Addis Ababa University (Since 1950)

COLLEGE OF NATURAL SCIENCES

SCHOOL OF EARTH SCIENCES

STREAM OF ENGINEERING GEOLOGY

Engineering Geological Characterization and Suitability Assessment of Sub-grade Embankment Fill Material, Sub-ballast, and Ballast Material of Addis Ababa City Light Railway Transit (AALRT); Case Study Ayat to Mexico Square, Addis Ababa, Ethiopia

A Thesis Submitted to

The School of Graduate Studies of Addis Ababa University

In Partial Fulfilment of the Requirements for the Degree of

Masters of Science in Engineering Geology

By

Dejene Fikadu

May, 2014

ADDIS ABABA UNIVERSITY
COLLEGE OF NATURAL SCIENCES
SCHOOL OF EARTH SCIENCES
STREAM OF ENGINEERING GEOLOGY

Engineering Geological Characterization and Suitability Assessment of Sub-grade Embankment Fill Material, Sub-ballast, and Ballast Material of Addis Ababa City Light Railway Transit (AALRT); Case Study Ayat to Mexico Square, Addis Ababa, Ethiopia

By:
Dejene Fikadu

Advisor:
Dr. Fekerte Arega

A Thesis Submitted to the School of Graduate Studies of Addis Ababa University in Partial Fulfillment for the degree Masters of Science in Engineering Geology.

Addis Ababa, Ethiopia

2014

ADDIS ABABA UNIVERSITY

COLLEGE OF NATURAL SCIENCES

SCHOOL OF EARTH SCIENCES

STREAM OF ENGINEERING GEOLOGY

Engineering Geological Characterization and Suitability Assessment of Sub-grade Embankment Fill Material, Sub-ballast, and Ballast Material of Addis Ababa City Light Railway Transit (AALRT); Case Study Ayat to Mexico Square, Addis Ababa, Ethiopia

By: Dejene Fikadu

Approval by Board of Examiners

Dr. Seifu Kebede ________________________________

Chairman, School of Earth Sciences

Dr. Fekerte Arega ________________________________

Advisor

Dr. Tarun K. Raghuvanshi ________________________________

Examiner

Dr. Alemgena ________________________________

Examiner

Addis Ababa, Ethiopia

2014

DECLARATION

I hereby declare that the thesis is my original work under the supervision of Dr. Fekerte Arega Yitagesu, School of Earth Sciences, Addis Ababa University during the year 2014 as part of Master of Science Program in Engineering Geology. I further declare that this work has not been submitted to any other University or institution for the award of any degree or diploma and all sources of materials used for the thesis have duly acknowledged.

All source of materials used for the thesis have duly acknowledged.

Dejene Fikadu

Signature ______________________________

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

ACKNOWLEDGEMENT

Foremost of all I would like to thank, appreciate and praise the merciful and the supreme ruler GOD in the name of Lord Jesus, who made my effort smooth and provides me indescribable grace for many up and downs in my life as well during the present study.

Subsequently, I would like to express my deepest gratitude to my advisor Dr. Fekerte Arega Yitagesu for guiding and supervising my research work. She was dedicated to advise and guide me with constructive and consistent follow-up in the present study. I am thankful also for Mr. Bayisa Ragassa (MSc.) for helping me, especially, in Engineering Geological Laboratory testing the samples and giving me valuable comments for the work.

I would also like to express my feelings of gratitude to Dr. Tarun K. Raghuvanshi and Dr. Trufat Hailemariam Gugsa, my instructors from the beginning and made a valuable role in teaching and guiding me for the concepts of engineering geological principles.

My warm appreciations and thankful gratitudes also go to SweRoad Consulting Engineers PLC and Ethiopian Railway Corporation (ERC) for their provision of the advises, supports, valuable ideas and test results and relevant manuals. I would like to extend my special thanks to Ato Mandella, Eng. Behailu, Eng. Henok, Eng. Iyob, W/ro Getenesh, Ato Workalemahu, Ato Addam and all staff members. My colleagues in Oromiya Water Mineral and Energy Bureau, especially my gratitude goes to Obbo. Alemayehu Gudeya Obbo. Zewudu Tadesse, Juhar, Tesfish, Alexo and all other staff members for creating encouraging condition for my stay in the study program and for my research work.

My deepest gratitude also goes to all my friends and my special friend (Aaddaasweet) for your all rounded support, encouragement, directly or indirectly.

Last but not least, I would like to convey my gratefulness to my father Fikadu Angessa, my mother Marge Disassa (Ayyoo), and all my family (Mimo, Moonee and Jiree) for their precious and non ending love. Love you!

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF PLATES

LIST OF ACRONYMS

AAiT	Addis Ababa Institute Of Technology
AASHTOO	American Association Of State Highway And Transportation Officials
ACV	Aggregate Crushing Value
AIV	Aggregate Impact Value
AREMA	American Railway Engineering Maintenance-Of-Way Association
ASTM	American Society For Testing And Materials
BS	British Standard
CBR	California Bearing Ratio
IS	Indian Standards
LL	Liquid Limit
LRT	Light Rail Transit
OD	Oven Dry
OMC	Optimum Moisture Content
PI	Plasticity Index
PL	Plasticity Limit
SG	Specific Gravity
SSD	Saturated Surface Dry
UIC	International Union Of Railways Way And Works Committee
USACE	United States Army Corps Of Engineers
USCS	Unified Soil Classification System

List of Annexure

Abstract

The present study was mainly focused on engineering geological characterization, assessment and the suitability analysis of the substructures of the Addis Ababa LRT alignment and to evolve possible engineering concerns or outlooks for the likely engineering geological gaps along the proposed alignment. For this field observation, investigation, sampling and laboratory tests for physical and strength properties assessment were performed on representative samples from sub-grade embankment material (Yerer-2 subgrade) and ballast stone materials (Galan Quarry). In all LRT project there is no subballast proposed and used by the contractor (CREC). Because instead of subballast, 5cm thick asphalt concrete is proposed in design specification of the LRT project.

Further, attempts were also made to identify and characterize the topographic classifications and surface drainage setup of the area by reviewing the relevant literatures and by using the elevation map of the study area. Based on the laboratory test Soil Classification such as; Grain size distribution and Atterberg Limits, Strength tests (California Bearing Ratio (CBR)), Moisture/ Density Relationship (modified proctor tests), and direct shear characterization, the sub-grade was found to be suitable. In addition to this it has been assessed that most of the identified subgrade fill material is granular in nature. Based on the index properties, with its AASHTO classification is A-1-a (0), A-1-b (0) and [GW, GP] USCS classification systems. Pertaining to its consistency limits, the subgrade material falls below A-line in the Cassagrade plasticity chart indicating that moderately plastic silt to low plastic. The bearing capacity of the subgrade material is above (ERA, 2002) specifications.

In laboratory gradation, specific gravity, water absorption, impact value, crushing value, abrasion resistance, soundness, Slake durability test and peterographic analysis were done for the ballast stone accordingly. Pertaining to the recommended standards and specifications set for ballast aggregate, all the engineering suitability results indicate that the studied material is suitable for intended use except for its water absorption values (which is above permissible) AREMA, (2010), and higher content in plagioclase feldspars which makes an aggregate more sensitive to weathering.

Key Words*: Sub-grade Embankment Fill Material, Subballast, Ballast Stone Material, Characterization, Index Properties, petrographic thin section.*

CHAPTER I - INTRODUCTION

1.1 Preamble

The Ethio - Djibouti Railways, founded in 1894, is the successor of the Imperial Railway Company of Ethiopia and jointly owned by the Governments of Ethiopia and Djibouti. In this regard, the two countries have a century old relation. Of course, Ethiopia is not new for train transport; nor is train transport from Ethiopia to a foreign country. At present, however, the service appears to have revived far better than the former with modern technology. In January 2010, it was announced that the Ethiopian Government had signed a memorandum of understanding with four different companies and in September 2010. Thus, design, planning and construction have begun on the project for the eight railway route lines (Fig. 1.1). Currently the Ethiopian Railway network construction project plan currently encompasses about eight route lines. These eight route lines do not include Addis Ababa Light Railway Transit (LRT) (www.erc.gov.et, December, 2013).

Experience from many countries, especially modern cities in the world has revealed that the railway systems play an important role in providing good transportation system in any country. Perhaps it is a symbol of modernization in transportation of big cities with large and growing number of population. In addition to Ethiopian capital city, the African Union (AU) Capital, context the importance of railway cannot be denied as the city is the living home for multi national and international peoples and organizations. According to AREMA MRE, (2010), Light Rail Rapid Transit (LRT) is the successor to the tram, trolley or streetcar, with running speeds up to 55 Miles per hour and stop spacing of 1/4 to 1-1/2 miles or more. A key characteristic is the ability to operate on city streets without significant station facilities and in mixed traffic.

The LRT track system will play an important role in providing good transportation system in connecting simply different towns round Addis Ababa, all around affordable transportation system for the citizens living in the city or else coming to the city and increasing the aesthetic value of the capital. In most of the countries, a very large portion of the annual budget to sustain the railway track system goes into track maintenance.

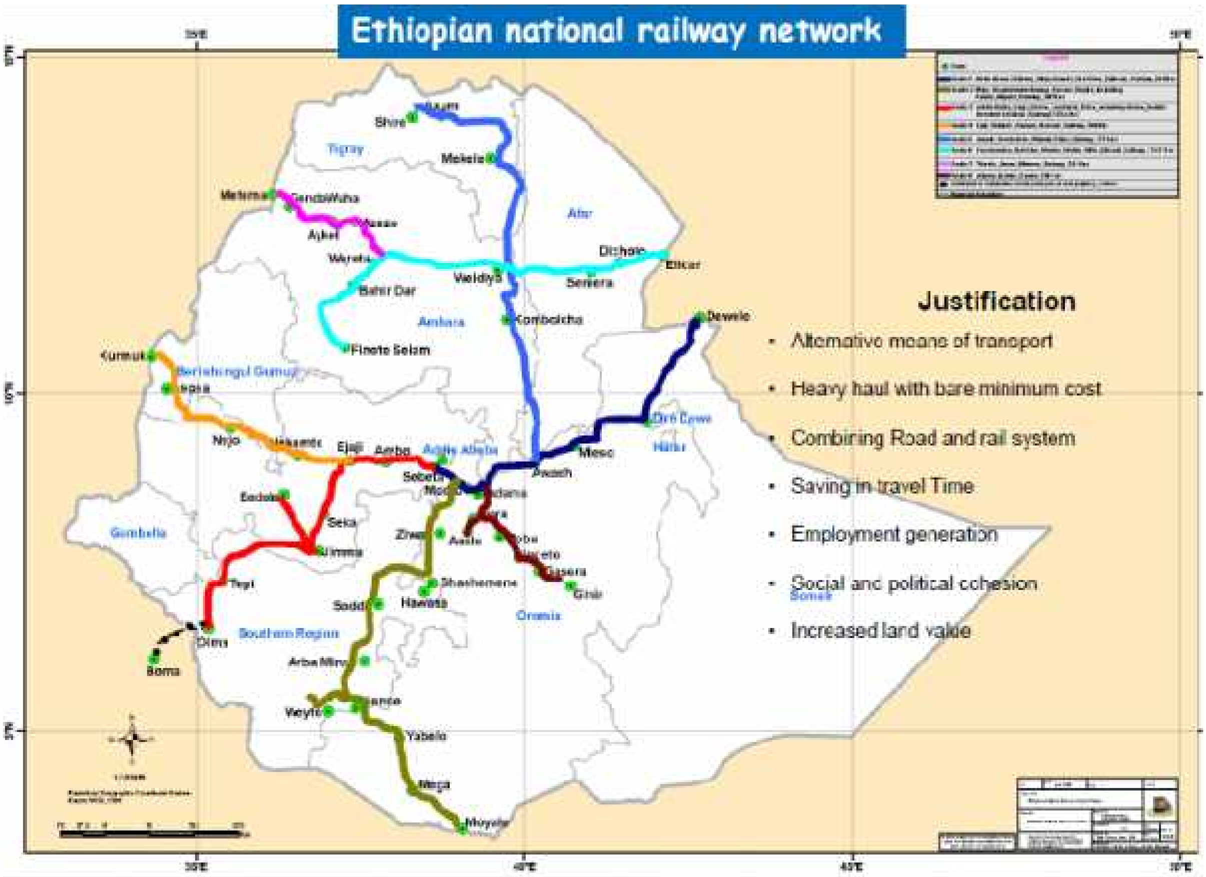


Fig. 1.1 National Rail Way Network of Ethiopia (Yehualaeshet Jemere., 2012)

Even though the sub-structure components have a major influence on the cost of track maintenance, less attention has been given to the sub-structure because the properties of the sub-structure are more variable and difficult to define than those of the super-structure (Selig and Waters, 1994). Therefore, this research will characterize, evaluate and analyze the suitability of the substructures, and locate valuable engineering out looks, concerns and gaps in order to minimize the country cost expense on maintenance.

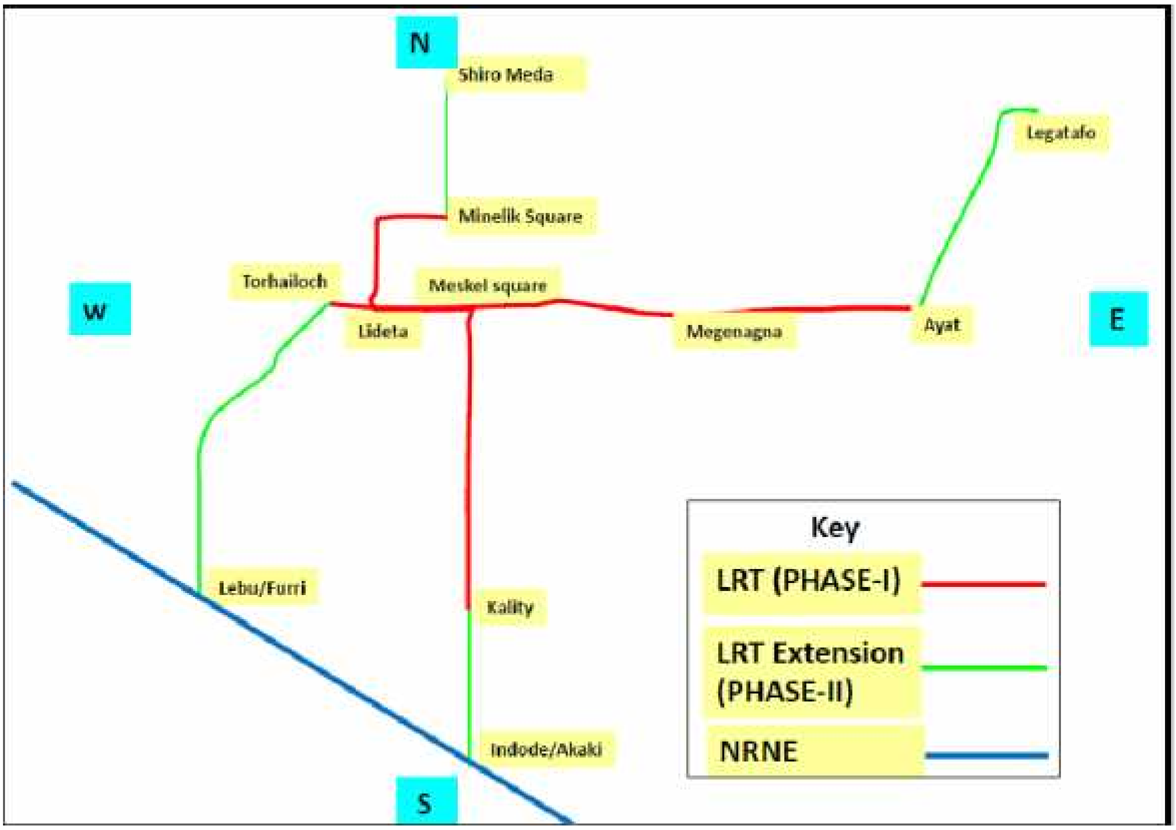


Fig. 1.2 Addis Ababa LRT Alignments Extension (Not to scale)

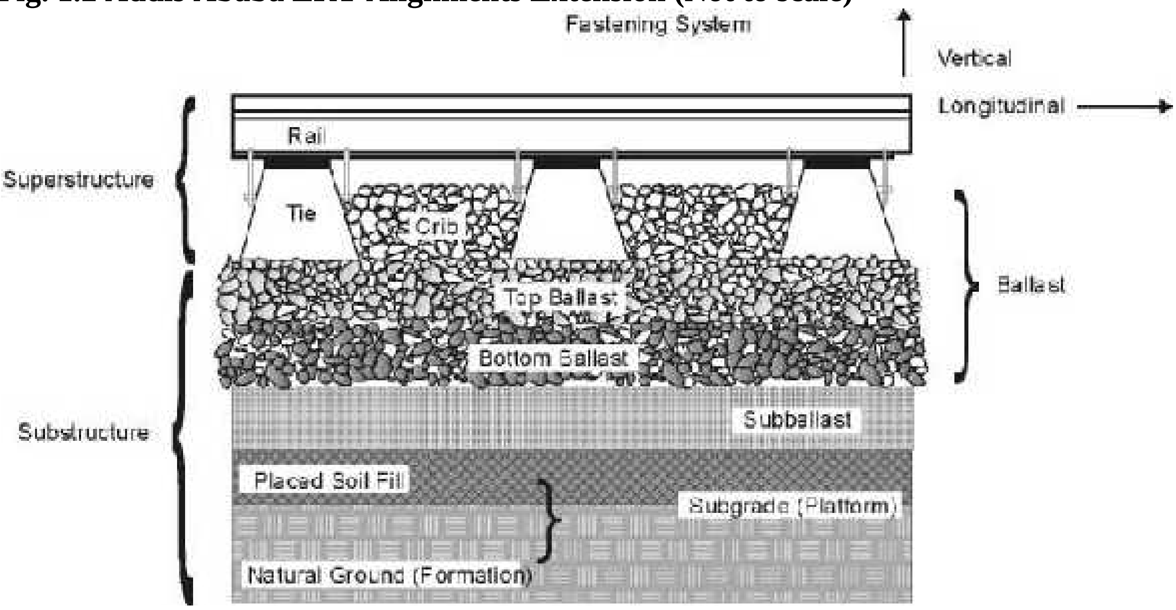


Fig. 1.3 Track structure components (Selig and Waters, 1994)

1.2 Justification of the Problem

Track substructure (ballast, sub ballast, and subgrade) conditions have an important influence in determining the potential for train derailments, which frequently cost millions of dollars per incident. A significant part of the railroad's track maintenance budget is driven by the rate of deterioration of smooth track geometry. Rough track is caused by movements in the substructure under repeated train loading. These are significantly affected by moisture accumulation and thickness of the roadbed layers. Thus, the hidden and hard to monitor substructure conditions are important for railway track performance (Selig and Waters, 1994).

On the other hand, Railway ballast provides the primary structural support for track structure and facilitates effective track maintenance, such as track leveling. Ballast is typically specified as angular, large (< 75mm) particles with a mineralogically and chemically sound composition; however, the mechanical behavior of the ballast deteriorates during the service life of rail track due to particle breakage and increasing fines (i.e., ballast stone fouling). The structural integrity of highly fouled ballast is compromised and can lead to track instability and, ultimately, train derailments (Selig and Waters 1994; Raymond and Bathurst 1994).

Fouling impacts the track performance by changing the mechanical properties of substructure layers through:

(1) Loss of effective drainage,

(2) Formation of “mud-holes”,

(3) Lack of resistance to lateral and longitudinal forces,

(4) Poor durability after maintenance, and

(5) Increasing rate of deterioration; therefore, costly ballast maintenance activities, such as undercutting, tamping, shoulder cleaning, or ballast replacement, are routinely performed by railway companies (Haque et al., 2008; Zaayma, 2006; Liu and Xiao, 2010).

1.3 Application of Results

It is obvious that Addis Ababa City Light Railway Transit is currently under construction by an international company called China Railway Engineering Corporation. However, this study will sufficiently add to the gaps in the engineering geological characteristics of rail way substructures. Especially, the qualities of the sub-grade embankment fill material, sub-ballast (even though the contractor doesn’t use it) and the quality of ballast stone material can be controlled both by the quality of material (source material) and production (crashing) condition.

The production condition may depend on the condition of the crusher; therefore this study was concentrated on the evaluation of the quality of the substructures by adding crucial engineering parameters those of which are not yet tested by the contractor. In addition to this, scientific study may be helpful for further railway line construction community with similar engineering and economic problems especially, for those projects that are expected to be constructed and currently under construction. Perhaps it is also helpful for Ethiopian Railway Corporation in adding the values in preparation of the country's railway manuals and specifications. Lastly, this paper will be help full as starting idea for further research for those questions do not answered and for enlightened research in the field of railways.

1.4 Objectives of the Study

1.4.1 General Objectives

The general objective of the present study is to characterize engineering geological suitability of the sub-grade, sub-ballast and ballast material of the Addis Ababa Light Railway Transit (LRT) project, case study Ayat to Mexico square (17.35Km) alignment. Finally, to evolve and quantify possible engineering geological good concerns along the proposed section.

1.4.2 Specific Objectives

Ayat to Mexico square railway line is aligned on soft, thick, with variable intercalation with loose tuffaceous lithologic unit underlain by highly weathered ignimbrite (especially from Magenagna to Mambis area) (Plate 1.1) and expansive sub-grade soil exposed on slightly

Plate 1.1 Natural subgrade excavated ignimbrite (Plate-A) and highly weathered tuffaceous material (plate-B)

undulating topography. Moreover the sub-grade embankment fill material, sub-ballast and ballast material characterization from engineering geological point of view so as to verify it's suitability along the proposed line. This will be through;

- Characterizing the general topographical feature along Ayat to Mexco square railway alignment by investigating the elevation and slope difference of the route.

- Visual characterization of natural sub-grade soils along the selected alignment.
- Engineering geological characterization and suitability assessment of the sub-grade embankment fills material for the selected quarry site.
- Engineering geological characterization and suitability assessment of the sub-ballast material for the selected quarry site.
- Engineering geological characterization and suitability assessment of the ballast stone material.
- To develop engineering geological map (structural map) in relation to Ayat to Mexico square along the proposed LRT alignment.
- To evolve and quantify possible engineering geological good concerns or out looks along the proposed section.

1.5 Genesis for the Present Study

There were various literatures pertaining to the proposed concepts, specification and general requirements for engineering suitability of Addis Ababa Light Railway Transit substructures were thoroughly reviewed during the present study. Thus, based on these literature reviews following methodologies were evolved to meet out the objectives set during the present study.

This is through adopting different approaches in characterizing the substructure materials for the proposed rail alignment. Characterization of prepared sub-grade (embankment) material, sub-ballast and ballast stone material both by its index and engineering properties were undertaken. Further, by conducting laboratory tests in the Engineering Geological laboratory, School of Earth Sciences and at Addis Ababa institute Technology, school of Civil Engineering laboratory for the determination of their suitability sensitive engineering properties were determined on the representative substructure samples. These include test for, Atterberg limit, gradation test, moisture – density relationships (Modified Proctor Tests), unit weight relationships, CBR, specific gravity test, water absorption test, impact value test, crushing value test, abrasion resistance test, soundness test, shape test, Slake durability test and polished value tests.

Later, for evaluation of the laboratory result different technical specification, standards and manuals were referred. On the other hand, a digital elevated map of the area and drainage conditions were also assessed and characterized using literature review and previous work done in the city administration of Addis Ababa. Finally, even though the project is currently ongoing, the study was also attempted to give solutions for the likely problems along the alignment based on the different manuals and guideline that reviews the design methodologies adopted in various World Railway systems.

1.6 Methodology

- In order to characterize proposed substructures, secondary data (the secondary data such as; geological map of the study area with the scale of 1:250,000 and topographic map of the study area with the scale of 1:50,000 and other published and unpublished geological documents of the site) were collected and reviewed.
- From previous works detail literature reviews and primary data to be generated through study will be utilized.
- For primary data generation field observation and laboratory analysis will undertake following different standards and specifications.
- Determining and characterizing the different engineering geological properties of the sub-grade embankment fill material through some internationally accepted standards.
- To determine the quality of ballast rock different physical, mechanical and chemical properties of the ballast rocks was evaluated in the field and laboratory. This was done to evaluate the following characteristics of the ballast material those helped to achieve its function properly.
- The weathering degree and geological structures of the quarry rocks such as; joints, fractures and other structures were evaluated and described.
- To determine the rock type, its texture, micro crack and its mineralogical composition that influences the engineering quality of the rock were examined by using hand specimen discretion and Petrographic thin section analysis.

1.7 Supporting Materials and Tools Used

- To determine the drainage condition of the study area and to serve as base map for the field work activity, topographic maps with scale of 1:50,000 have been used.
- Addis Ababa City Geological map with the scale of 1:250,000, which was done by Geological survey of Ethiopia was used.
- Analyzing data collected from the ERC or other representative using AREMA2009, AREMA MRE, 2010, China Railway Manual, AASHTO, USCS and ASTM

standards.

- Using some software for data processing, presenting, and preparation of maps with the help of; ArcGIS-9.3, Surfer and Micro Soft-Excel and etc.
- Geological hammer was utilized during the field work for sampling and estimation of rock strength in the field.
- To establish location details GPS was utilized during the field work to collect UTM coordinates of the proposed quarry sites.

1.8 The Scope of the Research

This study provides engineering geologically characterized Sub-grade (embankment fill material), sub-ballast and ballast material for the Addis Ababa City LRT project. The intended paper will indicate the strength, weakness and additional engineering good concerns only depending on the engineering geological characteristics and parameters. Furthermore, the study will help for the Ethiopian Railway Corporation in preparing manuals for further railway construction in the country. Lastly this paper will provide a starting idea for any research in a similar topic.

1.9 Limitation

It stated that, in the previous section that Addis Ababa Light Railway Transit (LRT) is the first in its type. Therefore, to characterize the railway substructures may face a shortage in experience especially in sub-ballast and ballast characterization.

As already known, there is very narrow experience in Ethiopia in railway construction. As a result, the researcher faces a limited amount of literatures and previous works in railway construction, in the review of researches in local engineering geological context. On the other hand, time and money constraints needed for the research may put an effect on the quality of the paper.

