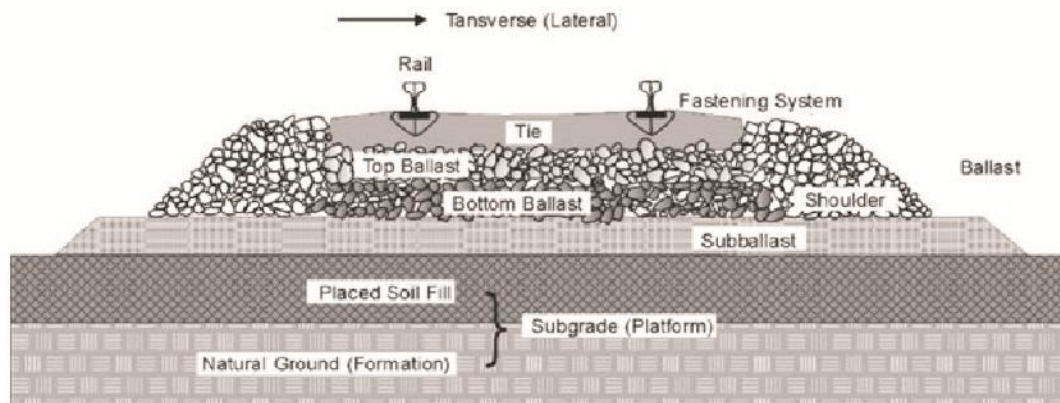




Addis Ababa University
school of Graduate Studies,
Faculty of Science

**Engineering Geological Characterization, Suitability Analysis
and Possible Remedial Measures For Sebeta-Ambo Railway
Project, Central Ethiopia**



A Thesis Submitted to
The School of Graduate Studies of Addis Ababa University
In Partial Fulfillment of the Requirements for the Degree of
Master of Science in Engineering Geology

By
Matewos Bekele

June 2012

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June 2012

DECLARATION

I hereby declare that the thesis is my original work under the supervision of Dr. Tarun Kumar Raghuvanshi, Department of Earth Sciences, Addis Ababa University during the year 2012 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.

Matewos Bekele

Signature _____

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

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ABSTRACT

Adequate knowledge and proper investigation of the sub-structure condition is important for effective assessment and control of deterioration of train track geometry. The present research study was conducted to characterize and assess the general suitability of the proposed route alignment of Sebeta–Ambo railway corridor. The present research was mainly focused on analyzing the engineering geological suitability of the railway alignment and to evolve possible remedial measures for the likely engineering geological problems along the general alignment. Besides, the construction material proposed to be used as embankment fill, blanketing and to be used as replacement material for sub-grade has also been characterized for its general suitability. In order to meet out the general objectives of the present research study attempts were made to characterize the natural sub-grade and embankment fill materials including blanketing layer. For this field observation, investigation, sampling and laboratory tests for physical and strength properties assessment were performed on representative samples from natural sub-grade and fill materials. Besides, settlement potential of the natural sub-grade, in particular was evaluated. The dynamic property of the embankment material was also assessed by reviewing relevant literatures. Different manuals and specifications were used to evaluate the suitability of the materials as well as to characterize and interpret the topography of the area. Further, attempts were also made to identify and characterize the hydrology and hydro-geological setup of the area by reviewing the relevant literatures and previous works carried out in the area.

Based on the characterization, the natural sub-grade was found to be unsuitable based on its low bearing capacity, settlement potential and high potential for volume change under moisture variation. In order to improve the foundation condition it has been suggested to remove the top natural undesirable soil and replace it with improved material. Besides, stage construction of fill materials can also be implemented. In addition to this it has been assessed that most of the identified fill material is granular in nature and can be designated as silty gravel (GM). This problem can be addressed by blending it with fine soil in appropriate proportion with the fill material. Blanketing materials, coarse, granular and well graded, was also used to improve the bearing capacity of the formation and hence similar materials as that of the embankment fill were used to characterize them. Based on the standard practices followed in other leading railway systems, it is evident that multi-layer formation system–layers with adequate strength and stiffness has to be adopted. Hence, according to Indian Railways Research Designs & Standards Organization, Two Layer System (Blanket & Prepared Sub-grade on Embankment Fill) of sub-structure formation design thickness has been suggested. Finally, this research project will provide an additional input for consultants participating in construction; will be the platform for future academic researcher and similar type of project.

CHAPTER I - INTRODUCTION

1.1 General

Transport infrastructure in Ethiopia has been neglected for decades, however since recent past it got priority by the Government of Ethiopia. Rail transport in Ethiopia currently encompasses only a line from Djibouti to Dire Dawa. This line continues from Dire Dawa to Addis Ababa, but no longer it's operational. 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. The railway has lost competitiveness, especially since Ethiopia has begun to improve its road network. Of course, Ethiopia is not new for train transport; nor is train transport from Ethiopia to a foreign country. In this regard the two countries have a century old relation. Despite such a long time, the poor train transport service in the country had, once, disrupted completely. 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, design, planning and construction has began on the project. The new network will be 5,000 km long, and radiate from the capital city of Addis Ababa. This network will be constructed in two phases, the first phase involving the construction of five lines (Fig. 1.3). Among these Addis-Ababa-Sebeta-Ambo railway corridor is one of them (ERC, 2010/11).

Experience from many countries has revealed that the railway systems play an important role in providing a good transportation system in any country. In Ethiopian context the importance of railway cannot be denied as the country is land locked and the transportation primarily depends on road and railway systems. The proposed new railway development project will not only link the capital, Addis Ababa, to various regions of the country but will also boost the country's trade flow after completion.

The railway track system plays an important role in providing a good transportation system in different countries. In most of the countries, a very large portion of the annual budget to sustain the railway track system goes into track maintenance. In the past, much of the attention has been given to the track super-structure consisting of the rails, the fasteners and the sleepers, and relatively less attention has been given to the sub-structure consisting of the ballast, the sub-ballast and the sub-grade (Fig.1.1). 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).

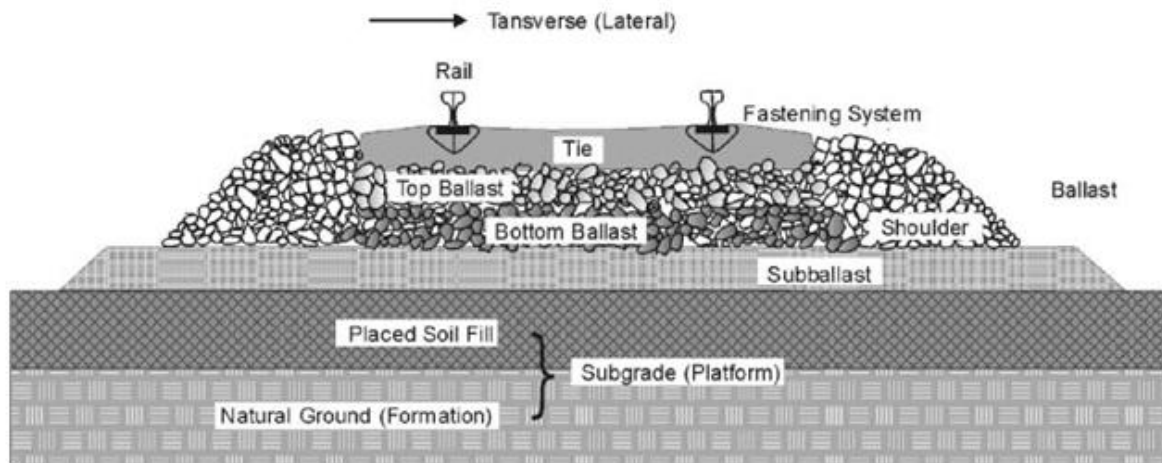


Fig. 1.1 Track structure components (Selig and Waters, 1994)

The character of soil and rock is useful for the foundation of any engineering structure. The foundation is the part of the engineered system that transmits to, and into, the underlying soil or rock the loads supported by the foundation and its self weight (Punmia et al., 2005).

The railway track structure combines a number of components in a structural system that is intended to withstand the combined effect of traffic loading and climate such that the sub-grade is adequately protected and railway vehicles operating cost, safety and comfort of passengers are kept within acceptable limits. The purpose of a railway track is to guide the train in a safe and economic manner. The track and the switches should allow smooth passage of trains. If the track is not perfectly leveled and aligned, the irregularities will cause oscillations or vibrations of the train, and this may induce discomfort for passengers and damage to goods.

The track carries the load of the train and distributes it over an area of the sub-grade that is as large as possible. Railway track consists of a super-structure and a sub-structure. The super-structure is composed of steel rails fastened to the cross-ties. The rails are designed to support and guide flanged steel wheels through their prescribed position in space. The superstructure is placed on a substructure. The substructure is composed of a layered system of materials known as ballast, sub-ballast and sub-grade. The rail ballast absorbs the train weight and distributes it from the rails to the sub grade, thereby avoiding any deformation. The railroad can thus keep its geometrical features. The sub-ballast, may or may not present in rail track,

also assist in distributing the loads transmitted by passing trains and to protect the embankment body from the seepage of rain-water and from seasonal thermal extremes (frost and thaw cycles).

Ultimately all the loads (statics and dynamics) placed on the track by trains are carried by the sub-grade. It is either compacted natural ground or an imported fill embankment, which provides a permanent way to support the ballast layer. The sub-grade is considered to be as important to track performance as the rail and ballast.

Each soil and rock deposit, which can be used as sub-grade material, is extremely variable and has its own characteristics which reflect its origin and the factors affecting it. As a result, investigation and testing are uniquely important if soils and rock are to be used economically and safely in engineering works (AREMA, 2009).

Railway construction, therefore, like any other engineering structures needs investigation of the natural properties of sub-grade material for its suitability for construction. This property may cause severe damages to infrastructure, unless proper measures are taken in their design.

1.2 Justification of the problem

Adequate knowledge of the sub-structural condition is important for effective assessment of the potential service interruptions and the need for slow orders. A significant part of a railroad's track maintenance budget is allocated to correct rough track that is caused by movements in the sub-structure under repeated train loading.

Deterioration of the track geometry has been recognized to be the main source of the need for track maintenance. This deterioration is mainly caused by the settlement of the sub-structure, which tends to depend on the site conditions and fill material properties. Many railway lines in the world including Ethiopia, are more than 100 years old, and are not designed in accordance to the requirements for modern railway traffic (Albalat, 2011). Due to the future demands for faster and heavier transports, railway structures can experience problems, such as reduced stability, increase of settlements, and possibility of extensive vibrations (Evstatiev et al., 1995; Porbaha et al., 1998; EUROSOILSTAB, 2000). These issues have an adverse effect on the safety, reliability, and economy of the railway operations. Thus, the hidden and hard-to-monitor sub-structure conditions are important to railway track performance. One of the important activities to be done to solve the problem is, therefore, proper investigation and

design of the railway line which considers both the actual ground condition and material properties of placed soil fill. Therefore, the present study was undertaken to characterize sub-surface in-situ material and embankment fills. Finally, an attempt has also been made to evolve necessary improvement techniques for in-situ sub-grade material by designing structural formation layer for Sebeta-Ambo railway corridor in Ethiopia. Thus, it is expected that the present study may probably facilitate in reducing the life cycle cost of railway infrastructure.

Like other developed or developing countries Ethiopia also needs a modern, economic, time saving and long lasting transportation system which will lead to economic growth at a faster pace. Perhaps, new railroad connecting the length and breadth of the Country seems to be an effort to accelerate such economic growth. Thus, the country needs to exert relentless efforts and allocation of a huge amount of resources to provide accessibility and to increase transport infrastructure throughout the country. In this context it is imperative that this effort should be accompanied by strict and professionally ethical quality assurance and supervision of construction.

1.3 History of railway line construction in Ethiopia and its Growth

The idea of constructing a railway line to link the Ethiopian capital with the coast appears to have been first conceived by Menelik's Swiss adviser, Alfred Ilg, who had first arrived in Ethiopia in 1877. The idea, however, was executed but a very slow progress was observed. On February 11, 1893, however, Alfred Ilg obtained a written decree from Menelik officially empowering him to study the project and set up a company. The plan was to start the railway line from the coast and to push inland bringing in supplies as the line advanced. Djibouti was considered more suitable than any other port on the Horn of Africa.

Ethiopia has now one narrow gauge railway connecting Addis Ababa to Djibouti port. This line is jointly owned by Ethiopia and Djibouti Governments and is operated by Chemin de Far Djibouti-Ethiopian (CDE). It is a historical line built at the initiative and the authority of Emperor Menelik II. For this construction began in 1889 E.C at the port city of Djibouti, and the final link reached Addis Ababa in 1910 E.C. The rail line passes through 32 towns and cities between Addis Ababa and Djibouti. It has 34 stations out of which Addis Ababa, Dire Dawa and Djibouti are the main stations and they provide both passenger and freight services, the rest provide passenger service only.

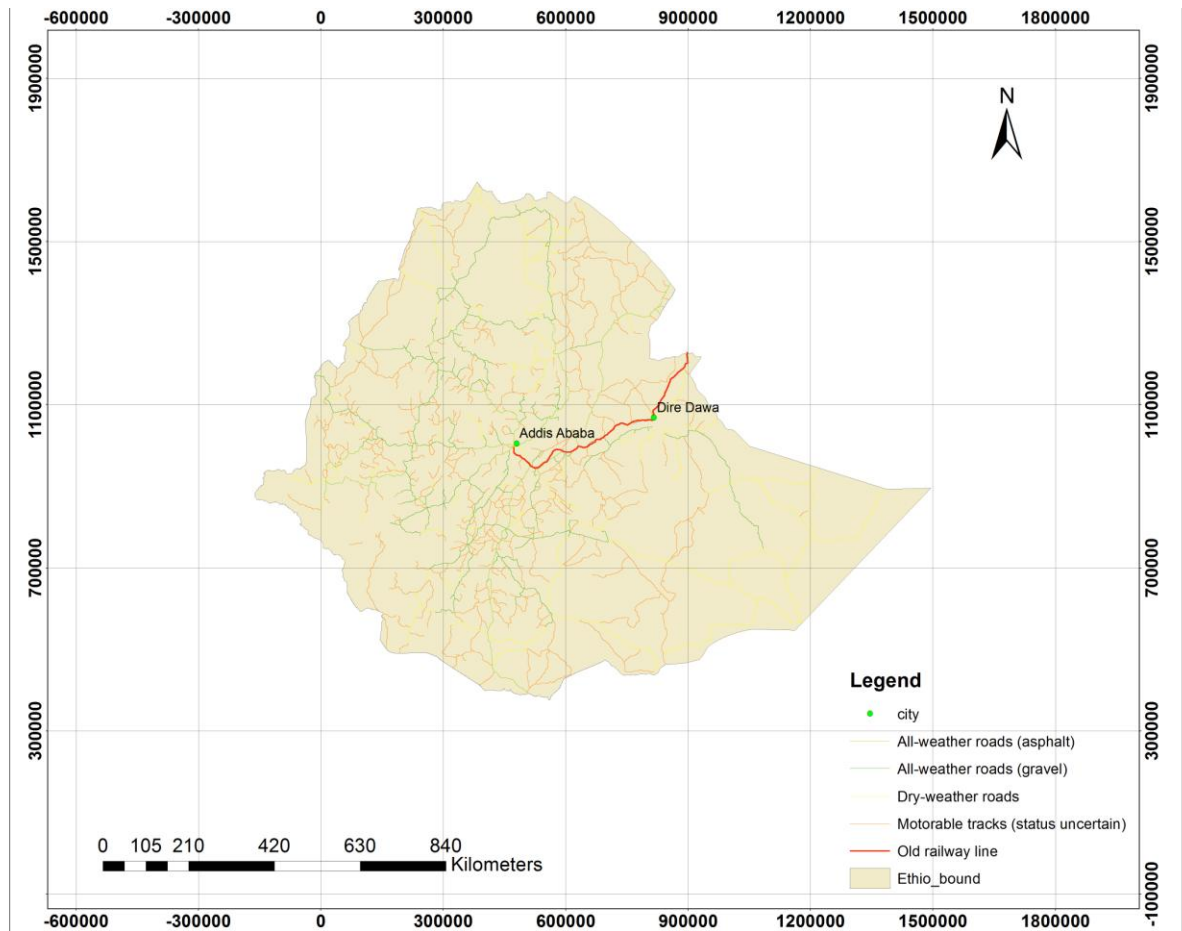


Fig.1.2 Addis Ababa – Djibouti Railway line

CDE line currently is in dilapidated condition and operates a skeleton service. In 2006/07, 44,634 tons of export commodities, 23,598 tons of import commodities, and 7,513 tons of domestic commodities were transported through this line (MTC, 2007).

1.3.1 Introduction to new national railway routes

The 781km CDE railway line is of great importance both to a landlocked country like Ethiopia as well as to Djibouti. More than 82% of the railway line traverses through Ethiopian land linking major Ethiopian trade and production centers. This line is also a strategic link for Ethiopia to access international maritime route and has the potential to carry up to 80% of the Ethiopia's international in-bound cargo. On the other hand, 90% of cargo in and out of Djibouti port requires an access route to disperse the cargo quickly to reduce port congestion and therefore, railroad is the right option for this (ERC, 2010/11).

In general, in addition to the above justification Ethiopian Railways Corporation (ERC) unequivocally justified introduction of railway system across Ethiopia, as primary national mass transport system (ERC, 2010/11) on the following grounds;

- (i) Geographical necessities
- (ii) Alternative means of transport
- (iii) Throughput in transportation system
- (iv) Combining Road and rail system
- (v) Operating Costs
- (vi) Saving in travel Time
- (vii) Employment generation
- (viii) Increase in demand
- (ix) Social benefits

1.3.2 Government of Ethiopia’s plan for railroad development

Ethiopian Railway Corporation (ERC) was set up to develop railway network in Ethiopia under the regulatory authority of Ministry of Transport and communications, by the council of Minister Regulation No.141/2007. ERC has further studied the proposed national railway network and based on the developmental goal of the Government of Ethiopia, prioritized the lines (ERC, 2010/11). The development indicators with respective weights assigned to them are shown in Table 1.1. Based on the reasoning as mentioned in Table 1.1, the railway lines have been planned across Ethiopia (Table 1.2, Fig.1.3 & 1.4) under the Ethiopian National railway Network (ENRN) plan.

The Sebeta–Ambo railway corridor, where the present research is carried out, is among one of the railway line, Addis-Ababa-Ijaji-Kombolcha-Guraferda-Dima including Kombolcha-Hara Gebeya, to be constructed in the first phase of the project. The present research was mainly focused on analyzing the engineering geological suitability of the railway alignment and to evolve possible remedial measures for the likely engineering geological problems along the general alignment by considering economical and technical factors.

Table 1.1 The development indicators with respective weights assigned (TAG, 2007)

Indicator	Weight (%)
Population Served By the Railway Link	20
The existing agriculture production and future potential of the Woredas crosses by the link	20
Existing livestock potentials for the Woredas	10
The Availability of mining and the types of mines soon ready for immediate production	10
Contribution of the link for National and International trade and access	5
Existing Road density and distribution on the Woredas crosses by the link	5
Investment cost for the link	30

1.4 Application of results

The findings and results from the present study mainly characterize sub-surface in-situ material and embankment fills. Finally, it evolves the necessary improvement techniques for in-situ sub-grade material with designing the structural formation layer.

