation of approximate homogenous sections, the 90% CBR value is also determined and the sub-grade bearing class is given by correlating with ERA Pavement design manual (2002).

The unit delineation by cumulative differences is computed using the following parameters:

1. the pavement response value, in this case, it is the CBR values
2. interval number (n) , the number values in increasing order
3. interval distance, (Δx_i), the distance between two successive CBR measurements, 200m is our interval distance
4. Cumulative interval distance ($\sum x_i$), 200,400,600m, etc.
5. The average interval response.

The detail calculation for the analysis unit delineation cumulative differences values are attached in annex 2.

The unit delineation chart is prepared only for sub-grade soils from 137+400 to 156+600, as shown in fig. 5.8. Data for the computations of the chart is attached as Annexure-2.

According to the Tanzania Pavement and materials Design Manual (1999) for homogenous Sections, the 90 % CBR value could be determined using the relation given as eq.5.5 and data thus computed for present study is presented as Annexure-1.

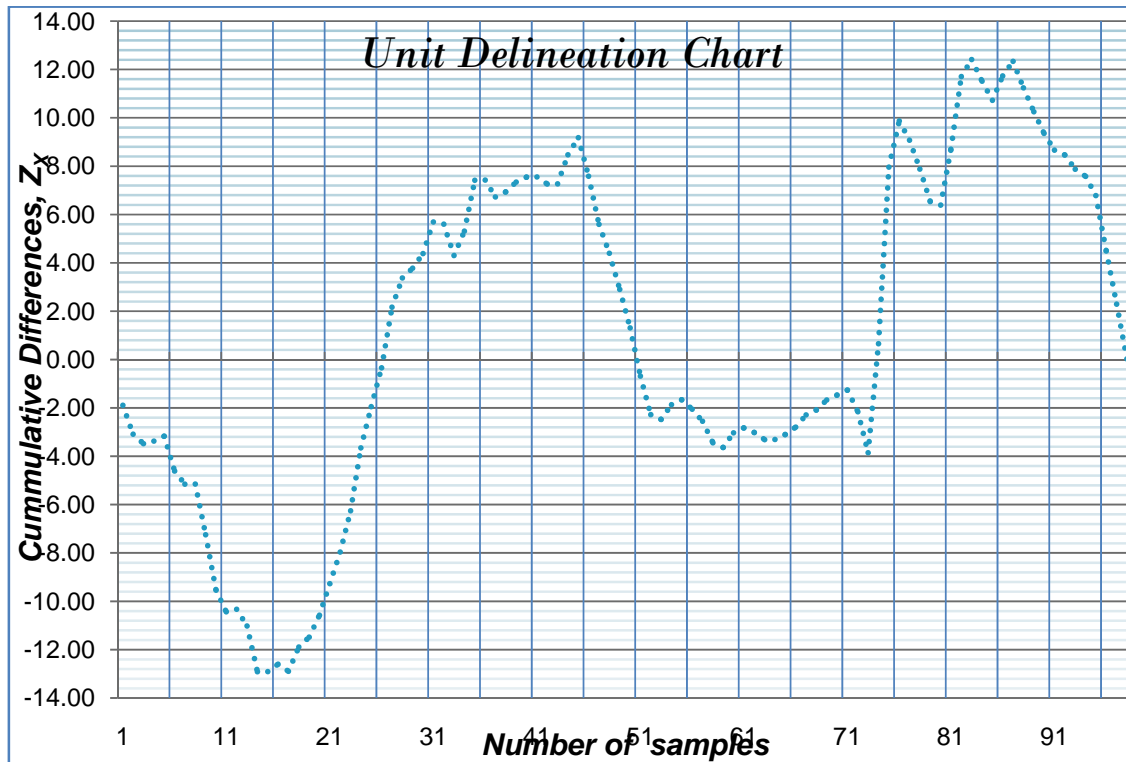


Fig. 5.8 Unit delineation for sub-grade

$$\text{Design CBR} = 1 + (n - 1)0.1 \quad \text{-----eq.5.5}$$

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

The homogenous sections were obtained 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. 5.8 where eight (8) sections are determined for the road length from km 137 + 400 to km 156+600.

Even though graphical representation are not shown, the remaining length of the road from km 157 + 200 to km 159 + 800, are also homogeneously categorized in to four (4) sections by the same methodology as the sections described in fig. 5.8.

In one homogenous section, at least three consecutive CBR values were considered and a single value that represents the Design CBR value may be determined as shown in Fig. 5.9.

5.6.2 Delineation of homogenous section

Based on AASHTO Guide for design for pavement structures (1993), the following Twelve (12) homogenous sections are identified from fig. 5.8 with analysis unit delineation by cumulative differences. Using the unit delineation chart, values having main slope changes have been considered as homogenous sections. Accordingly, those homogenous sections identified from the chart in Fig. 5.8 are summarized in table 5.7. For CBR values, 10,11,12,19 & 25, the design CBR is determined as follows in fig. 5.9 below.

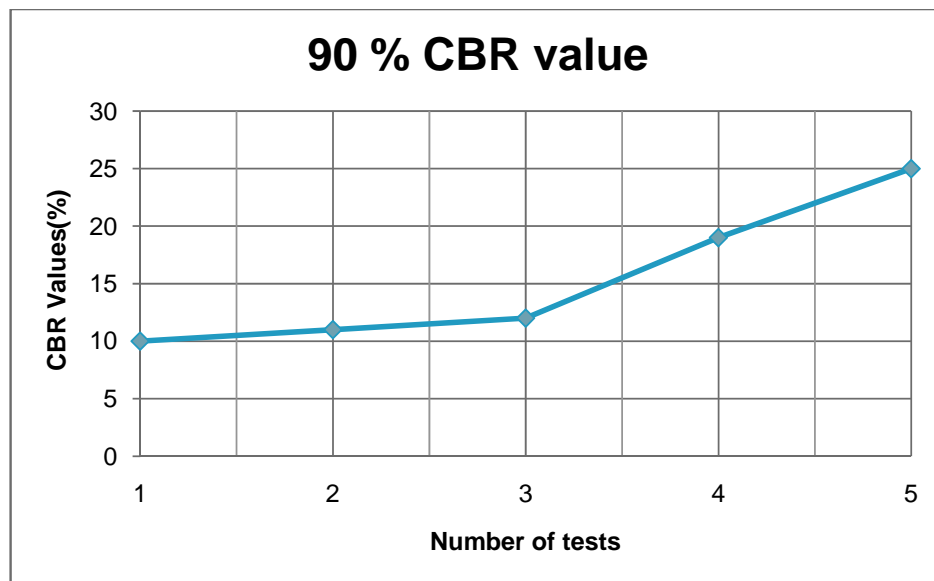


Fig. 5.9 90%-ile, Design CBR graph

Therefore, Using equation 5.5, the CBR shall be calculated as follows:

$$= 1 + (5-1)*0.1$$

$$= 1.4$$

CBR = 11.40% (from fig. 5.9)

Table 5.7 Identified Homogeneous sections

Homogeneous Sections (HS)	Chainage	CBR values	90% value	Sub-grade strength classes
HS1	137+400 - 140+400	5,12,13,17,14,0.5,23,6,2,4,16,14,8,2,27,5	5	S3
HS2	140+600 - 143+400	5,21,18,15,23,19,23,23,32,14,31,23,18,15,20	15	S5
HS3	143+400 - 146+400	22,6,10,29,21,8,14,17,16,15,14,12,17,24,12,1	8.30	S4
HS4	146+200 - 147+800	1,8,9,5,8,1	6.5	S3
HS5	147+800 – 151+800	12,15,20,11,14,10,10,17,18,13,14,12,17,14,18,16,15,18,13,18,2,10	10	S4
HS6	151+800 – 152+400	10,64,38,10	10.30	S4
HS7	152+400 – 154+600	11,6,9.5,18,34,25,11,9,12,27,8	6	S3
HS8	154+600 – 156+600	10,9,11,11,16,7,19,2,4,3	7.30	S4
HS9	157+200 – 158+200	20,5,11,15,16	7.4	S4
HS10	158+400 – 158+800	11,32,25,	13.8	S4
HS11	159+000 – 159+400	10,19,11,	10.40	S4
HS12	159+400 - 159+800	12,35,34	17.6	S5

BKS Group in association with BEZA consulting Engineers conducted the Design review for the Aposto ~ Wondo ~ Negelle Road Upgrading project in the year 2006 and the strength of the alignment sub-grade soils classes, Traffic classes and pavement structures; the results are presented in table 5.8.