1.10 Scheme of Presentation

This study is presented in seven chapters and the scheme of presentations of each chapter is as follows:

Chapter -1 presents the introduction which provides the general over view and importance of light railway transit to the city of Addis Ababa as well to the country and railway track system in general. It covers the introduction with the justification of the problem, objectives, methodology and material to be used, application of the study, limitation of the study and scheme of presentation.

Chapter -2 covers the literature review that relates to the present study and provides a detailed review of conceptual engineering geological parameters relevant to this paper.

Chapter-3 this chapter provides an over view of the study area and presents about location of the project area, characterization of the topography and general geomorphology of the area, drainage pattern and geologic setting of the area, soil and climate of the study area and seismicity condition.

Chapter - 4 Presents the engineering characterization of sub grade soil. This includes characterizing the sub grade material by testing it in the engineering geology llaboratory using some standards specified in chapter one and express these test results using different engineering parameters and index properties.

Chapter - 5 This chapter also presents Engineering Geological characterization of ballast stone material for the selected light railway transit (LRT) alignment.

Chapter -6 It presents the result and discussion on the suitability of the overall outcomes or the test results of the study regarding topographical, natural ground condition plus sub grade embankment fill material and ballast stone materials.

Chapter -7 this chapter briefly presents conclusions and technical Recommendations depending on the above six chapters.

CHAPTER II - LITERATURE REVIEW

2.1 Preamble

In this section to develop some detailed conceptual framework of the problems a systematic extensive literature review has been carried out. Thus, based on the observations from the relevant literature a systematic methodology was worked out for the present study. A summary of the literature review relevant to the present study is presented in the following paragraphs. For the present study, different literature sources were used. These include maps and reports, books, manuals, standards, journals and on-line materials available on the internet. Previous studies and practical works conducted in Ethiopia that are of a similar nature to the present research were assessed and included as part of the literature review. Experiences and manuals of other countries with similar physical characteristics to Ethiopia have also been referred and has been tried to accommodate in the literature review.

2.2 Review of Railway Superstructures

2.2.1 Rails

Rail is the most expensive material in the track. Rails are longitudinal steel members that are placed on spaced sleepers to guide the rolling stock. Their strength and stiffness must be sufficient to maintain a steady shape and smooth track configuration and resist various forces exerted by travelling rolling stock. One of their primary functions is to accommodate and transfer the wheel/ axle loads onto the supporting sleepers (Selig and Waters, 1994).

2.2.2 Fastening Systems

Among the components of railway track superstructures that connect sleeper with the rail. Its function is to retain the rail against force to the sleeper and resist the vertical, lateral, longitudinal and overturning movement of the rail. These fastenings can be grouped into three distinctive types.

- An elastic rail spike. This is driven into pre-drilled holes in sleepers and can be used with or without a steel or cast iron base plate.
- A spring clip bearing on the foot of the rail held down by a nut and bolt element tightened to a predetermined torque. This type of fastening is still used widely in France and Germany.
- A spring clip driven into a hole or slot in a ‘shoulder’, either cast into the sleeper or part of a base plate. The act of driving in the clip either twists or bends the clip thus creating a toe load on the rail.

2.2.3 Sleepers' Structure

According to (Bonnett, 2005; Remennikov and Kaewunruen, 2005; Lim, 2004) cited in Mathewos Bekele, (2012), Sleepers are the simplest and most secure structural method through which the required rail geometry for slab tracks serves. Sleepers may be hung together with rails to form the rail span or 16 they may be laid separately. An advantage for track geometry is the fact that the pre-finished sleepers used are required to have a constant level of quality. The basic functions of sleeper are the following. These are: to spread wheel loads to ballast, hold rails to gauge and inclination transmit lateral and longitudinal forces, insulating rails electrically, and provide a base for rail seats and fastenings. Sleepers can be wood (timber), concrete and steel based on the material it is made of.

2.3 Reviews on Engineering Geological Characterization of Substructures

2.3.1 Sub-grade

The sub-grade is part of embankment/cutting provided above subsoil by borrowed soil of suitable quality up to the bottom of blanket/ballast (IRDSO, GE-1, 2003). It is the platform upon which the track structure is constructed. Its main function is to provide a stable foundation for the sub-ballast and ballast layers. The influence of the traffic-induced stresses extends downward as much as 2-5 meters below the bottom of the ties. This is considerably beyond the depth of the ballast and sub-ballast. Hence, the sub-grade is a very important substructure component that has a significant influence on track performance and maintenance. Also, sub-grade is a source for rail differential settlement due to movement of the sub-grade from various causes. In general, the sub-grade is often the weakest sub-structure layer. Hence, the design method must consider the type and strength of the sub-grade soil, the distribution of dynamic wheel loads and number of repetitions, and the substructure layer resilient moduli (Selig, 2004).

The sub-grade may be divided into two categories: (i) natural ground (formation) and (ii) placed soil (fill material). Anything, other than soils existing locally is uneconomical to be used for the sub-grade. Existing ground should be used without disturbance as much as possible. However, techniques are available to improve soil formations in place if they are inadequate. Often some of the formation must be removed to construct the track at its required elevation, which is below the existing ground surface. This puts the track in a cut with the ground surface sloping downward toward the track. If the excavation intercepts the water table, slope erosion or failure can occur, carrying soil onto the track. Placed fill is used either to replace the upper portion of unsuitable existing ground or to raise the sub-grade surface to the required elevation for the super-structure and the remainder of the sub-structure (Mathewos Bekele., 2012).

2.3.1.1 Parameters to Characterize Sub-Grade (Embankment Fill Materials)

According to Gerald P. Raymond, (2005), the two main requirements of a stable subgrade are:

(a) The provision of sufficient granular or soil modified cover to ensure that overstressing does not occur, and

(b) The provision of a granular filter blanket to prevent the piping and thus loss of subgrade fines from below the track load bearing area. In order to ensure that overstressing does not occur, the track stresses need to be calculated. Also, the ultimate stability of the roadbed will be governed by the engineering characteristics and suitability of these soils (AREMA, 2010).

According to China Railway Manual (TB10012-2007), engineering geological investigations for ordinary sub-grade shall include engineering geological mapping which covers an area of 100 to 200 m from both sides of the line center and geological tests on representative geological cross-sections. The composition and thickness of the material and the drainage condition of existing upper 2-feet (0.61m) of the roadbed area are extremely important because of the high stress from track road and exposure to environmental factors (AREMA, 2010).

It also recommends remedial measures commonly employed to minimize the damages on the railway line sub-grade material. Practically, it is not possible to buildup sub-grades whose CBR value is less than or equal to 3 (UIC, 2005). Therefore, it is necessary to improve the CBR value either by installation of a capping layer or stabilization of the natural ground or embankment fills. Before acceptance as a foundation of the railway lines within the design depth these soils require special treatment. If the stability analysis indicates an adequate factor of safety for the foundation, the design will be based on the internal stability of the embankment. When the foundation is too weak to provide adequate support one or more of the following procedures could be adopted to achieve a stable fill/ foundation:

- Total or partial removal of unsuitable foundation material, displacement of this material, and replacement with compacted fill.

- Flattening of the slopes of the embankment section or the addition of berms at the toes of the embankment.
- Installation of a foundation drainage system to reduce pore water pressures.
- Stage construction of the embankment.
- Densification of sandy foundation soils.
- Use of light embankment materials (fills).
- Mechanical reinforcement or underpinning systems.
- Preloading and surcharging the fill area to accelerate consolidation of clay or organic soils. Soils used for improved sub-grade layers shall be non-expansive, non-dispersive and free from any deleterious matter (AREMA, 2010).

Table 2.1. Some Engineering Specifications of Sub-grade/Embankment fill Soil

Engineering properties	Value of the parameters	Manuals standards
Particles finer than 75 micron	≤50/40%	IS-2720/UIC ,2005
Particle size analysis	Cu > 4 and1 ≤ Cc ≤3 or Well graded soil	IRDSO, GE-1, 2003 and ASTM D422
LL	≤35/55-60	IS-2720, 1969/ERA, 2002
PI	≤15/20-25	IS-2720,1969/ERA, 2002
Cu (uniformity of coefficient)	> 7	IS-2720 ,(1987)
Embankment fill compaction density	≥95% at MDD of AASHTO T 180 D	IS-2720,1969/AREMA , 2010, AASHTO-2003
MDD	>1.8gm/cc	IS-2720, 1969
CBR	>4/5	UIC-2005/ERA, 2002

Consistency Limits

Atterberg, a Swedish scientist, considered the consistency of soils in 1911, and proposed a series of tests for defining the properties of cohesive soils. These tests indicate the range of the plastic state (plasticity is defined as the property of cohesive soils that possess the ability to undergo changes of shape without rupture) and other states. He showed that if the water content of a thick suspension of clay is gradually reduced, the clay water mixture undergoes changes from a liquid state through a plastic state and finally into a solid state.

Atterberg proposed liquid limit (LL), plastic limit (PL) and shrinkage limit (SL) (Fig.2.1), of consistency scientifically to classify the soils and understand the correlation between the consistency limits and index/engineering properties such as the compressibility, shear strength and consolidation. These limits represent the moisture holding capacity at different states of consistency.

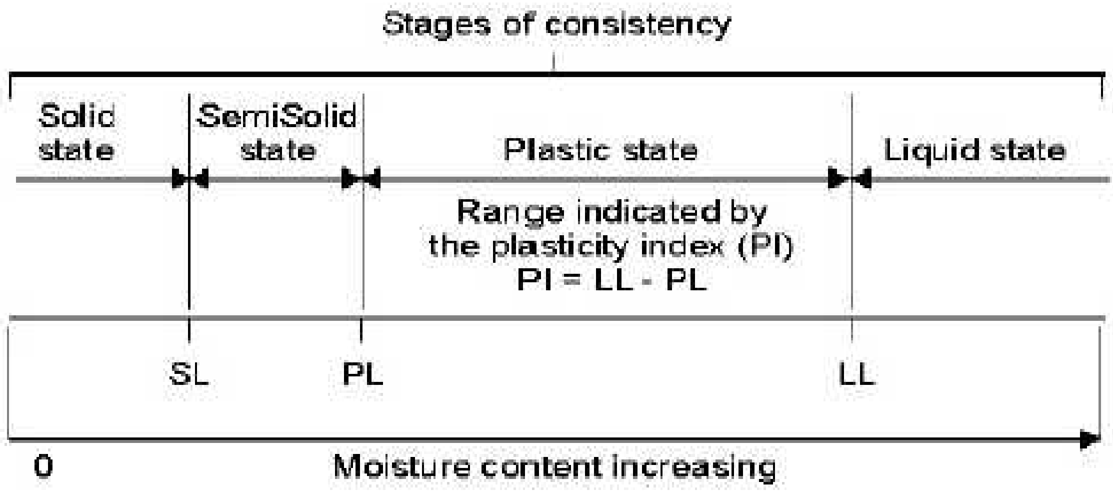


Fig. 2.1. Atterberg Limits and Soil Consistency Procedures (*Source:* USBR, 1968)

Plasticity index / indicates the degree of plasticity of a soil. The greater the difference between liquid and plastic limits, the higher is the plasticity of the soil. A cohesionless soil has zero plasticity indexes. Such soils are termed non-plastic. Fat clays are highly plastic and possess a high plasticity index. Soils possessing large values of w, and / are said to be highly plastic or fat. Those with low values are described as slightly plastic or lean (V.N.S, Murthy, 1990).

Table 2.2 Soil classifications according to Plasticity Index (after V.N.S, Murthy, 1990)

Plasticity index	Plasticity
0	Non-plastic
<7	Low plastic
7–17	Medium plastic
>17	Highly plastic

Fine-grained soil packing is grouped into silt, cohesive soil and organic soil. Silt, cohesive soil should be grouped into packing group in terms of liquid limit water content ratio W_L: when W_L <40% , grouped in C; when $W_L \geq 40$ %, grouped in D**.** Organic soil is grouped in E (China Railway code, TB10001, 2005).

The consistency of a fine-grained soil is the physical state in which it exists; it is related to a larger extent to water content. Consistency denotes the degree of firmness of the soil that is indicated by tests in the field as soft, firm, stiff or hard. Even though it is not possible to interpret the Atterberg limits and plasticity characteristics in fundamental terms, these parameters are of great practical use as index properties of cohesive soils. The engineering properties (uses) of fine grained soils are related to these index properties. The more plastic a soil means the more compressible, higher shrinkage-swell potential and the lower is its permeability will be (Abramson et al., 1996). The larger the plasticity index, the greater will be the engineering problems associated with using the soil as an engineering material, such as foundation support for residential building and road sub grades (Bowles, 1992).

Grain Size Distribution

According to Anon (1999), a full description of soil should provide data on its particle size, plasticity, particle characteristics and color, as well as its bedding, discontinuities and strength. Anon divides soils into coarse and fine types. Coarse soils are gravels and sands, and fine soils are silts and clays.

According to AASHTO (American Association of State Highway and Transportation Officials) soil classification system, (ASTM D-3242, AASHTO Method M 145) the revised method comprises seven groups of inorganic soils, A-l to A-7 with 12 subgroups in all. The system is based on the following three soil properties: Particle-size distribution, Liquid Limit and Plasticity Index. Hence, with the required data in mind, proceed from left to right in the chart. The correct group will be found by a process of elimination. The first group from the left consistent with the test data is the correct classification. The A-7 group is subdivided into A-7-5 or *A-l-6* depending on the plasticity index, Ip.

For A-7-5, Ip < wl- 30

ForA-7-6, Ip > wl - 30

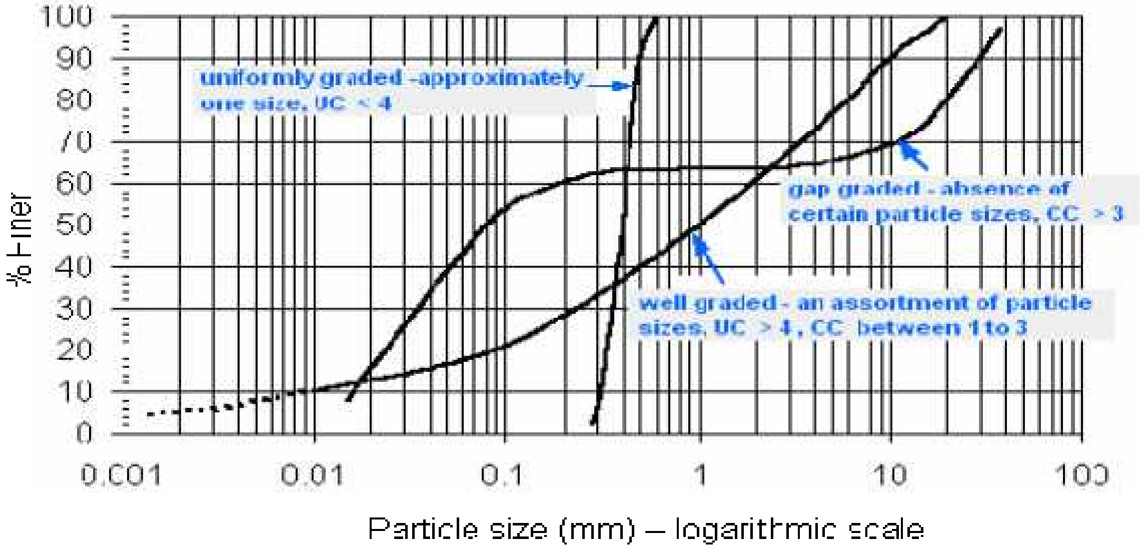


Fig. 2.2 Typical particle size distribution curves after (V.N.S, Murthy, 1990).

The USCS separates soils into two categories. One category is coarse-grained soils that are delineated if more than 50% of the soil is greater than 0.075 mm (No. 200 sieve). The other category is fine-grained soils that are delineated if more than 50% of the soil is finer than 0.075 mm. Coarse-grained soils are subdivided into gravels and sands while fine-grained soils are divided into silts and clays. Each soil type – gravel, sand, silt, and clay – are identified by grain size as shown in table…. The USCS does not differentiate silts from clays. Clays have particle sizes less than 0.002 mm. Two coefficients have been defined to provide guidance on distinguishing soils based on the distribution of the particles. One of these is a numerical measure of uniformity, called the uniformity coefficient, Cu, and defined as:

$$Cu = \frac{D_{60}}{D_{10}} \quad \text{……………………………………….....................................Eq. (2.1)}$$

where D_{60} is the diameter of the soil particles for which 60% of the particles are finer, and D_{10} is the diameter of the soil particles for which 10% of the particles are finer. Both of these diameters are obtained from the grading curve.

The other coefficient is the coefficient of curvature, C_C (other terms used are the coefficient of gradation and the coefficient of concavity), defined as:

$$CC = \frac{(D_{30})^2}{D_{10}D_{60}} \quad \text{………………………………………………………………Eq. (2.2)}$$

where D_{30} is the diameter of the soil particles for which 30% of the particles are finer. A soil that has a Cu < 4 contains particles of uniform sizes (approximately one size). The minimum value of Cu is 1 and corresponds to an assemblage of particles of the same size. The gradation curve for a uniform soil is almost vertical (Fig. 2.1). Higher values of Cu (> 4) indicate a wider assortment of particle sizes. A soil that has a Cu > 4 is described as a well-graded soil and has a flat curve (Fig. 2.1). The C_C is between 1 and 3 for well-graded soils. The absence of certain grain sizes, termed gap-graded, is diagnosed by a C_C outside of the range 1 to 3 and a sudden change of slope in the particle size distribution cure as shown in Fig. 2.1.

According to Hazen, (1893), cited in V.N.S, Murthy, (1990), the permeability of clean filter gravel/sands in a loose state can be correlated with numerical values designated D10, the effective grain size. The effective grain size corresponds to 10 percent fine particles.

Hazen found that the sizes smaller than the effective size affected the functioning of filters more than did the remaining 90 per cent of the sizes. To determine whether a material is uniformly graded or well graded, Hazen proposed the following:

Cu > 4 for well graded gravel,

Cu > 6 for well graded sand,

Cu< 4 for uniformly graded soil containing particles of the same size, and on the other hand the soil is said to be well graded if Cc lies between 1 and 3 for gravels and sands.

On the other hand Boulders, cobbles, gravels, sands, silts and clays are distinguished as individual groups on the basis of their particle size distribution (Table 2.2). Gravel, sand and silt have been subdivided into coarse, medium and fine-grained subgroups, and fine soils have been subdivided on the basis of plasticity. Coarse soils are described as well graded or poorly graded. Two further types of poorly graded coarse soils are recognized, namely, uniformly graded and gap graded (Fig. 2.2). Silts and clays are subdivided according to their liquid limits (F.G. Bell, 2007).

According to V.N.S, Murthy, (1990) a Group Index is introduced to further differentiate soils containing appreciable fine-grained materials. The characteristics of various groups are defined. The Group Index may be determined from the equation.

Group Index (GI) = 0.2a + 0.005ac + 0.01bd …………………………………..……..Eq 2.3

In which,

a = that portion of percentage of soil particles passing No. 200 (ASTM) sieve greater than 35 = (F-35).

b = that portion of percentage of soil particles passing No. 200 sieve,

greater than 15 = (F -15).

c = that portion of the liquid limit greater than 40 = (W_l -40).

d = that portion of the plasticity index greater than 10 = (I_p -10).

F = percent passing No. 200 sieve. If F < 35, use (F -35) = 0

It may be noted here that if GI < 0, use GI = 0. There is no upper limit for GI. When calculating the GI for soils that belong to groups A-2-6 and A-2-7, use the partial group index (PGI) only, that is :

PGI = 0.0lbd = 0.01(F - 15) (Ip - 10) …………………………………………………..2.4

Table 2.3. Particle size distribution of soils (F.G. Bell, 2007)

Types of material	Sizes (mm)

Boulder		> 200
Coble		60-200
Gravel	Coarse	20-60
	Medium	6-20
	Fine	2-6
Sand	Coarse	0.6-2
	Medium	0.2-0.6
	Fine	0.06-0.2
Silt	Coarse	0.02-0.06
	Medium	0.006-0.02
	Fine	0.002-0.006
Clay		<0.002

Unit Weight Relationships

A laboratory test using a pycnometer, was performed to determine the specific gravity of soil. Specific gravity is the ratio of the mass of unit volume of soil at a stated temperature to the mass of the same volume of gas-free distilled water at a stated temperature and its significance is for soil phase relationship of air, water, and solids in a given volume of the soil (ASTM D 854 2000). Specific gravity of the soil solids was determined using the following formula:

Specific Gravity, Gs = $\frac{W_o}{W_o+(W_A-W_B)}$ ……………………………………………...Eq. (2.5)

Where:

W_0 = weight of sample of oven-dry soil, g = W_{PS} - $W_{P,}$

W_A = weight of pycnometer filled with water,

W_B = weight of pycnometer filled with water and soil,

Strength Relation-ships

These are CBR, Triaxial Compression and Direct Shear Tests and Elastic Modulus of Subgrade Reaction (k-value). The k-value is used as a primary input in concrete pavement design thickness models, and can only be measured using a field test on top of the subgrade (AASHTO T- 193 test methods). According to ERA Manual, (2002) and UIC, (2005), especially for granular embankment fill material 4% CBR value is the minimum requirement for the specification.

There is no direct laboratory procedure for determining the k-value. The k-value can be estimated using any of the following methods:

Plate bearing tests on sub-grade. The 762mm (30 in.) diameter plate is loaded to a given pressure at a specified rate and the resulting deflection is measured.

$k = p/\Delta$ …………………………………………………………………...………Eq. (2.6)

Where: p = unit pressure on the plate, typically 69kPa (10 psi)

Δ = vertical deflection of the plate (mm (in.))

This test requires expensive equipment and is costly to perform; therefore Mn/DOT rarely performs it (China Railway Manual, 2010).

Deflection testing (Falling Weight Deflectometer – FWD) and back calculation of subgrade k-value. The FWD can be used to estimate the k-value by conducting the test on the top of subgrade or on the in-situ pavement structure and there is correlation with soil type and other soil properties or tests. Subgrade k-values can be estimated using soil classification, moisture level, dry density, CBR, R-Value, DCP or Mr. The Department uses a correlation developed from the results of plate load tests that conducted for Investigation (China Railway Manual, 2010, No. 183, Flexible Aggregate Base Design). The correlation is as follows:

$K\text{-value} = -1.17 + 63\sqrt{R\text{-Value}}$…………………………………...……………....Eq. (2.7)

Where: K-value is in psi/in or kPa/mm = psi/in * 0.271

Generally, once the desirable material properties have been identified, the next process in is building a quality embankment material for placement of the soil. The importance of soil preparation before rolling is not adequately appreciated. Blending of the soil to achieve a homogeneous composition and moisture content is essential for quality embankment construction (Chatwin et al., 1994).

Moisture-Density Relationship

According to AASHTO T180-93, modified proctor compaction test was mainly done to determine the relationship between the moisture content and dry density for a specific compaction effort. For the present study the optimum moisture content (OMC) and maximum dry density (MDD) was is determined. For this test a 10 lb rammer falling from a distance of 18 inches, and five equal layers of soil were subjected to 56 drops. The results obtained from proctor test, OMC and MDD, are utilized in CBR soaking and field density or percent compaction evaluation.

2.3.1.2 Engineering Geological Suitability of Rail Alignment

Topographic Analysis

Landform evaluation governs the distribution of superficial deposits, weathering profiles and slope processes. Thus, it exerts a profound influence on the engineering characteristics of the near surface materials in which the railway earthworks are constructed. Understanding the landform evolution requires an understanding of the regional tectonic evolution of the area, as this has controlled palaeolatitudes and rates of uplift and hence past climates, weathering processes and cycles of erosion (F. J. Baynes, 2005).

According to AREMA MRE (2010), transverse terrain properties are categorized into three classes, as follows:

Hilly Topography: This topography is more of unsuitable alignment, hilly and mountainous terrain where, transverse terrain slope is 10 % to 30%.

Rolling Topography: This is when the slopes more likely rise and fall (undulating) moderately and where occasional steep slopes are accustomed, accordingly, terrain slope from 5 % to 10%.

Flat Topography: This topography is more susceptible to water-sub-grade interaction and offers few obstacles to the construction of a railway, having continuously unrestricted horizontal and vertical alignment with terrain slope to the maximum of 5%.

According to AREMA MRE (2010), the following are the general guidelines regarding gradients:

- A rise of 1m vertically for every 100m traversed horizontally, in design line should also be determined by railway class, terrain conditions, and type of traction and transport demands.
- 0% to 1.0% is considered acceptable for freight, and passenger service.
- 1.1% to 2.0% - acceptable for combined passenger and freight service if they are in compliance with maximum grades elsewhere on the line.
- 2.1% to 3.0% - may be acceptable in passenger service and short ancillary freight service.
- 3.1% to 4.0% - may be acceptable in passenger service, preferably only for short distances such as flyovers.
- Grades above 4% are not recommended.
- 0% to 0.2% - preferred for maintenance and layover facilities.

On the other hand, in flat or slightly undulating areas, gradients ranging from 1% to 1.5% appear to be feasible. In hilly areas, however, gradient of up to 2% may have to be considered. Thus, 1% to 1.5 % gradient in general and 2% gradient in exceptional cases have been adopted. However, 2% gradient will, require multiple headings and /or banking depending on train load (ERC, 2011).

Drainage Condition and Requirements

This section deals with the surface and subsurface drainage of the roadway. Since water is the principal factor in influencing soil stability in roadbed, sub-grade and slopes control of surface and subsurface water is the most important aspect in railway design and maintenance.

Surface water from the roadway area, and sometimes surrounding topography, is usually handled by a system of ditches parallel to the roadbed with off take ditches where necessary. However, before deciding the hydraulic capacity to be provided in a structure, it is advisable to make a thorough search to determine what precipitation and stream flow (design flow in cubic feet per second) records are available in the general region of the project site. For structures on unstable soils, 3 feet per sec may be the maximum allowable velocity without damaging scour; generally, 3 to 6 feet per second will cause little, if any, scour in fairly good soils. Culverts and other paved waterways are frequently designed for flows as high as 10 feet per second (AREMA, 2010).