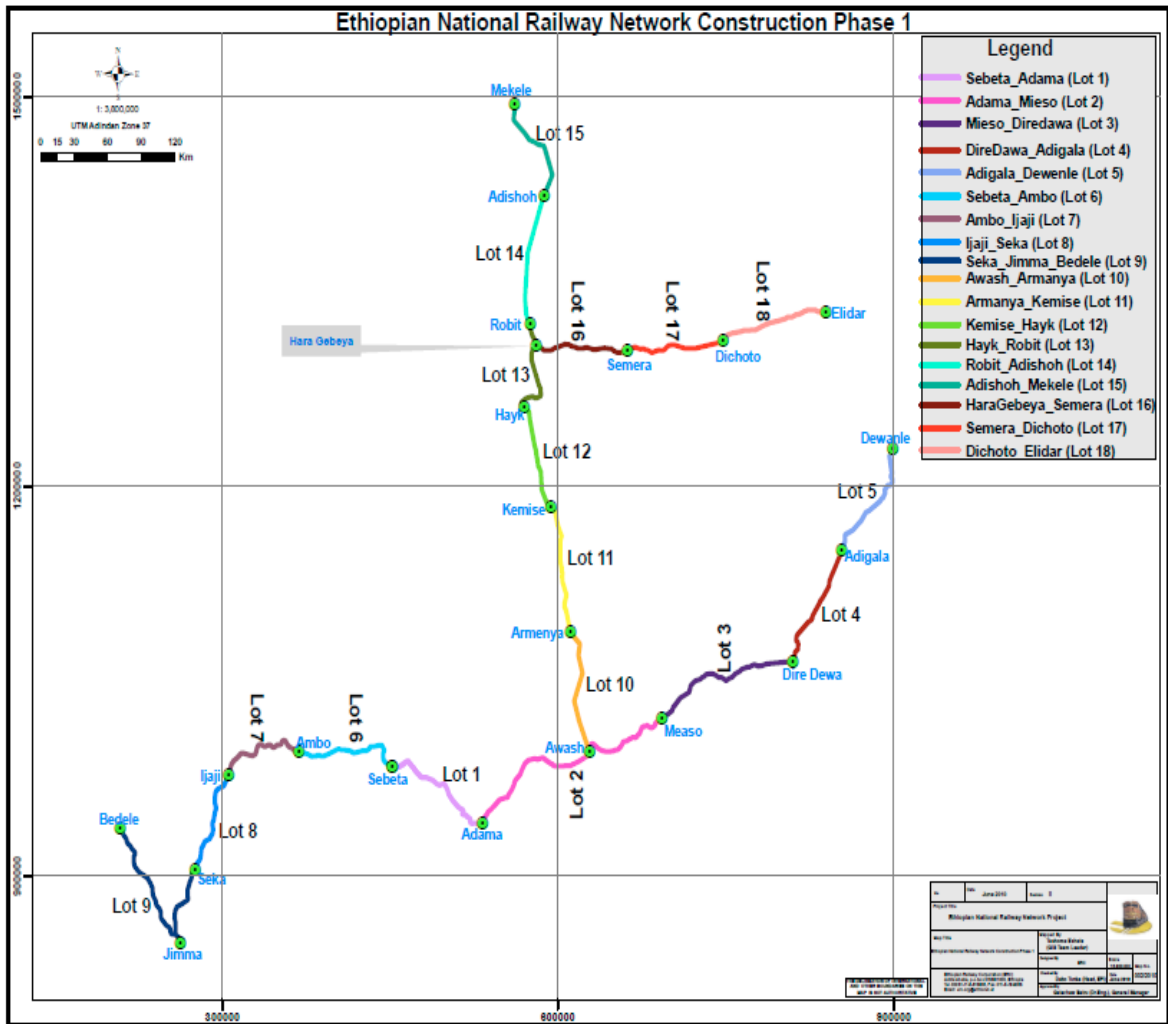


Fig. 1.3 Ethiopia National Railway Construction Phases 1

Table 1.2 The Ethiopian National railway Network plan (ENRN)

Route No.	Route name	Route Length (Km)
1	Addis Ababa-Modjo-awash-Dire Dawa-Dewanle	656
2	Modjo- Shashemene/Hawasa-Konso-Weyto(Including Konso-Moyale)	905
3	Addis-Ababa-Ijaji-Kombolcha-Gurafarda-Dima including Kombolcha-Hara Gebeya (direct to Boma with further extension to south Sudan)	637 115
4	Ejaji_Nekemt_Asoffa_Kurmuk	460
5	Awash_Kombolcha_Mekele_Shire	757
6	Fenoteselam_Bahirdar_Wereta_Weldia_semera_Elidar	796
7	Wereta_Azezo_Metema	244
8	Adama_Indeto_Gasera	248
Total		4815
Bird flying distances, subject to survey. Lines not in the order of priorities		

This scientific data may be helpful for the railway line construction community with similar engineering and economic problems especially, for those groups who are now trying to prepare the manuals for Ethiopian Railway Corporation.

1.5 Objectives of the Study

1.5.1 General Objectives

The general objective of the present study was to analyze engineering geological suitability of the railway alignment and to evolve possible remedial measures for the likely engineering geological problems along the general alignment.

1.5.2 Specific Objectives

Since the railway line is aligned on soft expansive sub-grade soil and rough topography, it will be a major objective to investigate and analyze this problem and to adopt different remedial measure for this problem.

- To characterize the general topography along the railway alignment by investigating the elevation and slope difference of the route.
- Engineering geological characterization of the in-situ sub-grade soils
- Engineering geological characterization of the embankment fills and blanket materials along the alignment.
- To evolve possible remedial measures for the likely engineering geological problems along the general alignment.
- To determine the safest design thickness of sub-surface structural formation in order to minimize the problem of formations.
- To suggest suitable recommendation based on the present study

1.6 Methodology and materials to be used

During the present study to characterize the sub-grade material AASHITO soil classification system has been adopted and in addition, USC soil classification system was also utilized to characterize the embankment and blanket material, for which various laboratory tests were considered. In order to characterize the materials, secondary data from previous works and primary data generated during the present study were utilized. For primary data generation field observation and laboratory analysis were undertaken following different standards and specifications. Fig.1.5 presents the conceptual work flow of the methodology followed during

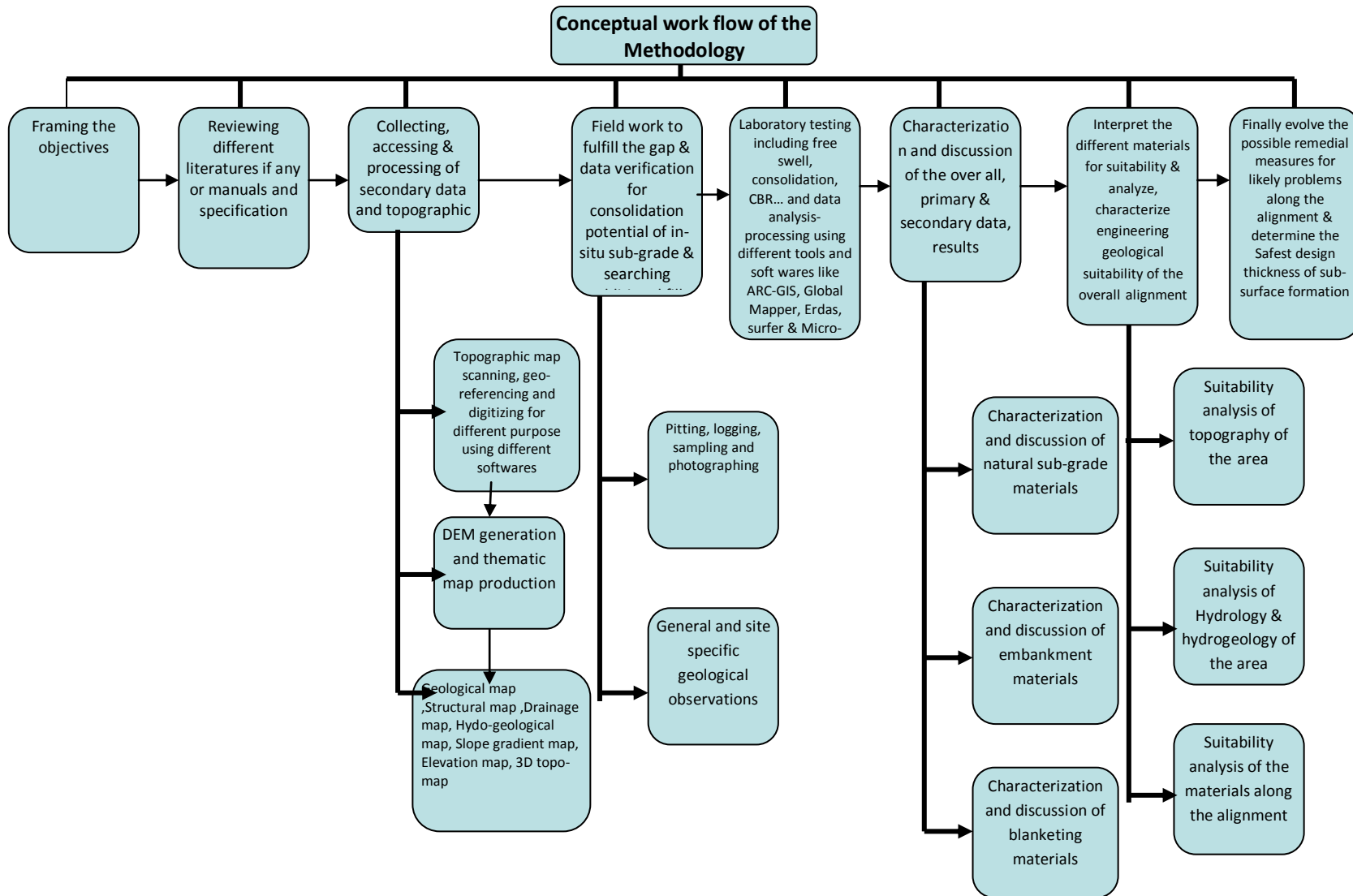


Fig. 1.5 Conceptual Work Flow of Methodology

the present study.

In general, in order to achieve the above objectives the following methodology was followed;

- Reviewing different literatures, manuals and specification of other countries related to the scope of the study.
- Literature review and previous work study to have the general idea of the regional and local geological and hydro geological information of the study area.
- Assessing and collecting information on the tectonics and structural setup of the present study area.
- Field works to overview the physiographic and geologic setup of the study area and to locate the position of sampling area using GPS and topographic and geological map.
- Investigation of test pits along the railway corridor and from logging to sampling of representative soils at each test points and taking photographs.
- Conducting Laboratory tests like; modified proctor compaction, consolidation test, California Bearing Ratio (CBR), swelling potential, Atterberg limits, grain size analysis, natural moisture content and dry density.
- Classification of the material based on the laboratory test results using AASHTO and USC systems.
- Characterizing the material by comparing and relating different parameters of engineering properties of soil using the AASHTO, USC and ASTM plasticity chart.
- Characterization of the material by integrating the results obtained from geotechnical field investigations and laboratory result analysis.
- Interpretation of the results of the tests and the classification in parallel to determining the index properties and swelling potential of the sub-grade soil along the corridor.
- Based on the literature review, engineering property, soil classification and dynamic characteristics, interpretations were made for the suitability of the construction material for embankment and blanketing.

- By using different design manuals attempts were made to provide the design considerations for minimizing the problems.
- Data processing, presenting and preparation of maps with the help of soft-wares; ArcGIS-9.3, ERDAS 9.2, AutoCAD, Global Mapper, Surfer and Micro Soft-Excel.

1.7. Limitation

During the present study there were limitations on availability of literatures and previous works as the country has no experience in railway construction. Also, railway line sub-surface as well as super structure characterization and construction has not been exercised so far in Ethiopia. Therefore, no sufficient references in local context were available. This has affected the scope and scheduling of the present study.

Despite these limitations, all efforts were made to perform the present research study in a logical and systematic manner. However, the limitations as mentioned above might have affected the quality of the results to certain degree. Therefore, it is strongly recommended that the results and the findings of the present study must be considered as indicative only. More elaborate systematic studies would be required before implementing the results from the present study.

1.8 Scheme of Presentation

The present study is presented in eight chapters and the scheme of presentation is as follows;

Chapter - 1 covers the introduction which provides the general over view and importance of railway to the country and railway track system in general. It covers the introduction with the justification of the problem, history of railway line construction in Ethiopia and its growth, objectives, methodology and material to be used, application of the study, limitation of the study and scheme of presentation.

Chapter - 2 presents the literature review that relates to the current research study and provides a detailed conceptual framework relevant to this research study.

Chapter - 3 deals with the study area and presents about location of the project area, characterization of the topography and general geomorphology of the area, drainage pattern and ground water, geologic setting of the area, Land use land cover, soil and climate of the study area and seismicity condition.

Chapter - 4 presents the engineering characterization of natural sub grade soil.

Chapter - 5 Covers Engineering Geological characterization of embankment fill soil for the selected railway alignment.

Chapter - 6 offer characterization of blanketing materials.

Chapter -7 presents the result and discussion on the suitability of the overall outcomes of the research regarding topographical, hydrology and hydrological aspects, natural ground condition plus embankment and blanketing materials. Besides, it encompasses economical recommendation for the ground improvement techniques of the natural sub-grade soil of the study area with designing sub-structural formation.

Chapter -8 includes Conclusion and Technical Recommendation.

CHAPTER II - LITERATURE REVIEW

2.1 General

The purpose of this chapter is to present literature review relevant to understand the general engineering geological suitability studies which have to be carried out along the rail alignment. For this, main emphasis was given to understand the engineering behavioral characteristic of expansive sub-grade soil. In addition, proper assessment of the material to be used for construction has been made. Also a thorough study was carried out to evolve the possible remedial measures to be adopted to improve engineering performance of the in-situ material and the construction material to be used along the proposed alignment. Besides, efforts were also made to know how to evaluate the quality of natural and prepared sub-grade material and to characterize it for intended engineering use. Further, literature related to geological and topographical requirements for rail alignment was also reviewed and is presented in this chapter. An attempt was also, made to understand, what methodologies were adopted in the previous similar studies and what the findings were. Here it is worth mentioning that in Ethiopian context none of such studies were carried out in past. However, all efforts were made to acquire understanding on related methodologies followed for similar studies conducted in other countries. Thus, based on all these above mentioned point general conceptual framework was developed and the methodology for the present study was evolved. A description on various relevant literature consulted during present study is summarized in the following paragraphs.

2.2 Review of Railway Track Structure

It would be meaningful to understand the railway track structure before discussing about other engineering geological suitability parameters. In order to familiarize to railway track structure and other related rail components a brief description is presented in the following paragraphs.

Railway track system in general consists of a super-structure and a sub-structure. The super-structure is composed of steel rails fastened to cross-ties. The rails are designed to support and guide flanged steel wheels through their prescribed position in space. The super-structure is placed on a sub-structure. The sub-structure is composed of a layered system of materials known as ballast, sub-ballast and sub-grade (Fig. 2.1) (Selig and Waters, 1994).

This part of the unit will provide a listing of design functions and description. For each of the main track components the functions are described by Selig and Waters (1994), Agarwal (1998) and Hay (1982).

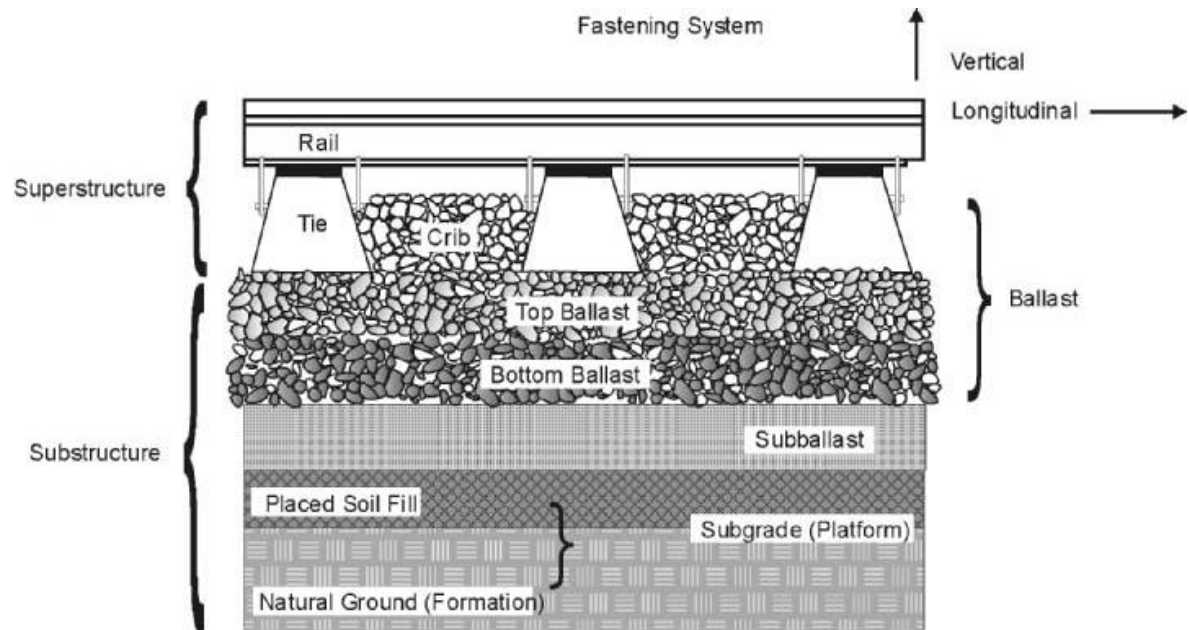


Fig. 2.1 Track structure components (Selig and Waters, 1994)

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 (Selig and Waters, 1994), 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.

2.2.2 Fastening Systems

The fastening system, or “fastenings,” includes every component that connects the rail to the sleeper. Fastenings clamp the rail gauge within acceptable tolerances and then absorb forces from the rails and transfer them to the sleepers. Vibration and impact from various sources e.g. traffics, natural hazards, etc. are also dampened and decelerated by fastenings. Fastenings sometimes act as electrical insulation between the rail and the sleepers.

2.2.3 Sleepers structure

Sleepers are the simplest and most secure method by which to project the required rail geometry for slab tracks. Sleepers may be hung together with rails to form the rail span or

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. This structure has different function such as; spreading the wheel load to the ballast, hold the rail to the gauge and from the inclination, and transmit the lateral and longitudinal dynamic forces of the train (Bonnett, 2005; Remennikov and Kaewunruen, 2005; Lim, 2004).

2.2.4 Ballast layer

Ballast or the track support layer keeps the rails and sleepers intact at the required position by resisting and dissipating the vertical, transverse and longitudinal forces transmitted by the sleepers. It distributes the loads to the layers below, protecting the sub-grade from high stress and attenuates the noise and shock in addition to providing immediate drainage. The ideal ballast must be of uniform quality, preferably angular in shape with hard corners usually with all dimensions nearly equal and clear and free from dust and other contaminants (Selig and Waters, 1994).

2.2.5 Sub-ballast layer

It is one of the track sub-structure components that are placed in between ballast and sub-grade. The importance of this layer is; to reduce the stress level from ballast to sub-grade, to prevent the interpenetration of ballast material into the sub-grade and to prevent emigration of fine sub-grade material into ballast layer which affects the drainage performance of ballast layer (Lim, 2004). Water falling on the track will enter the ballast and flow downward to the sub-ballast. Generally, the water will enter the sub-ballast, and this will be diverted to the sides of the track by the sub-grade surface unless the sub-grade is similar to the sub-ballast in its permeability. The sub-ballast also allows water flowing up from the sub-grade to discharge without eroding the sub-grade soil, which ballast cannot do because ballast is too coarse (Selig and Waters, 1994).

2.2.6 Sub-grade

The sub-grade 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). Anything, other than soils existing locally is generally 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.