Table 5.8 Strength of the alignment sub-grade soils classes, Traffic classes and pavement Structures of Aposto~Wondo~ Negelle Road upgrading project

Section		Traffic class	Soil class	Pavement structure (mm)			
From	To			Surfacing	Base	Sub base	Capping
92.2	120.5	T4	S2	50	175	200	232
120.5	147.60	T4	S3	50	175	200	96
147.6	155.70	T4	S4	50	175	225	128
155.70	173.5	T5	S3	50	175	200	64
173.5	184.5	T5	S3	50	175	225	128
184.5	201.40	T5	S2	50	175	200	296

(Source: BKS Group in association with BEZA consulting Engineers - the Design review, 2008)

The minimum CBR value for the sub-grade soils in the present research is taken as 5%. 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, the construction of Aposto ~ Wondo~ Negele Road upgrading project, Contract 2, Irbamoda ~ Wadera Road. Comparison of the sub grade soil classes between the Design Review consultant and present research findings is shown in table 5.9

Table 5.9 Comparison, the soils strength classes by the Design review consultant With the findings of the study

HS* By the Design consultant			Traffic classes	HS* by the Study		
From	To	Soil classes		soil classes	From	To
120.50	147.60	S3	T4	S3	137+400	140+600
		S3	T4	S3	146+200	147+800
		S4	T4	S3	152+400	154+600
147.60	155.70	S4	T4	S4	147+800	151+800
155.70	173.50	S3	T5	S4	154+600	159+400
		S3	T4	S4	143+400	146+200
		S4	T4	S4	151+800	152+400
		S3	T4	S5	140+600	143+400
155.70	173.5	S3	T5	S5	159+400	159+800

* HS = Homogenous sections

The CBR values of the following stations are, extracted from the homogenous sections, below the minimum requirements, i.e. 5%

- (i) 38+300 to 138+500
- (ii) 138 + 900 to 139+300
- (iii) 139 + 900 to 140 + 100
- (iv) 146+300 to 146+500
- (v) 147+300 to 147+ 500
- (vi) 151 +500 to 151 + 700
- (vii) 156+100 to 156+700

The Project Technical Specification (2008) stated that for unsuitable sub-grade soils, removal will be made up to 600 mm depth below the design grade and replaced with locally available suitable material. The material to be used for replacement must have CBR values greater than 5%, PI not exceeding 20% and swell values less than 1.5%. This is called providing granular bed and cover below and around the foundation. The method is implemented for eliminating of swelling of material near the bottom (invert) of the foundation.

According to ERA pavement design manual (2002), material with minimum CBR value of 5% is an 'S₃' type of sub-grade soil class which will be provided to eliminate the swelling of the sub-grade material. Swelling of clay soils are inversely related with depth of the structure but swelling potential of clay soils are proportional to plasticity index of the same (Chen 1988).

5.6.3 Pavement thickness determinations

The asphalt surfacing thickness is maintained constant for the entire length of the road as per the design review consultant, as shown in table 5.8. However, depending on the soil classes and traffic flow rate, the base course, sub- base and capping (improved sub-grade) layer thicknesses are variable. Taking the traffic density analysis done by design consultant in to account, thicknesses of the base course, sub base and capping layers are determined as shown in table 5.11. The pavement thickness selection is based on the combined system of Atkins (1983) & ERA, Design for flexible pavement, structural catalog (2002). The ERA design catalog in the manual offers eight different charts for alternative pavement structures using combinations of traffic and sub-grade classes.

The pavement structure has thickness variation both on, base, the sub-base and capping layers as compared with the thickness determined by the design review consultant (Fig. 5.8) Therefore; the total quantities of works will also vary for each increment in thickness for the layers.

5.7 Difference between the consultant design review & this study:

The thickness of the capping layer (selected improved sub-grade) falls in the range of 64mm, 96mm and 128mm. These thicknesses have practical inconveniences in quality controlling such as density checking. Usually, density checking for approval is taken at a depth in the range from 150mm to 200mm. It is also mentioned by ERA (2002), that the minimum practical layer thickness for an earth work is 100 mm and above with the coarser aggregate nominal size is 2/3 of this layer thickness.

However, the thickness of the capping layers determined by the design consultant will only be applicable for the following two conditions:

- (i) When the quality of the fill materials below it is same or
- (ii) When the materials above the capping layer (sub base) has uniform quality.

The general principle of a pavement structure is that the quality of material is increasing from bottom to top. More quality material is required at the top where the load imposed by traffic is highly concentrated. The intensity of load distribution will decrease from top to bottom and as per ERA (2002) at 1000 mm from the finished surface, the imposed traffic load will dissipate. Therefore, the pavement material may not be uniform from the angle of load distribution and bearing capacity sufficiency point of view.

It is specified in project technical specification that the design review consultant put the following compaction level for all earth work and pavement structures.

Sub-grade material	95%
Embankment material	95%
Improved sub-grade material	95%
Sub-base material	97%
Base course material	98%

The design review consultant has decided the requirement for compaction of sub-grade layers at 95%. In TRL, overseas Road note, 31 (1993), it is mentioned that the top 250 mm of all sub-grade should be compacted during construction to a relative density of at least 100% of MDD in the British standard (light) compaction test, 2.5 kg rammer method or at least 93% of MDD in British standard (Heavy) compaction test using the 4.5kg rammer. In addition the traffic classes also range from T₃ to T₅, i.e up to 6* 10⁶ Esa., the designing consultant could have decided the 93% of reference compaction (Density) for the sub-grade (road Bed) layer. For the following two main reasons:

1. The traffic classes are found in the intermediate category
2. The sub grade materials are dominated by illites (Micaeous soils) as mentioned in FHWA, NHI 05-037-2006, concerning the elastic rebound effects.

Eventually this could save time and compactive effort required to attain the minimum compaction.

The improved sub-grade layer (capping layer) is constructed as part of the earth work structure and is required to create homogenous layer throughout the entire length of the road. The embankment (ordinary fill) layer is constructed with better quality of material than the road bed (sub grade), usually lower quality than the improved sub-grade layer. The intensity and concentration of the load distribution is also variable from top down to the bottom.

In the contract document, it is clearly mentioned that material with minimum CBR value of 15% are required for the works of improved sub-grade (capping layer). When the in situ sub-grade material (native sub-grades) attain minimum CBR value of 15%, the subsequent fill layer can be omitted and the lower pavement structure layer may be placed directly after treating the top 200 mm thickness of the sub-grade by ripping, removing all loose and oversize material from the surface and compacting to the minimum density requirement. In this regard no clear statement is made from the design revising consultant.

In high fill areas with minimum fill height of 600mm & where both improved sub-grade and embankment layers are proposed, CBR values as low as 2% may be taken as suitable sub-grade materials. This is because, the load will dissipate before reaching the sub grade layer & moisture fluctuation will proportionally being reducing with depth.

In pavement structure, the material quality is becoming superior from bottom to top and so are the traffic load intensity and distribution. The required level of compaction is also higher

at top than the layer below it or bottom. To achieve this quality parameters and level of compaction, the costs of construction are also variable and increases from bottom to top and so is the tolerances limits. Eventually, it becomes more expensive when it reaches at the asphalt level & this can be supported with the data obtained from the Contract document for the captioned project as shown in table 5.10.

The unit rate per m³ for each item of work is shown in the table 5.10 just for the sake of comparison and ease of evaluation. These rates are extracted from Project Technical specifications (2008).

Table 5.10 the unit rate for the items of work (Project Technical specifications 2008)

S.No.	Pay Items no	Description of materials	Unit	Rate (Birr)
1	41.03(b)	Road bed preparation	M ³	31
2	42.01(a)	Embankment (ordinary Fill)	M ³	70
	42.01(c)	Rock fill	M ³	198
3	44.05	Selected sub grade (capping layer)	M ³	125
4	51.01(a)	Sub base (natural)	M ³	135
5	51.01(c)	sub base (Crushed)	M ³	250
6	52.01(b)	Base courses	M ³	250
7	64.02(a)	Hot mix asphalt (50mm thick)	M ²	167.20

5.8 Proposed Thickness for the Pavement Structure

In ERA structural catalog, chart 1 & 3, granular road base, the substitution ratio (thickness equivalency) for sub base to improved sub grade layer is 25:32. Up to 100mm of sub base may be substituted with selected fill provided the sub base is not reduced to less than the road base thickness or 200mm whichever is greater.

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

2 inch (mm) base	=	1 inch (mm) asphalt
2.7 inch (mm) sub base	=	1 inch (mm) (asphalt)

Therefore, base to sub base substitution ratio will be, 1 inch (mm) = 1.35 inch (mm)

From the structural catalog, base layer is defined to be 175mm for S₃ & T₄ soil strength & traffic classes. In addition, ERA (2002) it is pointed out that the minimum thickness of work for a layer in pavement structure is 100mm. Combining Atkins's structural thickness equivalency with practical workability point of view; base layer thickness is reduced to 150mm, which is more than ERA (2002) minimum lift thickness and the sub base layer to be 175mm. 100mm thick layer sub base & 25mm thick base is substituted with improved sub grade layer using ERA (2002) sub base to improved sub grade substitution ratio.

Then, base layer is defined to be 150mm, sub base 175mm & capping 170mm for S₃ soil strength & T₄ traffic classes.

The following problems are noticed in the project area with regard to quality, availability of material and workmanships:

- (i) Natural sub-base materials are scarce in the route corridor
- (ii) The sub-base material have more quality requirements than capping material
- (iii) The availability of capping material is comparatively easier along the route corridor than the sub-base materials.
- (iv) Using crushed sub-base is more expensive, from the angle of processing and transportations costs
- (v) Rain fall frequency in the study area is more and the sub-grade soil moisture is ever changing
- (vi) In most of the sections of the pavement structure, the thickness of the capping material is less than the usual thickness and thus, workability is difficult
- (vii) Since the area is categorized in the seasonally wet tropical regions, relaxation of the Plasticity index for the natural sub-base is impractical. TRL31, (1993)

Advantages of the proposed pavement (structural) thickness (table 5.11) as per the present research study are:

- (i) Decreases the thickness of the sub-base and increases (selected sub grade (capping layers).
- (ii) The total thickness of the base course & sub-base layer will decrease and so is the cost.
- (iii) Since the availability of capping material is better in the route corridor, transportation and processing costs for base course & sub base will proportionally be minimized.