On the other hand, according to China Railway Manual 2010, the subgrade shall be designed with complete and expedite drainage system. The drainage facility shall be properly arranged, and rationally connected with the drainage facilities of bridge, culvert, tunnel and station field, with enough drainage capability. Drainage facility shall be designed according to drainage area, surface shape, surrounding landform, geological condition, ground water condition and climate etc. Design of water drainage facility of subgrade shall be combined with comprehensive utilization of water and soil conservation, farmland, water conservancy. Drainage of subgrade in city area shall be coordinated closely with regional irrigation and drainage and sewage system.

The remaining water left from a surface water, infiltrates into the soil and becomes either ground water or capillary water. Where ground water is high, subsurface drainage may be needed to draw the water table down so that softening of the subgrade soils, sloughing, or instability of slopes will not occur. Hence, capillary water cannot be removed by drainage but can sometimes be controlled by lowering the water table. Lowering the water table will assist in reducing the amount of heaving track caused by frost, reduce the pumping and infiltration of soil into the sub-ballast and ballast sections, and reduce the potential for developing ballast pockets (AREMA, 2010).

2.3.2 Sub-Ballast Layer

It is one of the track substructure components that lay between ballast and sub-grade. The importance of this layer is: (1) to reduce the stress level from ballast to sub-grade, (2) to prevent the interpenetration of ballast material into the sub-grade and finally, (3) to facilitate free drainage of precipitation, (4) to prevent emigration of fine sub-grade material into ballast layer which affect the drainage performance of ballast layer (Raymond, 1985).

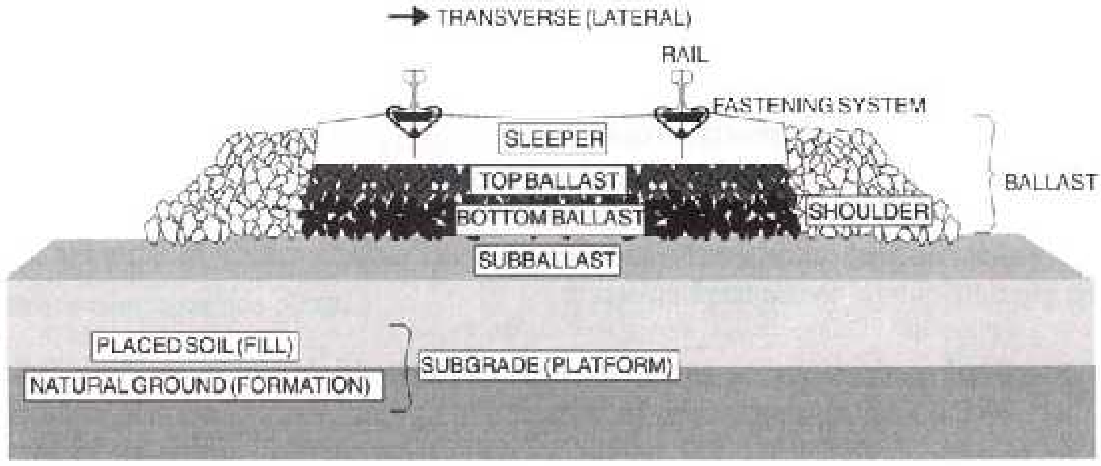


Figure 2.3. Track layout of a typical ballasted track- cross section (Selig & Waters, 1994).

The sub-ballast was compacted to a modified dry density of 95%. Slight fines content was required to allow satisfactory compaction and to develop a cohesive non erodable surface. Sub-ballast capping was carefully selected and commonly came from the distal portions of alluvial and colluvial fan systems (F. J. Baynes, 2005).

2.3.2.1 Engineering Geological Investigation of Sub-Ballast

Subballast should be either in conformity with the Manual for Railway Engineering (AREMA MRE) or be an aggregate screening of 19 mm maximum size, 3mm minimum size placed on a sublayer of sand in conformity to ASTM D1241 and one of its grading D, E or F (Gerald P. Raymond, 2005).

According to railways research designs and standards organization of India (RDSO, 2007), blanket/ sub-ballast is a layer of coarse grained material between ballast and sub-grade, spread over the entire width. On some other railway systems of the world, this layer is also called as sub-ballast. This manual also describes the functions of this substructure into two. These are:

Primary Function:

To reduce the stress - It reduces the traffic induced stresses on top of sub-grade to a tolerable limit. This function must be fulfilled to avoid track foundation failures.

Secondary Functions:

- **Separation Function:** It prevents penetration of ballast into sub-grade and also prevents upward migration of fine particles from sub-grade into ballast.
- **Drainage Function:** It should intercept water coming from ballast away from sub-grade and at the same time, permit drainage of water that is flowing upward from the sub-grade.
- **Prevention of Mud Pumping:** It prevents mud pumping by checking attrition of sub-grade particles by ballast.

And therefore, any sub-ballast would be accepted, if specifications of blanket material fulfill following criteria:

- The material is coarse grained, hard and well graded.
- Maximum percentage of fines (particle size less than 75 microns) present in blanket material is limited up to 10% to 12%. Allowing more fines in blanket material will lead to plasticity behavior of blanket material. Also, a minimum percentage of fines are required to give binding property to the blanket material so that erosion of blanket material does not take place due to high intensity rainfall.
- Material does not liquefy under vibrations caused by train movement and, therefore, is well graded.

2.3.2.2 Engineering Geological Parameters of Sub-Ballast

Grain Size Distributions

Prepare the gradation curve for the sub-ballast by plotting the grain size distribution for the sub-grade on a semi-logarithmic paper, using a logarithmic scale (as illustrated under subgrade material review above) for the grain sizes and the natural scale for percent passing. Determine the grain-sizes at 15%, and 50% points on the chart. Use these values with relevant ratios from Table 2.5 to compute the limiting grain sizes at the 15% and 50% passing lines on the chart. The maximum grain size of the sub-ballast must not exceed the maximum grain size of the track ballast. No more than 5% of the sub-ballast should pass the No. 200 sieve. Then, construct lines connecting the minimum and maximum points to set limits for the sub-ballast material (AREMA MRE, 2010). Soils prone to liquefaction falling in gradation zone and having coefficient of uniformity, $Cu < 2$ should be adequately designed to take care of this (RDSO, GE-1, 2003).

Physical and Textural Properties

The sub-ballast shall be a granular material so graded as to prevent penetration into the subgrade and penetration of track ballast particles into the sub-ballast zone. Applying the filter principle used in drainage to the grading of the subgrade material will determine the grain size distribution of the sub-ballast. Most state highway specifications include standard gradations for densely graded aggregate (DGA) and aggregate base course (ABC). These gradations may meet the requirements for use as sub-ballast (AREMA MRE, 2010).
Most, engineering geological characterizations and specifications stated above for subgrade material and for ballast material below, by different standards and manuals can properly work for the determination of subballast material as well (AREMA MRE, 2010, PAGE 107).

2.3.3 Ballast Layer

Ballast is a selected crushed and graded aggregate material which is placed upon the railroad roadbed for the purpose of providing drainage, stability, flexibility, uniform support for the rail and ties and distribution of the track loadings to the sub-grade and facilitating maintenance. There are distinct differences in the mineral composition of the various aggregate materials used for roadway ballast applications and the respective in track performance of those materials. Likewise, many variations exist in the mineral properties of aggregate materials within the same general nomenclature of the aggregates known as granites, trap-rocks, quartzites, dolomites, and lime stones. One particular aggregate material may possess most of the desirable characteristics for a good ballast material while a deposit of apparently similar material located in the same general geographical area will not meet the applicable specification requirements for railroad ballast (AREMA MRE, 2010). High standards must be established for railroad ballast to provide a quality track structure. Likewise, ballast required for concrete tie installations must exhibit some different behavioral and performance characteristics than those ballast materials which will provide satisfactory field performance for wood tie installations. Ballast is an integral part of the roadbed structure. The ballast section must react to track loadings in combination with the superstructure and subballast to provide supporting strength for the track and roadbed commensurate with specific railroad loadings and operating requirements. To provide track stability, the ballast must perform several well defined functions. The ballast must sustain and transmit static and dynamic loads in three directions (transverse, vertical and longitudinal) and distribute those loads uniformly over the subgrade. The prime function of the ballast is to drain the track system. The ballast must also perform a maintenance function to provide proper track alignment, cross level, and grade. According to AREMA MRE, (2010), the preferred ballast materials would be a clean and graded crushed stone aggregate and/or processed slag with a hard, dense, angular particle structure providing sharp corners and cubicle fragments with a minimum of flat and elongated pieces. These qualities will provide for proper drainage of the ballast section. The angular material will provide interlocking qualities which will grip the ties more firmly to prevent movement. Flat and elongated particles in excess of the maximum as specified in the specification could restrict proper consolidation of the ballast section. The ballast must have high wear and abrasive qualities to withstand the impact of traffic loads without excessive degradation. The stability of the ballast section is directly related to the internal shearing strength of the assembly of ballast particles. The material must possess sufficient unit weight (measured in pounds per cubic foot) as set out in the specification to provide a stable ballast section. The ballast must also provide high resistance to temperature changes, chemical attack, exhibit a high electrical resistance and low absorption properties. A ballast material should be free of cementing properties. Deterioration of the ballast particles should not induce cementing together of the degraded particles. Cementing reduces drainage capabilities, reduces resiliency, and provides undesirable distribution of track loads and in most instances results in permanent track and roadbed deformations. Cementing also interferes with track maintenance. Basically, ballast materials are placed and tamped in the ballast section in accordance with similar maintenance practices. The materials are then subjected to basic loading patterns, however, there are several factors that will materially affect the in track performance and stability of ballast materials. Drainage is the first and prime consideration in the roadbed maintenance and performance of ballast material.

Individual ballast particles must provide a free-draining and clean section for proper drainage of surface water to parallel side ditches or runoff areas. Excessive moisture in subgrades and ballast sections are the primary sources of track roadway problems. Side ditches should be free-draining and prevent standing water that could saturate the roadway subgrade. A wet ballast section reduces the shearing strength of the assembly of ballast particles and dirty, moist ballast sections will support the growth of vegetation that reduces the drainage capability of the ballast material. Drainage is a most important factor in contractive and expansive subgrade soil conditions which are prone to cause pumping conditions in the roadbed section.

2.3.3.1 Engineering Geological Investigation of Ballast Stone

According to Bele, (2007) Some common tests to evaluate the quality of aggregate materials are gradation test, specific gravity test, water absorption test, impact value test, crushing value test, abrasion resistance test, soundness test, shape test, Slake durability test and polished value tests.

Gradation of the Ballast Aggregate

Ballast testing has been carried out in accordance with the requirements of internationally accepted standards. In order to fix the crushing size for the rock samples, the gradation requirement for the ballast has to be initially selected. The AREMA 2010 No. 4A gradation for ballast, which is recommended for main track line, is adopted. Ballast, according to size AREMA 2010 No. 4A, shall fall within the graduation requirements presented in (Table 3.1).

According to Ionescu, (2004), cited in Bayisa Ragassa, (2011), the gradation of ballast stone for European and Australian systems the maximum particle size is 63 mm and minimum particle size is 13.2 mm; while in most of the North American railway systems the minimum size of 4.76 mm and maximum size of 51 mm size is preferred as a suitable ballast aggregate. However, this standard in this literature do not indicate the percentages of these maximum and minimum aggregate sizes.

Table 2.4 Gradation Standard size of AREMA No. 4A

Sieve size (inch)	Sieve size (mm)	Percentage passing by weight
2½	63.0	100
2	50.0	90-100
1½	37.5	60-90
1	25.0	10-35
¾	19.0	0-10
3/8	9.5	0-3

Table 2.5 Particle Grading on Top Ballast Belt (As to TB10082-2005, China manual 2010) page 28

Square mesh sieve (mrn)	37.5	30	25	20	15	10
Cumulative by mass passing sieve (%)	100	95~100	55~75	25~40	5~15	0~5

In addition to this the presence of very large grain size can cause the small numbers of ballast stone underside the sleeper which reduces the load distribution capacity of the ballast layer and affects the sub-grade (Bonnett, 2005). Therefore, Bonnett (2005) suggests overcoming gradation problem in ballast aggregate that “the sieve analysis, not more than 3% by weight should be retained on a 50 mm square mesh sieve and not more than 2% should pass through a 28mm sieve”.

Table 2.6 Standard Mechanical Chemical Tests for Ballast Aggregate

No	Engineering Properties	Limiting Values	Standards
1	Bulk Specific Gravities	≥2.6	ASTM C127 -91
2	Unit Weight (Kg/m^3)	≥1400	ASTM C29-91
3	Water Absorption	≤1.0%	ASTM C127-91
4	Soundness testing	≤5%	ASTM C88-91
5	Degradation/ Abrasion	≤25%	ASTM C535-91 and ASTM C131-91
6	Flakines and or Elongation Index	≤25% length to Width ratio greater than three (ASTM).	BS 812-105-1&2, ASTM D4791-91
7	Aggregate Crushing Value	≤25%	BS812-2000 Part 110
8	Aggregate Impact Value	less than 30%	(BS 812 Part 110, 1990).
9	Percent Material Passing No. 200 Sieve	1%	ASTM C 117
10	Test For Friable Materials	0.5%	(ASTM C 142)
11	Percent of flat and or elongated particles	≤5%	ASTM D4791

Specific Gravity Test

According to Raymond G.P., (1985), the specific gravity of the ballast stone is the ratio between its density and the unit weight of water. This property is the aggregate physical property that is related to the density of the rock material and controls the stability condition of the ballast material. As a result, it is one of the factors that control both vertical and horizontal holding capacity of the track. Also indicated by holding capacity of the ballast material is increased as the specific gravity (track mass) is increased.

Based on the facts and the procedure used during the test the specific gravity of an aggregate can be expressed as; oven-dry (OD) specific gravity, saturated surface-dry (SSD) specific gravity or as apparent specific gravity.

To calculate the specific gravity of aggregate following formulae can be used;

And Let:

A = mass of oven-dry test sample in air, g

B = mass of saturated surface-dry test sample in air, g

C = apparent mass of saturated test sample in water, g

Then:

$$Specific\ gravity\ (OD) = \frac{A}{B-C}$$ ……………………………………………Eq. 2.8

$$Apparent\ specific\ gravity = \frac{A}{A-C}$$ ……………………………………………Eq. 2.9

$$Specific\ gravity\ (SSD) = \frac{B}{B-C}$$ ……………………………………………Eq. 2.10

Unit Weight (Bulk Density) Test

According to ASTM C29, (1997) and Abebe Dinku, (2002) this aggregate testing method can be use to determine the unit weight (bulk density) of the ballast stone both in loose or compacted condition and also help to calculate the percentage of void content between the aggregate particles. Actually it is use only for the aggregates those have the maximum size of less than 125mm. as it is obvious, the unit weight of an aggregate is the weight (mass) of an aggregate per its unit volume. Therefore it required determining mass and volume of an aggregate and then it is possible to calculate the bulk density (unit weight) of an aggregate both at the dry and SSD condition and the percentage of void content between aggregate particles by using the following simple mathematical approaches.

And let:

M = Bulk density (unit weight) of the aggregate in dry condition (kg/m^3),

G = Mass of the aggregate plus the measure (kg),

T = Mass of the measure (kg),

V = Volume of the measure (m^3),

MSSD = Bulk density (Unit weight) of the aggregate in saturated surface dry condition,

A = percentage of water absorption, according to ASTM C 127 testing method,

S = Bulk specific gravity in dry basis, according to ASTM C 127 testing method,

W = Density of water (998kg/m^3),

Then, $M = \frac{G-T}{V}$ ……………………………………………Eq. 2.11

MSSD = $M\left[1 + \frac{A}{100}\right]$ ……………………………………………Eq.2.12

Water Absorption Test

This water absorption is done by immersing the oven dry ballast into the water (ASTM C 127 20011) standard testing procedure. It determines the ability of rock particles to retain water which could causes degradation of the ballast by freezing or facilitating the chemical reaction for chemical weathering (Raymond G.P., 1985). The track ballast faces different climatic change such as; rainy season and dry season; hence the water absorption test is s means to evaluate the ability of ballast material to withstand different climatic changes.

According to Nurmikolu, (2005), if procedurally, the same rock type with different degree of weathering; the rock with a high degree of weathering has high water absorption capacity. In another way the high water absorption value of the ballast material can show the existence of high porosity in the sample and its high susceptibility to frost weathering. It also shows a sign of chemical weathering in the aggregate.

Aggregate Impact Value Test

The aggregate impact value is a strength value of an aggregate that is determined by performing the Aggregate Impact Test on a sample of the aggregate in question. The test is fully described in, BS 812 (1990): Testing Aggregates: Part 112 - Method for determination of aggregate impact value. The AIV is the percentage of fines produced from the aggregate sample after subjecting it to a standard amount of impact. The standard amount of impact is produced by a known weight, i.e. a steel cylinder, falling from a set height, a prescribed number of times (usually 15 times), onto an amount of aggregate of standard size and weight retained in a mould. In the same as in ACV to calculate the AIV for the dry condition following equation can be used:

$$AIV=\frac{M2}{M1}x100 \quad \text{Eq. 2.13}$$

Where; 'M1' is the mass of the tested specimen in g and 'M2' is the mass of material passing the 2.36 mm test sieve in g (Abebe Dinku, 2002).

Aggregate Crushing Value Test

It is a measure of the ballast material due to gradually applied load given by British Standard Institution, British crushing value test, British standard 812 (1990). The specifications and standards for the limit of the quality of the ballast stone is as specified in the table 3-3 above. And to calculate the aggregate crushing value (ACV) following relation can be used;

$ACV=\frac{M2}{M1}x100$ …………………………………….……...……………….……..…Eq. 2.14

Where; ‘M1’ is the mass of the tested specimen in g and ‘M2’ is the mass of material passing the 2.36 mm test sieve in g (Abebe Dinku, 2002).

Abrasion Resistance Test

Aggregates must be tough and abrasion resistant to prevent crushing, degradation, and disintegration when stockpiled, placed with a paver, compacted with rollers, and subjected to traffic loadings. Gravels, which lack adequate toughness and abrasion resistance, may cause construction and performance problems (Mathewos, 2012). Degradation occurring during production can affect the overall gradation and, thus, widen the gap between properties of the laboratory designed mix and field produced mix. Los Angeles Abrasion test measures the hardness of an aggregate. In accordance with the requirements of the sample is placed in a drum with steel balls. The drum is rotated and the balls grind down the aggregate particles. Soft aggregates are quickly ground to dust, while hard aggregates lose little mass. Abrasion resistance applies only to coarse aggregates. The acceptable limits are set by the Los Angeles Abrasion Test AASHTO T-96. The limits vary from 30.0 to 50.0 percent, depending on the classification of the aggregate. The percentage is a measure of the degradation or loss of material as a result of impact and abrasive actions. In general, low values of L.A. abrasion loss are required. The LA Abrasion value (LAA) is then calculated using equation 3.4 (AASHTO T-96).

LAA= (Total weight - material retained on 1.7mm)/ Total weight * 100 ………… Eq. 2.15

Therefore, gradations containing particles retained on a 25 mm sieve shall be tested by ASTM C 535-95. Gradation with 100% passing on a 25 mm sieve shall be tested by ASTM C 131 - 95. On the other hand, according to Chanda & Krishna (2003), as cited in Bayisa ragassa, (2011), the Los Angeles Abrasion test, the stones with percentage loss less than 40% after 500 revolutions are qualified to be used as ballast.

Soundness Test

It is a typical aggregate test to evaluate the loss of particles after several immersions of the aggregates into sodium or magnesium sulfate. It is a cyclic aggregate test which involves immersing and drying the aggregates and performing sieve analysis and calculating the percent loss from each sieve size. Immersion and drying are considered to be one complete cycle. Care must be taken while performing soundness test since the result is affected by different cases, i.e. nature of the surface (degree of fracture), number of cycle used during the test, the porosity of the particle, amount and type of solution used and the chemical composition of the aggregates (Abateneh, 2011). Moreover, the limiting standard for both cases above is ≤5% (ASTM C88-99)

Shape Tests (Flatness or Flakiness)

These two names are indicated and used for the same shape characteristics in different standards and countries. Flaky materials, according to BR, are particles where the ratio of thickness to width is less than 0.6. So an index can be computed from the percent weight of flaky aggregate particles. According to U.S. Army corps of Engineers, a material is said to be flat, when a particle has a thickness to width ratio of less than $\frac{1}{3}$. Based on the U.S. Army corps, ASTM standards (T. Selig, et al, 1994) recently provides different ratios for flat particles. i.e., $\frac{1}{5}$., $\frac{1}{3}$., and $\frac{1}{2}$. The Australian railway ballast specification also recommends the amount of flaky particle on 6.7 mm sieve shall not exceed 30 % (ARTC, A. R., 2007, 01 04). The different limiting values of the elongation index are listed below according to different standards used in different countries (T. Selig, et al, 1994).

- British Standard – particle as one with a length to width ratio of more than 1.8.
- U.S. Army corps of Engineers – particle as one with a length to width ratio of greater than 3.
- Recent ASTM – based on corps of Engineers method, it provides a choice of three ratios: 2, 3 and 5.

Even though the limiting factor was not stated in brief, the shape factor is also another engineering characterization of the ballast stone. It is the ratio of the summation of all longest dimensions to smallest dimension of the particle. Its approximate evaluation of the flatness and elongation characteristics of the particles indicates how the shape of the particles looks like. Shape factor is designated by F_S.

$$F(S) = \sum_{i=1}^{\infty} \left(\frac{Li}{Ti}\right) \qquad \text{Eq. 2.16}$$

Where: Li and Ti are Longest and Smallest dimension of the particles respectively (Abateneh, 2011).

Slake Durability Test

Bele, (2007) emphasized that the existence of clay minerals such as montimorillonite in the rocks such as in dolerite and basalt can cause the rapid breakdown of the rocks due to theirs high water absorption (swelling) capacity. When the amount of clay minerals in rocks is measured, it determines the resistance of the rock to weakening and disintegration when it is subjected to wetting and drying cycles. The high amount of clay content in the rocks can influence this value.

Polished Value Tests

PSV is commonly used to measure the resistance of stone to surface polishing action (BS 812 part 114, 1990). It is to say that, resistance of an aggregate to lose their initial rough surface and develop a polished surface. It is one of the factors that can affect the surface skidding resistance of an aggregate. PSV is one of the factors that can affect the surface skidding resistence of an aggregate. According to Prentice (1990), cited in Bayisa Ragassa, (2011), an aggregate polished stone value (PSV) ranges from 20-70. Higher values show good resistance of an aggregate to polishing and low skidding. Most of igneous rocks, such as; granite, basalt, gabbros, dolerites have a PSV in the range of 35-65.

2.3.3.2 Petrographic Thin Section Analysis

Petrographic thin section analysis is the systematic description of aggregate rocks in a hand specimen and thin section. According to Raisanen et al., (2006), well assessed perception of the petrographic properties of a ballast stone helps to determine the properties of aggregate material in crushing. Further physical, chemical (ease for weathering) and mechanical tests are also needed to evaluate the engineering properties of an aggregate.

Coarse grain intrusive rocks such as granite are not suitable for an aggregate material like the fine grained, because of showing low aggregate crushing value. Very fine grained and glassy igneous rocks are also unsuitable for aggregate material, because during crushing such rocks form chips with sharp edges that are highly polished and tend to break into flakes that can affect the flakiness index value of the ballast stone (Bele, 2007; Raisanen et al., 2006).

On the other hand, the presence of micro cracks in individual grains in a rock can affect the susceptibility for dynamic wheel load degradation and finally for weathering. These lower the quality of engineering properties of a ballast stone (Bele, 2007). Therefore, different mineralogical and textural verifications have to be conducted as to the ASTM standard (ASTM C-227 and C-289, 1991).

CHAPTER III - THE STUDY AREA OVERVIEW

3.1 Preamble

Geologic, physiographic and climatic condition setups of an area have impact on the formation of natural sub-grade soils. The Addis Ababa Light railway transit (LRT) line stretches a total length of 34 km and stretches from Ayat square to Lideta (East-West Line) and from kaliti through meskel-square lideta to giorgis square (South –North line). As a part of the study area 2km buffer zone along the proposed Ayat- Mexico Square light railway transit line was also identified.

The primary reason to identify the present research problem was its potential for being problematic for the construction due to expansive sub-grade soil and the non availability of sufficient amount of fill material for the project. Thus, attempts are being made to recommend alternative way to overcome the problems along the corridor.

This chapter deals with the description of climatic, physiographic, geologic and hydrogeologic condition which influenced the formation of natural subgrade in the project site and the surrounding area. This section also includes description of the seismic condition and soil type distribution of the study area and its surrounding. Finally a brief description of the project site condition is included.

3.2 Location of the Study Area

The Addis Ababa city east-west light railroad transit, the study area, routes nearly at the center of the city (Fig.3.1). The city overlies at the western margin of the Main Ethiopian Rift and is a part of western Ethiopian highland. It is passing through the main squares such as Ayat, Meganagna, St. Urael church, Meskel Square, Mexico Square and lideta with the total length of 17.35km. The starting point of the research project road is geographically located at 997012m north & 486363m East with the an elevation of 2396m a.s.l. at Ayat square and ends at Mexico square which is geographically located at 995838m North & 471863m East with an elevation of 2363m a.s.l.