2.3 Engineering Geological Suitability of Rail Alignment

From the engineering geological suitability point of view, the major parameter that has to be evaluated are suitability of natural sub-grade, topography, geology and drainage along the rail alignment. Besides, availability of suitable construction material for ballast and fill material within the economic distance also influence the general engineering geological suitability of the rail alignment. Thus, in the following section a description of relevant literature related to the above mentioned points is presented.

2.3.1 Expansive soils

Expansive soils are normally excluded as an engineering construction material because of their potential to change in volume with a change in the moisture or suction condition of the soil. Volume changes causes deformation and cracking in light weight structures like railways and highways. This is, in part due to the low tensile strength of these structures when compared with the materials at greater foundation depth used in larger structures (Gourley and Scheiner, 1993).

As Holland and Richards (1982) presented this change in volume is manifested in seasonal vertical and horizontal movements of the soil. The horizontal movement leads to fissures

opening in the drier month of the year and closing during the wetter months while vertical movement lead to cyclic changes in the soil surface level. This movement decrease with depth down to a level below which no seasonal moisture change takes place and hence no volume change occurs.

Holland and Richards (1982) also suggested that when designing or constructing light weight structure on expansive soils attempt must be made to reduce the effective heave or movement of the soil beneath the structure to a level that will not affect efficiency and safety of the structure during performance stage. A potentially expansive sub-grade soil will undergo a volume change as a direct result of a change in soil moisture content.

International Union of Railways Way and Works Committee (UIC-2005) reviewed the nature of different residual soils and gives valuable information on how their peculiarities influence their testing and use in engineering. Materials showing a high degree of plasticity ($LL > 50$ or $PI > 18$) and low values of dry density (less than $1.5 - 1.6 \text{ t/m}^3$) have to be considered as potentially expansive, and the design shall include results of tests to quantify this property. UIC (2005) also suggested that if the swelling soil is located under the platform, within the active zone, the need for soil stabilization has to be considered, along the following ways:

- Substitution of the swelling soils by inert soils.
- Stabilization treatments, as lime stabilization.
- Ensuring constant water content in the unstable soil by avoiding the access of water through the platform or the removal of water through evaporation. The water impoundment at the contact between imported and natural swelling soil must be prevented. This can be achieved by ensuring an adequate cross fall in this contact (at least 4%) allowing water to be evacuated.
- If the platform foundation occurs in swelling soils within the active zone, the effects of carrying the works in humid or in dry season must be considered.
- The infiltrations through the sub-ballast or introduced by leakage of the drainage system must not affect the platform foundation, and adequate constructive measures have to be affected.

2.3.2 Engineering Geological Investigation of Sub-grade

According to China's Railway Manual (2005), 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.

American Railway Engineering and Maintenance way of Association, Manual for railway engineering (AREMA, 2009) describes; that the composition and thickness of the material and the drainage condition of existing upper 2-feet (0.61m) of the roadbed area is extremely important because of the high stress from track road and exposure to environmental factors.

This manual 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). It's therefore necessary to improve the CBR value either by installation of a capping layer or stabilization of the natural ground or embankment fill. Before acceptance as 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.

Based on different manuals the sub-grade material shall comply with the requirements shown in Table 2.1.

Table 2.1 Requirements for sub-grade soil

Engineering properties	Value of the parameters	Manuals standards
PI	Max 12	AREMA, 2009
LL	Max 30	AREMA, 2009
Compacted density	95% of ASTM D698T	AREMA, 2009
Allowable bearing capacity	Minimum 20psi (138kpa)	AREMA, 2010
Swell	Max 2	UIC, 2005
Grain size	Not more than 40% fines	UIC, 2005
CBR	Greater than or equal to 4	UIC, 2005

2.4 Characterization of prepared sub-grade (Embankment fill materials)

Fills are used to raise the existing ground surface when required to achieve the desired level for railway construction. Fills can also be used to cover unstable soil, in which case material with a higher bearing capacity is placed on top of the obstacle in order to carry the weight of the roadway or railway and reduce differential settlement. They may serve to elevate the grade above existing or predicted water levels, swampy area or snow depths; to bury obstructions, undesirable topographic variations, and to achieve design grades (AREMA, 2009).

Every railroad fill should be designed to satisfy the following requirements:

- (i) To assure the stability of the embankment under its own weight and superimposed load.
- (ii) To assure stability of the combined embankment and foundation system.
- (iii) To economically tolerate the magnitude of anticipated settlement

Cuts are made when excavations are required through hills to provide railway grades and to acquire materials for use in constructing fill sections. Cuts in cohesive soils need to be designed with caution. Considerations such as the proposed slope angle, drainage conditions, and moisture conditions and strength of the soils encountered in a cut are the most significant factors that influence the stability of earth slopes. Slope stability evaluations should generally be made to select the cross-section for cuts over 15 feet deep. In general, higher the cut section flatter the slope thus, slope attains more stable configuration. Cut slopes are rarely created greater than a slope of two to one (horizontal to vertical dimensions). Cut sections of roadway or rail are characterized by the railway being lower in elevation than the surrounding terrain (AREMA, 2009).

Based on the AREMA (2009) construction standards and consistent with the railway standards of major railways, typical grade construction over competent sub-grade should consider the following:

- The roadbed embankment should have side slopes of 2H: 1V or flatter. This will increase the stability of the sub-grade and reduce loss of granular material along the shoulder.
- Cut slopes in granular soil or stiff clay will generally support slopes as steep as 2H: 1V. Where seepage is noted or in areas with soft to firm clays or loose sand the slopes should be benched or flattened as required to maintain stability.

International Union of Railways Way and Works Committee (UIC-2005) also give standard value for each structure.

Embankments: for soils which are acceptable, a slope of 1:1.5 or 1:2 is normally used but 1:1 or 1:1.25 is also possible. In the case of coarse rock fill, also berms or toe walls, etc., may be required.

Cuttings: slope angles vary according to the type of ground:-

- Intact rock not susceptible to weathering and without unfavorable dip or cleavage: a slope of between 45° and 90° but with a stone trap at the base and benches having a width of about 1/3 of the height of each step produced;
- Granular soils: slope of 1: 1.5;
- Cohesive soils: slopes typically in the range 1:1.5 to 1:2 according to the height of the cutting, or even flatter, with benches if required.

Note - A soil embankment has three basic components: a center core, shoulders or shell, and a drainage system. In homogeneous embankments same materials for the core and shoulders are used. Site conditions may not require special drainage provisions other than side ditches. When the fill functions as an earth dam or levee, the core is constructed of relatively impermeable material while the shell will generally be constructed of a material that is more permeable than the core. However, in the present study only homogeneous embankment fills were considered.

Table 2.2 Requirements for fill materials

Engineering properties	Value of the parameters	Manuals standards
Particles finer than 75 micron	Not more than 50/40%	IS-2720/UIC 2005
LL	Less than 35/55-60	IS-2720/ERA 2002
PI	Less than 15/20-25	IS-2720/ERA 2002
Cu (uniformity of coefficient)	Above 7	IS-2720
Embankment fill compaction density	Not less than 95% at MDD of AASHTO T 180 D	IS-2720/AREMA 2009
MDD	Greater than 1.8gm/cc	IS-2720
CBR	Greater than 4/5	UIC-2005/ERA 2002

Note - Specification on construction materials is set by referring to AREMA (2009), Indian standard manual (IS: 2720, 1987), ERA Standard Technical Specification (ERA, 2002), and AASHTO Standard Specifications (AASHTO, 2006, Part IA).

Recommended side and back slopes

In an attempt to ensure the stability of slopes in the project area, the recommendation as specified in Table 2.3 and 2.4 may be used.

Table 2.3 Recommended side and back slopes (ERA, 2002)

Material type	Height of Slope	Side Slope		Back Slope	Zone Description
		Cut	Fill		
Earth or Soil	0.0 – 1.0m	1:3	1:3	1:3	Recoverable
	1.0 – 2.0m	1:3	1:2	1:2	Non - Recoverable
	Over 2.0m	1:1.5	1:1.5	1:1.5	Critical
Rock	0 – 2m	1:2	1:2	2:1	Critical
	Over 2m		1:1.5	4:1	
Black Cotton Soil	0.0 – 2.0m	-	1:6	-	Recoverable
	Over 2.0m		1:4		

Table 2.4 Recommended safe slope of embankment (AREMA, 2009)

Embankment material	Height of slope (ft)	Slope (H:V)
Sandy(cohesion less)+Cohesive materials		2:1 or flatter
Hard rock	Up to 50	1.5:1 or flatter
	>50	2:1 or flatter
Impervious soft rock	Up to 30	2:1 or flatter

2.4.1 Foundations of Fills

The foundation soil should have the strength to support the proposed embankment and live loads with an adequate safety factor.

It is occasionally necessary to remove and replace portions of weak or highly compressible foundation elements or to improve their characteristics by using stabilization procedures or controlled construction techniques (AREMA, 2009).

- AREMA MRE (2010, Volume 1, Part 2, Section 2.11.2.1 *Sub-grade Soils, b*, Page 1.2.20) recommends allowable bearing capacity of 20Psi (138kpa) for economical design in the absence of testing on sub-grade soils. Therefore, the sub-grade soils that have bearing values less than the recommended value should be treated with special considerations as mentioned above.

2.5 Characterization of blanketing material

Blanket is a layer of coarse grained material between ballast and sub-grade, spread over entire width. Blanket may be required over the formation where the soil is of poor quality, rainfall is heavy and traffic density is high, as the absence of blanket in such cases can lead to problems in service such as swelling or heaving of formation. The role of blanket layer includes improving the bearing capacity of sub-grade, reduction of induced stress on top of the sub-grade, prevent mud pumping and fouling of ballast, prevent damages of sub-grade by ballast, draining away of water from sub-grade and protection of sub-grade against erosion and climatic variation (RDSO, GE-35, 2000).

General recommended specifications of blanket material as suggested by RDSO, GE-0014, (2009) are given below;

- It should be coarse, granular and well graded.
- Skip graded material is not permitted.
- Uniformity Coefficient $C_u = D_{60}/D_{10} > 4$ (preferably > 7)
- Coefficient of Curvature $CC = (D_{30})^2 / D_{60} \times D_{10}$ should be 1 - 3.
- Fines (passing 75 microns): 3% to 10%.
- Los Angeles Abrasion value $< 35\%$.
- Minimum required Soaked CBR value 25 of the blanket material compacted at 100% of MDD
- The material for upper blanket layer, required for heavier axle load, shall be well- graded sandy gravel or crushed rock
- Filter Criteria should be satisfied with prepared sub-grade/sub-grade layer just below blanket layer, as given below:
 - Criteria–1: D_{15} (blanket) $< 5 \times D_{85}$ (sub-grade)
 - Criteria–2: D_{15} (blanket) > 4 to $5 D_{15}$ (sub-grade)
 - Criteria–3: D_{50} (blanket) $< 25 \times D_{50}$ (sub-grade)

- Filter Criteria is optional. This can be adopted with the experience gained by its compliance for different types of soils with blanket.

Note:- In exceptional cases on technical and economic considerations, LAA value may be relaxed up to 40%.

Table 2.5 Grading Percentage of Blanket Material

S. No	IS Sieve size (IS-2720)	Percent passing (by weight)
1	40 mm	100
2	20 mm	80-100
3	10 mm	63-85
4	4.75 mm	42-68
5	2.00 mm	27-52
6	600 micron	13-35
7	425 micron	10-32
8	212 micron	6-22
9	75 micron	3-10

2.6 Terrain Evaluation

The route upon which a train travels and the track is constructed is defined as an alignment. An alignment is defined in two fashions. First, the horizontal alignment defines physically where the route or track goes (mathematically the XY plane). The second component is a vertical alignment, which defines the elevation, rise and fall (the Z component) (AREMA, 2009).

In the present study the objective of the topographic analysis was to identify and compare the extent to which the proposed rail route alignment would require modifications. The topographic analysis is an important component for the general engineering suitability of the rail alignment.

2.6.1 Topographic analysis

The objective of the topographic analysis is to identify and compare the extent to which the proposed rail route alternatives would require modifications to the current topography to meet project design objectives.

2.6.1.1 Terrain classification

The geometric design elements of a railroad also depend on the transverse terrain through which the railroad passes. According to AREMA (2010), transverse terrain properties are categorized into three classes, as follows:

Flat terrain offers few obstacles to the construction of a railway, having continuously unrestricted horizontal and vertical alignment with terrain slope up to 5%.

Rolling terrain where the slopes generally rise and fall moderately and where occasional steep slopes are encountered, resulting in some restrictions in alignment, terrain slope from 5 % to 10%.

Hilly terrain: Rugged, hilly and mountainous terrain where, transverse terrain slope is 10 % to 30 %.

Having said that, the ruling grade, a rise of 1 foot vertically for every 100 feet traversed horizontally, in design line should also be determined by railway class, terrain conditions, type of traction and transport demands. According to AREMA (2009), the following are the general guidelines regarding gradients-

- ✓ 0% to 1.0% - generally 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.

However, Ethiopia Railway Corporation (ERC) is lately implementing the gradient depending on the topography of the area. 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 heading and /or banking depending on train load (ERC, 2011).

2.7 Drainage Condition Requirement

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 factor in railway design and maintenance. This section deals with the surface and subsurface drainage of the roadway.

Surface Drainage

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 on 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 sec 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 sec (AREMA, 2009).

In general, according to UIC (2005) any rainwater falling on the surface and likely to penetrate to the sub-grade must be quickly evacuated. This requires:

- (i) Correct cross fall of the sub-grade towards the longitudinal drainage system (cuttings) or towards the shoulder (embankment) in both straight and curved track. The cross fall should be in the range 3 % to 5 %.
- (ii) Correct dimensioning of the components of the line side drainage system. These must be capable of accommodating the run-off produced during the major design storm corresponding to the return period (at least ten years).

Subsurface Drainage

Only a portion of rainwater is handled by natural and man-made watercourses. The remaining water infiltrates 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 sub-grade soils, sloughing, or instability of slopes will not occur. 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).

According to UIC (2005) depending on local conditions, the Railways specify that the ground water table should be lowered to a minimum distance between 0.8m and 2.5m below the rail

top. The minimum value of 0.8 m is only permitted if the sub-grade under the ballast is good, frost does not penetrate into the ground and ground water flow is low.

2.8 Availability of Construction Material

The availability of suitable and potential site for construction material can determine the economic performance of the project (Zaruba and Mencl, 1976).

Suitable and potential construction material site must have suitable soil/or rock type and geologic structure, minimal overburden thickness, easy to access and it must have minimal environmental impact. The soil types can influence the quality of embankment fill materials, the rock type and its structure can determine the product of quarry material, while the overburden thickness may influence the profitability or economy of the site. However, due to the different factors it is impossible to establish the universally valid maximum admissible overburden thickness (Zaruba and Mencl, 1976). In general, the site for construction material should be close to the area of construction as much as possible.

2.9 Methodology Evolved for the present study

As stated above in this chapter various literatures pertaining to concepts, specification and general requirements for engineering suitability of rail alignment were reviewed during the present study. Thus, based on these literatures following methodology was evolved to meet out the objectives set during the present study.

The present study adopts different methodology in characterizing the different materials for the proposed alignment. Characterization of natural and prepared sub-grade both by its index and engineering properties were undertaken. Further, for the determination of their suitability sensitive engineering properties were determined on the representative soil samples by conducting laboratory tests. These include test for CBR, Atterberg limit, consolidation and unconfined compression strength. Later, for evaluation of the laboratory result different technical specification and manuals were used. In characterizing blanketing materials similar principles were followed however much emphasis for percentage passing for different sieve size of filter as per standards and shape of grains to uniformity were given in addition. In the similar fashion, the topography, hydrology and hydro-geology of the project area were also assessed and characterized using literature review and previous work done in the area. Further, evaluation was made according to the different manuals and specification. Finally, it was also attempted to give solutions for the likely problems along the alignment. This was

mainly done based on the recommendation and remedial measures suggested in different manuals and specification considering the site and our own country condition. In addition to this designing of sub-structural formation was also attempted. This was mainly suggested based on the different manuals and guideline that reviews the design methodologies adopted in various World Railway systems.

CHAPTER III - OVERVIEW OF THE STUDY AREA

3.1 Preamble

The present study area is a part of the Addis Ababa- Ambo-Ijaji-Kombolcha-Guraferda-Dima including Kombolcha-Hara Gebeya (direct to Boma with further extension to south Sudan) railway section that connects Addis Ababa with the Western part of Ethiopia.

The section which forms the study area connects the two cities namely; Sebeta and Ambo. The railway line stretches a total length of 108 km and is located in Shewa high land in Western highlands of Ethiopia. The project begins at 26 km from Addis Ababa and ends near the town of Guder. As a part of the study area 20km buffer zone along the proposed Sebeta-Ambo railway 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 presents a general overview of the study area and specifically deals with the general geologic, hydro-geologic and physiographic setup of the area in which the project railway alignment is planned to be constructed. It also includes the general climatic conditions and the soil type distribution of the study area.

3.2 Location of the study area

The intended study area is located in the central part of Ethiopia within the Oromia regional state starting from Sebet town, which is located 25km from Addis Ababa, to Ambo town, however entire project area extends up to Sudan boarder. Addis-Ababa-Sebeta-Ambo railway line project runs towards the western direction and continues along the towns of Ijaja, Nekemete, Nejo, Assosa and Kurmuk, Sudan boarder.

This new railway network would help a timely stream of goods across the nation and will help traders in saving their time, energy and transportation cost. The proposed case study railway corridor section has an approximate length of 108 km and is bounded by UTM coordinate of 451435.7m to 364520.3m E and 984085.7 to 993860.7m N (Fig.3.1).