Thus, considering construction costs, serviceability index & workability, the following thicknesses have been proposed for each pavement structure.

Table 5.11 proposed pavement structure thickness

Traffic classes	soil strength classes	Homogenous sections by this Research		Proposed Pavement structure (mm)		
	research findings	From	To	base course	Sub base	capping layer
By design Review Consultant						
T4	S3	137+400	140+600	150	175	170
T4	S3	146+200	147+800	150	175	170
T4	S3	152+400	154+600	150	175	170
T5	S4	147+800	151+800	150	175	150
T5	S4	154+600	159+400	150	175	150
T4	S4	143+400	146+200	150	175	150
T4	S4	151+800	152+400	150	175	150
T4	S5	140+600	143+400	150	175	-
T5	S5	159+400	159+800	150	175	-

- (iv) As the area is categorized in seasonally wet tropical region, the moisture fluctuation will be minimized due to the increment in total thickness of the capping layer, and eventually the vertical alignment will also be changed.
- (v) Crushed sub-base utilizations will be reduced parallel with the sub-base quantity.
- (vi) Workability problems in the capping layers due to small thickness will be avoided by increasing the capping layer thickness.

Table 5.12 Comparison of the two pavement structures

Pavement structure By Design Review			From	To	Proposed Pavement Structure		
base course	sub base	Capping layer			base course	sub base	Capping layer
175	200	96	137+400	140+600	150	175	170
175	200	96	146+200	147+800	150	175	170

175	200	96	152+400	154+600	150	175	170
175	225	128	147+800	151+800	150	175	150
175	200	64	154+600	159+400	150	175	150
175	200	64	143+400	146+200	150	175	150
175	225	125	151+800	152+400	150	175	150
175	200	64	140+600	143+400	150	175	-
175	200	64	159+400	159+800	150	175	-

5.9 Overall characterization of sub-grade and pavement structure design review

When the percent pass of the sub grade materials are higher, the swell values as well as the plasticity index will increase. The CBR and MDD values also are becoming higher for coarse grained materials. As explained by seed et al (1962), Swell potential and Plasticity index are linearly increasing. Similarly, higher values of group index & moisture content indicate the unsuitability of the materials for bearing stratum as well as construction materials. Thus, high plasticity index, liquid limits, group index values & moisture content will proof the general poor behavior of the materials. On the other hand, high dry density, CBR values and coarse grained texture shows that the suitability of the materials for engineering uses.

For pavement structure design, materials with higher value of CBR, low plasticity & swell value will require less thickness for each layer & vice versa.

Chapter Six: CONCLUSION AND RECOMMENDATIONS

6.1 Conclusions

The present study was carried out on Aposto ~ Wondo ~ Negelle Road upgrading project, Contract 2: Irbamoda Road that links the Town of Aposto and Negele via KibreMengist. The study area is found in Adola Rede woreda, Gujji Administrative zone of Oromia Regional state. The road project lies on the Hawassa - Kibre Mengist - Negelle Borena main road and starts at IrbaModa town, roughly 414km from Addis Ababa and ends after 522.5km at Wadera town.

The road project lies on the basement complex of the Adola belt that consists of different metamorphic lithologic units such as schist, quartzo feldspathic & boitite gneiss, amphibolites, graphite hornblende gneiss, & mica schist etc.

The main objective of the present study was to characterize the sub-grade soil for its general suitability for pavement design and as a construction material. Besides, a pavement structure review was also made to know the appropriateness of the pavement thickness determined by the Design Review Consultant.

In order to achieve the objectives of the present study a systematic methodology was followed. A total of 112 sub-grade soil samples were taken at 200 m interval and tests were conducted at project geotechnical field laboratory for the determination of Atterberg limits, grading, and MDD, OMC and CBR and swell values. Further, analyses have been made by integrating the primary results with secondary data obtained. Thus, based on the test results interpretations were made to meet the general objectives of the present study.

The test results indicated that 94.64% of soils possess Liquid limit having value less than or equal to 60%, whereas only 5.36% soils possess liquid limit values higher than 60%. Similarly, 35.71 % of soils possess PI having value greater than 30%, whereas 49.11 % soils possess PI values less than or equal to 30%. About 15.18% of soils are non plastic. The swelling potential for the sub-grade soils ranges between 0.171% and 43.37 %. About 77.68% of sub grade soils possess low to medium swelling potential and 22.32% of the soils show high to very high swelling potential. Further, the maximum dry density for sub-grade soil samples varies between 2.43 and 2.025gm/cc. Similarly, the moisture content varies between 7.2% and 29% & the CBR values for the sub grade materials range from 0.5% to 64%.

In addition, four empirical relations by Seed et al., (1962), Komornik et al., (1969), Chen (1988) and AASHTO M145 (2004), were utilized for predicting swelling potential, effects of moisture content on dry density and swelling pressure for unsuitable sub-grade soils. The geological formations and associated structures are also reviewed to interpret the geotechnical properties of the residual sub-grade soils in the project area.

Based on the analysis and interpretations of the test results, the sub-grade soils are classified as CL or OL, MH or OH, in the USCS; A-7-5, A-7-6, A-6, A-5 and A-4 on the AASHTO classification system. Further, the sub-grade soils are mainly composed of illites and kaolinites as per the Casagrande plasticity charts. This clearly indicates that the sub-grade soils in localized sections possess unsuitable engineering characteristics.

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

In order to give appropriate stabilization measures, the sub-grade material in the study area are categorized in to the following sub-units:

- (i) unsuitable sub-grade material, whose CBR values at 95% of MDD is less than 5%, maximum plasticity index of 30%, Maximum CBR swell values of 2%
- (ii) non plastic materials, whose plasticity index is 0, non swell and CBR values less (more) than the minimum required 5%
- (iii) Swampy areas.
- (iv) Rocky areas.

For design of pavement structure and sub-grade strength category, the road sub-grade is divided in to homogenous sections solely based on CBR. The division of the road in to homogenous sections has been carried out based on the method of cumulative differences as given in AASHTO (1993) Pavement Design Guide (Appendix J). Twelve (12) homogenous sections have been identified & the design CBR is computed using the Tanzania Pavement design manual approach.

After a thorough review and evaluation of the existing pavement structure thickness and (working) drawing, an alternative pavement structure design is also proposed. The proposed

pavement structure revised the thickness of the base course, sub base, and capping layers. This thickness determination is made by the combined methods of Atkins & ERA standards. The sub base to capping materials ratio is taken from ERA (2002) while the Base course to asphalt, & sub base to asphalt ratio are from Atkins proposal. The proposed thickness has advantages from workability, economic & safety point of view. The newly proposed pavement structure excludes the asphalt layer thickness.

The main advantages of the pavement structure thickness review and proposal over the approved pavement structure are:

- (i) Increases the thickness of selected sub grade (capping) layers.
- (ii) The total thickness as well as the cost of the base course and sub-base layer will decrease.
- (iii) Transportation and processing costs for base course and sub-base material will proportionally be minimized.
- (iv) The moisture fluctuation on the bottom of the sub-grade will be minimized due to the increment of the total thickness of the selected sub-grade (capping) layer,
- (v) Crushed sub-base utilizations will be reduced parallel with the sub-base quantity.
- (vi) Workability problems in the capping layers due to small thickness will be avoided

In general, except localized section, more than 80% of the sub-grade soils in the route corridor are suitable as bearing stratum as well as construction materials. On the other hand, the newly proposed pavement structure thicknesses are also determined in a reasonably site specific approach.

6.2 Recommendations

Based on the geotechnical characterization of sub-grade soils in general and for the subject road in particular, the above mentioned conclusions have been made. Based on the conclusion made above, the appropriate recommendations for Aposto ~ Wondo ~ Negele Road Upgrading, Contract 2: Irbamoda road are given here under.

- (i) For unsuitable sub-grade soils, whose CBR values at 95% of MDD is below 5%, maximum CBR Swell values of 2% and maximum plasticity index of 30%, it is recommended to excavate down up to 600 mm depth and replace it with a plastic non-expansive soil having a minimum CBR of 5%, 2% swell and PI between maximum 20 % which is equivalent to an S₃ type material indicated as per ERA, 2002. The work

shall be executed in three layers of equal thickness and must be compacted to the required minimum density.