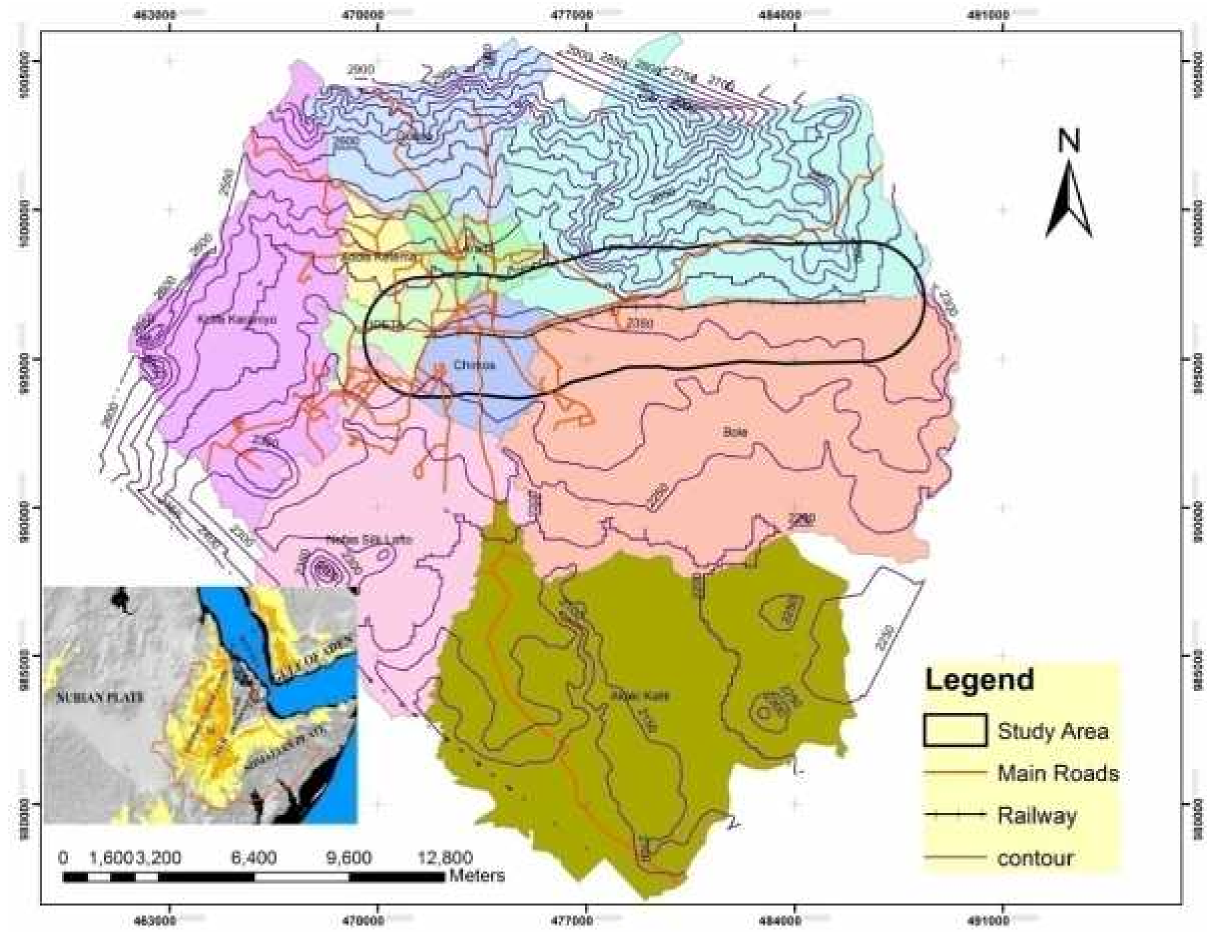


Fig 3.1 Location Map of the Study Area.

3.3 Physiographic Features and Classifications

Ethiopia can be divided into four major physiographic regions as it can be widely known as the Western plateau, Southern plateau, the Main Ethiopian Rift and Afar Depression (Mengesha Tefera et al., 1996).

Addis Ababa is located on a plateau with an elevation ranging from 2000 to 2800m a.s.l on the shoulder of the western Main Ethiopian Rift (MER) escarpment. The morphology of Addis Ababa is a direct reflection of the different volcanic stratigraphic successions, tectonic activities and the action of erosion between successive lava flows (Tamiru Alemayehu et al., 2006).

The physiographic map of Addis Ababa shows that the City is founded on an area with a well developed morphology. It is surrounded by high rising mountain systems in all directions and the center of the city lies on an undulating topography with some flat land areas. The urban area of the city is deeply dissected by numerous valleys formed by the river systems crossing the city from north to east.

Intoto mountain ridge forms the northern boundary of the city following the East-West trending Ambo – Addis Ababa major fault system. The elevation of this ridge ranges from 2600m to 3200m. The volcanic mountains; Mt. Wechecha chain to the west, Mt. Furi in the south - west and Mt. Yerer in the south east are the high massive volcanic centers rising to elevations of 3385m, 2839m and 3100m a.s.l, respectively (Fig. 3.2).

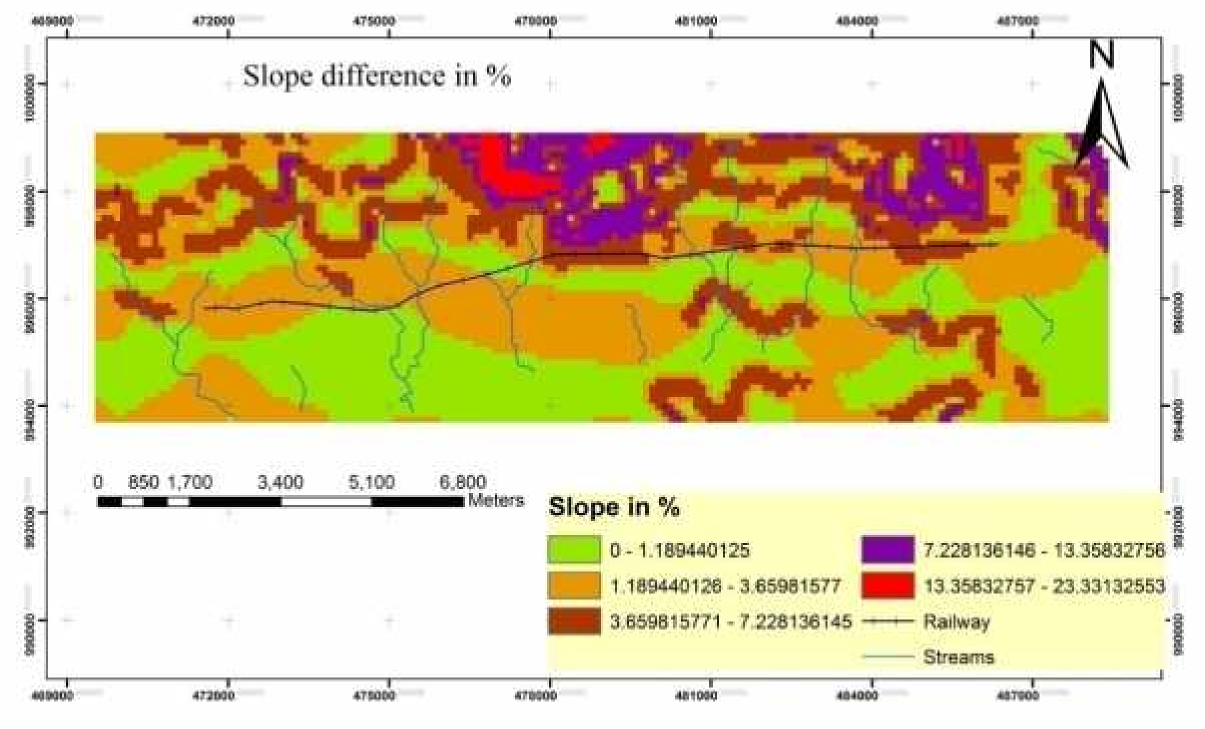


Fig. 3.2 Physiographic map of the study area

The center of the city lies on an undulating topography with some flat land areas. The topography is undulating and form plateau in the northern, western and southwestern parts of the city, while gentle morphology and flat land areas characterize the southern and south eastern parts of the city. Moreover, it is not uncommon to see sharp changes in the inclination of the slope and some flat land areas in different parts of the city (UNEP/UNESCO/UN-HABITAT/ECA SCIENTIFIC, 2003).

Topographic classifications and analysis's were to extract compare and contrast the extent to which the proposed light railway transit line pass to the current topographic features in order to meet project design as to internationally accepted standards and specifications. As a general understanding, rail way design and speed depends on the actual topography or physiographic features of the area (USACE, 2004). As can be observed in the physiographic map, the alignment of Light Railway Transit (East-West), the route line will therefore have to traverse two distinct types of topography, flat topography and track section in the range of rolling topography (section 2.3.1.2). This implies that gentle sloping (rolling) topography requires a greater amount of either embankment fill material, or cut during rail line construction than flatter topography. A perusal of Fig. 3.2 clearly indicates that much of the railway line (about 14.37km of the segment) would be relatively flat, mostly having slope gradient of less than 4%. Moreover, about 2.97km of the railroad will be gentle sloping topography with approximately slopes gradient greater than 4%. The total elevation range of the study area is varying from 2354m which is the minimum elevation on the alignment and to the maximum of 2490m with an average elevation of 2270.5m (Fig.3.2).

3.4 Geology of the Study Area

3.4.1 Regional Geology

Extensive areas of the highlands of Ethiopia on both sides of the rift valley are covered by Tertiary (Trap series) volcanic rocks which are mainly basalts with subordinate acidic rocks. In the rift valley, subsequent to the formation of the rift valley, the Trap series were overlain by a variety of younger volcanic rocks of basalts, ignimbrites and rhyolites (Tesfaye Chernet, 1988). The Tertiary Ethiopian volcanism occurred in three main stages separated by periods of quiescence. These are; the pre-Oligocene stage (Ashanti formation), the Oligocene-Miocene stage (Aiba, Alaji and Tarmaber formations) and the Miocene- Pliocene stage (Fursa, Balchi and Bishoftu formations) (Zanettin et al, 1978).

Addis Ababa is located in the western margin of the Main Ethiopian Rift Valley. The margin is more recent, shows Mio-Pliocene volcanic and is characterized by normal faults down thrown towards the rift. The upper (outer) boundary of the margin is marked by the large fault running approximately east-west immediately north of the Addis Ababa-Ambo road.

3.4.2 Geology of Addis Ababa

Addis Ababa is located in the western margin of the Main Ethiopian Rift Valley. According to Haileselassie Girmay and Getaneh Assefa (1989) cited in Habtamu Solomon, (2011), established the straigraphy of Addis Ababa area based of K/Ar absolute age determination taken from different literature and field work. The suggested Miocene-Pleistocene volcanic succession from bottom to top is Alaji Basalts, Intoto Silicics, Addis Ababa Basalts, Nazareth Group and Bofa Basalts.

3.4.2.1 Alaji Basalts

This unit is composed of basalts, which show variation in texture from highly porphyric to aphanitic. The unit is intercalated with gray and glassy welded tuff. The outcrops of Alaji basalts are situated in the North and Northeastern part of Addis Ababa. They form high topography and Rhyolites have more area extent than trachytes. Trachytes and rhyolites are contemporaneous in age. The Alaji basalt is underlain by tuffs and ignimbrites and is overlain by the Entoto Trachyites (Mohr, 1967).

3.4.2.2 Entoto Silicics (Early Miocene)

The unit is unconformably overlain by Addis Ababa basalt on the foothills of Entoto hills and is composed of rhyolites and trachytes with minor amount of welded tuff and obsidian (Hailesellase Girmay and Getaneh Asefa, 1989). The rhyolitic lava flow outcrop on the top and the foothills of the Entoto ridge, predominantly in the western side. It also outcrops in the eastern part of the town around Kokebe Tsebah School. The thickness is quite variable as it frequently forms dome structure. The thickness becomes maximum on the top of Entoto ridge and thin both towards the plateau and the plain east of Addis Ababa. The rhyolites are overlain by porphyritic trachytes and underlain by a sequence of tuffs and Ignimbrites. Tuffs and Ignimbrites are welded and characterized by columnar jointing. The trachytic lava flow outcrop on the top of Entoto Ridge and its foothills. The trachyte and the alaji aphanitic basalts are separated by paleosoil indicating time gap.

3.4.2.3 Addis Ababa Basalts

Stratigraphically, this unit is underlain by the Entoto silicics and overlain by lower welded tuff of the Nazareth group. It is porphyritec in texture and mainly present in the central part of the city (Hailesellase Girmay and Getaneh Asefa, 1989). Olivine porphyritic basalts outcrop around Merkato, Teklehamanote and sidest Kilo. The Lower Welded tuff overlies both types of basalt nearby the building college, the kolfe Police School, the Kokebe Tseba School and Yeka Mariam church. On the other hand, only in the gorge of the Ketchene stream the olivine porphyritic basalt is overlain by the plagioclase porphyritic basalt.

3.4.2.4 Nazareth Group

The units identified in this group are lower welded tuff, aphanitic basalt and upper welded tuff. Welded tuffs have been related to Wachecha and Yerer Volcanisms. The group is underlain by Addis Ababa basalt and overlain by Bofa basalts. The rocks outcrop mainly south of Filwoha fault and extended towards Nazareth (Hailesellase Girmay and Getaneh Asefa, 1989). The units in this group are:

i. Lower Welded Tuff

It outcrops as small discontinuous body in Filwoha, western parts of Addis Ababa and Sululta. Generally it is overlain by the aphanitic basalt and underlain by the olivine and plagioclase porphyritic basalt. The age of this unit overlaps with the period of the activity of wechecha trachyte volcanoes.

ii. Aphanitic Basalt

It covers the southern part of the city, especially the area of Bole International Airport and Lideta Old Airfield. The rock body shows vertical curved columnar jointing together with sub-horizonal sheet jointing. Along the course of Akaki River large amygdales of calcite occur in this basalt. Kaolinite lenses are present at the contact with the younger ignimbrite.

iii. Upper Welded Tuff

It outcrops all over the southern part of the city including Bole, Nefas Silk and Railway Station; nevertheless it is also present in the central and northern part of the city. It is gray colored, vertically and horizontally jointed and composed of sanidine, anortoclase, rebekite, quartz, pumice and unidentified volcanic fragments (Hailesellasie Girmay and Getaneh assefa, 1989). The welded tuff is underlain by aphanitic basalt and overlain by young olivine basalts.

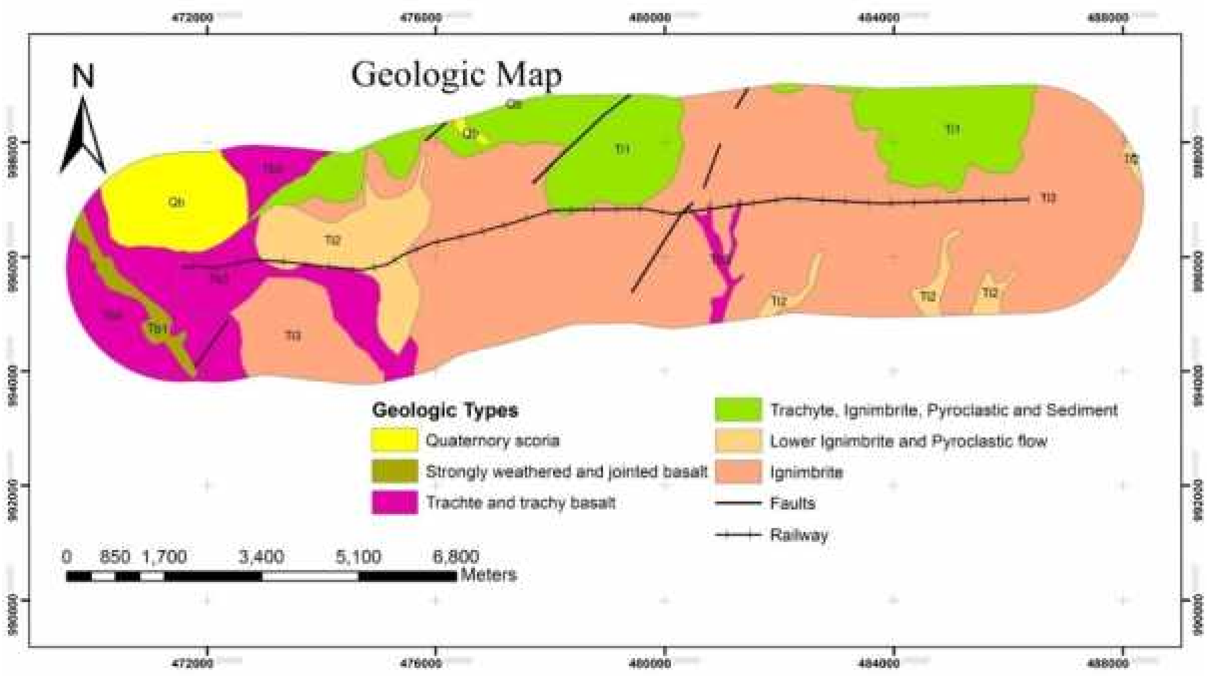


Fig. 3.3. Geological Map of the Area (Modified From Habtamu Solomon, 2011)

3.4.2.5 Bofa Basalts

Bofa Basalts outcrop southward from Akaki River where they appear in the form of boulders reaching a thickness of 10meters. They are restricted and dominated in the southeastern part of the city. This rock is characterized by big vesicles that are filled by calcite. This basalt is underlain by the tuffs which cover the welded tuff.

3.5 Soil Formations

Most of the soil formation in Addis Ababa area is as a result of the physical disintegration and chemical decomposition of volcanic rocks. The weathering products are either remain in places and form residual soils or transported and deposited in the low lying flat lands and depressions (Tamiru Alemayehu et al, 2006). The differences observed in the type and development of soils in the city depends mostly on the topography, parent rock and the degree of weathering. Although there is significant difference in the degree of weathering on the slopes, mostly soils are highly eroded and result in thin soil cover. In the localities where the topography is plain to gentle there is thick soil profile. The type of parent material and the length of time to which the parent material is subjected to weathering, control the variation in the thickness of soil. Thus, old basic and acidic rocks that outcrop in the central, western and southwestern parts of Addis Ababa are weathered and form thick soil profile. In places where young basalt and welded tuffs occur, the thickness of the soil cover is reduced.

The deterital materials that are derived from elevated areas of Entoto, Wechecha, Furi and Yerer are transported and deposited in the piedmont and along the stream courses of Addis Ababa. It covers most parts of Mekanisa, Ayere Tena, Kaliti, Akaki, Lideta, and Bole. The soil is black in color and the thickness varies from place to place primarily depending on the slope of the area. Kebede Tsehayu et al., (1990) classified the soil units of Addis Ababa based on their origin as alluvial, alluvial fan, colluvial, residual and lacustrine (Fig.3.3).

The alluvial soils which include channel and terrace deposits are found in some places along Akaki River in the west and southwestern parts of Addis Ababa and along Kebena River north of Bole area. The alluvial soils consist of more or less stratified deposits of gravel and clay transported by streams. The study indicated that sample taken from terrace deposits near Bole consists of 46% silt, 34% clay and 20% sand and classified as ML in USCS system. Alluvial fan is deposited where there is a decrease in gradient from a hill to a plain along a river section. It is coarser near the mouth of the river and become finer outwards and found in the Entoto region dissected by deep gullies Kebede Tsehayu et al (1990).

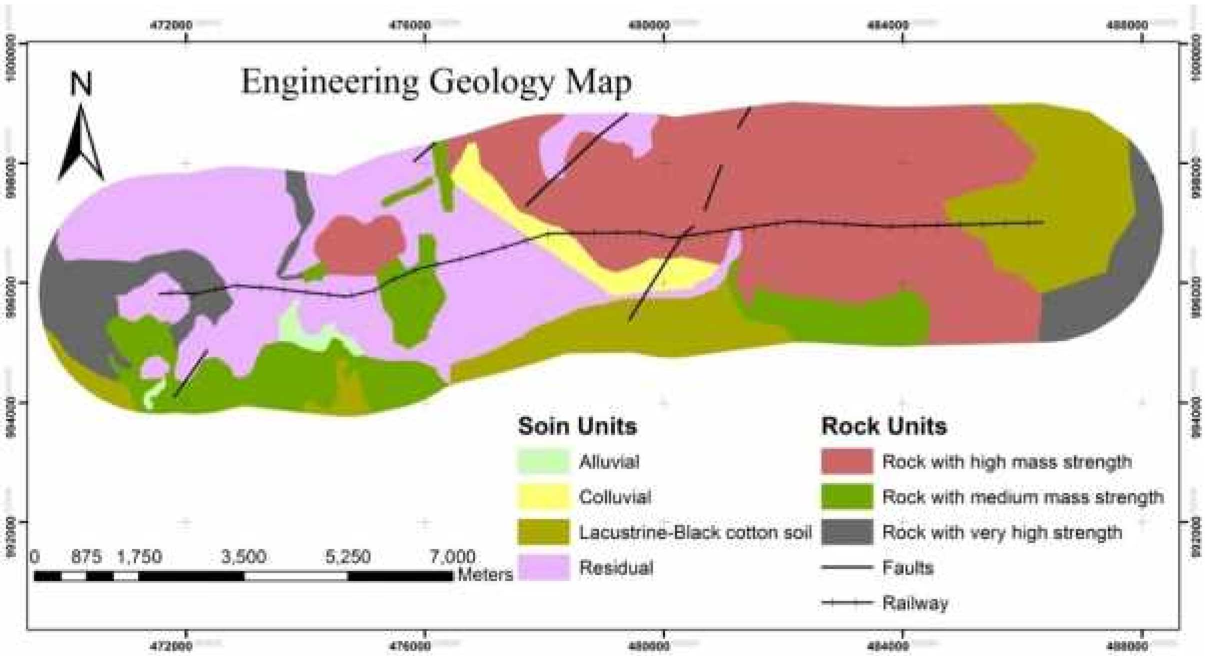


Fig. 3.4.Engineering Geological map of the study area (Modified from Kebede Tsehayu et al., 1990)

Residual soils developed in situ by the decomposition of rocks are mainly located in Gulele and Kolfe area. Sample tested provide grain size of 62% clay, 33% silt and 5% sand. In some localities reddish brown soil with a thickness of more than 10 meter is commonly seen.

Lacustrine soils, alternatively named as black cotton soil, are found in Bole, Lideta and Mekanisa areas as these areas are flat and relatively low lying (Kebede Tsehayu et al, 1990). In the same study samples taken from Bole and Mekanisa area provided 76% clay, 22% silt and 2% sand and according to USCS it is MH (rarely CH).
Observations and tests show that the low lying flat areas around Addis Ababa are dominated by black cotton soils. These soils have extremely high plasticity and very high degree of swelling as compared to the other identified soil types found in Addis Ababa. The thickness of this soil varies from place to place from 2m to 10m. The highest thickness is found in Bole area and in Beklo Bet area it is about 5m thick.

In areas where there is great contrast in the topography colluvial soils are found. These are loose and incoherent deposits, consisting of fine to coarse grain particles. Colluvial soils are mainly located at the foot slopes of northeastern part of Entoto silicics and other few places (Kebede Tsehayu et al, 1990).

3.6 Seismicity of the study area

Ethiopian seismicity characteristics is controlled and initiated by the active Ethiopian Rift System which divides the country into two along the NE–SW direction (Tilahun Mamo, 2005). The seismically and volcanically active northern Main Ethiopian rift (MER) and Afar rifts are virtually the only places worldwide where the transition between continental and oceanic rifting is exposed on land (Keir et al., 2006). The effect of earthquake is very significant on engineering structures like buildings, dams, rail tracks and roads. Hence, seismicity is highly associated with faults and fractures, which initiated problems on roads and railroads. The disconnection of the road/railroad due to the displacement of fault or fracture can cause great damages (Tilahun Mamo, 2005).

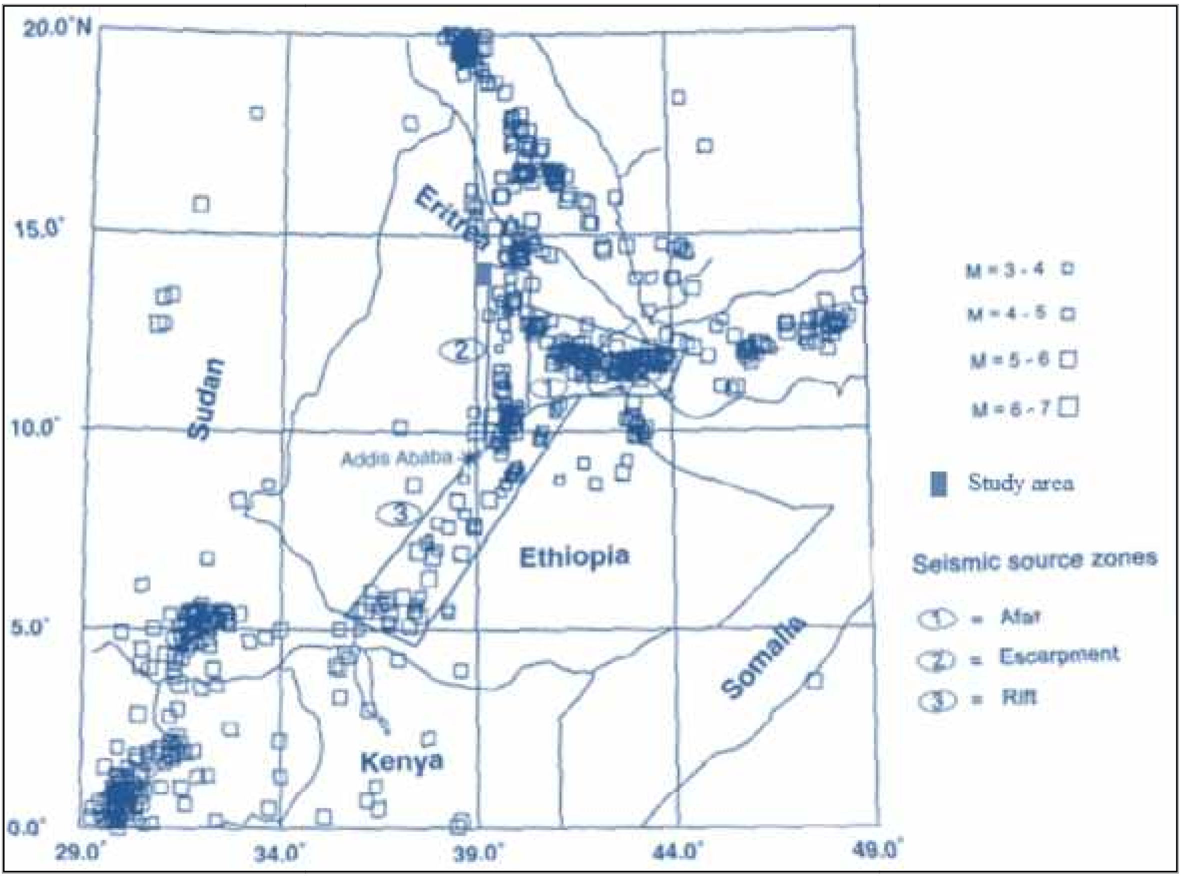
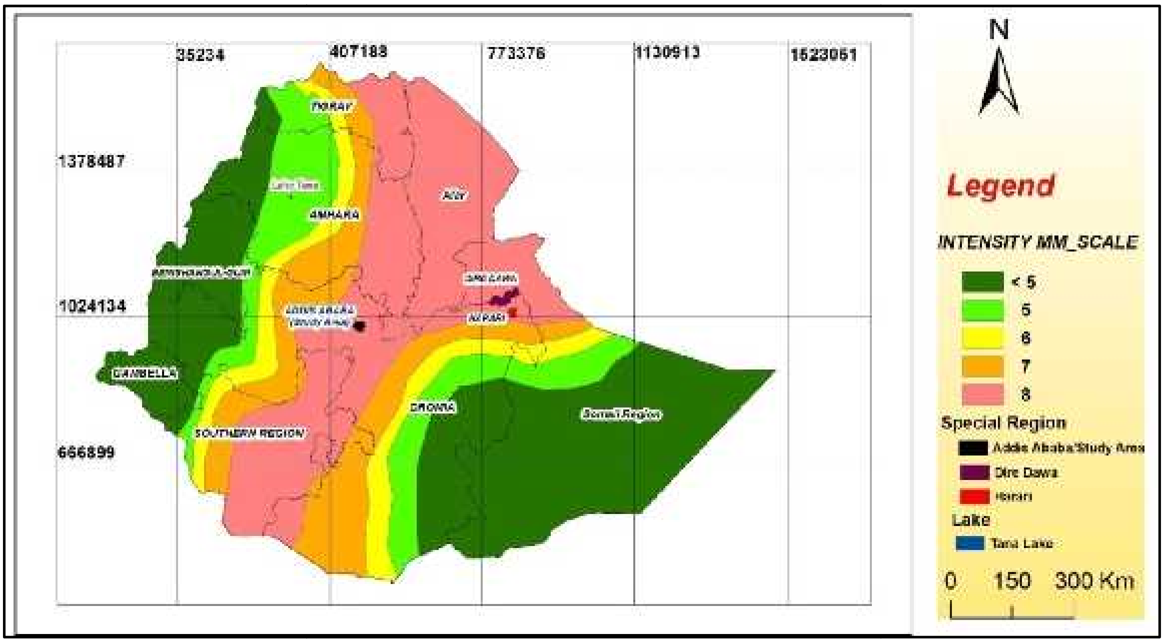


Fig. 3.5 Seismicity of the Region with the Seismic Source Zones (After Tilahun Mamo, 2005).