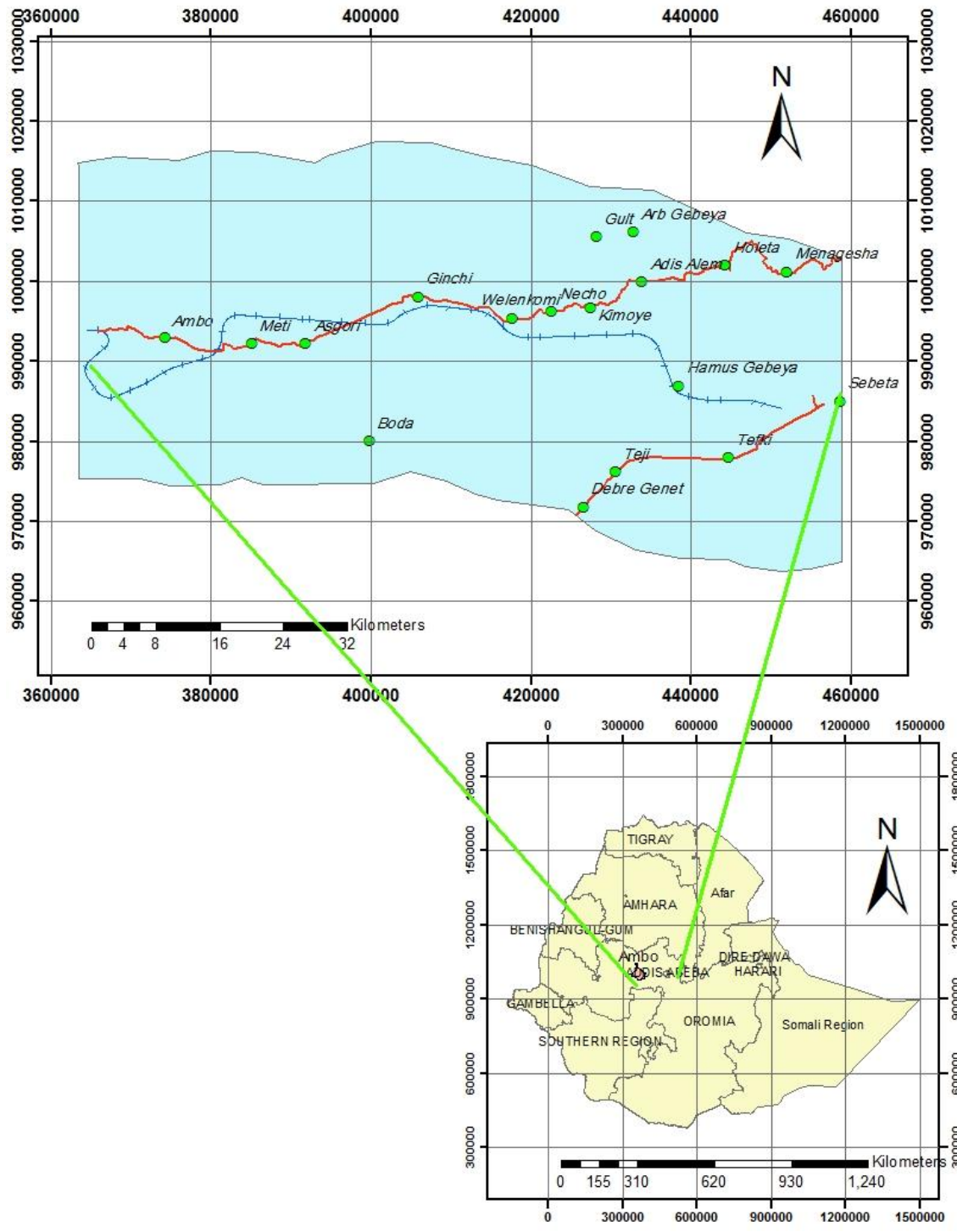


Fig.3.1 Location map of the study area

3.3 Physiographic features

3.3.1 Topographic classification

The objective of the topographic classification and analysis was to identify and compare the extent to which the proposed rail line alternatives would require modifications to the current topography in order to meet project design objectives. Grade changes are typically kept to a

minimum to maximize fuel efficiency and lessen long-term maintenance costs. Design speed of a railroad purely depends on the topography and the resultant design of the railroad (USACE, 2004). The proposed railway alignment is classified by slope percentage of the area through which the proposed railroad passes. The alignment of Railway system will therefore have to traverse two distinct types of topography, flat and rolling terrain with some hilly section. This implies that steeper terrain requires a greater amount of either fill, or cut during rail line construction than flatter terrain, and would therefore have a greater impact on topography. Normally, steeper the terrain is, the greater will be the impact. The topographic analysis of the study area consists of 20km right-of-way (ROW) corridor of the individual proposed rail line segments and segment combinations.

A perusal of Fig. 3.2 clearly indicates that much of the railway line (about 64km of the segment) would be relatively flat, mostly having approximately 90 to 95 % of their total lengths on slopes 0 to 5 %. Moreover, about 44 km of the railroad will be on rolling terrain with approximately 80 to 90 % of its length on slopes 5 to 10 % and the rest of the railroad on hilly terrain.

The current study area is located in the western Shewa zone which is part of western plateau in the region. The total elevation range of the study area is varying from 1566m-3379m on an average about 2366.87m (Fig.3.3). The minimum and maximum elevation of the study area shows that the study area has a varied topographic condition represented by flat land and mountainous areas. The area generally comprises of;

- Plateau zone (Part of the Western Plateau)
- Escarpment zone (Rift escarpment)

The plateau zone is the northern fringes of the vast western Ethiopian plateau, which extends westwards and includes the greater parts of the Oromiya region west and south west Showa zones. The escarpment zone area is dissected by deep eroded gorges of tributaries of Awash and Abay Rivers however, in the east the topography is smoother, rolling or flat (Fig.3.4). This zone which is topographically the watershed between the Awash and the Abay Rivers, falls in a series of tilted blocks most of which dip in different direction. Due to these varied physiographic conditions, cut and fill was planed at different location of the proposed track corridor.

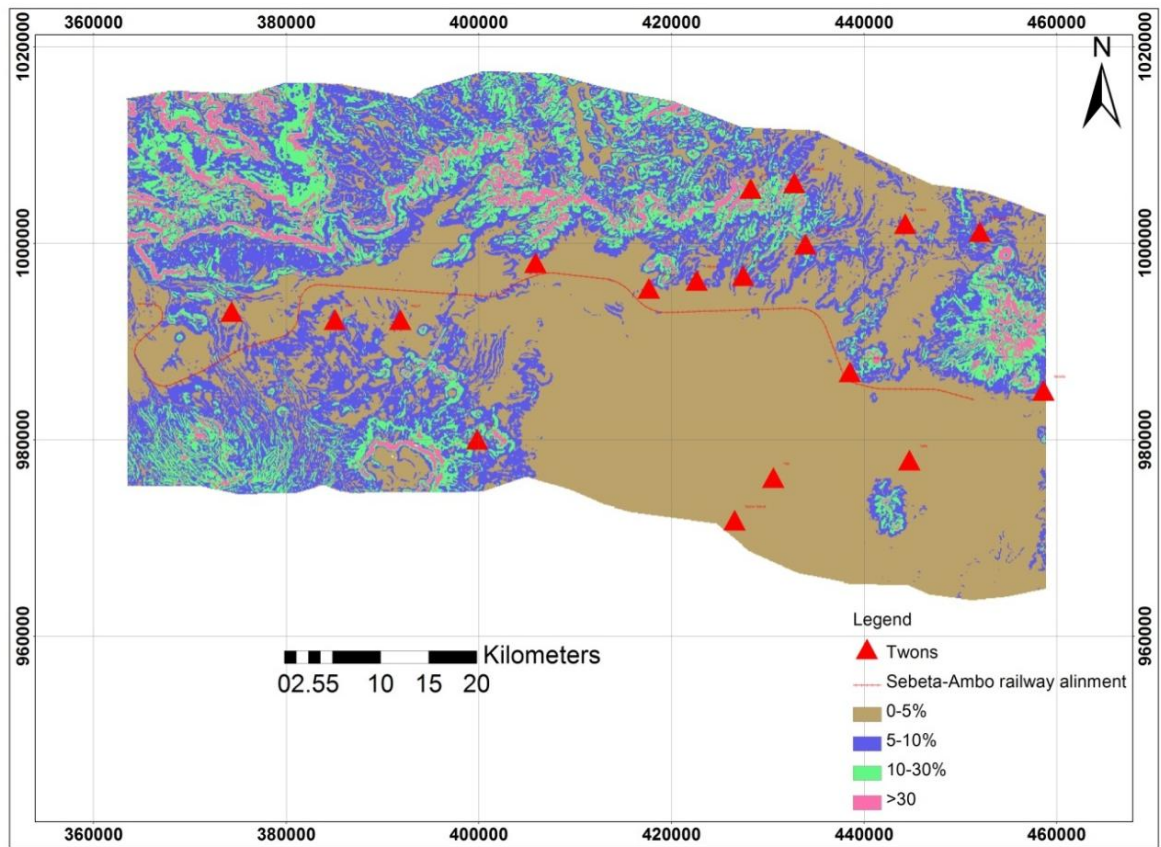


Fig.3.2 Slope classification of the area

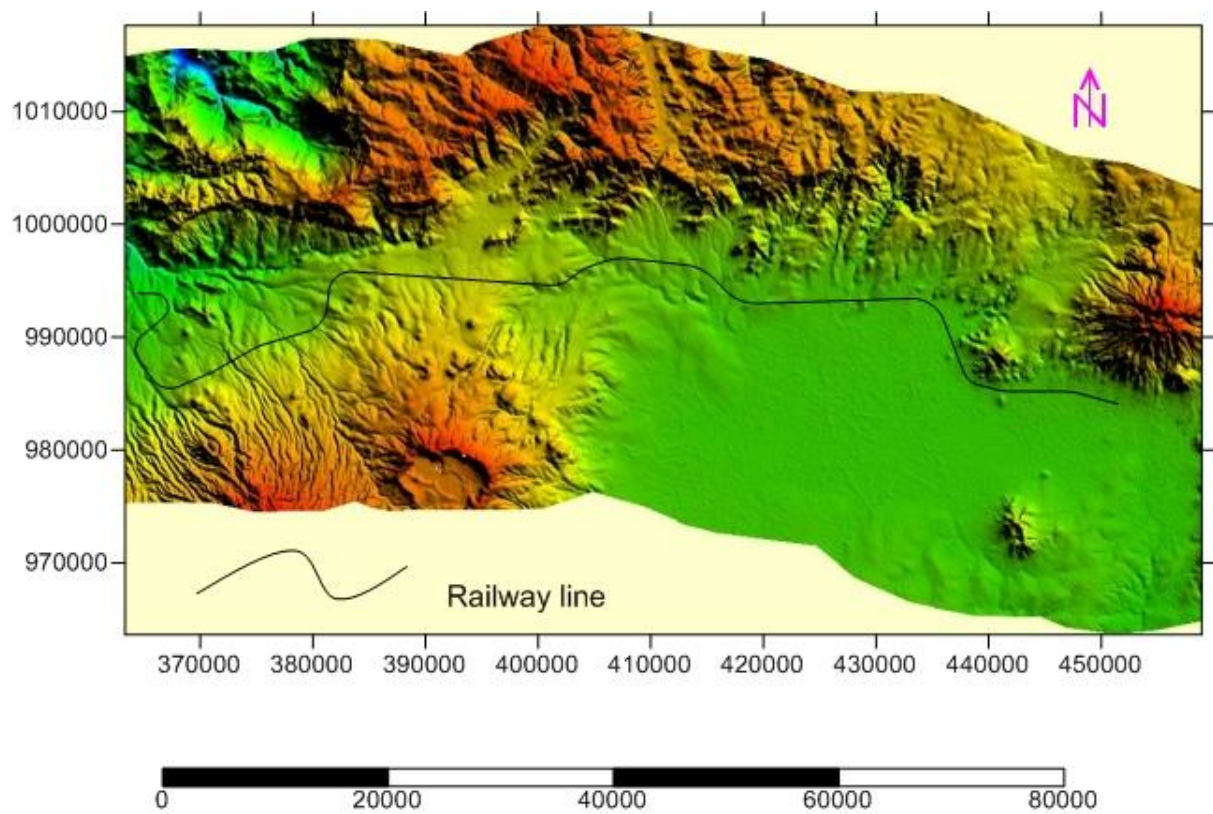


Fig.3.3 3D topographic view of the study area and railway alignment

3.4 Geology

3.4.1 Regional Geology Unit

Grave effort had been made to gather and analyze all pertinent previous geological works pertaining to the study area and its surroundings. The major geological formations and tectonic features of the study area are adopted from Adaa-Becho Groundwater Resource Evaluation Project report (WWDSE, 2008) and verification of data for geologic formation is also conducted during the present field work. The geo-structural findings are summarized in the following paragraphs;

Ethiopia can be divided in to four major phsiographic regions, widely known as the (i) Western plateau, (ii) Southeastern plateau, (iii) Main Ethiopian Rift (MER) and (iv) Afar Depression (AD). The Ethiopian plateau is underlain at depth by Precambrian rocks of the Afro-Arabian Shield. The Precambrian basement is covered for the most part by glacial and marine sediments of Permian to Paleocene period and Tertiary volcanic rocks with related sediments. The Precambrian basement exposures are found in the north (Tigray), in the west along the Sudan border (Gojam, Wollega, Illubabor, and Kefa), in the south (Sidamo and Bale) and in the east (Harerghe). The Precambrian rocks of Ethiopia are consisting of high-grade gneiss, metamorphosed volcano sedimentary rocks and associated ultramafic bodies and intrusives ranging from mafic to granitic composition. The outline of the geological history of the Afro-Arabian countries – Ethiopia is believed to be in the group – was sketched by Pierre Gouin (1979) as follows:

At the end of the Precambrian era, the crystalline basement complex of the present Afro-Arabian swell had been above sea level for a long time and remained for another 370 million years until the end of Paleozoic era. Such a long period of erosion and denudation left the earth's surface almost completely peneplained.

Crustal motion started in the beginning of Mesozoic era. During the late Triassic and early Jurassic periods, a regional epi-orogenic sinking of the crust commenced causing a progressive transgression of the ocean from the south east that is, from the Indian Ocean coast of present day Somalia in the general direction of Lake Tana in the North West Ethiopia. This downward crystal movement, concomitant with a sedimentation process, started a cycle of marine transgression and recession of Mesozoic sea. Within this large epi-continental sea, extensive layers of sediments were deposited to form hundreds of meters of

rocks consisting of sandstone, shale, gypsum, limestone and other varieties of sedimentary rocks.

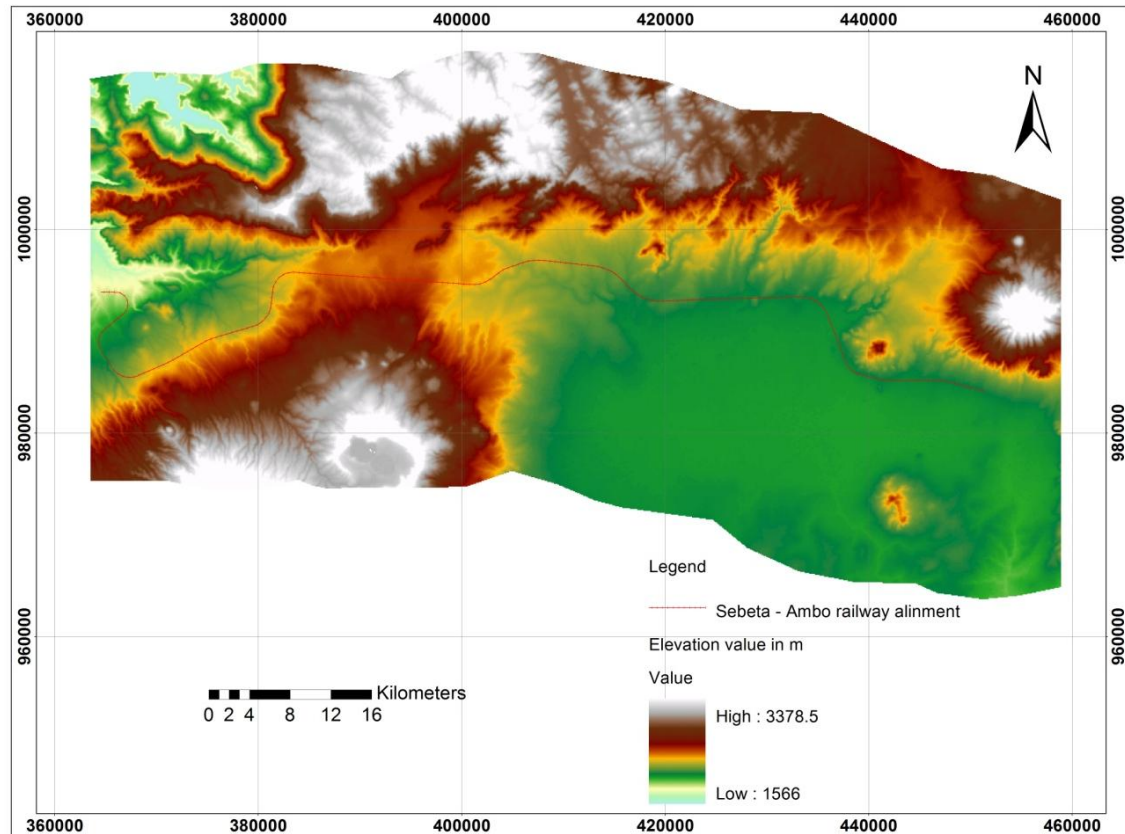


Fig.3.4 Elevation map of the study area

The crustal movement was reversed into the upward motion during the late Jurassic period, which brought the crust's surface up to sea level by marine regression in late Cretaceous period. The regional uplift resulted in wide spread crustal fracturing during the early Tertiary period. The crystalline and sedimentary rock layers were fissured mostly along or in the vicinity of the zone of maximum uplift, thus allowing outpouring of molten lava to cover the older rock layers.

Major fault displacement along the Rift Valley was initiated during middle Tertiary period. Subsidence of large crystal blocks along steeply inclined fault zones created grabben type depressions along the rift valley and at Lake Tana. Significant volcanic activity was associated with the formation of grabben and young volcanic rocks cover the old Tertiary volcanics in many depressions. Present day tectonic activity occurs along the Rift Valley as evidenced by numerous earthquakes. More recent volcanism, associated with tectonic activity, had been concentrated within this structure along the edge of the adjoining plateau. Accordingly, the geological setting of the present study area and its vicinity is represented as;

The Mesozoic era transgression and regression event depositions of sediments, like Adigrat sandstone, which rests un-conformably on the crystalline basement rock, Abay beds composed of gypsum and shale units, Antalo Limestone which conformably overly the Abay beds and Amba Aradam sandstone;

Tertiary and Quaternary volcanic units includes: - the Blue Nile basalts, Amba Aiba basalt, Alaji rhyolites, Tarmaber basalt, Rift volcanic. The Tectonic events of the rift system gave rise to the volcanic phenomenon of the area (mainly the eruption of basaltic lava flows and acidic fall deposits), the fault escarpments and the different physiographic units (the plateau, the escarpment and the depression area). Lacustrine and alluvial deposits and basaltic lava flows characterize the depression area.

3.4.2 Local Geology

The geological map (Fig.3.5) prepared covers a buffer zone of 20km from the railway corridor between Sebeta and Ambo towns. This geological map was instrumental for suitable potential construction material site selection and was also used to evolve general suitability of railway alignment.