- Side drains in such sections should be avoided or if this is not possible, they should be as shallow as possible and located as far away as practicable from the toe of the embankments. This helps to avoid infiltration of surface waters down the sub-grade layer.
 - The swelling potential and swelling pressure of expansive clays increase with increase in dry density as high as to limits of the volumetric shrinkages. From Casagrande plasticity chart, it is identified that, the dominant clay minerals in the soils are the illite groups whose compositions are rich in mica, are susceptible for elastic rebound effect. Therefore, the degree of compaction will have to be kept on the lower side, preferably 98 -100 % MDD (T99) or 93% in T180.
 - In addition, the road bed of the expansive clays should be kept moist during road bed preparation and should be covered by the appropriate fill/improved sub-grade without undue delays; culverts and drainage pipes shall not be directly laid on expansive soils; trees should not be planted and allowed to grow near the road.
 - The potential volume change decreases with increase in initial moisture content. Hence, it is good practice to maintain higher initial moisture content (on the higher side of the optimum moisture content, (2–3% above OMC). Also layers of expansive soil should not be left exposed to dry out; they should quickly be covered with the next layer. Dried surfaces should be allowed to moisten by applying water before carrying out subsequent construction on them.
 - Differential movement of the road surface under culvert structures could arise, as surcharge load on the sub-grade is lighter at culvert sites. Hence, culverts must not be cast and/or laid directly against expansive soils. The surrounds or side of the culvert structure must be placed with material of non-swelling, stable, impermeable and preferably gravely material.
- (ii) For Non plastic materials with 0 values of liquid limits and plasticity index: these materials are called cohesionless soils. Embankment and sub grade material need some cohesion to bind and fill those voids between aggregates. These materials need blending with plastic soils in order to incorporate into the pavement structures. The blending

ratio may depend on the plasticity index values of the imported and excavated materials on the road way that should not exceed from the plasticity index values of material intended for the ordinary fill (embankment) material.

- (iii) For Sub-grade on swampy areas; for poor drainage areas such as; in the flat terrains like km 144 + 000 to km 144 +500 & 151 +480 to 151 + 800, hard and sound rock fragments shall be laid and foundation drains such as perforated pipes encased with granular filter material. The drain lines shall be installed within the body of the rock fill but at a reasonable depth below the top of rock fill layer to avoid seasonal moisture changes in overlying sub-grade (embankment) materials. The granular filter material shall be again encased with geosynthetic material. The purpose of the geotextile material is just to separate the mixing of the overlying embankment material from the granular filter layer and allow only water movement on both directions. The top grade of the rock fill material shall be separated from the overlying embankment layer by geotextile material. The geotextile material will protect the intrusion of the overlying materials and ingress of the underlain rocks fragments.
- (iv) Sub-grade on rock foundations; in places where hard rock strata are encountered (overlap) with design grade of the sub grade layer or road bed. As mentioned in ERA 2002, the surface of the rock shall be excavated down to 500 mm minimum depth and replaced with suitable material equal to the S₃ and compacted in three equal layers. The roughened (excavated) rock surface may accumulate water and eventually create moisture variation on the overlying sub-grade layer, if the top of the rock surface is flat lying. In such cases, filter drain like in the swampy area shall be implemented.
- (v) For sections of the road alignment where sub-grade material having CBR values at 95% of MDD are more than 15%: its value for swell less than 1.5% and plasticity index of maximum 15%: can be taken as the level of improved sub-grade (capping) layer and other earth work parts intended to construct may be omitted and the sub-base layer can directly be put on the native sub-grade layer.

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



**Geological Survey of Ethiopia
Geosciences Laboratory Center
Result Form**

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

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

Client /Originator Name: - Ahmet Aydemir (Wondemeneh Fekadu)

Client Category: - Survey Gov. Pvt.

File name: - 14912/10PVT Area Ref:- No of Samples: - 4 Sample No. 2-167+200

Sample Type:-Rock Lab No: - 14913/10

Type of Analysis:-Petrography Preparation required: - Thin section Date Submitted: - 22/09/2010

I) Hand specimen Description: Grayish Pink in color & medium to fine grained in texture

II) Mineral composition

Mineral	Modal (%)	Texture
K-feldspar(Microcline)	53	Xenoblastic
Quartz	20	Xenoblastic
Biotite	12	Platy
Opaque(Fe-oxide)	10	Idio-Xenoblastic
Calcite	1	Xenoblastic
Muscovite	1	Platy
Chlorite	1	Platy
Apatite	1	Idio-Xenoblastic
Epidote	1	Xenoblastic
Zircon	Trace	Idioblastic
Sphene	Trace	Idioblastic



III) Textural Descriptions / Notes: Gneissose texture

Matrix of K-Feldspar (Microcline), quartz, biotite and opaque is strongly stretched and has parallel alignment. Discontinuous veins of quartz and k-feldspar are seen across the section. Opaque minerals are evenly distributed all over the section. Some grains of biotite are replaced by chlorite and muscovite.

IV) Rock Name: - Biotite Gneiss

Described By / Analysts	Checked by	Date Completed
1. Workelul G/K 2. Adise Mekonnen	<i>(Signature)</i> Workelul G/Kirstos	08/10/2010

Mineralogy & Physical Analysis
Case Team Co-ordinator



**Geological Survey of Ethiopia
Geosciences Laboratory Center
Result Form**

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

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

Client /Originator Name: - Ahmet Aydemir (Wondemeneh Fekadu)

Client Category: - Survey Gov. Pvt.

File name: - 14912/10PVT Area Ref:- No of Samples: - 4 Sample No. 1-167+200

Sample Type:-Rock Lab No: - 14912/10

Type of Analysis:-Petrography Preparation required: - Thin section Date Submitted: - 22/09/2010

I) Hand specimen Description: Grayish Pink in color & medium to fine grained in texture

II) Mineral composition

Mineral	Modal (%)	Texture
K-feldspar(Microcline)	62	Xenoblastic
Quartz	25	Xenoblastic
Calcite	6	Xenoblastic
Muscovite	3	Platy
Opaque(Fe-oxide)	4	Idio-Xenoblastic



III) Textural Descriptions / Notes: Gneissose texture

Matrix of xenoblastic K-Feldspar (Microcline), quartz, and calcite is strongly stretched and has parallel alignment. Vein filled by large crystals of quartz, opaque and calcite is seen across the section.

IV) Rock Name: - Alkali Gneiss

Described By / Analysts: 1. Workelul G/K, 2. Adise Mekonnen

Checked by: Workelul G/Kirstos

Date Completed: 08/10/2010

Mineralogy & Physical Analysis

Case Team Co-ordinator

Client		Contractor	
Ethiopian Roads Authority, ERA		Ahmet Aydenis - KMC (JV)	
Consultant		Contractor	
Grontmij/Carl Ibro in Association with Gondwana Engineering, PLC		Ahmet Aydenis - KMC (JV)	
Test Result for Sub grade soils in the stations from km 137+400 to 159+800			
Testing Location: Kibre Mengist Site Laboratory			

Annex 1

S/N	Sample Location	Visual Description	Lab. No	Depth Sampled (m)	Laboratory Tests Result											Remark			
					2.00mm	0.425mm	0.075mm	Soil Class	LL, %	PL, %	PI, %	MDD	OMC, %	CBR, Dry Density			Steell at 65 blows		
														10	30			65	
1	137+400	brownish lateritic soils	A/LAB-SE-0086		99.2	97.7	83.1	A-7.5(24)	59	25	24	1.625	22	1.405	1.563	1.65	5	5.75	Higher swell value
2	137+600	reddish brown silty clay	A/LAB-SE-0096	0.6	93.98	75.95	50.28	A-7.5(12)	65	26	29	1.600	22	1.296	1.526	1.663	12	1.03	
3	137+800	brownish silty clay	A/LAB-SE-0079		99.69	97.15	88.19	A-7.6(29)	55	26	29	1.508	27.6	1.316	1.489	1.542	13	0.48	
4	138+000	reddish brown silty clay	A/LAB-SE-0078		80.49	72.23	59.52	A-7.6(12)	51	28	23	1.74	20.5	1.569	1.694	1.735	17	0.15	
5	138+200	reddish brown silty clay	A/LAB-SE-0077		91.57	81.91	53.8	A-7.5(11)	54	29	25	1.64	21	1.276	1.514	1.637	14	1.89	Higher swell value
6	138+400	Yellowish decomposed	A/LAB-SE-0080				>35	A-4			NP	1.618	14	1.416	1.528	1.629	0.5	13.15	Higher swell value
7	138+600	Brown Micaceous silty	A/LAB-SE-0075				>35	A-4			NP	1.93	9.5	1.674	1.828	1.912	23	0.02	
8	138+800	Yellowish decomposed	A/LAB-SE-0074		90.09	76.98	47.94	A-6(6)	34	13	21	1.882	12.8	1.569	1.775	1.888	6	2.71	Higher swell value
9	139+000	whitish decomposed	A/LAB-SE-0073				>35	A-4			NP	1.59	16.5	1.298	1.483	1.565	2	4.61	Higher swell value
10	139+200	Yellowish brown	A/LAB-SE-0072		83.86	73.23	60.56	A-7.6(11)	43	20	23	1.759	17	1.518	1.677	1.699	4	4.83	Higher swell value
11	139+400	dark brown silty clay	A/LAB-SE-0071		97.66	88.54	59.42	A-7.6(5)	32	19	13	1.82	15	1.601	1.764	1.851	16	0.92	
12	139+600	yellowish brown silty	A/LAB-SE-0070		95.21	88.83	68.39	A-7.6(18)	48	19	29	1.71	20.5	1.517	1.679	1.72	14	0.16	
13	139+800	micaceous clayey silt	A/LAB-SE-0069		93.91	79.69	52.49	A-7.5(5)	44	31	13	1.756	17	1.524	1.737	1.763	8	2.41	Higher swell value
14	140+000	reddish brown micaceous clay	A/LAB-SE-0068				>35	A-4			NP	1.726	14.5	1.363	1.593	1.735	2	7.73	Higher swell value
15	140+200	reddish brown silty clay	A/LAB-SE-0067		83.12	50.13	39.17	A-7.6(2)	45	29	16	1.99	11.7	1.748	1.906	1.978	27	0.43	Higher swell value
16	140+400	pinkish micaceous with	A/LAB-SE-0066				>35	A-4			NP	1.78	15.6	1.502	1.694	1.813	5	3.77	Higher swell value