(Source:Laike Mariam Asfaw, 1986)

Fig. 3.6 Seismic risk map of Ethiopia 100 year return period, 0.99 probabilities

Therefore, based on their intensity, according to Laike Mariam Asfaw (1986), the seismic risk map of Ethiopia has five different zones (Fig 3.4). According to this map the present study area falls into the highest seismic risk zone of intensity 8. However, it requires consideration and a systematic study to know exactly the damage an earthquake can cause to this structure. Such study is beyond the scope of the present research. On the other hand

3.7 Drainage Pattern of the Study Area

Surface water condition in the study area includes intermittent and non intermittent streams and non organized municipal waste water. Several perennial and intermittent streams emerging from Yekka and Intoto ridges are located in the north, north-east and central part just across the proposed structure. As can be observed from Fig 3.5, most of the streams in the study area have radial pattern while some streams from northern ridges flow parallel to sub-parallel with indicated lineaments (Fig 3.3 and Fig. 3.5.)

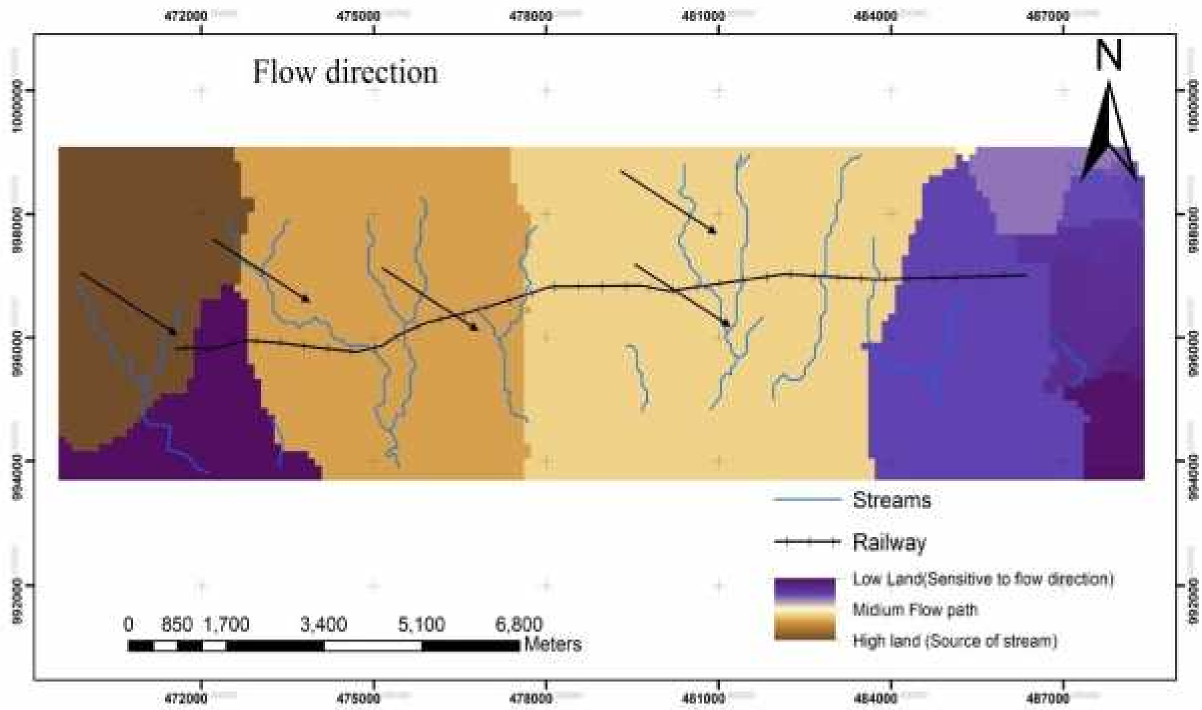


Fig. 3.7 The Drainage patterns in the Study Area

3.8 Climate of the Study Area

Ethiopia is classified into five climatic zones (National Atlas of Ethiopia, 1981). These include "Kur" (Alpine), above 3000m mean sea level; "Dega" (Temperate), 2300m to about 3000m; "Weina Dega" (Sub tropical), 1500m to about 2300m; "Kolla" (Tropical), 800m to about 1500m and "Bereha" (Desert), less than 800m. Most parts of Addis Ababa fall under the Weina Dega (Sub tropical) category.

Addis Ababa City is located at 09^0 02' N Latitude and 38^0 44' E Longitude. It is built on the steep escarpment of Mt. Intoto in the North (2900 m) to the South with an average altitude of 2400 m. This varying topography of the city has affected its spatial expansion favoring the relatively flat landscape in the south, north east and central part as a major factor contributing more the type of formation of the soil (black cotton, lacustrain, colluvial, alluvial), modes of drainage pattern and variation in weather conditions (Zeleke., 2013).

3.8.1 Rain fall in the study area

The variation in the seasonal distribution of rainfall in Ethiopia can be attributed by the reference to the position of the Inter-Tropical Convergence Zone, the relationship between upper and lower air circulation, the effects of topography and the role of local convection currents and the amount of rainfall (Daniel, 1977).

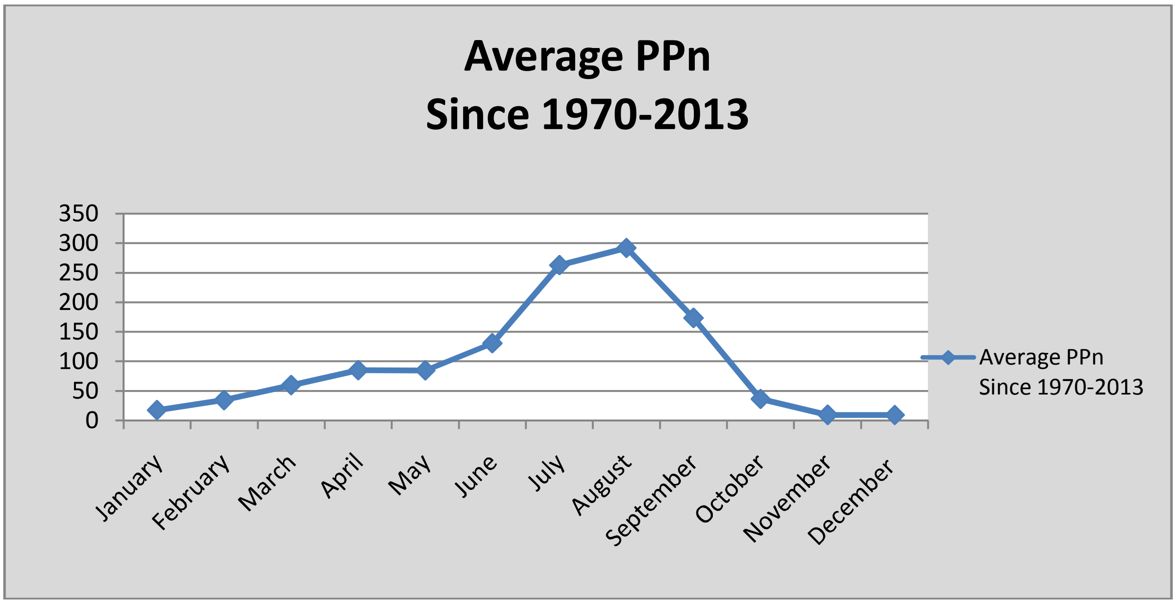


Fig. 3.8 Mean monthly rainfall of Addis Ababa (*Source: National Meteorological Services Agency as of Apr, 2014*)

The heaviest rain fall occurs during August whereas the minimum rain fall occurs in December and January at Addis Ababa Bole station (Fig 3.8). Furthermore, Bole station, records greater amounts of mean annual rainfall at the month of August. On the other hand, we can have minimum annual temperature when in the year we have got minimum precipitation.

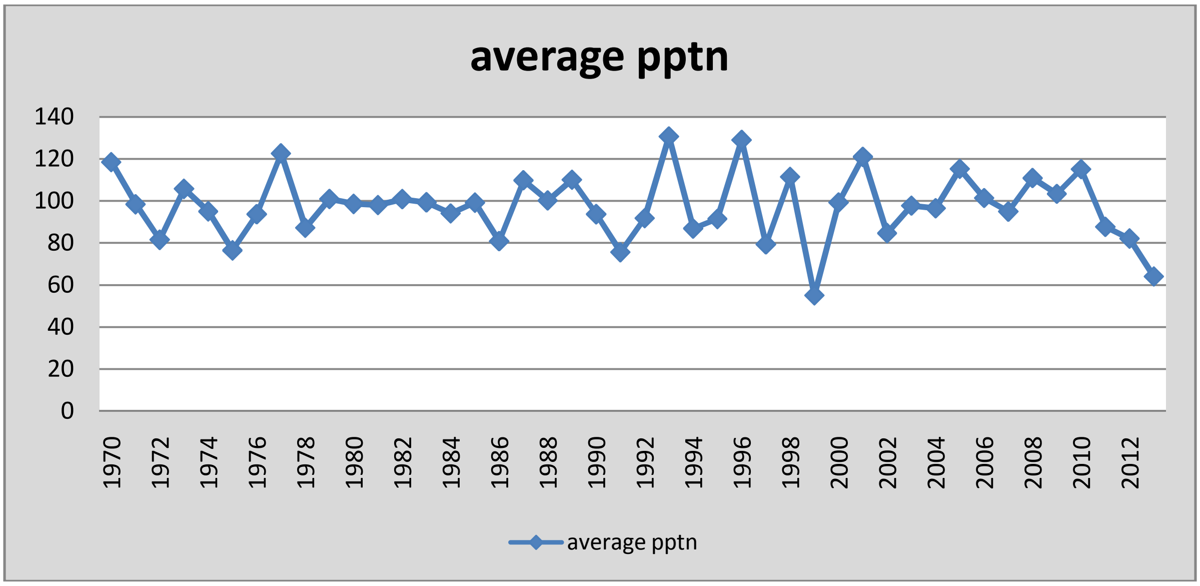


Fig. 3.9 Mean Annual rainfall of Addis Ababa (*Source: National Meteorological Services Agency as of Apr, 2014*)

For this research, the monthly total rain fall record of one station from 1970 to 2012 for Addis Ababa Bole observatory was utilized to analyze monthly and annual mean rain fall (Fig. 3.9).

As indicated in (Fig. 3.8) the precipitation occurs throughout the year and shows variation in amount from month to month. The monthly mean records of rain fall shown for each station indicates that the mean annual rain fall at Addis Ababa Bole station is 99.8 mm.

3.8.2 Temperature

To understand the temperature conditions of the study area, maximum and minimum monthly temperature in different years for the bole station of the study area was utilized (Source Ethiopian National Metrology Agency, April 2014) to calculate the general maximum, minimum and mean temperature of each station in the study area.

Hence, the diverse rain fall and temperature condition in the country is mainly because of the product of its location in the Africa's tropical zone and the complex topographic condition of the country as characterized by different settings of massifs.

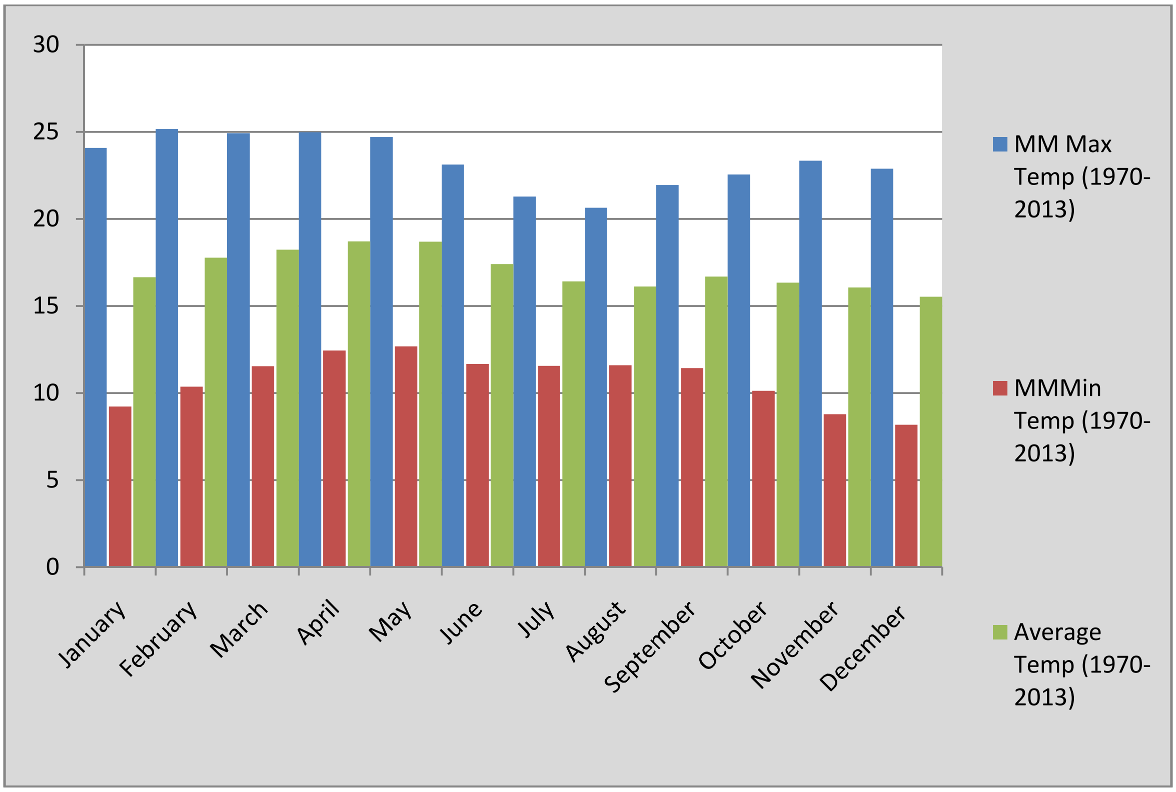


Fig. 3.10 Mean monthly Temprature of Addis Ababa (*Source: National Meteorological Services Agency as of Apr,* *2014*)

CHAPTER IV - CHARACTERIZATION OF SUBGRADE EMBANKMENT FILL MATERIAL

4.1 Preamble

The subgrade is comprised of the uppermost materials placed in the road bed embankment or the soil remaining at the base of a cut. The subgrade soil is often referred to as the foundation or road bed soil. This foundation component is usually constructed of native inorganic soil often in combination with imported soils from select borrow sources, and would be compacted to a specified density and moisture content (AASHTO Pavement Manual, 1997).

In cohesion less, disturbed samples and standard penetration test values are usually adequate. In cohesive soils it may be necessary to obtain undisturbed tube samples for tests which include classification, water content, and shear strength. Most importantly, exploration should be carried out at least below the bottom of the proposed cut or deeper if recommended by the geotechnical engineer (AREMA MRE, 2010).

For reasons of poor bearing capacity of formation, inadequate factor of safety against slope stability, excessive settlement, ballast attrition, sub-grade attrition due to mud pumping and loss of soil from formation on account of erosion etc, railway structure may develop instability. Formation failure may be on one account or in combination. Existence of one or more of these causative factors may lead to development of others (RDSO GE-039, 2003).

4.2 Over All Visual Investigation of Natural Sub-Grade Soil

In order to emphasize the alignment of Addis Ababa LRT (Ayat to Mexico) and to group homogeneous sections, the visual survey of natural sub-grade soil extension along the alignment has been carried out during the field work prior to the laboratory analysis of subgrade embankment fill material. This was carried out to assess the nature, type and extent of existing natural sub-grade soil that makes the railroad.

Surface examinations and descriptions, geological and geomorphologic description, can provide useful information about the type of soils. The morphological description includes color of the soil, soil consistence, soil texture, soil structure, texture groups etc. (China Railway Manual, 2010).

Plate. 4.1 Natural Subgrade Material (A-Lem Hotel area, B-Mexico Area, C-Megenagna Area and D Golagul Bldg Area).

The type and extent of natural sub-grade soil visualized along the route line is found to be mainly dependent on manually transported and reworked for several years. This is mainly to the depth of two to two and half meters. The other factor is the topography and geology of the project area. From Ayat square to Tele Garage and Lem Hotel to wuha limat area are characterized by very thick slightly weathered and columnar jointed ignimbritic rock unit exposure overlain by thick over burden of transported, reworked and black cotton soil. Furthermore, highly weathered ignimbrite overlain by thick (greater than 4m) from St. urael church to Bambis areas are magnificent features. This selected railway alignment generally lies more of on flat region. As a result, erosion effect is minimal along flat line sections and thus development of thick residual soil is facilitated.

From the visual inspection, during the field data verification, the natural sub-grade is well described. In most of the cases it was black cotton soil however, since the proposed railway alignment was reconstructed on formerly asphalted line, it is difficult to purely visualize the natural subgrade. In spite of this the area is described by black cotton expansive soil by many researchers so far. At most places the color of the soil varies from gray brown to dark gray mixed with transported granular soil and dark organic municipally transported materials. During the soil extension survey, clear distinctions between the individual soils types can be made due to those lines were under excavation. But there are also non distinct soil exposures due to many times construction and their similarity in origin and soil properties, and gradual transitions from one soil type to the other.

In general visually the Addis Ababa LRT route line can be divided into:

(1) From 22+400 (Ayat square) to16+300 (megenagna) and then to 13+100 (H&M Bldg) is land pass type route line.

Engineering geologically this area is mainly characterized by thick dark grey sediment of expansive (black cotton) soils (approximately greater than 4m) with slight gentle sloping topography. This area seems more nearer to the rift escarpment as the Yerer Mountain can see from in front the way to Ayat. For this route line there seems no change in geomorphology, geological out crop and soil profile up to the ring road. After the ring road there is an exposure of highly altered volcanic ash and in some scattered places of this line highly intercalated with organic soil (may be decomposed municipal waste) and expansive soil. Actually the drainage problem in this area also a magnificent as the seepage from the municipal waste percolates to the excavated natural sub-grade material.

(2) From 11+200 (H&M Bldg) - 11+590 (Golagul Bldg) is under pass type of route line.

This line begins with very thick fluvial deposits at exiting River Bridge. River Ginfille is dominant in this area for fluvial soil formation. As a result, there need to be great attention for the foundations of civil structures to be constructed. The way to Golagul Bldg is characterized by highly thick, highly jointed and slightly weathered ignimbritic rock exposures with 1m to 1.5m over burden of expansive soils. The ignimbrite is intercalated with thin yellowish tuff, which is pinched out in the expansive soils. Beyond the Golagul Bld thick highly jointed and slightly weathered ignimbritic rock pinches out and finally ends up in thick expansive clayey soil underlain by highly weathered tuff.

(3) From 12+300 (Zerihun Bldg) to 11+400 (St. Urael church) is ground pass type route line.

On gently sloping topography, the dark grey thick expansive clayey soil underlain by highly jointed ignimbrite. In this route section, the predominance of paleo soil is very high up to Ginfille River.

(4) From 10+523 (Bambis) to 10+100 (St. Estifanos church) -meskel square-stadium-legehar to 8+300 (Debrework Bldg) is over pass type of route line.
This area can be characterized by slightly undulating topography from urael church and running flat then after up to stadium with thick dark grey black cotton soil underlain by highly weathered welded tuff.

(5) From 8+300 (Debrework Bldg) to 7+900 (Mexico square) and then to 6+700 (Lideta) is under pass type of route line.
In this area we can observe the same fetures of ignimbrite rock feture exposed under route line (2) above.

4.3 Index Properties of Sub-grade Fill Material

There are different properties under different conditions those affecting the performance and use of soil for various purposes such as; foundation material, embankment material and filter material. So that, soils have to be properly sampled and subjected to various tests so as to understand their properties. For the present study the sub-grade soil samples which are taken from yerer-2 quarries were subjected to various tests to determine their physical and engineering properties. The type of tests carried out in laboratory includes; Soil Classification tests such as; Grain size distribution and Atterberg Limits, Strength tests (California Bearing Ratio (CBR)), Moisture/ Density Relationship (modified proctor tests), and direct shear tests.

The minimum data needed to evaluate the subgrade soils classification should be (which requires Atterberg limits and gradation as appropriate) and strength (lowest expected). The following current ASTM test designations may be used in developing the necessary data where appropriate for design (AREMA, 2010):

Grain Size Analysis . . . . ……………ASTM D421 (Sample Preparation)
ASTM D422 (Test Procedure)
Plastic Limit and Plasticity Index . . . . ………………………………….ASTM D4318
Moisture Density Tests . . . . ………. . …...ASTM D698 and ASTM D1557
Where cohesive soils exist in the subgrade, resulting of an unconfined compression test of the compacted cohesive material (saturated) will give cohesion or shear strength for use in design. It may not be necessary to develop shear values from tests for some non-cohesive soils but where necessary standard tests may be performed. However, when subgrade

support is marginal and/or where the liquid limit of the subgrade soil exceeds a value of 30 or the plasticity index exceeds 12; special attention should be given to that soil (AREMA, 2010).

4.3.1 Grain Size Distribution Analysis

Soils may be divided by geological genesis into residual soil, slope wash, colluvial soil, alluvial soil, marine soil, lacustrine soil, silting soil, glacial soil and eolian deposit, etc. General soil may be divided into rubbles soil, sandy soil, silt and cohesive soil by shapes of soil grain and gradation or plasticity index (China Railway Manual, 2010).

For this project there are about three quarry sites (Yerer-1, Yerer-2 and Gelan) were identified by the contractor (CREC) and only yerer-2 quarry /borrow site is approved by the consultant (SewRoad Consulting engineers Plc.). Furthermore, sampling and access to the quarry site were made with material engineer assigned by the consulting office. For the soil classification purpose one representative sample was collected from the potential embankment sub-grade fill material borrow site.

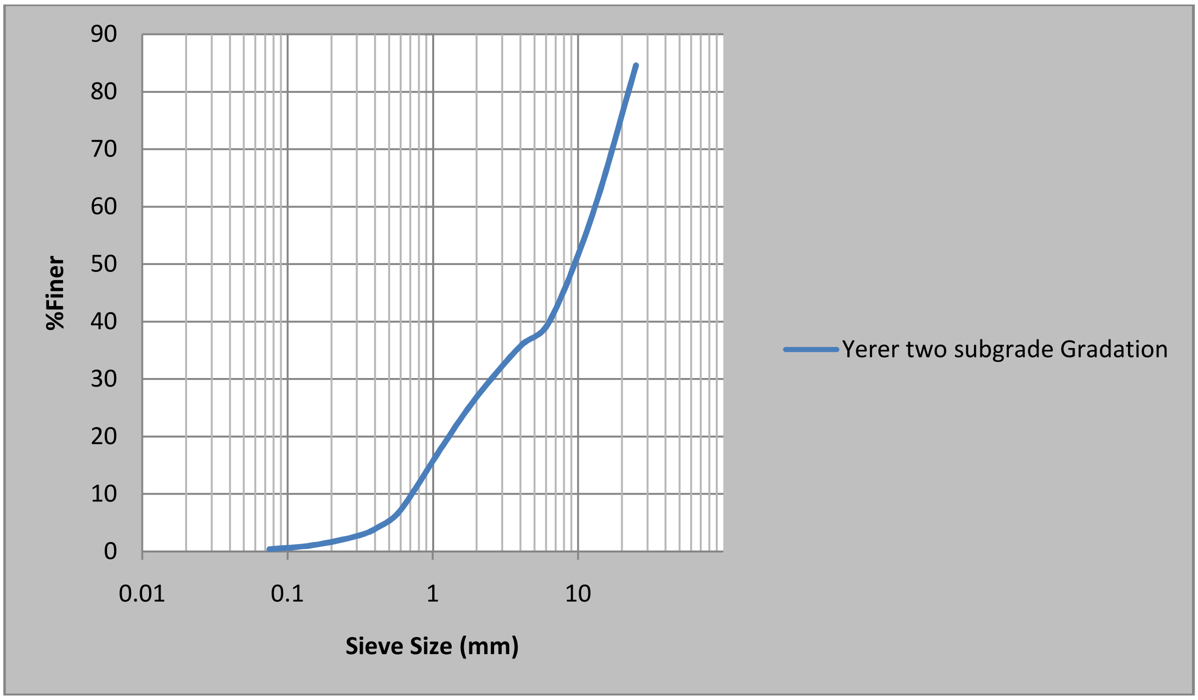


Fig 4.1 Subgrade Enbakment Material Grain Size Distribution

Moreover, according to USCS system most of the soil sample is gravel fraction (that constitutes 73.15%). In addition to this, the sand grain size in the soil is about 25.76%. Finally, only 1% of the subgrade material constituent is silt and clay (Annex One).