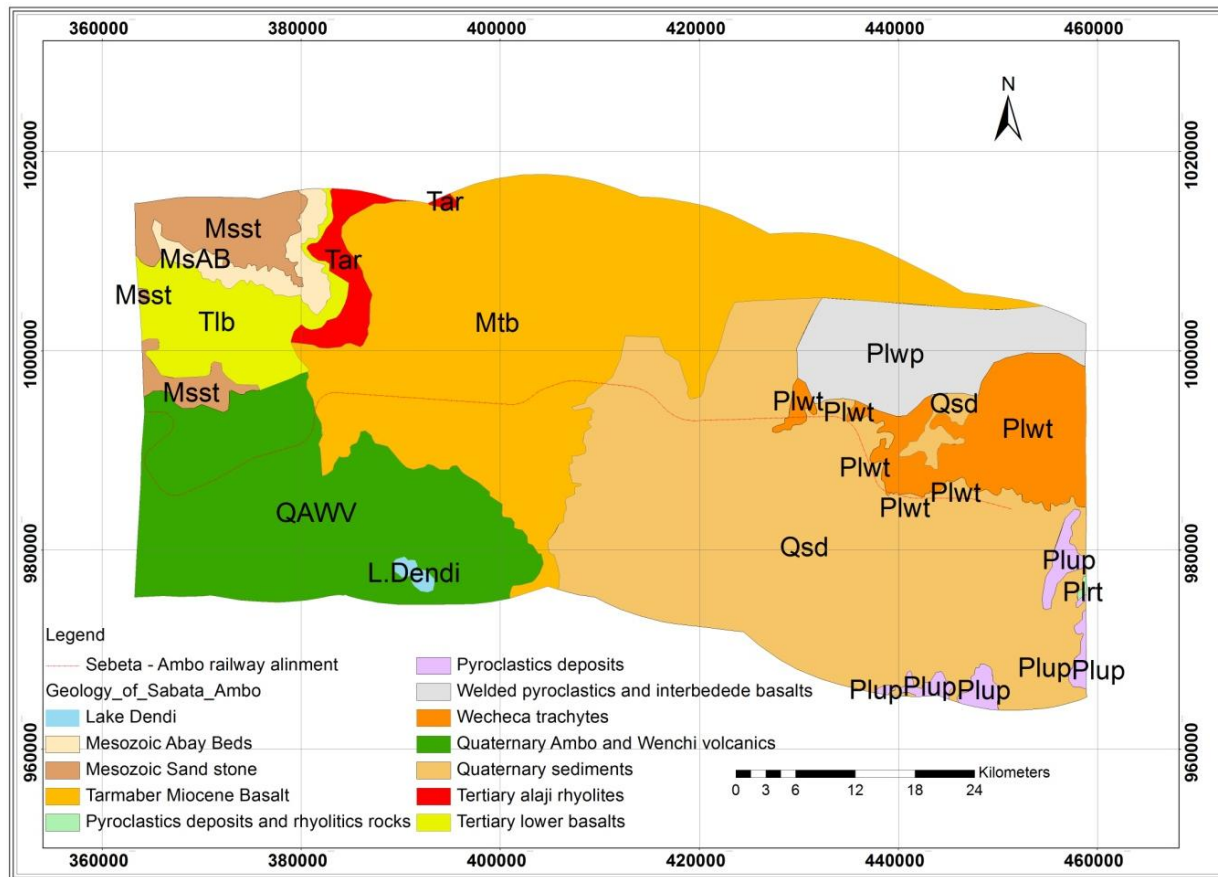
Table 3.1 The major geological formation crossed by the proposed Sebeta-Ambo railway corridor

No	Corridor length (km)	The major geological formation	Total length
1	0+000-26+000 & 27+800-46+800	Quaternary sediment that overlay the quaternary basalt	45km
2	26+000-27+800	Wechecha trachyte mixed with ignimbrite rock and welded pyroclastic deposit materials	1.8km
3	46+800-77+600	Highly weathered and fragmented Tarmaber Miocene basalt	30.8km
4	77+600-(end)	Quaternary Ambo-wonchi volcanic containing basaltic lava flow, scoria cone and pyroclastic deposit	29.4km

Tertiary Volcanic Rocks

- **Scoria & Scoraceous basalt (Tarma Ber Basalt)** are the dominant units exposed in the eastern and central parts of the study area and in watershed divide area of the Awash and Abay river basins. It is highly weathered, fractured, and pinkish to grayish in color and Miocene in age (Tesfaya Kidane, 2010).
- **Central Volcanic Unit - Wonchi volcanic** are mainly trachytic lavas exposed at Wonchi and Gurage mountains. South western part of the study area forming an elevated ridges or mountain picks. It is grayish color, fine to medium grained trachyte with subordinate ash

falls and ignimbrite. Wachecha trachytic lava flows are also identified in the eastern parts of the study area. It covers the south of Menagesha, surrounding Sebeta town.



(Map source: 1:250,000 map of the area compiled by Geological Survey of Ethiopia)

Fig 3.5 Local Geology of the proposed railway corridor

Quaternary sediments deposit

- **Quaternary Ambo and Wenchi volcanics** - This lithologic unit contains basaltic lava flows, scoria cones and pyroclastic deposit within it. It covers area around Ambo town and Eastern parts of the Guder town. These quaternary volcanic rocks resulted from the Wenchi and Dendi central volcanoes. In the surrounding of Ambo Area the young olivine basalt are also exposed. The age of this young basalt is determined as less than 100,000 years old (Tsagahe Abebe et al.1998).
- **Alluvial Cover** - mainly outcropped above the Tertiary volcanic on the plateaus and eastern parts of the study area, Becho plain and consisting of regolith, black to reddish brown clay soils, talus and alluvium.

3.5 Structure and seismicity of the study area

Geological faults and earthquakes are mostly interrelated by nature; therefore a breaking up of plates may be caused by earth dynamism or displacement of detached fractured plates which causes earthquake. Due to the location of the country, Ethiopia, right on one of the major tectonic plates, i.e., the African and Arabian plates, earthquakes have been a fact of life for a very long period of time (Pierre Gouin ,1979).

In the 20th century alone, a study done by Pierre Gouin (1979) suggests that as many as 15,000 tremors, strong enough to be felt by humans, had occurred in Ethiopia proper and the Horn of Africa. A similar study by Fekadu Kebede (1996) indicated that there were a total of 16 recorded earthquakes of magnitude 6.5 and higher in some of the Ethiopia's seismic active areas in the 20th century alone.

The most significant earthquakes of the 20th century were 1906 Langanu earthquake, the 1961 Kara Kore earthquake, the 1983 Wondo Genet earthquake, the 1985 Langanu earthquake, the 1989 Dobi graben earthquake in central Afar, and the Nazret earthquake in 1993 were all felt in Addis Ababa, and the other major cities of Nazret and Awassa (Samuel Kinde, 2002).

Some of the Ethiopia's major cities like Addis Ababa, Nazret, Dire Dawa and Awassa are very near to main fault lines such as the Wonji fault, the Nazret fault, the Addis-Ambo-Ghedo fault, and the Filwoha fault lines along which numerous earthquakes of varying magnitude have occurred over the past years (Fekadu Kebede, 1996). According to Tsagahe Abebe et al. (1998) and Tesfaye Kidane (2010) the area is mainly affected by several Ambo lineaments that have different orientation (Fig.3.7). However, the Ambo fault that has E-W strike orientation was also observed in the study area and out of the buffer zone of the study area. The faults are kilometers away from the rail way corridor and more or less parallel to the railway lines. This might not cause a significant impact on the railroad. However, seismic risk map of Ethiopia (Fig. 3.8) 100 year return period, 0.99 probabilities, (Lakemariam Asfaw, 1986) shows that the proposed railway alignment (present study area) will be crossing through seismic zones of MM scale 7 and 8. Thus, the present study area falls in seismic active zone.

The Geologic structures and potential seismic active zone forms an important design parameter particularly for rigid engineering structures. However, for the present study these parameters were not covered as these falls beyond the scope of the present study.

3.5 Soil Extension of the Study area

According to Berhanu Debele (1985), the Expansive soils in Ethiopia covers about 12.6 million ha, or 10.3% of the total area. A perusal of Fig.3.10, adopted from Srivastava et.al. (1984) clearly show the distribution of vertisols in Ethiopia. Although these soils are suitable for agriculture but they are unsuitable to be used for civil structure due to their expansive behavior as characterized in the following chapter.

According to Food Aid Organization (FAO, 1984) soil classification, railroad crosses three major pedological soil types namely; Eutric Vertisols (clay, sub angular/ angular blocky, hard firm, very sticky and very plastic), Chromic Luvisols (clay/silty clay, sub angular, friable to firm, sticky and slightly plastic), and Humic Nitosols. The perusal of Plate 4.1 (Chapter 4) indicates that the area is generally one of the largest places in Ethiopia which are dominantly covered by soil with expansive nature. This type of soil is expected to be originated from the highly dominant parent basaltic rocks of the area (Geological Survey of Ethiopia and Aquatest, 2004). The soil map as presented in Fig. 3.9 shows the relation between the railroad and the soil extension.

Sub-grade soil extension evaluation helps narrow soil stretches from being overlooked especially in the presence of problematic soils. The dominant sub-grade materials in the proposed railway line include black cotton soils and silty clay soils with some sands.

3.7 Drainage pattern and hydrogeology of the study area

Surface water condition in the study area includes streams and lake. Several perennial and intermittent streams emerging from ridges are located in the north, south-east and western sides just parallel to sub-parallel with the regional structure, Ambo fault belt and lineaments. Most of the streams in the study area have radial pattern while some streams from northern ridges flow parallel to sub-parallel.

Even if Awash and Abay are the dominant basins in the study area, the south-west ridge is a kind of triple junction that shed to Omo, Abay and Awash basin. It is therefore clearly seen that geologic structure are the main drainage controlling physical features (Fig.3.11). Some of the major perennial streams include Huluka, Bajis and Guder rivers from western part of the study area around Ambo town and Kella, Jemjem (Plate 3.1), Jeliwan and Awash rivers from

central part of the study area around Ginchi and Welenkomi are characterized by high drainage pattern and/or density.

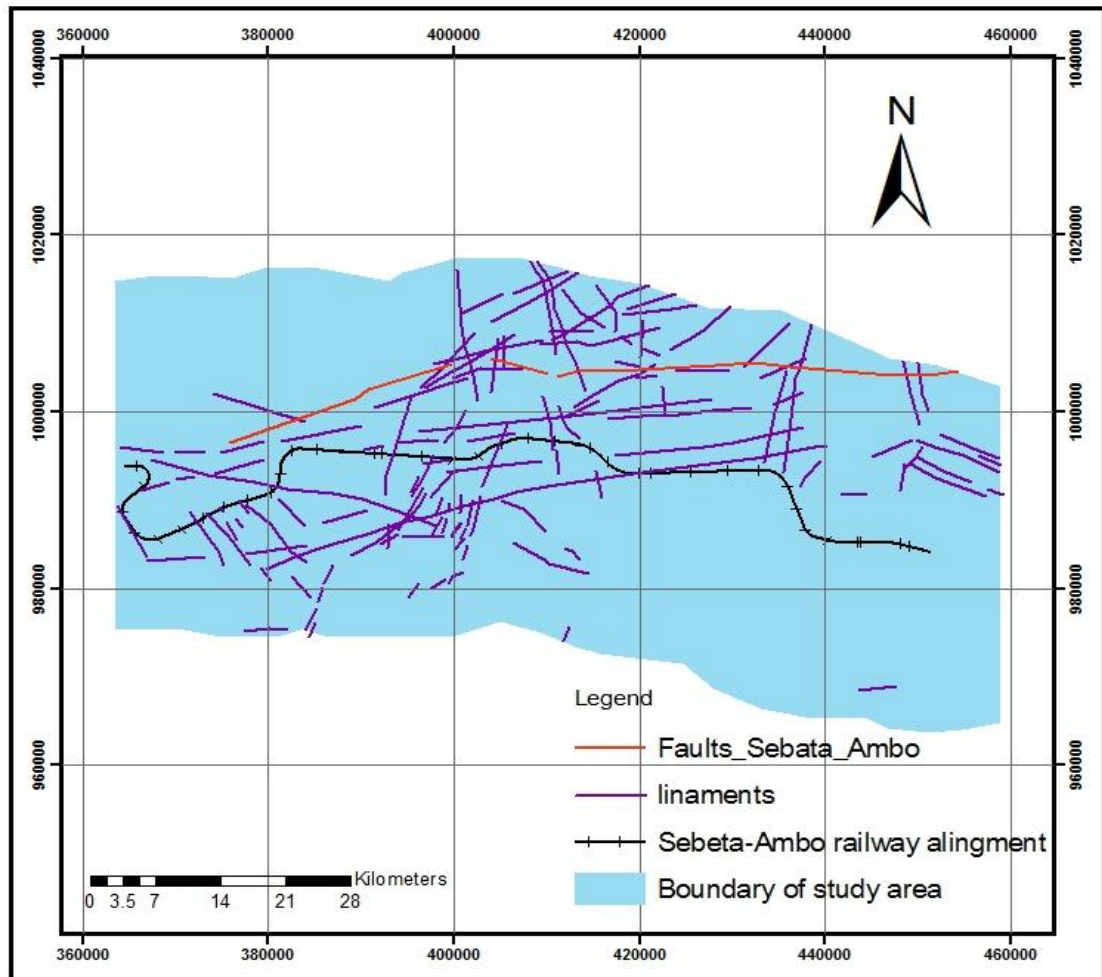


Fig.3.6 Geologic structures in the project area

The drainage system is primarily governed by the regional and local geologic structures. Perennial streams and seasonal springs occur along the fault lines, fractures and contacts. Presence of intensive faulting has contributed for the emergence of numerous springs like; Beredo spring which is in between Ginchi and Welcomi Towns, beside it has also enhanced occurrence of ground water. Lake Dandi, which is an 8km wide crater lake, is also other important surface water resources in the area. However, the high elevated parts of the study area in the north and eastern border and also some central area of the study area is characterized by low drainage pattern and / or density. As explained earlier in the local geology two terrain types were identified in the study area these are volcanic and sedimentary units. The latter unit is a minor unit which pinches out near to the Ambo town towards west end of the study area. The sedimentary unit constitutes Mesozoic sandstone and Abbay beds with minor lime stone on the way to Guder from Ambo.

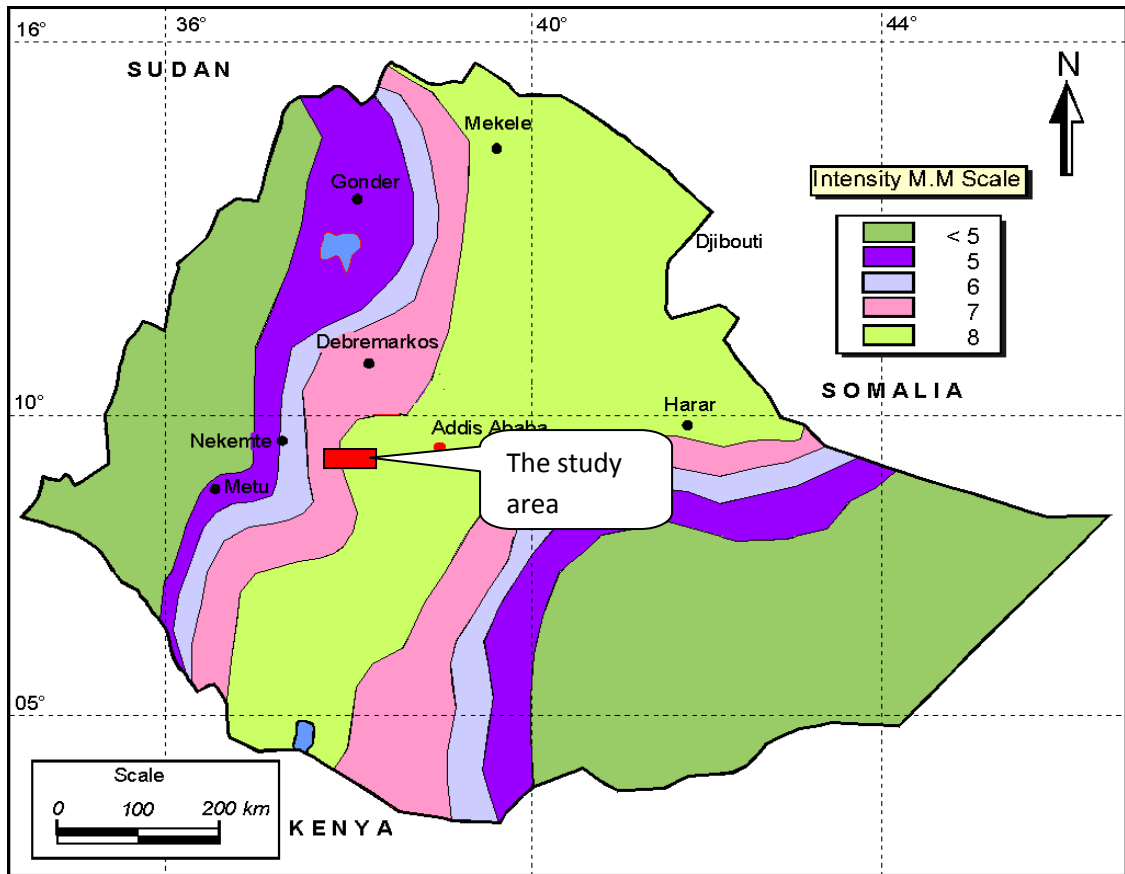


Fig.3.7 Seismic Risk Map (After, Lakemariam Asfaw, 1986)

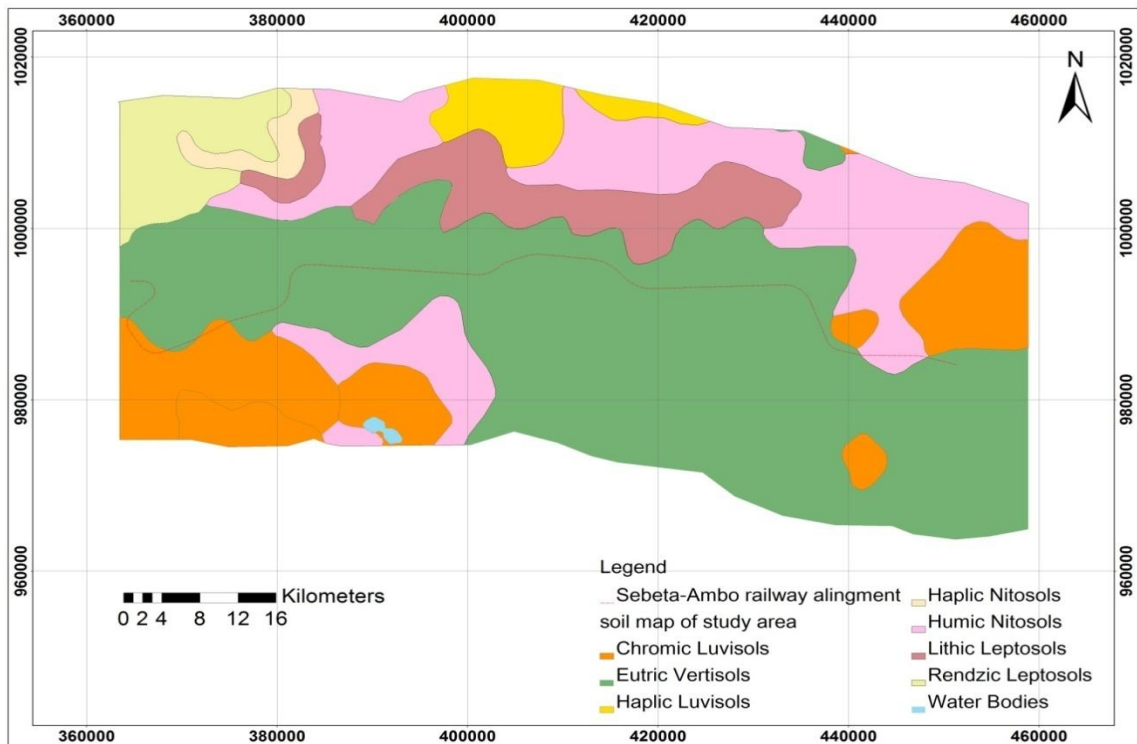


Fig. 3.8 Soil map of the study area (FAO 1984)