Aposto Wendo Negele Road Upgrading Project	
Contract 2: Irbomoda Wadera Road Construction	
Client	Contractor
Ethiopian Roads Authority, ERA	Ahmet Aydoniz - KMC (JV)
Consultant	
Groutmij/Carl bro in Association with Gondwana Engineering, PLC	
Test Result for Sub grade soils in the stations from km 137+400 to 159+800	
Testing Location: Kibre Mengist Site Laboratory	

S/N	Sample Location	Visual Description	Lab. No	Depth Sampled (m)	Laboratory Tests Result											Remark		
					2.00mm	0.425mm	0.075mm	Soil Class	LL, %	PL, %	PI, %	MDD	OMC, %	CBR, Dry Density			Swell at 65 blans	
17	140+600	reddish brown silty clay	AA/LAB-SE-0065		98.91	95.41	80.57	A-7.6(18)	49	28	21	1.61	23.2	1.367	1.525	1.574	21	0.23
18	140+800	reddish brown silty clay	AA/LAB-SE-0064		100	95.99	84.2	A-7.5(20)	55	36	19	1.6	24	1.31	1.543	1.647	18	0.51
19	141+000	reddish brown silty clay with	AA/LAB-SE-0063		99.77	89.53	68.53	A-7.5(12)	48	30	18	2.025	12	1.847	1.967	2.058	15	0.05
20	141+200	Reddish brown silty clay			98.05	85.54	55.81	A-7.5(10)	51	31	20	1.59	23	1.337	1.432	1.594	23	0.96
21	141+400	Reddish brown silty clay			100	91.67	52.48	A-7.6(7)	46	28	18	1.61	24	1.312	1.527	1.626	19	0
22	141+600	Reddish brown silty clay			99.73	94.72	62.32	A-7.5(12)	53	34	19	1.692	24	1.391	1.529	1.637	23	0
23	141+800	Reddish brown silty clay		2.0m.	98.84	86.92	56.27	A-7.6(8)	46	29	17	1.64	22.5	1.423	1.611	1.705	23	0.06
24	142+000	Reddish brown silty clay		2.0m.	99.77	89.53	68.53	A-7.5(12)	55	39	16	1.525	26.5	1.276	1.453	1.562	32	2.27
25	142+200	Reddish Brown Silty		0.8m.	99.8	88.44	54.25	A-7.5(9)	56	39	17	1.49	29	1.25	1.47	1.53	14	0.1
26	142+400	Reddish Brown Silty		0.8m.	100	91.24	59.73	A-7.5(9)	54	40	15	1.59	24.5	1.21	1.46	1.55	31	0.25
27	142+600	Reddish Brown Silty		0.8m.	100	97.27	83.19	A-7.5(19)	57	39	18	1.56	25	1.26	1.46	1.59	23	0.5
28	142+800	Reddish Brown Silty		0.6m.	100	97.52	83.85	A-7.5(15)	51	38	13	1.52	27.5	1.32	1.49	1.57	18	0.17
29	143+000	Reddish Brown Silty		0.8m.	100	97.52	83.85	A-7.5(21)	53	32	21	1.55	26	1.33	1.53	1.58	15	0.04
30	143+200	Red silty Clay	AA/LAB-SE-0191	0.8m.	99.9	95.93	77.78	A-7.5(15)	59	46	13	1.55	25	1.2	1.35	1.57	20	0.56
31	143+400	Red silty Clay	AA/LAB-SE-0191	0.6m.	89	77.61	67.35	A-7.5(14)	51	31	20	1.7	20.5	1.54	1.73	1.76	22	0.3
32	143+600	Yellowish decomposed	AA/LAB-SE-0191	0.80m.	97.4	91.6	67.4	A-7.6(21)	53	20	33	1.74	16	1.366	1.604	1.72	6	4.4



Client		Contractor	
Ethiopian Roads Authority, ERA		Abinet Aydemiz - KMC (JY)	
Consultant		Contractor	
Grootmij/Carl bro in Association with Gondwana Engineering PLC		Abinet Aydemiz - KMC (JY)	
Test Result for Sub grade soils in the stations from km 137+400 to 159+800			
Testing Location: Kibre Mengist Site Laboratory			

S/N	Sample Location	Visual Description	Lab. No	Depth Sampled (m)	Laboratory Tests Result										Remarks			
					2.00mm	0.425mm	0.075mm	Soil Class	LL, %	PL, %	PI, %	MDD	OMC, %	CBR, Dry Density		CBR at 95%	Swell at 65 blows	
33	143+800	Whitish & yellowish	AA/LAB-SE-0192	0.80m.	99.8	94.9	74.6	A-7-6(21)	46	16	30	1.706	17.5	1.389	1.634	1.727	10	0.88
34	144+000	Reddish with some gravel	AA/LAB-SE-0207	2.0m.	87.8	73.2	55	A-7-6(11)	51	26	25	1.88	13.6	1.581	1.737	1.824	29	0.56
35	144+200	Red clay	AA/LAB-SE-0191	2.0m.	94	85.2	66.9	A-7-6(22)	52	28	24	1.747	19.8	1.496	1.682	1.714	21	0.07
36	144+400	Whitish tuff	AA/LAB-SE-0196	0.6m.	99.2	86.1	51	A-6(8)	36	13	23	1.881	12.8	1.636	1.823	1.89	8	0.34
37	144+600	Reddish clay	AA/LAB-SE-0195	0.6m.	98.3	87.3	61.3	A-6(8)	35	18	17	1.82	15.6	1.636	1.823	1.89	14	0.3
38	144+800	Red clay	AA/LAB-SE-0197	0.6m.	100	95.7	74.2	A-7-6(19)	53	28	25	1.565	24.6	1.23	1.462	1.592	17	0.22
39	145+000	Red clay	AA/LAB-SE-0199	0.6m.	99.8	96.6	87.6	A-7-6(18)	42	22	20	1.685	18.8	1.484	1.633	1.741	16	0.2
40	145+200	Red clay		1.0m.	99.9	97.7	81.8	A-7-6(24)	55	28	27	1.538	25.3	1.294	1.475	1.581	15	0.4
41	145+400	Red clay		0.80m.	99.8	96.4	86.7	A-7-5(21)	55	35	20	1.521	26.5	1.305	1.474	1.515	14	0.01
42	145+600	Red clay		0.6m.	95.8	74.7	54.4	A-6(7)	38	20	18	1.8	15.6	1.583	1.742	1.85	12	0.29
43	145+800	Red clay		0.70m.	98.6	92.4	77.8	A-7-6(19)	48	24	24	1.64	20.2	1.338	1.567	1.646	17	1.2
44	146+000	Red clay		1.0m.	99.9	97.1	78.6	A-7-6(18)	46	24	22	1.685	20	1.414	1.596	1.706	24	0.46
45	146+200	Dark Clay		1.0m.	99.9	97.6	63.3	A-6(8)	32	15	17	1.83	14.4	1.401	1.608	1.692	12	0.03
46	146+400	Yellowish decomposed		1.0m.	97.2	89	79.8	A-7-6(24)	52	23	29	1.725	18.4	1.283	1.479	1.625	1	11
47	146+600	Yellowish with grey		0.7m.	99.1	93.4	64.1	A-6(8)	32	15	17	1.805	15.1	1.605	1.758	1.851	8	1.03
48	146+800	Yellowish with whitish		0.6m.	97	86.5	72.8	A-7-6(19)	48	21	27	1.768	17.8	1.472	1.671	1.771	9	0.51



Aposto Wendo Negele Road Upgrading Project	
Contract 2: Irbameda Wadera Road Construction	
Client	Contractor
Ethiopian Roads Authority, ERA	Ahmet Aydeniz - KMC (JV)
Consultant	
Grontmij/Carl biro in Association with Gondwana Engineering, PLC.	
Test Result for Sub grade soils in the stations from km 137+400 to 159+800	
Testing Location: Kibre Mengist Site Laboratory	