Further, based on distribution of particle size (gradation), coefficient of uniformity (Cu) was also computed for most of the granular soil samples by using equation (2.1) and coefficient of curvature, C_C otherwise termed as the coefficient of gradation or the coefficient of concavity equation (2.1).

Accordingly, the sample satisfy the requirement set by IS-2720, Cu >7, as shown below. C_u =12.6/0.75= 18 on the other hand the value for Cc will be Cc= $(2.8)^2$/ (12.6)*(0.7) which is equal to 0.93. And this value is approximately nearer to the boundary 1 to 3.

Table 4.1 Summary of Index Properties of Sub-grade Embankment Fill Material

Sieve Size (mm)	25	12.5	6.3	4	2	1.18	0.6	0.425	0.3	0.15	0.075
% Pass	84.6	58.5	39.9	35.7	26.8	18.5	7.2	4.3	2.7	1.1	0.38

Atterberg limits (%)	LL	PL	PI
	43	41.98	1.1
AASHTO(GI) Classification	A-1-a(0) or A-1-b(0)		
USC classification	GW,GP		
OMC	11.65%		
MDD	2.05g/cc		
CBR(One Point)	58.47%		

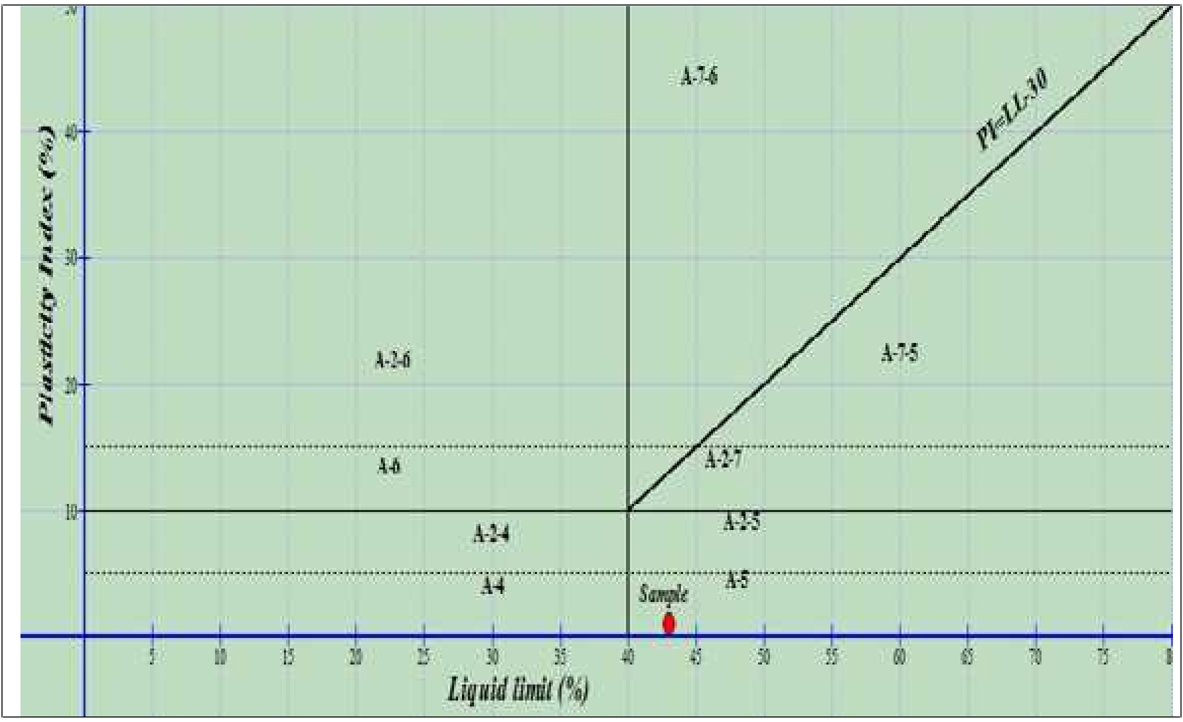


Figure 4.2 Chart for use in AASHTO soil classification system

4.3.2 Group Index

Group Index (GI) further helps to differentiate soils containing appreciable fine-grained materials. The GI is determined by using Equation below

GI= (F-35) (0.2) + (0.005) (LL-40) +0.01(F-15) (PI-10).. Eq. 4.1

Where; 'F' is the percentage passing 0.075mm (No. 200) sieve expressed as a whole number, "LL" is the liquid limit and *'PI'* is the plasticity index.

Then, (0.38-35) (0.2) + (0.005) (43-40) + 0.01 (0.38 -15) (1.1-10) = -5.6 so that, the group index is equal to zero (0) (V.N.S, Murthy, 1990 page 92).

4.3.3 Consistency Limits

The apparatus was discovered by A. Casagrande (1932) and the procedure for the test is called the Casagrande cup method. Plasticity index and liquid limit are the important factors that help an engineer to understand the consistency or plasticity of clay. Shearing strength, though constant at liquid limits, varies at plastic limits for all clays.

As can be illustrated from the LL and PL tables (Annex One), the result for both analyses shows more of similar figure (Table 4.1). Highly plastic clay (sometimes called fat clay) has higher shearing strength at the plastic limit and the threads at this limit are rather hard to roll whereas lean clay can be rolled easily at the plastic limit and thereby possesses low shearing strength. On the other hand, there are some fine grained soils that appear similar to clays but they cannot be rolled into threads so easily. Such materials are not really plastic. They may be just at the border line between plastic and non-plastic soils. In such soils, one finds the liquid limit practically identical with the plastic limit and PI=0.

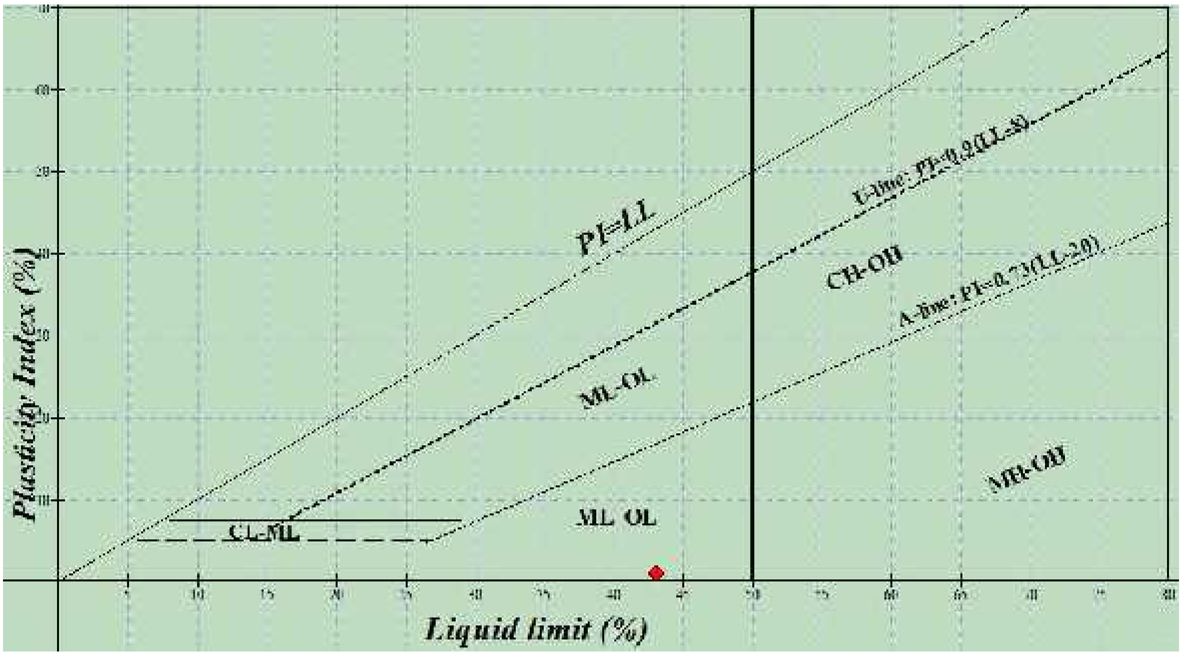


Fig 4.3 Cassagrande's LL-PI chart and respective clay minerals

According to Nelson et al., (2010), sited in Mathewos Bekele., (2012), soils that exhibit plastic behavior over wide ranges of moisture content and that have high liquid limits have greater potential for swelling and shrinking. Then according to the above test results the plasticity index (PI = LL –PL) of the yerer-two subgrade embankment material is 1.02% which is low plastic material (V.N.S, Murthy, 1990).

4.3.4 Moisture/ Density Relationship (Modified Proctor Tests)

In order to determine the compaction characteristic of the sub-grade fill material representative sample from borrow area was tested in Engineering Geology laboratory. For this, Modified Proctor test was conducted. Through Proctor test, optimum moisture (OMC) and maximum dry density (MDD) of the soil was determined. Therefore the optimum moisture content is 11.65% at the maximum dry density of 2.05g/cc (Annex Two and Fig. 4.4)

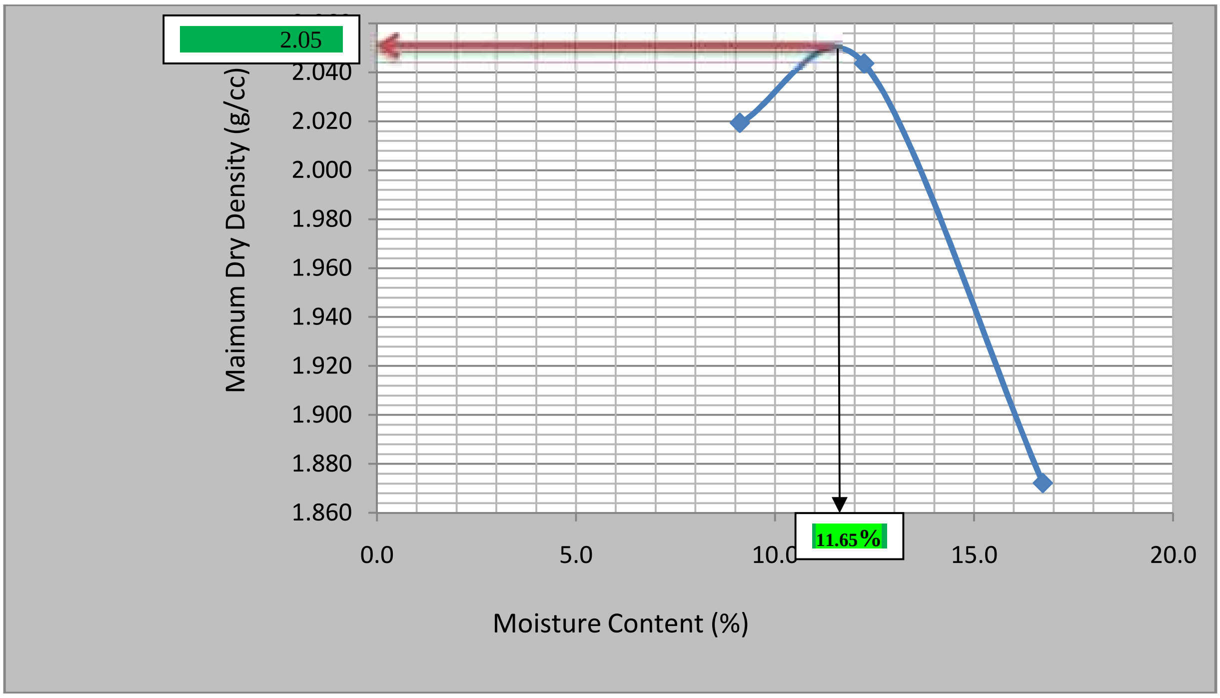


Fig 4.4 MDD-Moisture Relationships

4.3.5 California Bearing Ratio Tests

The CBR values will be calculated for penetration of 2.5mm and 5mm after 4 days soaked soil sample. Generally, the CBR value at 2.5 mm penetration will be greater than that at 5mm penetration and in such a case; the former shall be taken as the CBR value for design purpose. This test has done using AASHTO T- 193, (1993) and BS 1377: Part 4: 1990) test methods. If the CBR value corresponding to a penetration of 5mm exceeds that for 2.5mm, the test shall be repeated (RDSO/GE/014 : SR-0029, 2011). For the present study CBR test was performed on yerer-two embankment fill material soil sample and the test results are presented in (Annexure- three). Accordingly, the one point CBR results at 95% MDD for the subgrade material is 58.47 %. Especially for granular fill material 4% CBR value is the minimum requirement of the specification (ERA, 2002; UIC, 2005).

CHAPTER V - CHARACTERIZATION OF BALLAST STONE MATERIAL

5.1 Preamble

Ballast is laid upon the sub-ballast, the surface on prepared surface of the roadbed. A good ballast material should be workable, durable and strong enough to resist crushing, and angular in shape to resist movement and to permit drainage when compacted. Ballast supports the crossties resting upon it, holding them firmly in place. It thereby maintains a liniment, provides uniform support for the track, and, if properly structured, distributes traffic weight evenly on the roadbed otherwise the substructures under it. Ballast provides a flexible base so that ties can be moved to adjust rails to proper alignment and surface. It reduces dust and inhibits the growth of vegetation along the tracks. The following three paragraphs discuss ballast materials, their selection, and the ballast section.

The suitability condition of the ballast material can be depending on both the suitability condition of ballast rock and the aggregate production condition. This means even if most of the ballast engineering properties are controlled by the quality of ballast rock, some of the ballast material engineering properties such as; gradation and grain shape can be affected during the aggregate production (Bayisa Ragassa., 2010).

As to different suitability standards of ballast material, different laboratory tests are required for the proposed material as already been discussed in detail in Chapter Three above. For the present study the tests which were carried out are; grain size distribution test, specific gravity test, Water absorption test, Slake durability test, sodium sulfate soundness test, Los Angeles Abrasion value test, aggregate crushing value test, aggregate impact value test and petrographic thin section analysis. These laboratory tests were conducted using the standards of Chinese Railway Manuals (China, 2010) and "American Railway Engineering Maintenance-of-way Association Manual" (AREMA, 2010). For the testing standard, as specified by AREMA, (2010) ASTM standard testing methods were also followed. According to AREMA (2010), manual the recommended limiting values of tasting for ballast material from basalt rock that were adopted for present study, are presented in Table 3.5.

In general all laboratory tests including petrographic thin section analysis were carried out in both Engineering Geological laboratories, School of Earth Sciences, Addis Ababa

University and Addis Ababa Institute of Technology, School of Civil Engineering Laboratory.

5.2 Engineering Geological Characterization of Ballast Stone

5.2.1 Gradation Test

In previous chapter this study considered different specification and standards in different countries so as to meet the specific objectives stated for the research. According to Bonnett, (2005), to overcome gradation problem in construction, not more than 3% by weight should be retained on the 50 mm square mesh sieve and not more than 2% should pass through the 28mm sieve.

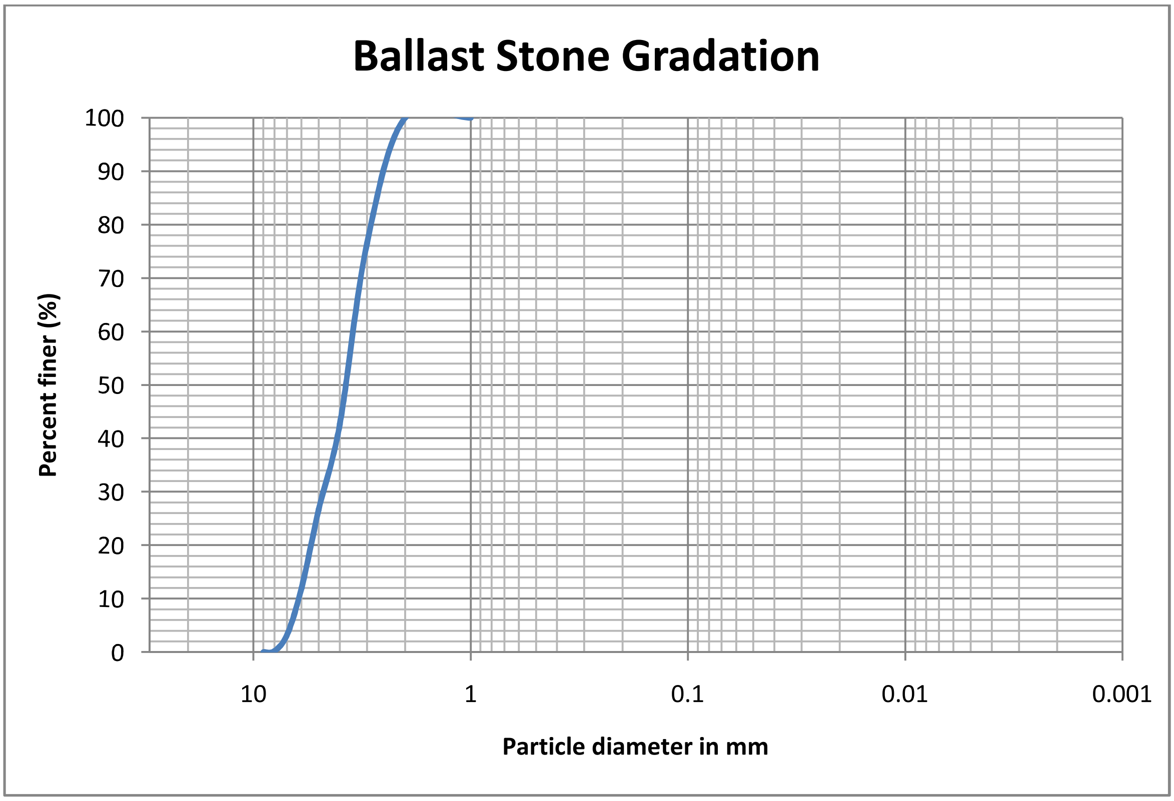


Fig 5.1 Ballast Stone Grain Size Distributions

And therefore, depending on the above grain size distribution curve, the coefficient of uniformity ($Cu=D_{60}/D_{10}$) for the ballast stone is equal to 0.6. On the other hand, the coefficient of curvature ($Cc= (D_{30})^2/D_{60}*D_{10}$) is equal to 1.04. The gradation curve is semi-vertical to vertical in logarithmic distribution curve. Aggregates of similar or the same size have assimilated for the LRT project. And this indicates that uniformly graded ballast stone material (Annex Four).

5.2.2 Specific Gravity Test

The specific gravity is one of the aggregate properties that relates to its density. Specific gravity can control both vertical and horizontal dynamic load bearing capacity of ballast layer in the railway track. This test can help to determine the average density of coarse aggregate particles without including the volume of voids between the particles (ASTM C127, 2001).

As a result, the specific gravity of ballast stone can be described in three ways as; oven-dry (OD) specific gravity, saturated surface dry (SSD) specific gravity and apparent (Bulk) specific gravity. In the present study tests were conducted at engineering geology laboratory and AAiT, Civil Engineering laboratory (Annexure Five).

As can be observed from Table 5.1, the average specific gravity for all representative rock samples from Galan quarry site satisfies the standard specification proposed by AREMA (2010) as the average values are above 2.9.

5.2.3 Unit Weight (Bulk Density) Test

In the present study the unit weight of representative rock sample from Galan ballast stone quarry site was conducted according to ASTM C29 (1997) standard. From this test the unit weight in dry condition, unit weight in SSD (Saturated surface dry) condition and the percentage of void content between the aggregate particles for sample was determined (Table 5.1 and Annex Five). Accordingly, the bulk density of the ballast as to the average value for both trials is 3000kg/m^3 that the Unit weight (Bulk density) for the representative rock sample the standard specification proposed by AREMA (2010) as the values is above 1400 (kg/m^3).

Table 5.1 Summary of Ballast Material Physical Tests

Test Number	sample 1	sample 2	Mean
Specific gravity, γ_d	2.7	3.0	2.9
Apparent (Bulk Density) specific gravity, γ (ap)	2.9	3.1	3.0
SSD specific gravity, γ(SSD)	2.7	3.1	2.9
water absorption, w	1.5	1.2	1.4

5.2.4 Water Absorption Test

It is one of the ballast aggregate testing methods that measures the ability of an aggregate material to absorb water and which help to determine the sensitivity of an aggregate for weathering or degradation. Thus, it implies that if the aggregate has high water absorption capacity, its swelling and shrinking capacity will be high based on the seasonal fluctuations. In this paper water absorption test was conducted following ASTM C 127 (2001) testing standard for representative samples collected from Galan quarry site and the results are presented in (Table 5.1) above. In the table above the water absorption value for the rock samples is 1.4% to an average (Annex Five). This result is slightly above the specified limit of ≤1%, as proposed by AREMA Manual (2010).

5.2.5 Impact Value Test

Impact value test is one of the tests for the strength of the ballast stone. According to this specification during the AIV test the aggregate sample with 10-14 mm size is subjected to 15 blows and the percentage of fine material passing through 2.36 mm sieve is calculated which ultimately provides AIV. The small AIV indicates the high resistance of an aggregate to the impact force and thus represents the high quality aggregate material (BS 812 part 110, 1990). As result the average value is 15.55% (Table 5.2 and Annex Six).

5.2.6 Aggregate Crushing Value Test

Like that of the aggregate impact value, the aggregate crushing value (ACV) test is the mechanical properties of an aggregate that is used to measure the resistance of an aggregate to crushing or degradation under a gradual applied compressive load or static load and follows the same procedure (BS 812 part 110, 1990).

As a result, the average value of ACV from (Table 5.2 and Annex Six) for the rock sample is 13.89%. This is the permissible result as to the specified standard set in chapter 2.

5.2.7 Abrasion Resistance Test

The percentage degradation or loss of the ballast material is 18% (Annex six). In general, low values of abrasion loss are required for a ballast stone in railway construction. Los Angeles Abrasion tests also one of strength test measures for the ballast aggregates. Procedurally, in accordance with the requirements a sample is placed in a drum with steel balls. The drum is rotated and the balls grind down the aggregate particles. Soft aggregates are quickly ground to dust, while hard aggregates lose little mass. Abrasion resistance applies only to coarse aggregates (Ballast). The acceptable limits are set by the Los Angeles Abrasion Test and the limits vary from 30.0 to 50.0 percent, depending on the classification of the aggregate (AASHTO T-96).

5.2.8 Soundness Test

In this paper to conduct soundness test the anhydrous sodium sulfate was used according to ASTM C88 (1999) and AASHTO-T104 testing standards. Furthermore, according to AREMA, (2010) which was using ASTM standard the permissible limit for Soundness percent loss by sodium sulfate is ≤ 5.0%. The test conducted during present study on sample from Galan quarry site showed that the Soundness percent loss is only 0.9% (Annex Seven).

Table 5.2 Summary of Crushing Resistance of the Ballast

Specimen identification No.		Trial -01	Trial -02	Average
Aggregate impact value (%)	M2/M1 X100	13.37	17.73	15.55
Aggregate Crushing value (%)	M2/M1X100	13.7	14.1	13.89
Los Angeles Abrasion				18%
Soundness Test				0.9%
Slake Durability Value				98%

5.2.9 Slake Durability Test

This test method covers the determination of the slake durability index of a shale or other similar rock after two drying and wetting cycles with abrasion. The test method is used to estimate qualitatively the durability of weak rocks in the service environment. The specimen shall consist of ten representatives, intact, roughly equi-dimensional shale fragments weighing 40 g to 60g each ASTM D4644 (1987).

This was primarily to establish a longer trend of weight loss as the rock continued to subject to more cycles of scrubbing in the wet drum. Temperature of the water in the trough was 25°C. The drum was turned 20 revolutions per minute for 10 minutes. The test was intended to assess the impact of water on the weathering process for the trap rock type. The dry-testing specimens were placed in the oven for 24 hrs for each cycle. Before testing they were cooled down to ambient temperature. The weight loss calculation for both wet and dry testing follows the ASTM (D4644-08) standard practice. The water absorption of rock specimens under degradation simulation in laboratory test and under actual conditions was determined using the ASTM (D6473-10; C127-04) standard test method.

Accordingly, the average slake durability result of both cycles is 98% which is illustrated in the (Annexure Six).

5.2.10 Petrographic Thin Section Analysis

For this study an attempt for the petrographic analysis being done to understand the index properties of the intact basaltic rock that can be used to evaluate rock forming constituent mineral behavior and to evaluate the micro cracks through thin section analysis. These parameters are; rock type, mineralogical composition, grain size, grain shape, existence of micro crack, secondary minerals and degree of weathering.

Just to achieve this, during the present study the thin sections were prepared for the representative rock samples from Galan ballast stone quarry site. Hence, two samples were selected as representative depending on their physical property. In the ballast quarry, some parts of the basaltic rock exposures were observed as cleavage forming fine grained outcrop. And this was taken as sample one. On the other hand, the other parts of rock exposures were massive fine grained basaltic groups taken as sample two. For these two identified samples the detailed petrographic descriptions of rock samples from Galan ballast stone quarry site is given in the following paragraphs.

Sample 1

From the hand specimen observation the rock may be described as basaltic rock with dark colored, fine grain, forming layer of banding with moderately weathered surface. As can be observed from the (Plate 5.1) below, under the microscopic observation the rock can be described as; abundant amount of elongated minerals of plagioclase feldspar, opaque minerals and volcanic glasses of which the interstices are filled with the few smaller minerals of olivine form a framework in this rock. Texturally this rock form the aphanitic texture surrounded by the ground mass of plagioclase feldspar and opaque mineral. Further, micro crack perhaps alteration was also observed within the ground mass.