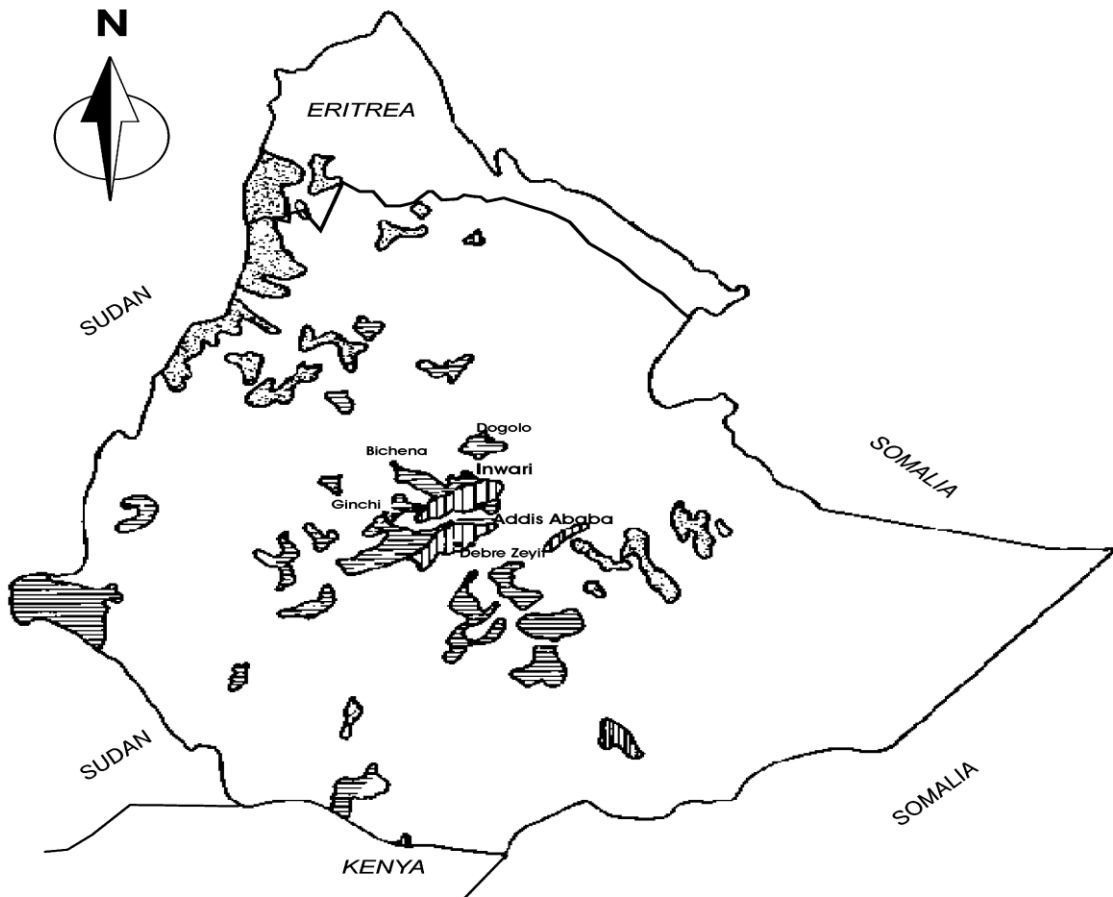


Fig.3.9 The distribution of expansive soil in Ethiopia (Srivastava et.al. 1984)

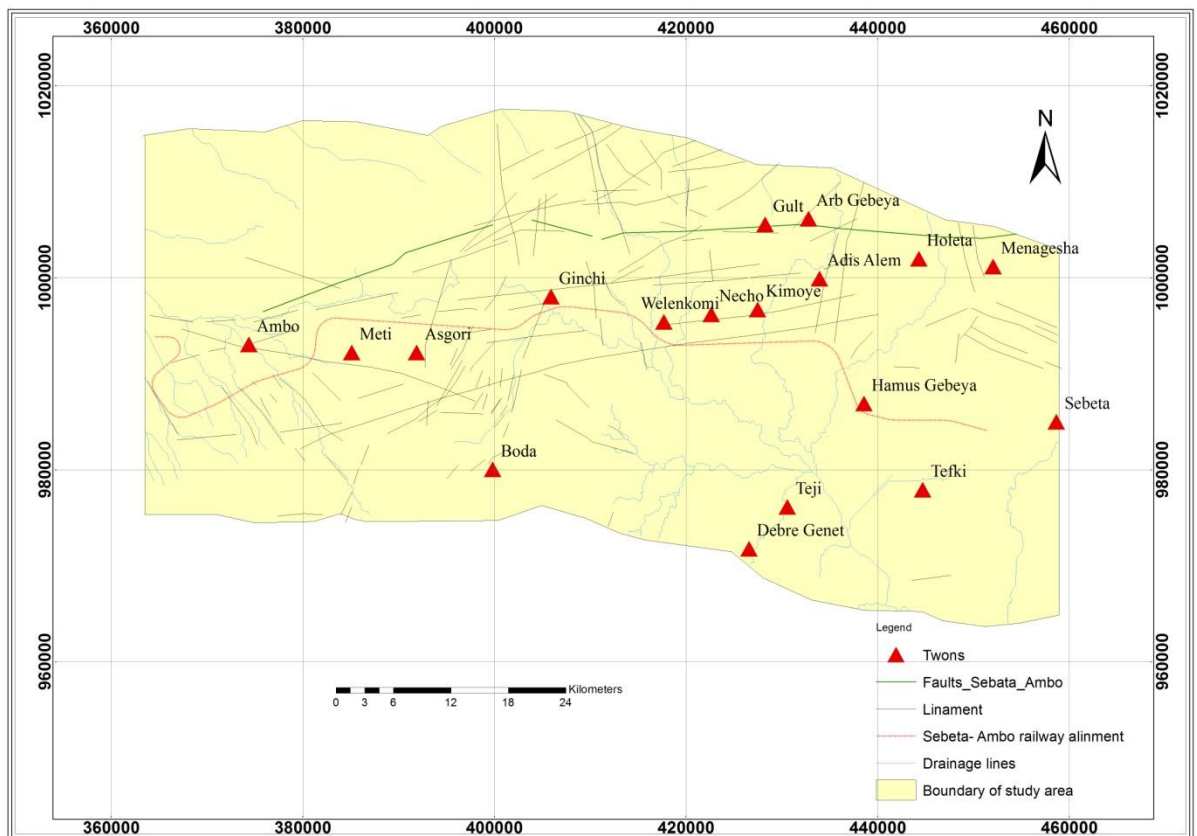


Fig.3.10 Showing inter-relationship of drainage line and structures

Table 3.2 Sub-grade extension along the project route

Chainage	Visual Description
0+000-2+500	Black cotton soil
2+500-3+500	Light brown silty clay soil
3+500-6+500	Black cotton soil (marshy area)
6+500-7+800	Grayish silty clay soil
7+800-9+200	Black cotton soil
9+200 -9+600	Black cotton soil
9+600-10+000	Light reddish to grayish silty clay soil
10+000-12+500	Black cotton soil (marshy area)
12+500-13+400	Light grayish silty clay soil (marshy area)
13+400-18+000	Black cotton soil
18+000-19+200	Reddish brown silty clay soil
19+200-21+000	Black cotton soil (marshy area)
21+000-21+400	Reddish brown silty clay soil
21+400-28+200	Black cotton soil
28+200-29+300	Brownish silty clay soil
29+300-33+400	Black cotton soil(marshy area)
33+400-33+700	Dark brown silty clay soil
33+700-37+200	Black cotton soil
37+200-38+400	Reddish brown silty clay soil
38+400-40+500	Black cotton soil
40+500-41+000	Reddish silty clay soil
41+000-46+500	Black cotton soil
46+500-47+000	Light brown to grayish silty clay soil
47+000-50+000	Black cotton soil
50+000-51+500	Yellowish to grayish silty clay soil
51+500-51+800	Black cotton soil
51+800-52+500	Yellowish to brownish silty clay soil
52+500-53+800	Black cotton soil
53+800-54+300	Light reddish silty clay soil
54+300-55+000	Black cotton soil
55+000-55+500	Reddish to yellowish silty clay
55+500-58+000	Black cotton soil
58+000-59+400	Grayish silty clay soil
59+400-62+800	Black cotton soil
62+800-63+400	Grayish silty clay soil
63+400-66+900	Black cotton soil
66+900-67+300	Reddish silty clay soil
67+300-79+800	Black cotton soil
79+800-80+450	Brown silty clay soil
80+450-80+900	Black cotton soil
80+900-81+300	Brown silty clay soil
81+300-83+800	Black cotton soil
83+800-85+700	Light reddish to yellowish silty clay soil
85+700-87+900	Black cotton soil
87+900-92+300	Reddish brown silty clay soil
92+300-95+450	Grayish silty clay soil
95+450-99+800	Reddish brown silty clay soil
99+800-101+350	Black cotton soil
101+350-103+400	Reddish brown silty clay soil
103+400-106+500	Black cotton soil
106+500-107+200	Reddish brown silty clay soil
107+200-107+700	Black cotton soil
107+700-108+060	Reddish silty clay soil

Lemmassa Mekonta (2001) study shows that, the sand stone unit is considered as confined aquifer, the source formation of Ambo mineral water, having the confining layer as Abbay bed (mudstone-shale unit) and basalt rock.

Volcanic products spread over the large portion of the study area, particularly from the east-west running volcanic centers, south border and north counterpart of Ambo lineaments. Ground water occurrence is largely influenced by the basic and acidic volcanic rocks particularly the basalt rock is the main source of aquifer apart from sedimentary rocks. Occurrence of plenty of springs around basaltic rock unit strengthens its suitability as a media to transfer water in to and out of the system.

According to Adaa-Becho Groundwater Resource Evaluation Project (WWDSE, 2008) alluvial deposit, in the eastern margin, (Becho-plain) has significant contribution for shallow ground water. Also, this deposit has significant function in providing shallow ground water, allowing recharge to the lower acidic overlain scoracious formation which serves as potential aquifer. Finally, what matters the most is the variation of the groundwater table rather than the presence of ground water.



Plate 3.1 Jemjem River, Awash tributary, at station 53+000

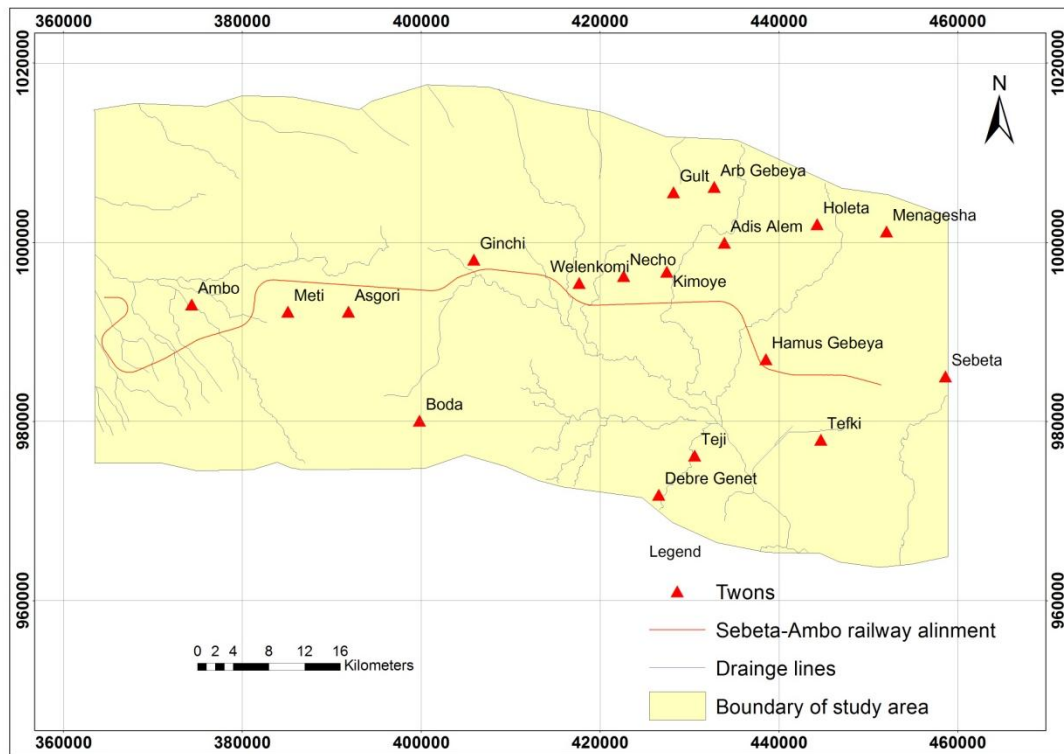
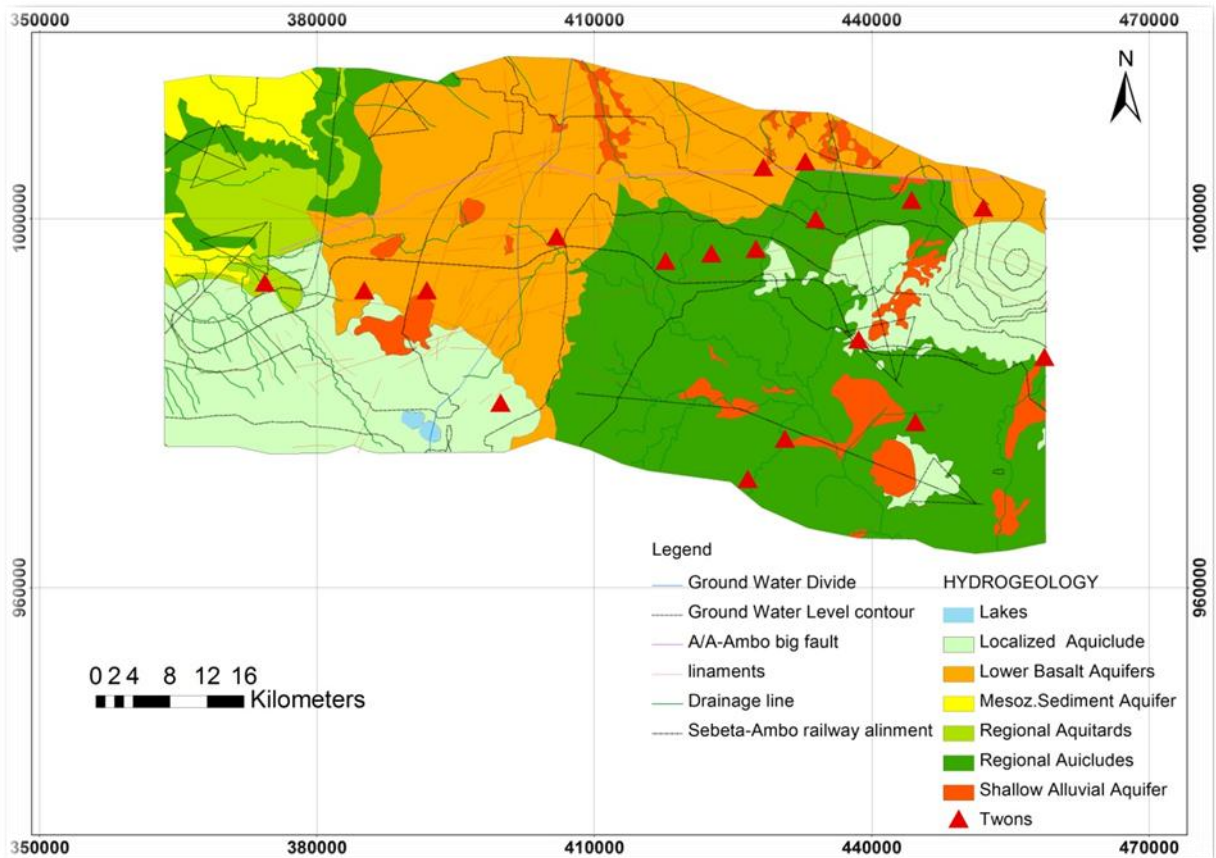


Fig3.11 Drainage pattern of the study area



(Source: Extracted from *Adaa-Becho Groundwater Resource Evaluation Project, WWDSE, 2008*)

Fig. 3.12 Hydro geological map of the study area

3.8 Land use and land cover

In and around the study area traditional farming is one of the major activities to generate income among the rural inhabitants. As can be seen in Plate 3.2, the land is intensively cultivated by the indigenous people for production of subsistence crops specially teff, maize etc. Because of the increasing population size traditional farming is advancing at the expense of deforestation. As a result hill side, mountains and plain lands are stripped of their forest and grass cover. Taking advantage of this, erosion is also washing away most fertile soils out of the area from all direction every year. Bush, shrubs, big trees and grass has covered most parts of the study area. Unlike past, the vegetation density is too little but there is some spared remnants forest along the hill side. However, these are also at verge of extinction due to many reasons such as replacement by eucalyptus tree which is sold for a good amount of money, farm land expansion, burning for charcoal etc. Generally, unwise natural resource utilization and mismanagement is endangering and complicating the ecosystem.

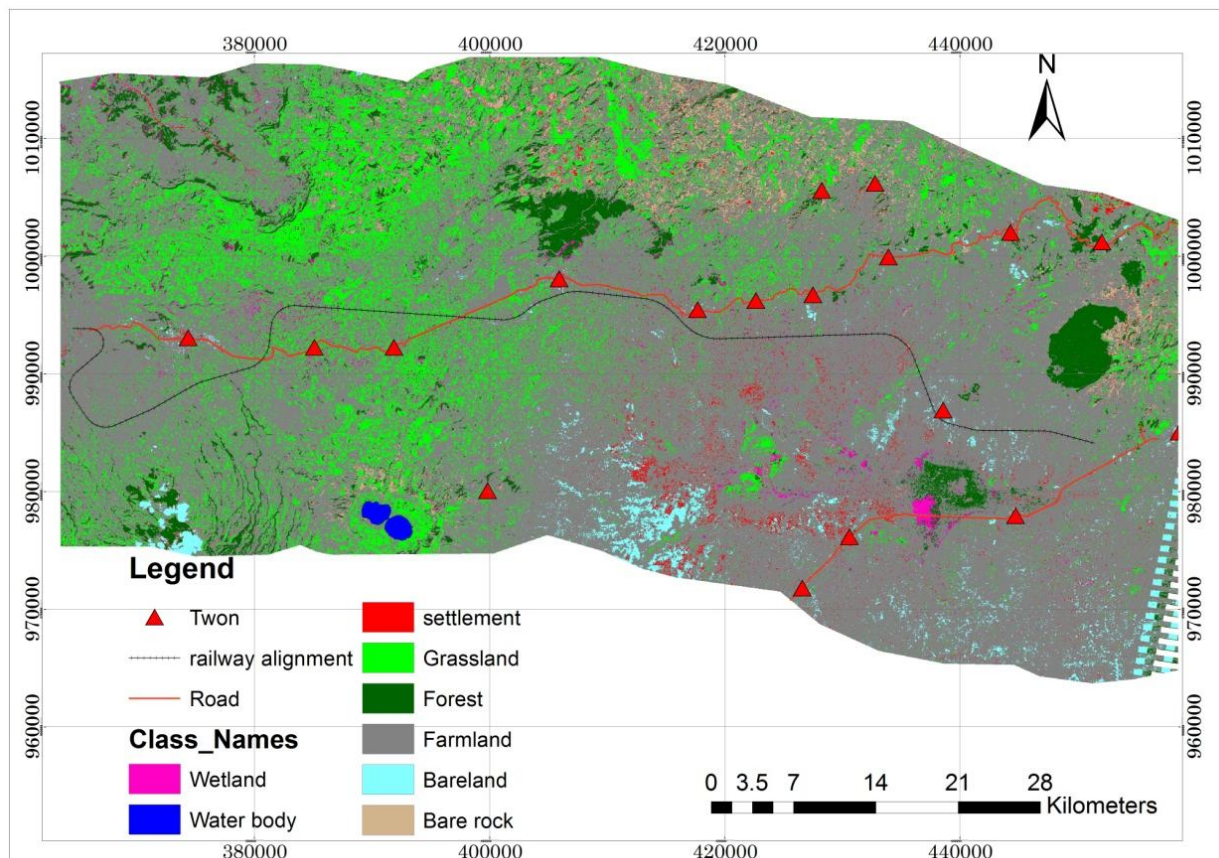


Fig 3. 13 Land use and Land cover map of the study area



Plate 3.2 Land use land cover

3.9 Climate of the study area

Seasonal classification over the country is mainly based on the average rainfall distribution pattern over the year. Hence, Ethiopia has three seasons namely Bega, Belg and Kiremete. Bega (dry season) is from October to January. It is characterized by dry and cool period. Belg (small rainy season) is from February to May. It is characterized by varying dry and wet days. Kiremet (main rainy season) is from June to September violent thunderstorms in the highlands are common in late June.

The study area forms a part in the central plateau, its relief is very essential in controlling the climatologically condition. Considerable rainfall is received in the west central part due to High Point Mountains that poses orographic influence.

Rain fall and temperature data obtained from National Metrological Service Agency (NMSA) was analyzed to understand the metrological nature of the area and is presented in the following paragraphs.