Annex I

S/N	Sample Location	Visual Description	Lab. No	Depth Sampled (m)	Laboratory Tests Result										Remark				
					2.00mm	0.425mm	0.075mm	Soil Class	LL, %	PL, %	PI, %	MDD	OMC, %	CBR, Dry Density			Swell at 65 blows		
49	147+000 B	Dark with brown clay		0.30m.	98.9	92.8	85	A-7-6(37)	68	29	39	1.57	23.4	1.341	1.429	1.565	5	2.66	
50	147+200 A	Brown clay		0.7m.	95.6	73.5	52.7	A-6(6)	33	14	19	1.8	15.5	1.668	1.788	1.819	8	0.8	
51	147+400	Black centom (Dark)		0.80m.	99.7	97.8	91.8	A-7-6(47)	68	22	46	1.596	18.7	1.403	1.533	1.593	1	10.7	
52	147+600	Dark Clay		0.8m.	98.4	90.9	71.9	A-7-6(17)	44	19	25	1.701	18.6	1.516	1.651	1.68	12	0.75	
53	147+800	Reddish gravel	269	0.8m.	86.1	70.2	54.7	A-7-5(13)	54	25	30	1.96	11	1.704	1.946	1.99	15	0.02	
54	148+000	Red Silty Clay		0.8m.	97.2	79	50.4	A-4(2)	26	17	9	1.88	13	1.605	1.742	1.909	20'	0.1	
55	148+200	Brown clay		0.6m.	96.5	78.7	61.9	A-6(8)	35	18	17	1.751	16.2	1.534	1.694	1.784	11'	0.4	
56	148+400	Red clay		0.6m.	100	97.7	85.21	A-7-6(20)	44	22	22	1.6	20.4	1.369	1.49	1.597	14'	0.4	
57	148+600	Red clay		0.6m.	100	97.1	81.1	A-7-6(17)	44	24	20	1.62	21.2	1.327	1.564	1.659	10'	0.2	
58	148+800	Red clay		0.6m.	99.9	98	89.2	A-7-6(32)	60	29	31	1.519	27.5	1.331	1.481	1.548	10'	0.04	
59	149+000	Red clay	AA/LAB-SE-0169	0.6m.	99.2	93.5	84.3	A-7-6(19)	45	23	22	1.64	20	1.364	1.509	1.663	17	1.22	
60	149+200	Red clay	AA/LAB-SE-0145	0.6m.	99.8	96.7	91.2	A-7-5(23)	52	31	21	1.535	26.5	1.257	1.449	1.591	18	0.07	
61	149+400	Red clay		1.0m.	99.9	95.2	82.4	A-7-6(25)	54	26	28	1.592	21.5	1.201	1.418	1.641	13	0.34	
62	149+600	Red clay	AA/LAB-SE-0144	0.6m.	100	98.9	94.2	A-7-6(27)	53	29	24	1.55	24.5	1.119	1.311	1.558	14	1.24	
63	149+800	Red clay	AA/LAB-SE-0142	0.8m.	98.3	90.3	77.2	A-7-6(19)	47	23	24	1.65	20.7	1.372	1.586	1.689	12	0.82	
64	150+000	Red clay	AA/LAB-SE-01	0.6m.	99.8	90.1	82.1	A-7-6(20)	50	28	22	1.62	21.2	1.336	1.511	1.6	17	0.82	



Aposto Wendo Negele Road Upgrading Project	
Contract 2: Irbameda Wadera Road Construction	
Client	Contractor
Ethiopian Roads Authority, ERA	Ahmet Aydeniz - KMC (JY)
Consultant	
Grontmij/Carl bro in Association with Gondwana Engineering, PLC.	
Test Result for Sub grade soils in the stations from km 137+400 to 159+800	
Testing Location: Kibre Mengist Site Laboratory	

Annex I

S/N	Sample Location	Visual Description	Lab. No	Depth Sampled (m)	Laboratory Tests Result											Remark			
					2.00mm	0.425mm	0.075mm	Soil Class	LL, %	PL, %	PI, %	MDD	CBR, Dry Density				Swell at 65 blows		
													OMC, %	10	30			65	
65	150+200	Red clay	A-4/LA-B-SE-162	0.6m.	99.1	94.1	84.8	A-7-6(23)	51	27	24	1.6	22.2	1.266	1.514	1.615	14	0.42	
66	150+400	Red clay	148	0.6m.	99.2	94.2	84.3	A-7-6(19)	47	26	21	1.632	21.3	1.337	1.507	1.647	18	0.34	
67	150+600	Red clay	149	0.6m.	98.9	92.3	77.5	A-7-6(16)	45	25	20	1.785	15	1.317	1.534	1.657	16	0.73	
68	150+800	Red clay	150	0.8m.	100	97.6	87	A-7-6(21)	49	27	22	1.57	25	1.278	1.487	1.59	15	0.13	
69	151+000	Reddish Brown Gravelly	0.041	0.6m.	90.7	81.43	47.78	A-6(3)	35	24	11	1.55	27	1.53	1.77	1.9	18	1.66	swell is more
70	151+200	Red Silty Clay		0.6m.	99.7	47.78	62.32	A-7-5(10)	52	35	17	1.55	27	1.37	1.51	1.6	13	0.02	
71	151+400	Dark Brown Clay		0.8m.	99.9	62.32	70.75	A-7-6(15)	47	24	23	1.57	26	1.43	1.58	1.58	18	0.3	Unsuitable
72	151+600	Dark Clay		0.8m.	99.5	70.75	89.87	A-7-5(61)	91	33	58	1.43	25.3	1.27	1.45	1.52	2	15.1	swell is more
73	151+800	Dark Grey Silty Clay		0.7m.	98.7	89.87	54.78	A-6(6)	30	13	17	1.91	12.8	1.68	1.84	1.92	10	7.61	swell is more
74	152+000	Dark Brown Sandy Silt		0.8m.			>35	A-4			NP	2.04	9.05	1.78	2.03	2.18	64	0.51	
75	152+200	Red Clayey Silt		1.2m.	97.4	89.77	55.64	A-7-5(7)	48	37	12	1.58	23	1.29	1.5	1.65	38	0.11	Unsuitable
76	152+400	Red Clayey Silt		2.3m.	99.7	94.72	62.32	A-7-5(11)	56	38	16	1.71	18.6	1.47	1.68	1.74	10	4.14	swell is more
77	152+600	Red Micaceous Silt		2.4m.			>35	A-4			NP	1.73	18.5	1.46	1.67	1.78	11	4.04	swell is more
78	152+800	Red Micaceous Silt		1.9m.			>35	A-4			NP	1.73	15	1.37	1.62	1.95	6	7.53	swell is more
79	153+000	Red Micaceous Silt		1.4m.	100	91.3	58.45	A-7-6(8)	45	29	16	1.67	21.8	1.33	1.48	1.611	9.5	1.49	
80	153+200	Red Clayey Silt		2.0m.	99.6	89.1	61.86	A-5(6)	43	32	10	1.6	23.5	1.48	1.57	1.68	18	0.42	



Aposto Wendo Negele Road Upgrading Project	
Contract 2: Irbamoda Wadera Road Construction	
Client	Contractor
Ethiopian Roads Authority, ERA	Ahmet Aydimis - KMC (JV)
Consultant	
Grontmij/Carl Iero in Association with Gondwana Engineering, PLC	
Test Result for Sub grade soils in the stations from km 137+400 to 159+800	
Testing Location: Kibre Mengist Site Laboratory	

Annex 1

S/N	Sample Location	Visual Description	Lab. No	Depth Sampled (m)	Laboratory Tests Result											Remark			
					2.00mm	0.425mm	0.075mm	Soil Class	LL, %	PL, %	PI, %	MDD	OMC, %	CBR, Dry Density			CBR at 95%	Swell at 65 blones	
81	153+400	fill section		2.0m.	9.7	95.96	83.44	A-7.5(21)	55	34	21	1.58	25	1.32	1.5	1.63	34	0.4	
82	153+600	Red Silty Clay		1.5m.	99.8	94.45	75.43	A-7.5(18)	55	34	21	1.49	29	1.35	1.49	1.56	25	0	
83	153+800	Reddish Brown Silty		1.5m.	100	88.15	39.157	A-5(0)	41	33	8	1.48	26	1.317	1.55	1.608	11	0.16	
84	154+000	Reddish Brown Silty		2.0m.	99.55	83.91	66.39	A-5(7)	49	41	8	1.82	13.3	1.445	1.73	1.8	9	2.95	Higher swell value
85	154+200	Light Grey Decomposed		1.5m.	96.73	73.15	38.42	A-5(0)	41	33	8	1.832	15.5	1.609	1.752	1.832	12	2.27	swell is more
86	154+400	Red clay		1.0m.	87.7	85.34	72.31	A-7.6(15)	48	28	20	1.575	24	1.33	1.52	1.65	27	0.05	
87	154+600	Reddish Brown Silty		2.0m.	97.24	82.84	62.07	A-7.5(10)	61	47	13	1.7	19.4	1.406	1.563	1.663	8	3.85	swell is more
88	154+800	Reddish Brown Silty		1.5m.	100	88.05	41.74	A-5(1)	42	33	9	1.59	22.8	1.288	1.424	1.558	10	1.17	
89	155+000	Reddish Brown Silty		1.5m.	100	94.66	37.9	A-5(1)	50	40	10	1.58	23.5	1.228	1.404	1.573	9	1.22	
90	155+200	Red Silty Clay		1.5m.	100	93.56	66.24	A-5(6)	48	42	6	1.58	24.2	1.379	1.518	1.57	11	0.3	
91	155+400	Red Silty Clay		1.0m.	100	93.59	54.22	A-5(4)	47	38	9	1.53	25	1.396	1.516	1.588	11	0.05	
92	155+600	Red Silty Clay		1.5m.	99.64	92.92	62.72	A-5(5)	46.38	38	8	1.62	22.6	1.321	1.509	1.628	16	0.19	
93	155+800	Reddish Brown Silty		2.0m.	98.57	91.18	68.32	A-7.5(12)	52	35	17	1.7	18.8	1.364	1.58	1.658	7	0.52	
94	156+000	Decomposed Micaceous Silt		2.0m.	-	-	>35	A-4	-	-	NP	1.83	13.5	1.535	1.752	1.893	19	0.97	
95	156+200	Pinkish micaceous Silt		2.0m.	98	87.69	45.74	A-5(2)	44	34	10	1.67	16	1.344	1.585	1.691	2	1.65	
96	156+400	Pinkish micaceous Silt		2.0m.	99.73	94.66	74.82	A-7.5(23)	69	44	25	1.57	16.7	1.284	1.439	1.533	4	6.53	