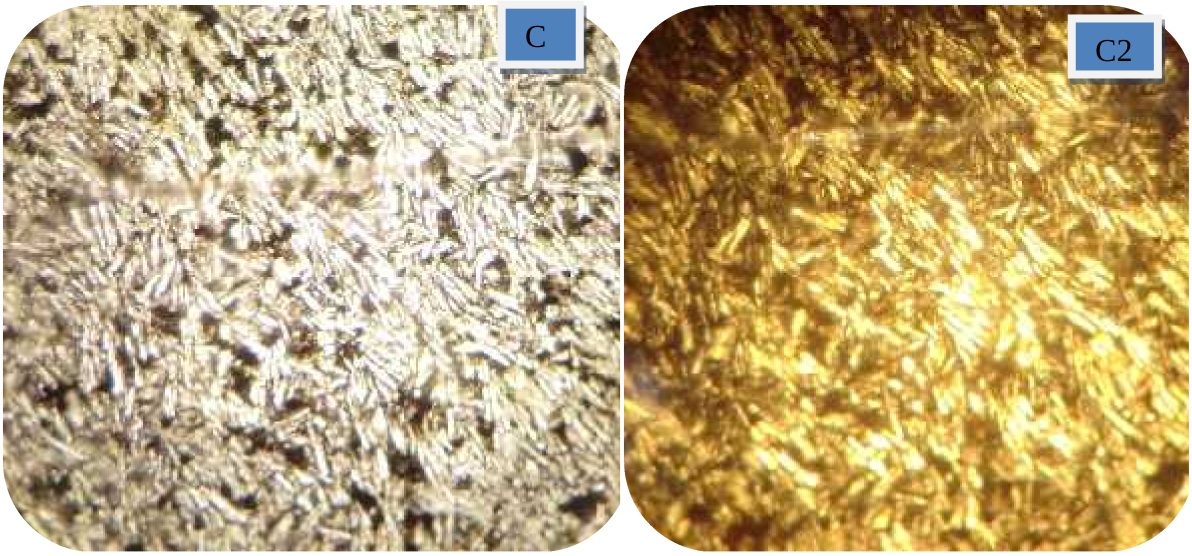


Plate 5.1 Tabular elongated plagioclase feldspars, opaque minerals and micro crack or alteration with 10X magnifications under plane polarized light (PPL)-(**C2**) and cross polarized light (XPL) –(C1).

Based on the visual observation the thin section, each mineral in the rock was estimated and presented. As a result, the plagioclase feldspar needles constitute about 70% percent, the opaque minerals constitute about 28% and the rest 2% is occupied by volcanic glass and few grains of olivine.

Sample 2

Like that of sample one, the same mineral constituents reside in sample two except that the presence of micro fracture otherwise alteration in sample one (Plate 5.2). The rock may be described as dark colored fine grain massive basalt with moderately weathered surface. Microscopic thin section study revealed that the rock is mainly composed of; elongated needles of plagioclase feldspars, opaque minerals, very few olivine and volcanic glass. The estimated percentage of mineralogical composition of rock sample is plagioclase feldspar needle constitute about 70% percent, the opaque minerals constitute about 28% and the rest 2% is occupied by volcanic glass and few grains of olivine.

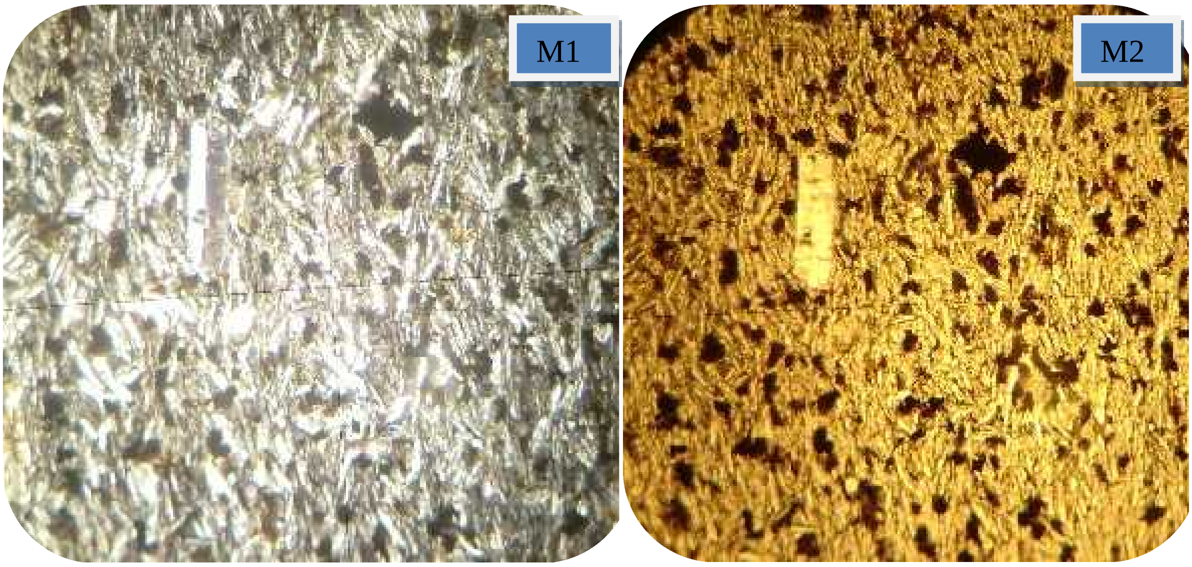


Plate 5.2 The phenocrysts of, plagioclase feldspars, opaque minerals for massive ballast stone sample 10X magnifications under plane polarized light (PPL)-(**M2**) and cross polarized light (XPL) –(**M1**).

In addition to this, no micro cracks were observed in this sample both in the individual mineral grains and within the ground mass (Plate 5.2). In general, based on the thin section mineralogical composition and textural condition the ballast rock proposed may be named as aphanitic basalt.

CHAPTER VI – DISCUSSION AND SUITABILITY ANALYSIS

6.1 Preamble

Suitability of the soil for a particular use usually depends on one or more engineering properties, parameters and indexes. The performance of engineering works will depend on the correct assessment through characterizing engineering properties, parameters and indexes to determine suitability and to predict performance of a subgrade soil, subballast and ballast stone material for its intended use.

In this paper, the physiographic environment, Regional and local geological settings, seismicity issues, a systematic engineering geological characterization and suitability analysis of different substructure materials that may influence the proposed alignment are discussed well. Finally, it provides the safest suitability analysis from engineering geological point of view has discussed in detail in order to minimize the problem of sub-grade embankment fill material and ballast stone. In addition to this, main engineering geological problems associated with the substructure materials encountered for the proposed light rail transit in general are evaluated and its possible remedial measures are provided.

To assess quality of stone ballast aggregate, there are ten laboratory tests done and to be discussed below. These are: grain size distribution test, specific gravity test, Bulk density test, Water absorption test, Slake durability test, sodium sulfate soundness test, Los Angeles Abrasion value test, aggregate crushing value test, aggregate impact value test, petrographic thin section analysis. A detailed discussion and interpretation on the test results is presented in the following paragraphs.

6.2 Suitability of Geological and Physiographical Environment

The study area is characterized by ignimbrite, trachyte and trachy basalt, trachyte ignimbrite, pyroclactic and sediments of which are exposed at foot edge of the mountain yekka. This segment is mainly overlain by varying thickness of expansive black cotton soil. Even though the route continues marching across expansive soil throughout the corridor, it also encounters volcanic rocks and pyroclastic welded and loose tuff deposits underlain by the lower ignimbrite (Fig 3.3).

The geological structures such as; lineaments and a big fault which trends nearly NE-SW was observed in the central and Northern corridor the study area. The said lineaments have been mapped on the geological map of the study area (Fig 3.3). Fault-related seismicity usually has its most significant effect on civil structures. The foundations of civil structures can be engineered to be safe even in earthquake-prone areas.

It has been discovered that, in this study area the fault and proposed railway corridor are almost aligned across the rail structure especially one lineament observed on the geological map and tends to cross the rail alignment around Tele- Garage-Gurd Shola area (Fig. 3.3). So that the presence of this fault across the alignment may produce too many design effects to the railway structure. As previously recorded earthquake data shows, local earthquake effects associated with this fault have been minor in recorded history. Actually in this case, assessment of possible seismic effect due to this fault on the proposed Addis Ababa City Light Railway Transit (LRT) alignment is beyond the scope of the present study. Therefore, it is highly recommended to carry out additional studies in the later studies.

Generally, the LRT physiographic alignment analysis shows that 82.8% of the corridor is connected to the flat topography or below 4% (see section 2.3.1.2 and Fig. 3.2). This indicates the permissible figure according to (AREMA, 2010). While the rest 17.2% of the alignment is above 4% which is not the acceptable slope gradient in passenger and short ancillary freight service (AREMA, 2010). Thus, most of the proposed route (82.8%) passes through flat topographic lands, less or equal to 4% gradient (Fig. 3.2).

6.3 Suitability Analysis of Sub-Grade Material Based On the Laboratory Test Results

During the present study, characterization of sub-grade embankment fill material depending on different parameters and index properties for suitability has been made. The laboratory investigations were carried out at both Engineering geological laboratory, School of Earth Sciences and Addis Ababa institute of Technology, school of civil Engineering laboratory. In general, the following consideration shall be given during final sub-grade characterization stage; quality of the material, so as to comply with the specifications for intended use.

6.3.1 Grain Size Distribution Analysis

To verify the characteristic behavior of soil grain size distribution, soil classification (AASHTO and USC) for the proposed sample collected from yerer-2 subgrade embankment fill material was conducted. The results indicate that the sub-grade material from yerer-2 pits (Plate 6.1) is mostly composed of granular material. Therefore, a procedural laboratory tests were undergone to the granular fill material to assess its suitability. Accordingly, the result of characterization of the sub-grade material from chapter four, section (4.3.1) shows that the uniformity coefficient (Cu) is 18 and the coefficient of curvature (Cc) is 0.93. This result clearly shows that well graded grave sub-grade material (Table 4.1 and figure 4.2).

Plate 6.1 Sub-Grade Embankment Fill Material from Yerer-2 Pits (A&B)

On the other hand according to AASHTO classification system this subgrade material lies in the group classification of A-1-a (0) and A-1-b (0). As a broad classification in this system, it lies in well graded granular materials of 35% or less of total sample passing No. 200.

The other classification system considered in this test is the Unified Soil Classification System (USCS) which is modified from the version of Casagrande's Airfield Classification (AC) System developed in 1942 for the US Corps of Engineers. As a result for the subgrade material is well-graded gravels, gravel-sand mixtures, little or no fines with the group name of GW (Table 4.1 and Figure 4.1).

This implies that, the sub-grade material contains usual types of significant constituents of Stone fragments, gravel and sand which can achieve the suitability grading of excellent to good.

6.3.2 Group Index Analysis

The combined result for gradation test and consistency analysis show the group index is negative 5.6. This implies that according to AASHTO manual (1993), when the value is negative, the group index shall be taken as zero. Therefore the result clearly shows that, a good sub-grade material suitable for the proposed project. In general, the higher the value of the GI, the poorer is the quality of the material within its own group (Arora, 1997).

6.3.3 Consistency Test Results Analysis

Casagrande plasticity chart is also helpful to differentiate a soil type in accordance to unified soil classification system. 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 (Casagrande, 1948).

The liquid limit is 43% and the Plastic limit is 42% (Table 4.2 and Table 4.3). These results for liquid limit and plastic limits indicate different meanings for different researchers. For example, high swelling potential (Chen, 1983), medium swelling potential (Holtz and Gibbs, 1956; Ladde and Lambe, 1963; Daksana Murthy and Ramana, 1973). The plastic index for this material is 1%. Then, according to Holtz and Gibbs, (1956), and Seed et al., (1962), it fall under low swelling potential. Finally, pertaining to the above consistency limit values, the sampled soil is suitable for prepared sub-grade material (ERA, 2002).

For this study, the samples tested for plasticity index were plotted on Cassagrande plasticity chart as shown in (Fig. 4.3). Then, by using this chart it has been attempted to plot the liquid limit and the plasticity index values of the subgrade soil. A consideration of the chart clearly shows that the soil sample is below “A”-line. Thus, the soil sample can be classified as moderately plastic silt to low plastic silt.

6.3.4 Moisture/ Density Relationship (Modified Proctor Tests)

In order to determine the compaction characteristic of the embankment fill material the sample from yerer-2 borrow area was tested in laboratory. For this Proctor test was conducted. Through Proctor test optimum moisture (OMC) and maximum dry density (MDD) of the soil can be determined. The results thus obtained are presented in Table 4.4 and Fig.4.4. Accordingly, the test result illustrated in (Table 4.4) shows that, the maximum dry density is 2.05g/cm^3 and the optimum moisture content is 11.65%. Hence, according to Indian standards, IS 2720, material greater than 1.8g/cm^3 is suitable for prepared sub-grade soil. Thus, sub-grade material in the present case can satisfy the general suitability requirements (AASHTO T 180; AREMA, 2010).

6.3.5 California Bearing Ratio Tests Analysis

The outcomes of laboratory findings, as discussed in chapter four, revealed that the sub-grade soil is generally classified as a suitable material to be used as a railroad bed depending on its strength property (Table 4.5). The CBR value for this strength analysis is 58.47%, which is under the category of the maximum required value grouped under strength class (S3) at modified compaction (ERA, 2002).

6.4 Suitability Analysis of Ballast Material Based On the Laboratory Test Results

In general it is a known fact that the quality of the quarry improves with depth as the degree of weathering decreases with depth. At this case, the quarry site ballast stone excavations and bastings were under intensive operation since the railway construction was already under intensive progress. In this paper after evaluating the general suitability of Galan quarry ballast stone as per the proposed test standards, the representative samples were collected from the quarry site (Plate 6.2) for the laboratory test analysis. Finally, based on these results engineering quality of the rock was evaluated as per the ballast stone aggregate specifications.

Plate 6.2 Galan Quarry site for Ballast Stone (A&B)

To determine the overall suitability of each quarry site based on the nine engineering test parameters with equal weights were given to the parameters and later their average value was calculated. The evaluated rating for each parameter is already presented in (Table 2.5).

6.4.1 Grain Size Test Results

Ballast stone gradation testing has been carried out in engineering laboratory in accordance with the requirements of internationally accepted standards (AREMA MRE, 2010 and China Manual, 2010). To fix the crushing size differences and assess the suitability conditions for the aggregate samples, the graduation requirement for the ballast stone has to be initially selected.

Therefore, as it can be observed from the analysis of (Table 6.1) it shows that there is gap in satisfying the gradation specifications of sieve size of 50mm and 37.5mm for American Railway Engineering and Maintenance-of –way Association (AREMA, 2010) standard and gradation of sieve size of 37.5mm and 25mm mesh size in the (Chinese manual, 2010). On the other hand, depending on the coefficient of uniformity (Cu) for the ballast (Cu = D_{60}/D_{10} = 0.6) as illustrated under section 5.2, the Cu is less than 4. This indicates that uniformly graded ballast stone (as stated in the review of chapter two, approximately particle of the same size). And the coefficient of curvature (Cc) for the proposed ballast stone (Cc = $(D_{30})^2/D_{60}*D_{10}$ = 1.04) which is described in the range of 1 up to 3.

Table 6.1 Ballast Gradation Suitability Analyses as to Different Standards

AREMA MRE, 2010			China Manual, 2010		Actual Test Result In the lab		Remarks for the Test results
Sieve size (inch)	Sieve size (mm)	Percentage passing by weight	Sieve size (mm)	Percentage passing by weight	AREMA	China	
2½	63.0	100	37.5	100	100	42.7	Meet AREMA
2	50.0	90-100	30	95-100	76.5	-	Doesn't meet AREMA
1½	37.5	60-90	25	55-75	42.7	26.7	Doesn't meet Both
1	25.0	10-35	20	25-40	26.7	-	Meet AREMA
¾	19.0	0-10	15	5-15	12	-	Meet AREMA
3/8	9.5	0-3	10	0-5	3.2	0.2	Meet Both

This can be supported by; freshly placed ballast is narrow-graded material (i.e. contains limited range of particle sizes), consisting of a large amount of open pore space and a permeable structure. As stated earlier, the Current thinking is that a narrow gradation would be the best meet the requirements for railway ballast grain size. However, there is a disagreement within the railway system with regard to the maximum and minimum particle size that would offer the best performance for a ballast material, i.e. lower deformation and degradation of ballast layer (Ionscu, 2004).

6.4.2 Specific Gravity Test

This test method is basically used to determine the unit weight of ballast stone that controls aggregate bearing capacity of the. The aggregate with high unit weight has good holding capacity than the aggregate with low unit weight. According to the AREMA MRE (2010) specification the unit weight of the ballast aggregate that is produced from trap rock such as; basalt must be greater than or equal to 1400kg/m^3.

Accordingly, the test result for this study show 2.75 to an average (Table 5.1) which is greater than the standard given by AREMA (2.60). Indraratna et al. (2006), as cited in Bayisa Ragassa, (2011), emphasized that the higher specific gravity of an aggregate rock the greater will be the bearing capacity and the lower will be the degradation of the ballast aggregate.

6.4.3 Unit Weight (Bulk Density) Test

The test result of the unit weight of an aggregate sample has been evaluated according to the illustrations given in the (Table 5.1) and is greater than 1400kg/m^3. The specific gravity of an aggregate depends both on the void content and the mineralogical composition of the source rock. This implies that the bulk density is dependent on the particle specific gravity and the void ratio. Therefore, in order to have higher bulk density of ballast aggregate it is desirable to use the rock type with low void ratio and denser particles. The basic igneous rock that has more iron content can be characterized by high unit weight than the rock that is dominated by silicate minerals.

6.4.4 Water Absorption Test Results

The deterioration of the ballast aggregate material both by chemical weathering and mechanical disintegration can be estimated through this test result. Specification for the water absorption of railway ballast aggregate from trap rocks must be less than or equal to 1.0% (AREMA MRE, 2010).

Trapped water leads to increased pore water pressure and subsequent loss of shear strength, stiffness and elasticity. Such a condition will lead to a reduction of track stability and a continued deterioration of track components over time (this condition may even occur in arid regions if water migrates upwards and accumulates under the largely sealed ballast). When ballast is no longer able to perform its intended function, it is said to become ‘fouled’ (Ionscu., 2004).

According to the test results of ballast material (Table 5.1), it has got a slight potential for absorption of water. This implies that, if the ballast aggregate is more susceptible to mechanical and chemical weathering it may result into deterioration of the engineering properties of ballast stone material. Thus, the safest and life time or duration of the performance of ballast stone layer will be affected. On the other hand, as it was discussed the climatic condition of the study area, under this chapter, the wet and dry variation conditions are very high. Hence, the ballast layer has to face higher changes between evaporation and saturation, as a result if the ballast stone material has got high water absorption capacity it can easily be disintegrated by mechanical and chemical weathering, and this will result in lowering bearing capacity of the track (Raymond G.P., 1985).

6.4.5 Impact Value Test

This test method is the way by which an aggregate sample is subjected to the standard amount of impact through known weight, i.e. a steel cylinder, falling from a set height, a prescribed number of times (normally 15 times), onto an amount of aggregate of standard size and weight retained in a mould (2.36mm sieve size) (BS 810, 1990: Testing Aggregates: Part 110).

The test result of an impact value for the proposed ballast stone material is 15.55% (Table 5.2) **to** an average. And it is a permissible according to the standard illustrated in (Table 2.5). This indicates that the higher resistance of an aggregate to the impact force and thus represents the high quality aggregate material (BS 812 part 110, 1990).

6.4.6 Aggregate Crushing Value Test Result

In this case the same procedure will be applied in the aggregate crushing value (ACV) test like that of aggregate impact value tests. This is through the resistivity of an aggregate to mechanical engineering properties to crushing under a gradual applied compressive load or static load of 400KN for ten minutes (BS 812 part 110, 1990).

Then, the average aggregate crushing value for the tested ballast in the three consecutive trials is 13.89% (Table 5.3). This result is above the permissible specification mentioned in (Table 2.5) which indicates very sound ballast stone for this test result.

6.4.7 Abrasion Resistance Test Results

In accordance with the requirements of AASHTO T-96, (1977), it measures the hardness of an aggregate material for different construction purposes especially for an aggregate susceptible for mechanical disintegration.
This is when a sample is placed in a drum with eleven or twelve steel balls. Then the drum is rotated and the balls grind down the aggregate particles. Soft aggregates are quickly ground to dust, while hard aggregates lose little mass. Abrasion resistance applies only to coarse aggregates (Abebe Dinku, 2002).

The abrasion resistance of ballast rocks is dependent on the type of rock, quality of minerals in the rock and the bond between the mineral grains in the rocks (Chanda and Krishna, 2003). The Los Angeles Abrasion value will be calculated according to the percentage of aggregate material passing a 1.7mm sieve after the completion of the test.
The result of Los Angles Abrasion test shows that 18% as illustrated in (Table 5.4). Therefore, according to specification (Table 2.5), the aggregate material from galan quarry site is suitable as ballast stone material.

6.4.8 Soundness Test Result

An aggregate prone to weathering from the water interaction otherwise of other dissolved pollutants those can result from the environmental waste and organic waste from the train itself have great influence where the aggregate is utilized. According to AREMA (2010) which was using ASTM C88, (1999) standard specification permissible limit for Soundness percent loss by sodium sulfate is ≤ 5.0% (Table 2.5). In if the soundness loss value is high it indicates highly susceptibility of an aggregate to weathering while the low percentage loss value shows the high resistance to the weathering. Therefore, the ballast stone soundness test result for the proposed project of Addis Ababa LRT shows only 0.9% as illustrated in (Table 5.4). Thus, it may safely be concluded that the ballast stone from Galan quarry is suitable as far as their potential for degradation to weathering action is concerned.

6.4.9 Slake Durability Test

According to Brown, (1981) as cited in Chaowarin et al., (2012) Slake durability index test has long been used to identify the durability and water sensitivity of rocks as subjected to engineering requirements under in-situ conditions. The test has been widely accepted and standardized by the American Society for Testing and Materials (ASTM D4644) in 1987, and included as part of the ISRM suggested methods by the International Society for Rock Mechanics (Brown, 1981).

The primary objective of this study is to investigate the weathering and degradation characteristics of some trap rocks by performing slake durability index testing. The results are compared with those obtained from the ASTM standard method (mentioned above). An attempt at predicting the rock strength degradation with time has also been made. The results can be useful for the design and indicating the gap for long term stability analysis of rock foundations, embankments, and support system (i.e. by explicitly considering rock degradation in the design parameters) (Chaowarin et al., 2012).

The test result for this specific ballast stone about 98% (Annex Six). This implies that about 2% weigh loss has recorded. So that it indicates that, the quality of an aggregate is good from the slake durability point of view.

6.4.10 Petrographic Analysis and Their Implications

The strength of a particular rock type mainly depends on its mineralogical composition, the size and shape of its minerals, presence or absence of micro cracks in it and the degree of weathering. Hence, the petrographic analysis in the chapter five shows, the ballast stone is mineralogically composed of plagioclase feldspars (70%), opaque minerals constitute about 28% and the rest 2% is occupied by volcanic glass and few grains of olivine.

The continuous reaction in the Bowen's Series, those minerals which crystallize first is the plagioclases. Plagioclase minerals have the formula (Ca, Na) (Al, $Si)_3O_8$. The highest temperature plagioclase has only calcium (Ca). The lowest temperature plagioclase has only sodium (Na). Therefore, according to this concept, the high temperature minerals are unstable at low temperature surface. This implies that, susceptibility for weathering is higher for calcium rich plagioclase minerals than sodium rich plagioclase feldspars. On the other hand, the petrographic analysis for cleavage forming basaltic sample (sample-1) shows micro crack or else alteration. And this will increase the chance for mechanical weathering during the dynamic loading of the rail track. Finally, this study shows that, the ballast rock is more prone to mechanical and chemical weathering due to the above combined petrographic investigations. In general, fine grained, hard-mineral and un weathered aggregates are best as ballast materials.

CHAPTER VII - CONCLUSION AND RECOMMENDATIONS

7.1 Conclusion

In this study, the subgrade embankment fill material has been characterized in terms of their; index and engineering properties and their relative response to the site conditions. The general suitability analysis based on standard specifications has also been analyzed for; topographic classifications, drainage condition of the area. In order to characterize the proposed subgrade material, systematic methodology has been adopted, which of these include; detailed literature and previous works review, and various field visual observation of the natural subgrade and laboratory tests.

In general, this paper is focused on characterization and suitability assessments of subgrade embankment fill material, ballast stone material and site condition in general along the proposed alignment. However, among the three substructures to be assessed, there is no subballast material used by the contractor (CREC). This is because the company is using 5cm thick concrete asphalt in place of subballast material. Further this will decrease the fouling degree of ballast by preventing the segregation of the subgrade fine material in to the ballast bed.

For characterizations of the ballast stone material several laboratory tests were conducted on selected samples. These tests are; gradation test, specific gravity test, unit weight test, water absorption test, slake durability test, crushing value test, impact value test, soundness test, abrasion test, polished value test, petrographic thin section analysis etc.

For subgrade embankment material and ballast stone, detailed description and interpretations on these tests have already been presented in Chapter IV Chapter V and chapter VI. Moreover, possible procedural engineering geological characterizations for the likely good outlooks of engineering geological concerns along the proposed line are also worked out.

The east-west LRT alignment contains both flat topography (82.87% of the corridor) and gentle sloping with approximately slopes gradient between 4% and 7% (Fig.3.2). This implies that, 17.37km of the corridor will suffer a great interaction of surface water with subgrade material except an alignment designed for an over pass. The alignment also crossed by some perennial and intermittent rivers like Yekka, Kebenna, Gifille and Gion. As a result, areas along these rivers may face deep rooted alluvial sediments. As per the standard requirements the ground water table should be at least at a minimum depth of 0.8 m below the rail top to avoid problems related to moisture.