3.9.1 Rain fall in the study area

To determine the rain fall condition of the study area the rain fall data for different years from various metrological stations (Sebeta, Kimoya, Weloncomi, Ginchi and Ambo stations) was collected and analyzed. Accordingly, from Ambo station 11 years (1999-2009) data, from

Ginche station 10 years (1999-2008) data, from Weloncomi stations 9 years (1999-2007) data, from Kimoya station 9 years (1999-2007) data and from Sebeta stations 11 years (1999-2009) rain fall data was collected and analyzed. Thus, the mean annual rain fall for Ambo, Ginchi, Weloncomi, Kimoya and Sebeta stations was computed as 954.5mm, 1191.1mm, 891.2mm, 949.6mm and 938.9mm, respectively (Fig.3.15, Table 3.4).

A perusal of graph (Fig. 3.14) shows that the high rain fall in the study area is recorded during the months June, July and August while the minimum rain fall in the study area was recorded from October to February. In order to see the fluctuation of the rain fall for each year, the annual rain fall for consecutive years at different stations of the study area were also evaluated.

From annual rain fall graph (Fig. 3.15) the rain fall condition shows almost constant from year to year in each station. However, in the year 2006 Ginchi station shows an exceptionally high value of rainfall. This implies that there is a possibility of sudden increase in rain fall in future.

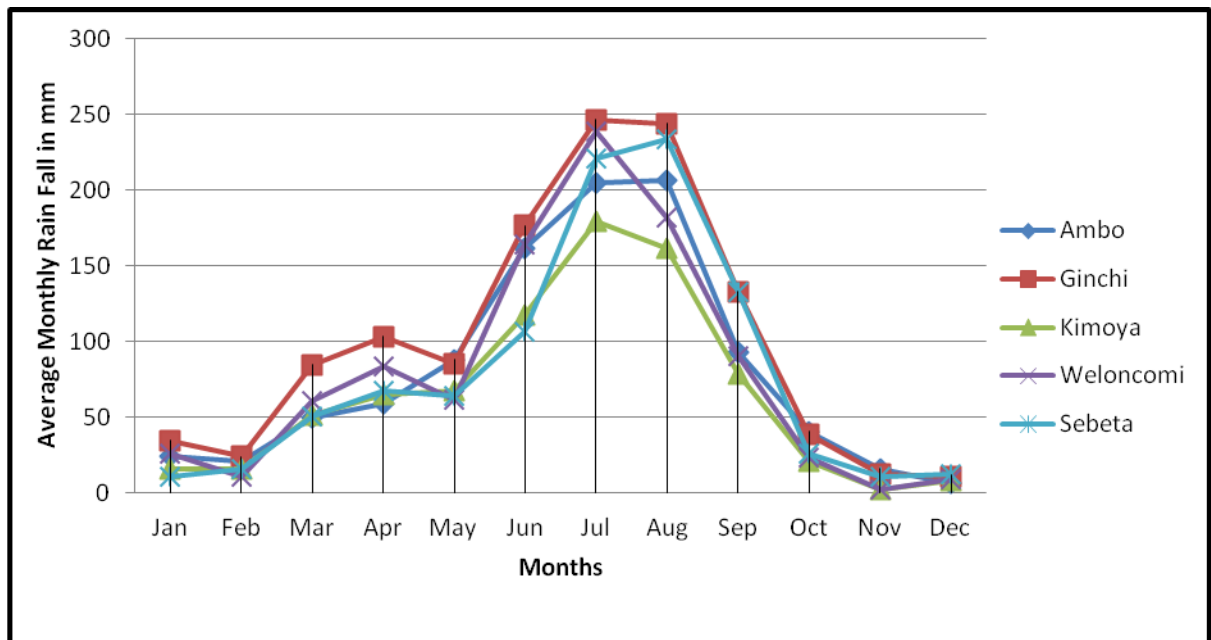


Fig. 3.14 Mean monthly rain fall of the study area based on the five stations

Table 3.3 Mean monthly rain fall of different station in the study area

Stations	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Ambo	24.5	20.6	49.9	59.2	88.1	161.5	204.8	206.6	93.0	40.5	15.9	6.5
Ginchi	34.5	23.9	84.7	103	85.2	176.7	246.3	244	133	38.2	12.1	10.2
Kimoya	15.3	15.3	51.7	64.8	67.8	117	179	161.4	78.5	21.2	2	8.2
Weloncomi	26	10.3	61	83.4	61.6	163.6	238.6	182	90	23.3	2.19	9
Sebata	11	15.8	50.8	67.1	64.4	106.8	220.9	233.9	132.3	26.2	10.6	12.2

Table 3.4 Different year's annual rain fall of each station in the study area

Stations	1999	2000	2001	2002	2003	2004	2005	2006	2007	2008	2009
Ambo	822.5	903.2	1022	769	889	944	856.6	1104	1163	1160	866.4
Ginchi	1124	1026.7	1105	1024.6	1076.8	1137	1148	2152.6	1090.7	1024	-
Kimoya	769.6	946.9	1014	861.7	915	1083	990.4	988.9	891	-	-
Weloncomi	743.3	624.4	975	905.8	1016	876	1008	-	771	-	-
Sebeta	860.9	857.5	967	794	1034.5	1031	906.7	1043	835	1093	906

3.9.2 Temperature

In order to know the temperature condition of the study area, maximum and minimum monthly temperature in different years for three stations (Ambo, Addis Alem, and Kimoya stations) of the study area were utilized (Source Ethiopian National Metrology Agency). From Ambo station 11 years (1999-2009) data, from Addis-Alem station 11 years (2000-2010) data and from Kimoya station 9 years (1994-2002) monthly maximum and minimum temperature data was collected and used to calculate the general maximum, minimum and mean temperature of each station in the study area.

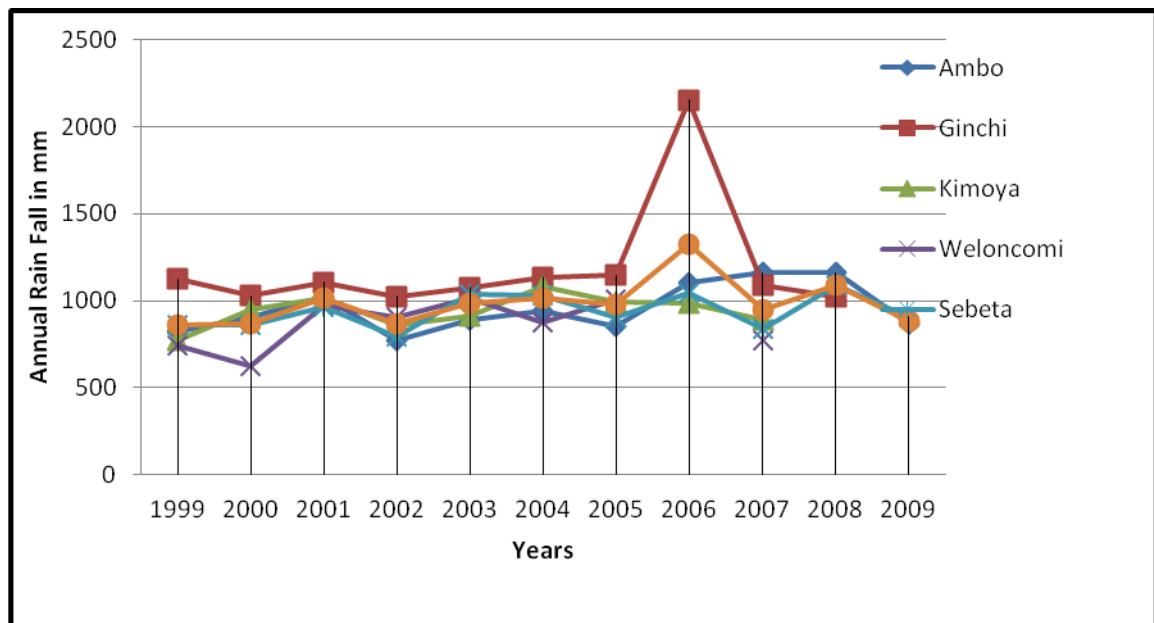


Fig. 3.15 Annual rain fall distribution in the study area

Generally, the temperature in Ethiopia shows variation from place to place and also from time to time. The diverse rain fall and temperature condition in the country is mainly because of the result of its location in the Africa's tropical zone and the complex topographic condition of the country.

In the present study area temperature variation is not much when compared on yearly basis. However, significant temperature range variation can be observed if compared (by taking maximum and minimum) for different stations on monthly basis.

Table 3.5 The maximum, minimum and mean temperature* of four stations in the study area

Stations	Ambo	Addis-Alem	Kimoya
Minimum	11.9	9.5	8.5
Average	18.95	15.9	17
Maximum	26	22.2	25.5

Finally, it may be concluded that two prominent climatical conditions prevailing in the study area are wet season (June to September) and dry season October to May with slight Belg rainfall from April to May. Based on this, it can be said that the ground water level of the area will follow the rainfall intensity in the area. From engineering point of view temperature and rain fall variation in the study area may result in the fluctuation of the moisture content within the soil.

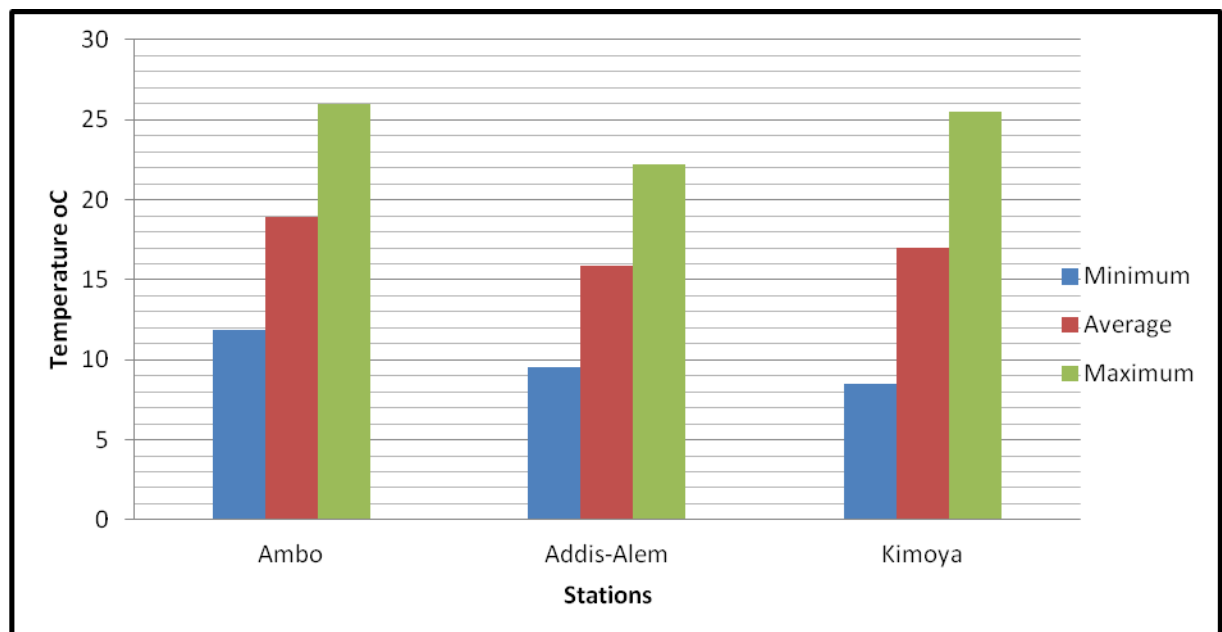


Fig.3.16 Maximum, minimum and mean temperature* of four stations in study area

In other words the swelling of the sub-grade soil is expected to occur during the wet season, from June to September and shrink during the dry season.

4.1 Preamble

Sub-grade is the load-bearing layer of a railway structure and road, either compacted natural ground or an imported fill embankment, which provides a permanent way to support the ballast layers or embankment fill. The aim of this section is to provide a basic platform for a better description of the natural ground or natural sub-grade. Natural Sub-grade investigation and characterization of its engineering properties are fundamental for foundation of any light weight structure. Hence, it is important for site selection, design and construction. The description made in the present chapter refers to natural sub-grade which is found in-situ on the ground or ground soil.

Natural sub-grade investigation is performed by pitting from the existing ground level along the alignment route. The aim of the investigation is to assess the nature of the natural ground soil characteristics, to assess the suitability of the soil so as to integrate for the embankment fill design and identify the location and nature of problematic natural sub-grade soil section along the proposed alignment and finally to suggest possible remedial measure.

The natural sub grade investigation in the present study area was performed at 500 m interval as per the project specification. Sampling and test pit logging was performed by the Classic Consulting Engineers P.L.C, (CCE, 2011/12) however, some pitting and logging was also performed during the present field verification and sampling of soils for fine fills and for settlement potential of the natural sub-grade.

4.2 Natural sub-grade field investigation

During the natural sub-grade investigation in the field, description of the soil extension and sampling of the natural sub-grade for laboratory analysis was performed. Sampling of natural sub-grade was performed for classification and strength evaluation. The classification of natural sub-grade soil was performed at 0.5 km intervals and at 216 locations. Strength test (California Bearing Strength (CBR)) was carried out on an average of 1km interval along the proposed alignment and 108 samples of soil were taken for analysis (CCE, 2011/12). Field data verification for some important soil tests as well as undisturbed soil sampling (UDS), for consolidation test, from representative pitting (Plate 4.1) was also performed during the present study.

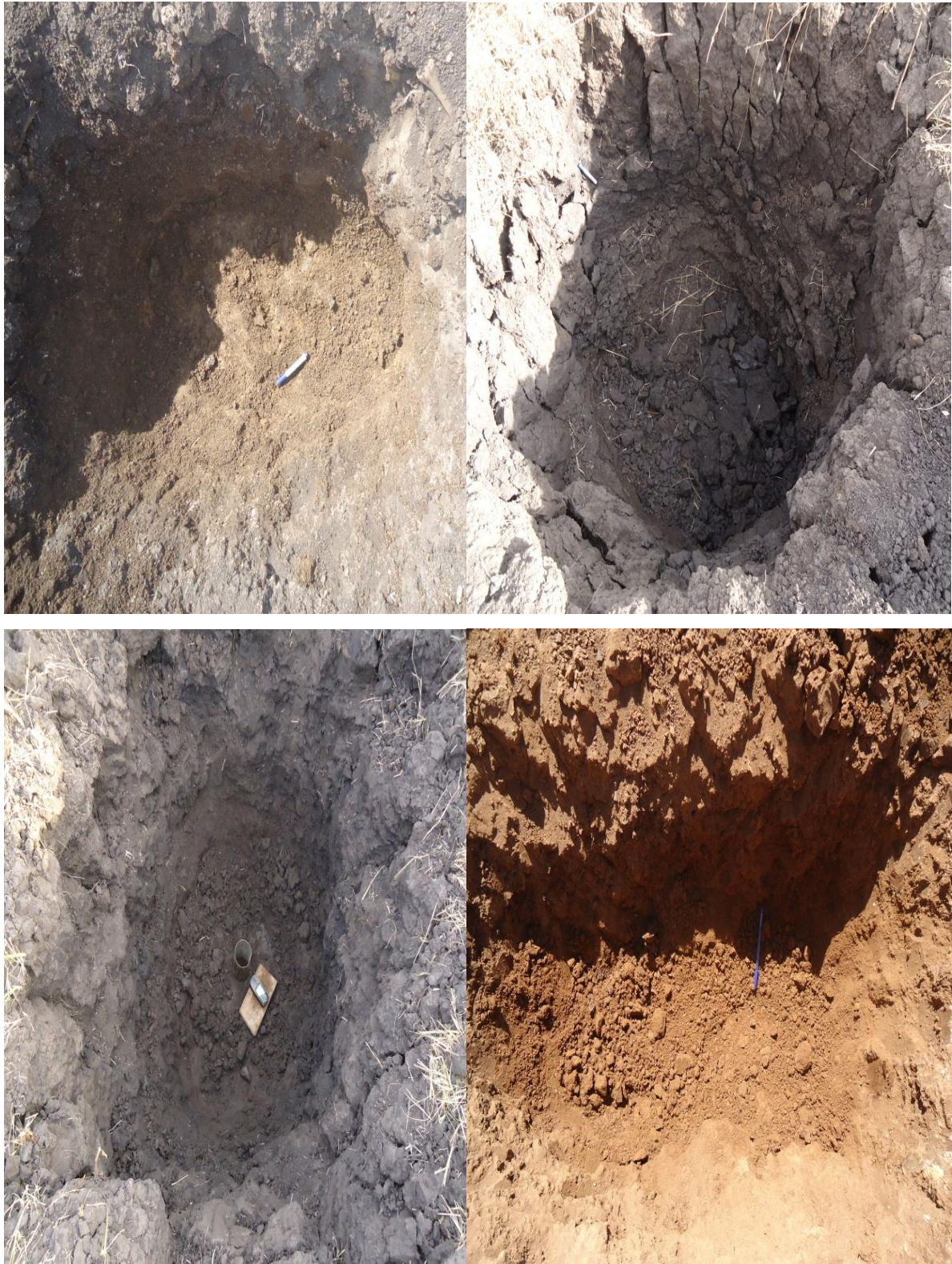


Plate 4.1 **Excavated representative test pits**

4.2.1 Visual natural Sub-grade Soil Investigation

In order to group homogeneous sections, during the field work, the visual natural sub-grade soil extension survey along the alignment has been carried out. This was carried out to assess

the nature, type and extent of existing natural sub-grade soil that makes the railroad. Natural sub-grade soils with similar soil type were grouped together and their extent was also determined.

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

The type of natural sub-grade soil encountered along the route corridor is found to be mainly dependent on topography and geology of the project area. The railway alignment generally lies from rolling terrain to a flat region. As a result, erosion effect is minimal along flat terrain sections and thus development of thick residual soil is facilitated.

From the visual inspection, during the field data verification, the sampled natural sub-grade is well described. In most of the cases it was black cotton soil however, at places the colour of the soil varies from reddish brown to yellowish brown and grayish silty clay in some of the stretches. During the soil extension survey, clear distinction between the individual soil types could not be made due to their similarity in origin and soil properties, and gradual transitions from one soil type to the other.

4.2.2 Natural sub-grade sampling

Further investigation on soil type, strength of the soil (CBR) and swelling potential was conducted by taking representative samples. For soil classification analysis samples were taken at an intervals of approximately 0.5 Km whereas, samples for strength and swelling potential tests were taken at every 1 km along the proposed alignment. However, at specific locations samples were also taken at 2 km interval. The interval of the sampling stations was based on the project specification (CCE, 2011/12).

4.3 Laboratory Investigation for natural Sub-grade Soils

The wide range of properties under different conditions affects the performance and use of soil for various purposes such as; foundation 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 natural sub-grade soil samples were subjected to various tests to determine their physical properties. The type of tests carried out in laboratory includes; Soil Classification

tests such as; Grain size distribution and Atterberg Limits, Strength tests, Moisture/ Density Relationship (Compaction), California Bearing Ratio (CBR), and Swell tests. Consolidation test was also carried out for selected soil samples.

4.4 Soil Classification

Though there are number of soil classification systems in practice, for the present study soil groups were worked out using American Association of State Highway and Transportation Officials (AASHTO) soil classification method and based on which determination of natural sub-grade quality was assessed. Index properties are indicator in predicting the engineering properties of any soil material and helps in classifying the soil. The two important index properties of soils are; (i) Grain size distribution and (ii) Consistency limits (Arora, 1997).