Aposto Wendo Negele Road Upgrading Project	
Contract 2: Irbamoda Wadera Road Construction	
Client	Consultant
Ethiopian Roads Authority, ERA	Grontmij/Carl bro in Association with Gondwana Engineering PLC
Contractor	
Ahmet Aydeniz - KMC (JV)	
Test Result for Sub grade soils in the stations from km 137+400 to 159+800	
Testing Location: Kibre Mengist Site Laboratory	

Annex 1

S/N	Sample Location	Visual Description	Lab. No	Depth Sampled (m)	Laboratory Tests Result												Remark		
					2.00mm	0.425mm	0.075mm	Soil Class	LL, %	PL, %	PI, %	MDD	OMC, %	CBR, Dry Density				CBR at 95%	Swell at 65 blows
														10	30	65			
97	156+600	Reddish Brown		2.0m.	95.6	87.52	55.32	A-7.5(5)	44	33	11	1.72	16	1.436	1.602	1.681	3	1.12	
98	156+600	Whitish Tuffaceous		2.0m.	-	-	≥ 35	A-4	-	-	NP	1.71	16.7	1.53	1.7	1.82	3	1.06	
99	156+800	line shift / high fill																	
100	157+000	Swamp/line shift areas																	
101	157+200	Red Clayey Silt		2.0m.	100	90.43	42.75	A-7.5(2)	46	35	11	1.61	18.5	1.293	1.476	1.573	20	0.59	
102	157+400	Red Clayey Silt		2.0m.	97.5	85.36	47.03	A-7.5(3)	48	35	13	1.43	11	1.167	1.315	1.635	5	1.85	Higher swell value
103	157+600	Red Clayey Silt		2.0m.	100	96.92	48.99	A-6(4)	40	28	12	1.707	16.7	1.496	1.694	1.76	11	0.7	
104	157+800	Reddish Brown Silty		2.0m.	-	-	≥ 35	A-4(0)	-	-	NP	1.74	13.6	1.611	1.685	1.747	15	0.21	
105	158+000	Reddish Brown Silty			-	-	≥ 35	A-4(0)	-	-	NP	1.86	11.6	1.681	1.803	1.913	16	0.5	
106	158+200	Reddish Brown Silty			95.6	87.52	55.32	A-6(2)	34	21	13	1.94	11.4	1.648	1.819	1.924	11	0.33	
107	158+400	Reddish Brown Silty			93.49	82.62	45.8	A-4 (4)	28	20	7	2.03	9.8	1.766	1.914	2.05	32	0.33	
108	158+600	Reddish Brown			83.99	66.37	40.25	A-4(3)	25.25	19.38	6	2.15	7.2	1.9862.066	2.066	2.183	25	0.21	
109	158+800	Brown Gravelly Silt			87.52	76.98	42.92	A-4(2)	26	20	6	2.03	8	1.876	2.027	2.0229	10	0.59	
110	159+000	Reddish Brown Silty		2.0m.	-	-	≥ 35	A-4	-	-	NP	1.75	11	1.604	1.679	1.746	19	0.73	
111	159+200	yellowish sandy gravel		2			≥ 35	A-4			NP	1.95	11	1.704	1.871	1.951	11	1.11	
112	159+400	yellowish to white sandy		2			≥ 35	A-4			NP	1.925	12.4	1.726	1.857	1.898	12	0.48	



Aposto Wendo Negele Road Upgrading Project	
Contract 2: Irbamoda Wadera Road Construction	
Client	Contractor
Ethiopian Roads Authority, ERA	Ahmet Aydeniz - K.M.C. (JV)
Grontmij/Carl bro in Association with Gondwana Engineering, PLC	
Test Result for Sub grade soils in the stations from km 137+400 to 159+800	
Testing Location: Kibre Mengist Site Laboratory	

Annex 1

S/N	Sample Location	Visual Description	Lab. No	Depth Sampled (m)	Laboratory Tests Result							Remark							
					2.00mm	0.425mm	0.075mm	Soil Class	LL, %	PL, %	PI, %		MDD	OMC, %	CBR, Dry Density	CBR at 95%	Seell at 65 blows		
113	159+600	pinkish micaceous silt		2			≥ 35	A-4			NP	1.93	11.8	1.679	1.77	1.878	35	0.73	
114	159+800	greyish soils with quartz		2			≥ 35	A-4			NP	1.94	9.8	1.697	1.791	1.871	34	0.16	



SABA Engineering Plc.
P.O. Box 62668 Tel. 39 06 93139 10 65 / 39 16 17 / 391733 Fax. 391230 / 391617 E-mail sava.eng@telecom.net.et Addis Ababa, Ethiopia
LAB. NO. 4675/09

CLIENT Ahmet Aydeniz Construction Co.Inc

PROJECT IbraModa- Wadera Road

SAMPLE OF Rock Sample

SAMPLE AND TEST ORDER SUBMITTED BY The client

SPECIFIED BY The client

TEST FOR Quality

TEST RESULT REPORTED TO The client

On :15/07/09

On :15/07/09

On :04/08/09

Summary Test Results of Rock Sample

Sr No.	SOURCE	LAA Loss AASHTO T 96-83 %	ACV Part 110- 1990 %	TFV KN	AIV BS812 Part 112 1990 %	coated aggregate with 85/100 AC % AASHTO T 182-84	Sodium sulphate Soundness Loss AASHTO T104-86 %	Specific gravity		
								Bulk SSD	AASHTO T 85-85 Apparent GS Absorption %	
1	122+000	10	19	226.7	18	Above 95	0.74	2.90	3.00	1.76

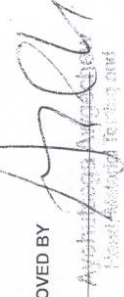
REMARK :

REPORTED BY

LAB ENGINEER



APPROVED BY


Avshur
15/07/2009

Aposto Wendo Negele Road Upgrading Project

Contract 2: Irbamoda Wadera Road Construction

Ethiopian Roads Authority, ERA

Grontmij/Carl bro in Association with Gondwana Engineering, PLC

Ahmet Aydeniz - KMC (JV)

Annex 2

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

S/N	station (Distance)	Pavement Response value (CBR)	Interval Number(n)	Interval Distance(Δxi)	Cumulative Interval distance ($\sum xi$)	Average interval response	Actual Interval Area(ai)	Cumulative Area($\sum ai$)	Zx Value
1	137+400	5	1	0.2	0.2	5	1	1	-1.89
2	137+600	12	2	0.2	0.4	8.5	1.7	2.7	-3.09
3	137+800	13	3	0.2	0.6	12.5	2.5	5.2	-3.48
4	138+000	17	4	0.2	0.8	15	3	8.2	-3.38
5	138+200	14	5	0.2	1	15.5	3.1	11.3	-3.17
6	138+400	0.5	6	0.2	1.2	7.25	1.45	12.75	-4.61
7	138+600	23	7	0.2	1.4	11.75	2.35	15.1	-5.16
8	138+800	6	8	0.2	1.6	14.5	2.9	18	-5.15
9	139+000	2	9	0.2	1.8	4	0.8	18.8	-7.24
10	139+200	4	10	0.2	2	3	0.6	19.4	-9.54
11	139+400	16	11	0.2	2.2	10	2	21.4	-10.43
12	139+600	14	12	0.2	2.4	15	3	24.4	-10.33
13	139+800	8	13	0.2	2.6	11	2.2	26.6	-11.02
14	140+000	2	14	0.2	2.8	5	1	27.6	-12.91
15	140+200	27	15	0.2	3	14.5	2.9	30.5	-12.91
16	140+400	5	16	0.2	3.2	16	3.2	33.7	-12.60
17	140+600	21	17	0.2	3.4	13	2.6	36.3	-12.90
18	140+800	18	18	0.2	3.6	19.5	3.9	40.2	-11.89
19	141+000	15	19	0.2	3.8	16.5	3.3	43.5	-11.48
20	141+200	23	20	0.2	4	19	3.8	47.3	-10.58
21	141+400	19	21	0.2	4.2	21	4.2	51.5	-9.27
22	141+600	23	22	0.2	4.4	21	4.2	55.7	-7.97
23	141+800	23	23	0.2	4.6	23	4.6	60.3	-6.26
24	142+000	32	24	0.2	4.8	27.5	5.5	65.8	-3.65

Aposto Wendo Negele Road Upgrading Project

Contract 2: Irbamoda Wadera Road Construction

Ethiopian Roads Authority, ERA

Grontmij/Carl bro in Association with Gondwana Engineering, PLC

Ahmet Aydeniz - KMC (JV)