During the present study, characterization and suitability assessments of subgrade embankment material both by their index and engineering properties of the materials had been done. Based on the laboratory test results data, the subgrade fill material was classified as A-1-a(0) and A-1-b(0) class as per ASSHITO classification System whereas as per USC soil classification system the soil equivalence class is [GW]. However, analysis made through Cassagrande LL-PI chart indicates that the fine soil within granular fill material is dominantly inorganic silts of medium compressibility fraction. Hence, the most probable soil class for most of the granular subgrade fill material is silty gravel [GM]. The analysis for subgrade material properties indicates GI of negative value which indices zero, low PI value, (less than 15/25), the one point CBR value greater than 4 and permissible MDD-OMC values for the sample. Beside to this, the subgrade material was also characterized following soil properties such as; particles size distribution, coefficient of uniformity, coefficient of curvature and soil strength. And these results fall within the recommended values. Hence, much of the material are coarse and well graded, as a result they are suitable to be used as subgrade embankment fill material. However, the material needs finer fraction to satisfy the required percentage pass as well as to meet the required compaction level (as the percentage passing Sieve No. 200 is only 0.4%).

On the other hand, as per the formulated standards and specifications, representative ballast rock sample was taken for the detailed laboratory testing. The laboratory tests were mainly conducted to appraise the engineering suitability of an aggregate as the source for ballast stone. These tests are; gradation test, specific gravity test, Water absorption test, Slake durability test, sodium sulfate soundness test, Los Angeles Abrasion value test, aggregate crushing value test, aggregate impact value test, and petrographic thin section analysis.

As per the recommended standards and specifications set in Table 2.5 for ballast aggregate, all the engineering suitability results obtained are within permissible limits mentioned standards and specifications except for water absorption values (1.4%) that is slightly higher than the AREMA (2010) specification. Even though the water absorption value of the tested ballast is above the limiting value, it is anticipated that the basaltic rock will be more suitable from other engineering geological point of view.

Another effort was made to evaluate the role of petrographic parameters (grain texture, mineralogical composition and micro crack) of the basaltic rock. This examination results implying that, fine grained texture, elongated needles of plagioclase feldspar, opaque minerals, very few olivine and volcanic glasses. Whereas, parts of the basaltic rock exposures, that observed as cleavage band forming (sapmle-1) outcrop has alteration and contain some micro cracks in it. Except micro crack, perhaps alteration, both petro-graphically analyzed samples are mineralogically and texturally similar. The presence of abundant constituents of plagioclase feldspars may make the ballast stone sensitive for chemical weathering as the alkaline feldspars are unstable at surface temperature than alkali feldspars. More over the plagioclase feldspars are tabular elongated with poor inter locking characteristics of the constituent material. Further, this decreases the the mechanical strength of the aggregate material. The proposed trap rock can be named as aphanitic basalt which is characterized as fine grained, hard-mineral and unweathered aggregates.

7.2 Recommendations

The above discussed conclusions have been made following detailed engineering geological characterization; analysis and interpretation of the overall subgrade embankment fill material from yerer-2 borrow site and ballast stone material from Galan quarry. Therefore, based on the test results of primary, secondary data and site condition in general, the following recommendations are given:

- It is recommended that more additional parameters of settlement potential and mineralogical tests (Activities of clay minerals) should be performed to have more realistic results.
- Deep cuts for under pass area (especially megenagna, Golagul Bldg and St. Urael Church area) need special attentions and precaution as the level for the ground water and in depth to municipal waste seepages are the series problems in soaking the sub-grade embankment material and the retaining walls with aggressive mixtures. So that still it is recommended that depending on local conditions, the ground water table and waste water seepage level should be lowered to a minimum depth of between 1m and 2.50m below rail top.
- The geologic and physiographic environments have a great impact on the functionality and strength of any civil structures. As a result, in previous chapter it is observed some lineaments in the geological map of the area. This is especially at

Tele-Gurdshola area. Therefore, it is strongly recommended that special attention has to given for either the foundation strength or compaction parameters and for the ground water level otherwise the presence of seepages or fissures etc.

- Except for the test result of water absorption, the ballast stone material is suitable from engineering geological point of view. Therefore it is strongly recommended that the ballast aggregates of uniformly graded have to be packed well under the slippers so that water droplets will quickly flush down to the ditch.

REFERENCES

AASHTO (1996), (1997). *American Association of State Highway and Transportation Officials. AASHTO Guide for Design of Pavement Structures*, Washington, D.C U.S.A.

Abateneh Yitayew Alemu, 2011, *Survey of Railway Ballast Selection and Aspects of Modelling Techniques MSc thesis,* SE-100 44 Stockholm.

Abebe Dinku, 2002, *Construction material testing manual, Department of Civil Engineering,* Addis Ababa University Press.

American Association of State Highway and Transportation Officials (AASHTO) (1993). *Design of Pavement Structures*. American Association of State Highways and Transportation Officials, Washington D.C, USA.

American Society for Testing and Materials (ASTM) standard manual volume I and II

American Railway Engineering and Maintenance-of-Way Association, AREMA (2009). Manual for Railway Engineering. Landover, MD.

American Railway Engineering and Maintenance-of-Way Association AREMA (2010). Manual for Railway Engineering. Landover, MD.

Arora, K.R, (1997), *Soil mechanics and Foundation Engineering.* A.K. Jain, standard Publishers. India.

ARTC, A. R. (2007, 01 04). *Australian Rail Truck Corporation LTD (ARTC).* Retrieved 04 20, 2011, from http://extranet.artc.com

ASTM C 29 (1997), *Standard test method for Bulk Density ("Unit Weight") and Voids in Aggregate, ASTM Committee C09 on Concrete and Concrete Aggregates*, United States, 4 Pp.

ASTM C 88 (1999a), *Standard Test Method for Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate, ASTM Committee C09 on Concrete and Concrete Aggregates,* United States, 5 Pp.

ASTM C 127 (2001), *Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Coarse Aggregate, ASTM Committee C09 on Concrete and Concrete Aggregates*, United States, 6 Pp.

ASTM C 131 (2001), *Standard Test Method for Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine, ASTM Committee C09 on Concrete and Concrete Aggregates*, United States, 4 Pp.

ASTM C127-04. *Standard test method for density, relative density (specific gravity), and absorption of coarse aggregate. Annual Book of ASTM Standards, Vol. 04.02. Philadelphia:* American Society for Testing and Materials.

ASTM D4644-87. *Standard test method for Slake durability of shale and similar weak rocks. Annual Book of ASTM Standards, Vol. 04.08.* Philadelphia: American Society for Testing and Materials.

Bayisa Ragassa, 2011, *Potential Quarry Site Selection and Ballast Rock Characterization for Ambo-Ijaji Railway Project, West Showa None of Oromia Region,* Unpublished M.sc Thesis, Addis Ababa University, Faculty of science, Addis Ababa,Ethiopia

Bell, F.G. (2007), *Engineering geology, second edition*, Elsevier 581 Pp.

Bonnet, C.F (2005), *Practical Railway Engineering, 2nd Edition, Imperial College press*, London 190 Pp.

Brennan Bailey & D. Jean Hutchinson, 2011. *Assessment of embankment fouling from geotechnical testing of railway ballast samples Pan-Am CGS Geotechnical conference, CN Rail, Montreal, Quebec, Canada* CN. 1996.

Brown, E.T. 1981. *Rock Characterization, Testing and Monitoring – ISRM Suggested Methods.* Oxford: Pergamon.

BS 812 (1990), *Testing Aggregates, Part 112: Methods for determination of Aggregate impact value (AIV),* Board of BSI, UK, and 15 Pp.

BS 812 (1990), *Testing Aggregate, Part 110: Methods for determination of Aggregate Crushing Value (ACV),* Board of BSI, UK, 13 Pp.

Canadian National Railway Specification for Crushed Rock Ballast. 12-20B. Montreal, QC, Canada.

China's Railway Manual (2005/2010). *National railway standard of the people's republic of china*, China Railway Publishing House, Beijing China.

Daniel Gemechu, 1977, *Aspects of Climate and Water Budget in Ethiopia, A Technical Monograph Published For Addis Ababa University*, Addis Ababa University Press, Ethiopia.

ERA, 2002, *Pavement design Manual, Volume I and II, Flexible Pavements and Gravel Roads*, Addis Ababa, Ethiopia.

Ernest T. Selig and John M. Waters. (1994). *Track Geotechnology and Substructure Management.* London: Thomas Telford Publications.

Government of India, ministry of railways research designs and standards organization, RDSO, No. GE-1, (2003). Guidelines of Earthwork in Railway Projects, India.

Government of India, ministry of railways research designs and standards organization, RDSO, No. GE-0011, (2007). Guidelines of Earthwork in Railway Projects, India.

Habtamu Solomon (2011). *Chemical stabilization of Expansive subgrade soil performance evaluation on selected road section in northern Addis Ababa*. Unpublished M.sc Thesis, Addis Ababa University, Faculty of science, Addis Ababa, Ethiopia.

Haile Sellasie Girmay and Getaneh Assefa, 1989, *Addis Ababa-Nazareth Volcanics: A Miocene- Pleistocene Volcanic Succession in Ethiopian Rift Ethiopian Journal of Science*, Addis Ababa, Ethiopia.

Holtz,W.G. and Gibbs, H.J., 1956, *Engineering Properties of Expansive Clays, Transactions,* American Society of Civil Engineers,vol.121,pp.641-677.

Indian standard methods of test for soils IS-2720 (1997), *first revised, bureau of Indian standards manak bhavan, 9 bahadur shah zafar marg,* New Delhi, Indian.

International Union of Railways (UIC), UIC Code, 1994R 1994

International Union of Railways (UIC), UIC Code 719R-2005.

Ionescu, D. 2004. *Evaluation of the engineering behavior of railway Ballast. Ph.D thesis, University of Wollongong,* Wollongong, Australia.

Jiunnren Lai On Spring 2004 *Direct Shear Testing Originally Prepared By Dr. Roy E. Olson On Fall 1989*

Joseph E. Bowels, 1992: Fourth Ed. Engineering Properties of soils and their measurements, McGraw Hill, USA.

Karsten Therman et al., 2006 *Shear strength parameters from direct shear tests influencing factors and their significance*. IAEG 2006 Pp 484.

Kebede Tsehayu & Tadesse H.Mariam, 1990, *Engineering Geological Mapping of Addis Ababa, Ethiopian Institute of Geological Survey, Addis Ababa, Ethiopia.*

Keir, D., Ebinger, C., Stuart, G., Daly, E., Ayele, A., 2003, *Seismicity of the Northern Main Ethiopian and Southern Red Sea Rifts: EAGLEII,* American Geophysical Union, NASA ADS.

Matewos Bekele (2012). *Engineering Geological Characterization, Suitability Assessment and Possible Remedial Measures for Sebeta – Ambo Railway Project, Central Ethiopia*. Unpublished M.sc Thesis, Addis Ababa University, Faculty of science, Addis Ababa, Ethiopia.

Mengesha Tefera, Tewodros Chernet and Workneh Haro, (1996), *Geological Map of Ethiopia (1:2,000,000), Second Edition and Explanatory notes.* Bulletin N0. 3. GSE.

Mittal. S. and J.P. Shukula 2003. *Soil Testing For Engineerers, for KHANNA Publishers 2-B, Nath Market, Nai Sarak,* Delhi-India.

Murthy, V.N.S, 1990. *Geotechnical Engineering Principles And Practices Of Soil Mechanics And Foundation Engineering.* Marcel Dekker Inc, 270 Madison Avenue, New York, 10016.

Raisanen M., Mertamo M., STENLID L. and Viitanen, J. (2006), *Laboratory crushing of rock aggregates, Paper number 411,* Geological *Survey of Finland*

Raymond, G.P. (1985). *"Railroad Ballast Specification and Evaluation," Lecture Notes. Selig, E.T. (2004). Railway Track Design. In Handbook of Transportation Engineering*. Edited by Kutz, M. The McGraw-Hill Companies, Inc., McGraw-Hill.

Selig, E.T., and Waters, J.M. 1994. *Track Geotechnology and Substructure Management*, Thomas Telford Services Ltd, London.

Tamiru Alemayehu, TenalemAyenew, Dagnachew Legese, Yirga Tadese, Solomon Waltenigus and Nuri Mohamed, 2006, *Ground water Vulnerability Mapping of the Addis Ababa Water Supply Aquifers,* Addis Ababa, Ethiopia.

Tilahun Mammo (2005). *Site-specific ground motion simulation and seismic response analysis at the proposed bridge sites within the city of Addis Ababa, Ethiopia:* Elsevier engineering geology, pp. 127-150.

Tutumluer, E. Dombrow, W., and Huang, H. 2008. *Laboratory characterization of coal dust fouled ballast behavior, Journal of the Transportation Research Board,* 2117: 93-101.

UNEP/UNESCO/UN-HABITAT/ECASCIENTIFIC (2003). *The Ground Water Vulnerability Mapping of the Addis Ababa water supply Aquifers, Ethiopia*. Addis Ababa University, Department of Geology & Geophysics (AAU) and Addis Ababa Water & Sewerage Authority (AAWSA),Addis Ababa, Ethiopia

www.erc.gov.et accessed on Nov.10-2013

www.nra.ie/Publications/DownloadableDocumentation/Environment/file,16183,en.pdf accessed on March 27-2014

ANNEX ONE

INDEX PROPERTIES FOR SUB-GRADE EMBANKMENT FILL MATERIAL

Particle Size Analysis of Soils (ASTM D 422)

Date Tested: 31/03/2014
Tested By: Dejene Fikadu
Project Name: Msc Thesis
Sample Description: Subgrade Embankment Fill Material
At: School of Earth Sciences, Engineering Geology laboratory

Sieve opening	Sieve size (mm)	Weight of soil retained (g)	% retained	Cumulative % retained	% Finer
25mm	25	447.92	15.43	15.43	84.56
12.5mm	12.5	755.97	26.04	41.48	58.51
6.3mm	6.3	541.34	18.65	60.13	39.86
4.00mm	4	120.36	4.14	64.28	35.71
2mm	2	257.48	8.87	73.15	26.84
1.18mm	1.18	240.88	8.29	81.45	18.54
0.6mm	0.6	329.68	11.35	92.81	7.18
0.425mm	0.425	84.37	2.90	95.72	4.27
0.3mm	0.3	46.6	1.60	97.32	2.67
0.15mm	0.15	46	1.58	98.91	1.08
0.075mm	0.075	20.6	0.70	99.62	0.37
pan		11	0.38	100	0
total		2902.2	100		

Liquid Limit Determination (BS 1377:Part 2 : 1990, ERA: 2002 and IS :2720)

Trial No.	1	2	3
Mass of can + moist soil (grams)	65.6	43.5	46.84
Mass of can + dry soil (grams)	53	39.2	42.2
Mass of Can (grams)	27	27.6	29.2
Mass of Dry Soil	26	11.6	13
w = Water content, w%	48.46	37.07	35.69
No. of drops (N)	24	26	30

From the chart Liquid Limit (LL) = 43.0%

Plastic Limit Determination ((BS 1377:Part 2 : 1990, ERA: 2002 and IS :2720)

Trial No.	1	2	3
Mass of can + moist soil (grams)	23.4	24.4	21.54
Mass of can + dry soil (grams)	20.9	21.7	20.32

Mass of Can (grams)	16.2	15.8	15.8
Mass of Dry Soil	4.7	5.9	4.52
w = Water content, w%	53.19	45.762	26.99
Plastic Limit (PL) = Average w % =	41.98178389		

ANNEX TWO

PROCTOR TEST (Modified, PROCTOR TEST ASTM D698-1991)

Date Tested: 03/04/2014

Tested By: Dejene Fikadu With Mr. Bayisa Ragassa(MSc)

Project Name: Msc Thesis

Sample Description: Subgrade Embankment Fill Material

At: School of Earth Sciences Engineering Geology laboratory

	TRIAL NUMBER			1	2	3	4
DENSITY	WEIGHT OF SOIL + MOLD	(g)	W_1	4,137	4,223	4,120	
	WEIGHT OF MOLD	(g)	W_2	2057.59	2057.59	2057.59	
	VOLUME OF MOLD	(Cm^3)	V	943.9	943.9	943.9	
	WEIGHT OF WET SOIL	(g)	$W_3 = W_1 - W_2$	2,080	2,165	2,063	

MOISTURE	WET DENSITY OF SOIL	(g/Cm^3)	$W_d = W_3/V$	2.203	2.294	2.185
	CONTAINER NUMBER			l	v	P
	WET SOIL + CONTAINER	(g)	a	386	359.1	337.57
	DRY SOIL + CONTAINER	(g)	b	357.1	324.4	295.0
	WEIGHT OF CONTAINER	(g)	c	40.4	40.89	40.4
	WEIGHT OF WATER	(g)	e = a-b	28.9	34.7	42.6
	WEIGHT OF DRY SOIL	(g)	d =b-c	316.7	283.5	254.6
	MOISTURE CONTENT	**(%)**	**m= (e/d)*100**	9.1	12.2	16.7
	DRY DENSITY OF SOIL	**(g/Cm3)**	**$D_d = W_d/(100+m)*100$**	2.019	2.044	1.872

ANNEX THREE

CALIFORNIA BEARING RATIO (CBR) Test Results (AASHTO T- 193, (1993) and BS 1377: Part 4: 1990)

DATA SHEET

California Bearing Ratio (CBR) (One Point)
Date Tested: 23/04/2014
Tested By: Dejene Fikadu and Ato Mastewal
Project Name: Msc Thesis
Sample Description: Subgrade Embankment Fill material
At : AAiT , Civil Engineering Laboratory

swelling gauge reading = 4+22.5
swelling of the gauge reading after 4daays = 4+23.6
mold + Soil(compacted) = 9015g
mold + Soil(wet) = 9150g
wt of mold =

Ring calib. factor, N/Div	27.707	Rammer wt. (kg)	4.54	OMC	
Plunger Area, mm2	1935	Blow /Layer	56	MDD	
Rate of strain, mm/min	1.27			Swell (%)	3.8

penetration of plunger (mm)	Load dial Reading (Div.)	Load (KN)	stress (KN/mm2)	stress (N/mm2)	Corr. Load (KN)	CBR (%)
0	0	0	0	0	0	
0.64	120	3.32484	0.001718264	1.718263566	3.3	
1.27	172	4.765604	0.002462844	2.462844444	4.8	
1.91	226	6.261782	0.003236063	3.236063049	6.3	
2.54	280	7.75796	0.004009282	4.009281654	7.8	58.47076
3.81	372	10.307004	0.005326617	5.326617054	10.3	
5.08	402	11.138214	0.005756183	5.756182946	11.1	55.5
7.62	575	15.931525	0.008233346	8.233346253	15.9	
10.16	660	18.28662	0.00945045	9.450449612	18.3	
12.7						

ANNEX FOUR

GRADATION TEST FOR THE BALLAST STONE (BS: 812, Part 103.1: 1985 and ASTM D 422)

Date Tested: 08/04/2014
Tested By: Dejene Fikadu
Project Name: Msc Thesis
Sample Description: Ballast Stone Material
At: School of Earth Sciences Engineering Geology laboratory

Sieve No	Sieve opening	Weight of soil retained (g)	% retained	Cumulative % retained	% Finer
	75	0	0	0	100
	63	0	0	0	100
	50	1599.5	23.50015	23.50015	76.49985
	37.5	2302.467	33.82827	57.32841	42.67159
	25	1088	15.9851	73.31351	26.68649
	19	994.9	14.61725	87.93076	12.06924
	12.5	603.6	8.868202	96.79897	3.201034
	10	204.2333	3.000633	99.7996	0.200401
	Pan	13.64	0.200401	100	0

ANNEX FIVE

TEST FOR SPECIFIC GRAVITY, BULK DENSITY AND WATER ABSORPTION test for ballast Stone (ASTM C127, 2001)

Date Tested: 09/04/2014
Tested By: Dejene Fikadu with Ato Mastewal
Project Name: Msc Thesis
Sample Description: Ballast Stone Material
At: School of Earth Sciences Engineering Geology laboratory and AAiT ,
School Of Highway Engineering Laboratory

Test Number	sample 1	sample 2	Mean
Mass of saturated surface dry aggregate in air (A)	454.4	519.8	487.1
Massof saturated aggregate + Basket in water (B)	308	369.2	338.6
Mass of basket in water (C)	17	17	17
Mass of oven dried aggregate in air (D)	447.4	511.5	479.45
Specific gravity, rd= D/[A-(B-C)]	2.738066095	3.051909308	2.894987702
Apparent specific gravity, r(ap)=D/[D-(B-C)]	2.860613811	3.210922787	3.035768299
SSD specific gravity, r(SSD)=A/[A-(B-C)]	2.780905753	3.101431981	2.941168867
water absorption, w=(A-D)/Dx100	1.56459544	1.622678397	1.593636919

ANNEX SIX

BALLAST STONE MATERIAL RESISTANCE TO MECHANICAL DEGRADATIONS

Laboratory Impact Value for Ballast Stone (BS 812: Part 112, 1990)

Date Tested: 23/04/2014
Tested By: Dejene Fikadu With Ato Getnet
Project Name: Msc Thesis
Sample Description: Ballast Stone Material
At: AAiT, School of Civil Engineering , Highway Laboratory

	Standard fraction tested	separating sieve		
	14-10mm	2.36mm		
Specimen identification No.		Trial -01	Trial -02	Average
Number of blows (commonly 15 blows)	N			

Mass of tray + specimen (g)		1194	1012	1103
Mass of tray alone (g)		881.3	709.7	795.5
Mass of original test specimen (g)	M1	312.7	302.3	307.5
Mass of tray + material passing 2.36mm sieve (g)		46.7	58.5	52.6
Mass of tray alone (g)		4.9	4.9	4.9
Mass of material passing sieve size 2.36mm (g)	M2	41.8	53.6	47.7
Mass of tray + material retained on 2.36mm sieve (g)		275.2	253.4	264.3
Mass of tray alone (g)		4.9	4.9	4.9
Mass of material retained on sieve size 2.36mm (g)	M3	270.3	248.5	259.4
Mass passing + retained on separeting sieve (check versus M1)	M2+M3	312.1	302.1	307.1
Aggregate impact value (%)	M2/M1 X100	13.37	17.73	15.55
AIV mean value (%)	**15.55**			

Laboratory Crushing values for Ballast Stone (BS 812: Part 110, 1990)

Date Tested: 24/04/2014
Tested By: Dejene Fikadu with Ato Fikiru Badada
Project Name: Msc Thesis
Sample Description: Ballast Stone Material
At: AAiT, School of Civil Engineering , Material Laboratory

	Standard fraction tested	**separating sieve (Retained On)**	
Test Procedures	14-10mm	2.36mm standard	
Trial Number		Trial -01	Trial -02
Mass of tray + specimen (g)		3606.3	3640.8
Mass of tray alone (g)		188.2	188.2
Mass of original test specimen (g)	M1	3418.1	3452.6
Mass of tray + material passing 2.36mm sieve (g)		656.7	674.7
Mass of tray alone (g)		188.2	188.2
Mass of material passing sieve size 2.36mm (g)	M2	468.5	486.5
Mass of tray + material retained on 2.36mm sieve (g)		3136.8	3153.5
Mass of tray alone (g)		188.2	188.2

Mass of material retained on sieve size 2.36mm (g)	M3	2948.6	2965.3
Mass passing + retained on separeting sieve (check versus M1)	M2+M3	3417.1	3451.8
Aggregate Crushing value (%)	M2/M1X100	13.7	14.1
ACV mean value	%	13.89	

Laboratory Los Angeles Abrasion Test Results (AASHTO T-96, 1977 method)

Date Tested: 01/05/2014
Tested By: Gondwana Engineering Plc
Project Name: Msc Thesis
Sample Description: Ballast Stone Material

Sieve Size Passing (mm)	Sieve Size Retained (mm)	No. of Spheres	Weight of Surcharge gm.	Weight of sample before, gm.	Weight of sample after, gm.	Weight loss gm.	Percent loss %
37.5	25.0			1250			
25.0	19.0			1250			
19.0	12.5			1250	2500		
12.5	9.5			1250	2500		
9.5	6.30						
6.30	4.75						
4.75	2.36						
Total		12	4592.5	5000	4125	875	**18**

Slake Durability Test (ASTM D4644-2008 Method)

Date Tested: 02/04/2014
Tested By: Dejene Fikadu
Project Name: Msc Thesis
Sample Description: Ballast Stone Material
At : School of Earth Sciences, Engineering Geological Laboratory

Slake Durability Ballasat 1 (Trial -1)		Slake Durability Trial 1 Ballasat 2 (Trial -1)	
Wt of can + Sample(g)	590.16	Wt can + Sample(g)	533.28
Wt of Can(g)	60.96	Wt of Can(g)	54.45
Wt of Sample(g)	529.2	Wt of Sample(g)	478.83

After Oven Dry (Trial 1)

Slake Durability Ballasat 1(Trial - 1)		Slake Durability Ballasat 2 (Trial - 1)	
Wt of can + Sample(g)	581.03	Wt of can + Sample(g)	525.09
Wt of Can(g)	60.96	Wt of Can(g)	54.45
Wt of Sample(g)	520.07	Wt of Sample(g)	470.64

Wt loss	**9.13**	Wt loss	**8.19**
	After Oven Dry (Trial 2)		
Slake Durability Ballasat 1(Trial - 2)		Slake Durability Ballasat 2 (Trial - 2)	
Wt of can + Sample(g)	578.77	Wt of can + Sample(g)	522.08
Wt of Can(g)	60.96	Wt of Can(g)	54.45
Wt of Sample(g)	517.81	Wt of Sample(g)	467.63
Wt loss	**2.26**	Wt loss	**3.01**

ANNEX SEVEN

BALLAST STONE RESISTANCE TO CHEMICAL DEGRADATION

Soundness Test Results (AASHTO T-104, 1996 method)

Date Tested: 01/05/2014
Tested By: Gondwana Engineering Plc
Project Name: Msc Thesis
Sample Description: Ballast Stone Material

Passing Sieve Size	Retained On Sieve Size	Grading Of Test Sample, % ret.	Container No.	Wt. Of Specimen before test (gm)	Wt. Of Specimen after test (gm)	Percent passing after test	Corrected % Loss
63mm	50mm						
50mm	37.5mm						
37.5mm	25.0mm	73.3	1	1000.00	991.70	0.8	0.61
25.0mm	19.0mm	14.6	2	501.00	497.80	0.6	0.09
19.0mm	12.5mm	8.9	3	669.00	661.10	1.2	0.11
12.5m	9.5mm	3.0	4	330.00	325.20		0.04

						1.5	
9.5mm	4.75mm	0.2	5	300.00	292.70	2.4	0.00
Total							**0.9**