Annex 2

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

S/N	station (Distance)	Pavement Response value (CBR)	Interval Number(n)	Interval Distance(Δxi)	Cumulative Interval distance ($\sum xi$)	Average interval response	Actual Interval Area(ai)	Cumulative Area($\sum ai$)	Zx Value
25	142+200	14	25	0.2	5	23	4.6	70.4	-1.95
26	142+400	31	26	0.2	5.2	22.5	4.5	74.9	-0.34
27	142+600	23	27	0.2	5.4	27	5.4	80.3	2.17
28	142+800	18	28	0.2	5.6	20.5	4.1	84.4	3.37
29	143+000	15	29	0.2	5.8	16.5	3.3	87.7	3.78
30	143+200	20	30	0.2	6	17.5	3.5	91.2	4.38
31	143+400	22	31	0.2	6.2	21	4.2	95.4	5.69
32	143+600	6	32	0.2	6.4	14	2.8	98.2	5.60
33	143+800	10	33	0.2	6.6	8	1.6	99.8	4.30
34	144+000	29	34	0.2	6.8	19.5	3.9	103.7	5.31
35	144+200	21	35	0.2	7	25	5	108.7	7.41
36	144+400	8	36	0.2	7.2	14.5	2.9	111.6	7.42
37	144+600	14	37	0.2	7.4	11	2.2	113.8	6.73
38	144+800	17	38	0.2	7.6	15.5	3.1	116.9	6.93
39	145+000	16	39	0.2	7.8	16.5	3.3	120.2	7.34
40	145+200	15	40	0.2	8	15.5	3.1	123.3	7.54
41	145+400	14	41	0.2	8.2	14.5	2.9	126.2	7.55
42	145+600	12	42	0.2	8.4	13	2.6	128.8	7.26
43	145+800	17	43	0.2	8.6	14.5	2.9	131.7	7.26
44	146+000	24	44	0.2	8.8	20.5	4.1	135.8	8.47
45	146+200	12	45	0.2	9	18	3.6	139.4	9.18
46	146+400	1	46	0.2	9.2	6.5	1.3	140.7	7.58
47	146+600	8	47	0.2	9.4	4.5	0.9	141.6	5.59
48	146+800	9	48	0.2	9.6	8.5	1.7	143.3	4.39

Aposto Wendo Negele Road Upgrading Project

Contract 2: Irbamoda Wadera Road Construction

Ethiopian Roads Authority, ERA

Grontmij/Carl bro in Association with Gondwana Engineering, PLC

Ahmet Aydeniz - KMC (JV)

Annex 2

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

S/N	station (Distance)	Pavement Response value (CBR)	Interval Number(n)	Interval Distance(Δxi)	Cumulative Interval distance ($\sum xi$)	Average interval response	Actual Interval Area(ai)	Cumulative Area($\sum ai$)	Zx Value
49	147+000 B	5	49	0.2	9.8	7	1.4	144.7	2.90
50	147+200 A	8	50	0.2	10	6.5	1.3	146	1.31
51	147+400	1	51	0.2	10.2	4.5	0.9	146.9	-0.69
52	147+600	12	52	0.2	10.4	6.5	1.3	148.2	-2.28
53	147+800	15	53	0.2	10.6	13.5	2.7	150.9	-2.48
54	148+000	20	54	0.2	10.8	17.5	3.5	154.4	-1.87
55	148+200	11	55	0.2	11	15.5	3.1	157.5	-1.66
56	148+400	14	56	0.2	11.2	12.5	2.5	160	-2.06
57	148+600	10	57	0.2	11.4	12	2.4	162.4	-2.55
58	148+800	10	58	0.2	11.6	10	2	164.4	-3.44
59	149+000	17	59	0.2	11.8	13.5	2.7	167.1	-3.64
60	149+200	18	60	0.2	12	17.5	3.5	170.6	-3.03
61	149+400	13	61	0.2	12.2	15.5	3.1	173.7	-2.83
62	149+600	14	62	0.2	12.4	13.5	2.7	176.4	-3.02
63	149+800	12	63	0.2	12.6	13	2.6	179	-3.31
64	150+000	17	64	0.2	12.8	14.5	2.9	181.9	-3.31
65	150+200	14	65	0.2	13	15.5	3.1	185	-3.10
66	150+400	18	66	0.2	13.2	16	3.2	188.2	-2.80
67	150+600	16	67	0.2	13.4	17	3.4	191.6	-2.29
68	150+800	15	68	0.2	13.6	15.5	3.1	194.7	-2.08
69	151+000	18	69	0.2	13.8	16.5	3.3	198	-1.68
70	151+200	13	70	0.2	14	15.5	3.1	201.1	-1.47
71	151+400	18	71	0.2	14.2	15.5	3.1	204.2	-1.27
72	151+600	2	72	0.2	14.4	10	2	206.2	-2.16

Aposto Wendo Negele Road Upgrading Project

Contract 2: Irbamoda Wadera Road Construction

Ethiopian Roads Authority,ERA

Grontmij/Carl bro in Association with Gondwana Engineering,PLC

Ahmet Aydeniz - KMC (JV)

Annex 2

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

S/N	station (Distance)	Pavement Response value (CBR)	Interval Number(n)	Interval Distance(Δxi)	Cumulative Interval distance ($\sum xi$)	Average interval response	Actual Interval Area(ai)	Cumulative Area($\sum ai$)	Zx Value
73	151+800	10	73	0.2	14.6	6	1.2	207.4	-3.85
74	152+000	64	74	0.2	14.8	37	7.4	214.8	0.65
75	152+200	38	75	0.2	15	51	10.2	225	7.96
76	152+400	10	76	0.2	15.2	24	4.8	229.8	9.87
77	152+600	11	77	0.2	15.4	10.5	2.1	231.9	9.07
78	152+800	6	78	0.2	15.6	8.5	1.7	233.6	7.88
79	153+000	9.5	79	0.2	15.8	7.75	1.55	235.15	6.53
80	153+200	18	80	0.2	16	13.75	2.75	237.9	6.39
81	153+400	34	81	0.2	16.2	26	5.2	243.1	8.70
82	153+600	25	82	0.2	16.4	29.5	5.9	249	11.70
83	153+800	11	83	0.2	16.6	18	3.6	252.6	12.41
84	154+000	9	84	0.2	16.8	10	2	254.6	11.51
85	154+200	12	85	0.2	17	10.5	2.1	256.7	10.72
86	154+400	27	86	0.2	17.2	19.5	3.9	260.6	11.73
87	154+600	8	87	0.2	17.4	17.5	3.5	264.1	12.33
88	154+800	10	88	0.2	17.6	9	1.8	265.9	11.24
89	155+000	9	89	0.2	17.8	9.5	1.9	267.8	10.24
90	155+200	11	90	0.2	18	10	2	269.8	9.35
91	155+400	11	91	0.2	18.2	11	2.2	272	8.66
92	155+600	16	92	0.2	18.4	13.5	2.7	274.7	8.46
93	155+800	7	93	0.2	18.6	11.5	2.3	277	7.87
94	156+000	19	94	0.2	18.8	13	2.6	279.6	7.58
95	156+200	2	95	0.2	19	10.5	2.1	281.7	6.78
96	156+400	4	96	0.2	19.2	3	0.6	282.3	4.49

Aposto Wendo Negele Road Upgrading Project

Contract 2: Irbamoda Wadera Road Construction

Ethiopian Roads Authority,ERA

Grontmij/Carl bro in Association with Gondwana Engineering,PLC

Ahmet Aydeniz - KMC (JV)

Annex 2

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

S/N	station (Distance)	Pavement Response value (CBR)	Interval Number(n)	Interval Distance(Δxi)	Cumulative Interval distance ($\sum xi$)	Average interval response	Actual Interval Area(ai)	Cumulative Area($\sum ai$)	Zx Value
97	156+600	3	97	0.2	19.4	3.5	0.7	283	2.29
98	156+600	3	98	0.2	19.6	3	0.6	283.6	0.00
99	156+800	0		0					
100	157+000	0		0					
101	157+200	20	99	0.2	0.2	10	2	2	-1.41
102	157+400	5	100	0.2	0.4	12.5	2.5	4.5	-2.33
103	157+600	11	101	0.2	0.6	8	1.6	6.1	-4.14
104	157+800	15	102	0.2	0.8	13	2.6	8.7	-4.96
105	158+000	16	103	0.2	1	15.5	3.1	11.8	-5.27
106	158+200	11	104	0.2	1.2	13.5	2.7	14.5	-5.99
107	158+400	32	105	0.2	1.4	21.5	4.3	18.8	-5.10
108	158+600	25	106	0.2	1.6	28.5	5.7	24.5	-2.81
109	158+800	10	107	0.2	1.8	17.5	3.5	28	-2.73
110	159+000	19	108	0.2	2	14.5	2.9	30.9	-3.24
111	159+200	11	109	0.2	2.2	15	3	33.9	-3.66
112	159+400	12	110	0.2	2.4	11.5	2.3	36.2	-4.77
113	159+600	35	111	0.2	2.6	23.5	4.7	40.9	-3.49
114	159+800	34	112	0.2	2.8	34.5	6.9	47.8	0.00

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