4.4.1 Grain size distribution

In order to determine the grain size distribution of the natural sub-grade wet sieve analysis was employed in accordance with AASHTO T-88 Test Method for Particle-Size Analysis of Soils. For the present study, wet sieve analysis was carried out to determine the grain size distribution of natural sub-grade soil and later the grain size distribution was used for the classification of the soil type. The grain size distribution analysis for various soil samples are presented in Annexure - two.

4.4.2 Atterberg limits

In the year 1911, Atterberg proposed the limits: liquid limit (LL), plastic limit (PL) and shrinkage limit (SL) (Fig.4.1), of consistency in an effort to classify the soils and understand the correlation between the limits and engineering properties like compressibility, shear strength and permeability. The limits represent the water holding capacity at different states of consistency (Casagrande, 1932).

This test determines the nature and response of the natural sub-grade soils upon change to moisture content. Consistency limit indicates the properties of soils such as; volume changing behavior of soils due to moisture change, strength properties due to plasticity and shear strength loss during wet time. Expansive clays exhibit higher shrinkage and swelling upon change in moisture content. The degree of their expansiveness can be obtained by conducting shrinkage limit test (USBR, 1968).

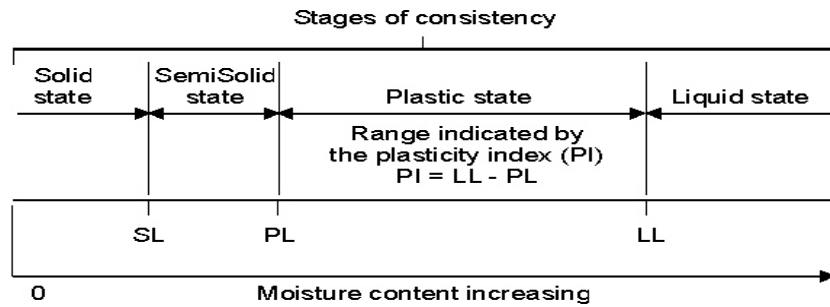


Fig.4.1 Atterberg limits and the stages of soil consistency (Source: USBR, 1968)

The Atterberg Limit depends on the type of predominant material in the soil. If Montmorillonite is the predominant mineral the liquid limit can reach or even exceed 100% (Fig.4.2). It is also expected that the Atterberg Limit is less for Illite soil materials and even lesser for Kaolinite soils (Chleborad, 2005).

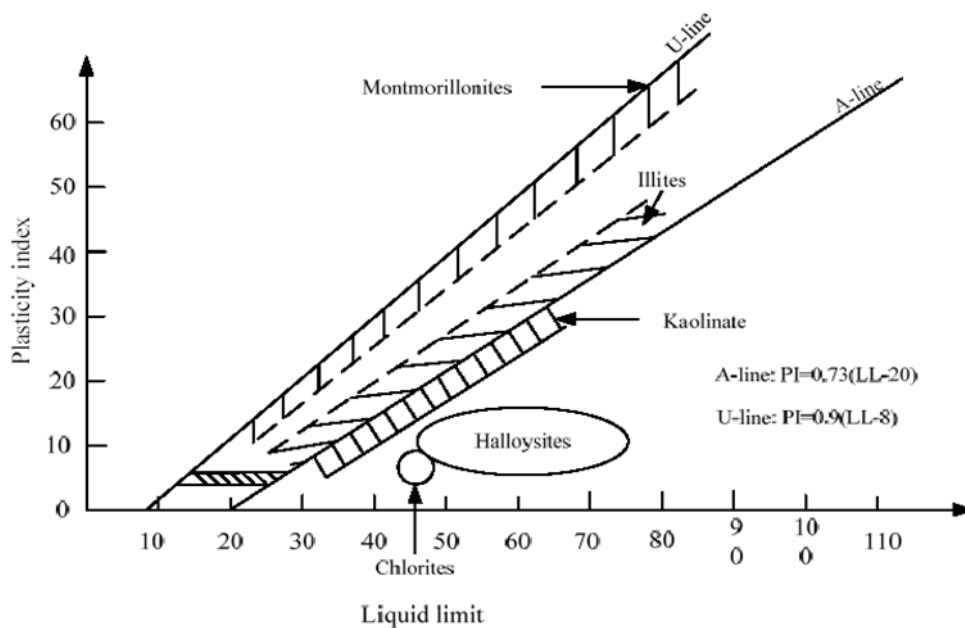


Fig 4.2 Cassagrande's LL-PI chart and respective clay minerals

In general, soils that exhibit plastic behavior over wide ranges of moisture content and that have high liquid limits have greater potential for swelling and shrinking (Nelson et al., 2010). For this reason, the present research has focused on the sub-grade soils that may result into higher plasticity index. Further, shrinkage limit was carried out to determine the weighted expansiveness nature of the natural sub-grade soils.

4.4.3 Compaction test (Moisture-Density Relationship)

The compaction test, also called Proctor tests, was conducted on representative samples from natural sub-grade soils of the study area. The 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 determined according to AASHTO T180 modified proctor test. For this test a 10 lb hammer falling from a distance of 18 inches, and five equal layers of soil was 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. For the present study the compaction test was conducted on 54 soil sample from the natural sub-grade and the test results are presented in Annexure – two.

4.4.4 Consolidation test

Any structure built on the ground, as for example from the load applied by the construction of an embankment of the railroad, causes increase of pressure on the underlying soil layers. The soil layers are unable to spread the stresses laterally as the surrounding soil strata confine them. Hence, there must be adjustment to the added new pressure by vertical deformation. The compression of the soil mass leads to the decrease in the volume of the mass, which result in the settlement of the structure, built on the mass. Hence, this test helps to predict the amount and rate of settlement of a structure and also helps in understanding the stress history of soil deposit (Arora, 1997)

For the present study consolidation test was carried out on selected undisturbed soil samples taken from different soil units (Welencomi to Asgori towns). Later, different consolidation parameters were analyzed from the test results. The test was conducted according to ASTM D 2435 on standard one-dimensional odometer. The standard one dimensional consolidation test is usually carried out on saturated specimen using an Odometer. In this test a small representative sample of soil is carefully trimmed and fitted into a rigid metal ring. The soil sample is mounted on a porous stone base and a similar stone is placed on top to permit water, which is squeezed out of the sample to escape freely at the top and bottom. Prior to loading, the height of the sample should be accurately measured. Also, a micrometer dial is mounted in such a manner that the vertical strain in the sample can be measured as loads are applied.

According to ASTM D 2435, the consolidation test apparatus is designed to permit the sample to be submerged in water during the test to simulate the position below a water table of the prototype soil sample from which the test sample was taken.

Loads are applied in steps in such a way that the successive load intensity, P , is twice the preceding one; the load intensities commonly used being 1/4, 1/2, 1, 2, 4, 8, 16 kg/cm². Each load is allowed to stand until primary consolidation is practically ceased. The dial readings are taken at elapsed time of 0, 0.25, 0.50, 1, 2, 4, 8, 15, 30, 60 minutes.....24 hours. After the greatest load required for the test has been applied to the soil sample, the load is removed in decrements to provide data for plotting the expansion curve of the soil in order to learn its elastic properties and magnitude of plastic or permanent deformation.

4.4.5 Strength Test

The most commonly adopted strength determination laboratory test for soil material is California Bearing Ratio (CBR) Test. For the present study 3-point CBR Test was performed. For this three CBR moulds were prepared by 10, 30 and 65 hammer drops according to AASHTO T- 193 test methods. The CBR value at 95% of the MDD was determined to be as the CBR value. Further, CBR test also determines the swelling capacity of the material and gives swell percent by attaching swell gage in the CBR mould on the top of surcharge. For the present study CBR test was performed on 54 soil samples and the test results are presented in Annexure- two.

4.5 Laboratory analysis of Test Results

As described in the previous sections different laboratory tests has been conducted on representative samples which includes; Grain Size distribution and Atterberg Limits for the purpose of soil classification, determination of optimum moisture content, maximum dry density and 4-days soaked 3-Point CBR and swell. In this section, all the test result are discussed and evaluated accordingly.

The laboratory analysis is grouped into three; the first deals with the soil classification, the second deals with the strength related parameters such as; optimum moisture content (OMC), Maximum Dry Density (MDD), California Bearing Ratio (CBR) and swell potential (SP) and the third deals with the settlement related parameters.

4.5.1 Soil classification

For engineering purposes AASHTO and Unified Soil Classification System (USCS) are the two most widely used soil classification systems. The AASHTO system of soil classification is based on particle-size distribution, liquid limit and plasticity index. It comprises of seven groups of inorganic soils from A-1 to A-7 with 12 subgroups in all. On the other hand the USCS divides the soil in to three major divisions, coarse grained soils, fine grained soil and highly organic soils based on the recognition of the type and predominance of the constituents considering grain-size, gradation, plasticity and compressibility (Casagrande, 1948). However, for the present study the natural sub-grade soil groups were worked out using AASHITO soil classification method and the summery of the classification with its test results are presented in Annexure - two.

The soil classification made for the natural sub-grade soil in the present study confirms that AASHTO soil classification determines almost all of the natural sub-grade soil samples falling in to A-7-5 (about 91.2%) soil group whereas, some of the soil samples fall into A-7-6 (about 8.4%) only, one sample falls into A-2-7 soil group. Detailed sample description on soil classification is presented in Annexure - two.

From the above test results it can be said that the nature of the natural sub-grade soil along the proposed alignment are susceptible to volumetric change when subjected to moisture variations. Therefore, it can be said that the nature of the natural sub-grade soil along the proposed alignment is of poor quality.

Group Index

Group Index (GI) further helps to differentiate soils containing appreciable fine-grained materials. The GI is determined by using Eq. 4.1:

$$GI = (F - 35) (0.2 + 0.005 (LL - 40)) + 0.01 (F - 15) (PI - 10) \quad \text{..... Eq. 4.1}$$

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.

When the value is negative, the group index shall be taken as zero which indicates a good natural sub-grade material and when the value is greater than or equal to 20, the group index shall be reported as twenty, it indicates a very poor sub-grade material (AASHTO, 1993). In

general, the higher the value of the GI, the poorer is the quality of the material within its own group.

Along the proposed rail alignment GI value was determined for all samples and the results are presented in Fig. 4.3. A Perusal of Fig. 4.3 indicates that all the samples along the proposed alignment show GI values greater than 20. This indicates that all the section contains unsuitable soil to be used as foundation of the embankment fill. These results coincide with the AASHTO soil classification result where the natural sub-grade soils fall under A-7-5 and A-7-6 soil classes.

In addition to this, the expansive nature of natural sub-grade soils of the proposed alignment was further determined by plasticity index (PI). The PI value of the natural sub-grade soil is in the range of 20-52, as shown on the Fig. 4.4.

In general, according to Holtz and Gibbs (1956) the soils having PI values in the range of 15 – 28 have medium potential swell whereas soils having PI values from 25 – 41 have high potential soil. The soils which have PI value greater than 35 have very high potential swell. Thus, the natural sub-grade soils have a potential for volume change when they are subjected to moisture variations (AASHTO, 2000). This is also an indication that the soils along the proposed alignment, in general, are of poor quality as far as swelling potential is concerned and thus, unsuitable to be used as foundation material.

4.5.2 Moisture Density Relations of the natural Sub-grade Soil

Proctor density test was conducted in order to determine the relationship between moisture content and dry density for a specific compaction effort. This test was conducted according to AASHTO T180. In total Proctor Density test was carried out for 53 samples collected at an interval of 2 km along the proposed alignment.

A perusal of Fig. 4.5 and Fig. 4.6 indicates that the OMC ranges from 1.24 % – 1.45% whereas the MDD is found to be in the range of 24 gm/cc - 38 gm/cc.

4.5.3 California Bearing Ratio (CBR) and swell (%)

The strength of the natural sub-grade material has also been determined using CBR value. A three point CBR test at 10, 30 and 65 blows were conducted according to AASHTO T193 and the CBR values at 95% MDD have been interpolated from the CBRs at densities

obtained from different compaction level and by interpolating from single compaction level. In total CBR test was conducted for 53 soil samples and the results thus obtained are shown in Fig.4.7. The result showed that the natural sub-grade has very low CBR value, less than 4%, which does not satisfy the minimum requirement as sub-grade material (UIC, 2005).

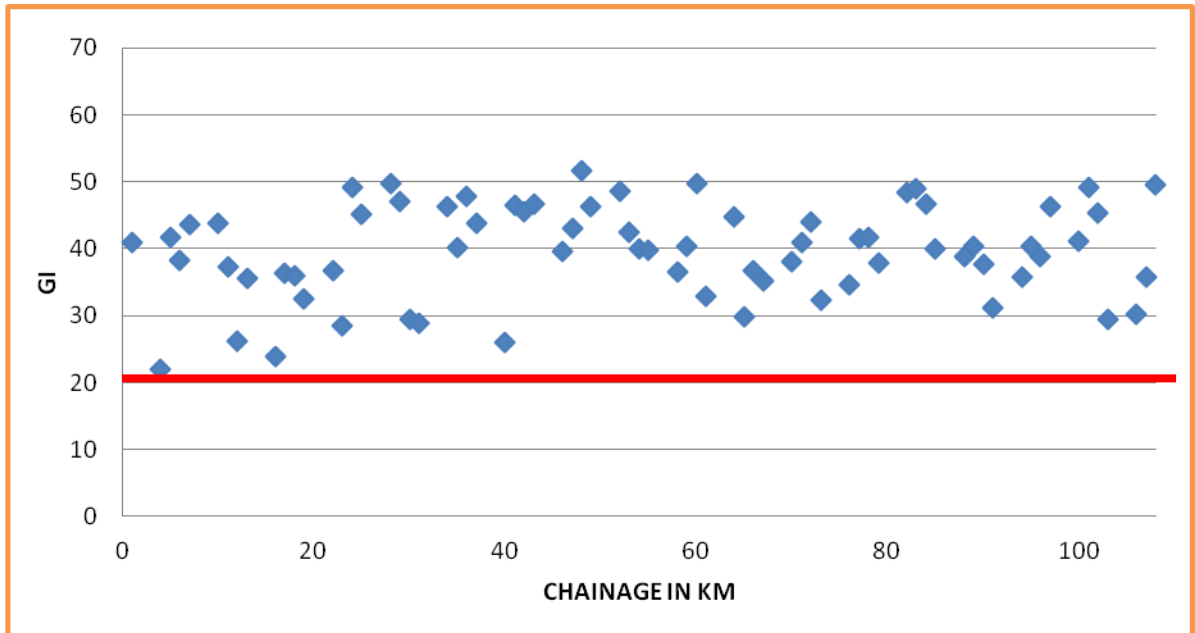


Fig. 4.3 Group Index Values for the natural Sub grade Soil along the railway alignment

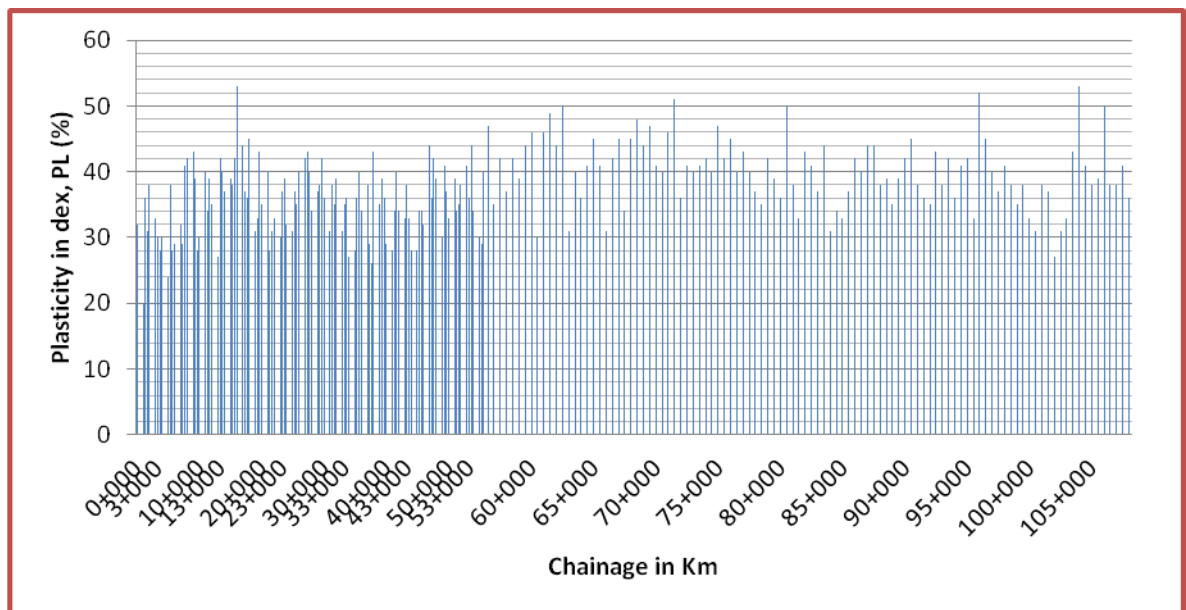


Fig. 4.4 Plasticity Index for the natural Sub grade Soil along the road alignment

The Fig.4.8 shows the swell values determined from a swell gage adjusted on the CBR mould that is soaked for 4 days in the worst condition. The least requirement for having suitable sub-grade material is having swelling % of less than 2% (UIC, 2005). However, as can be seen from Fig.4.8 all samples show value of swell % above 2% therefore soils are considered to be unsuitable.

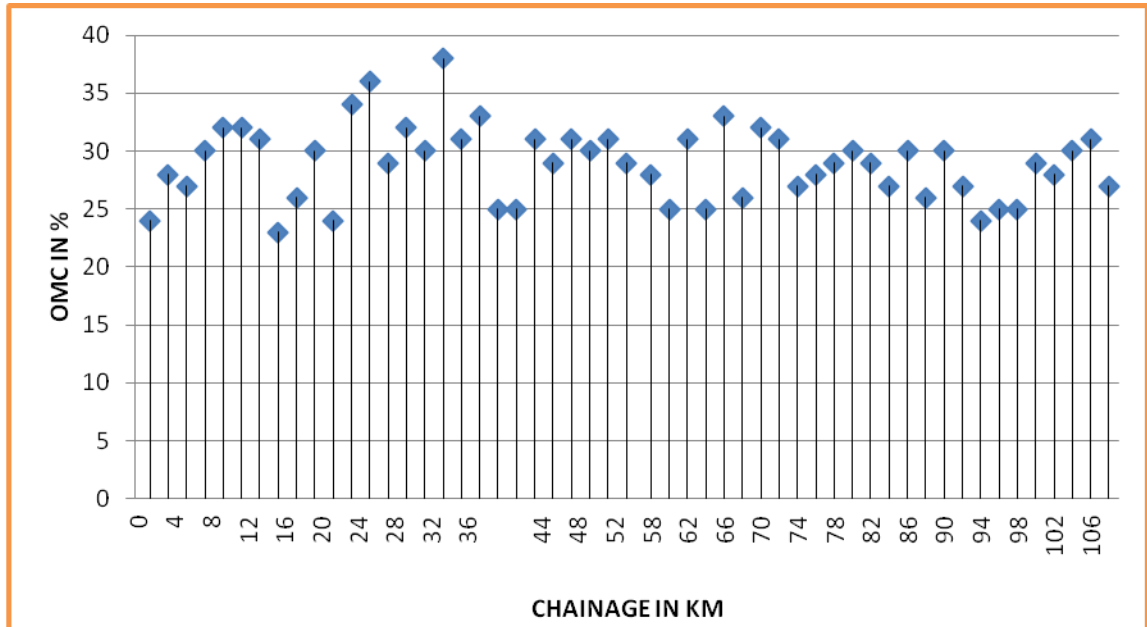


Fig.4.5 Optimum Moisture Content along the railroad alignment

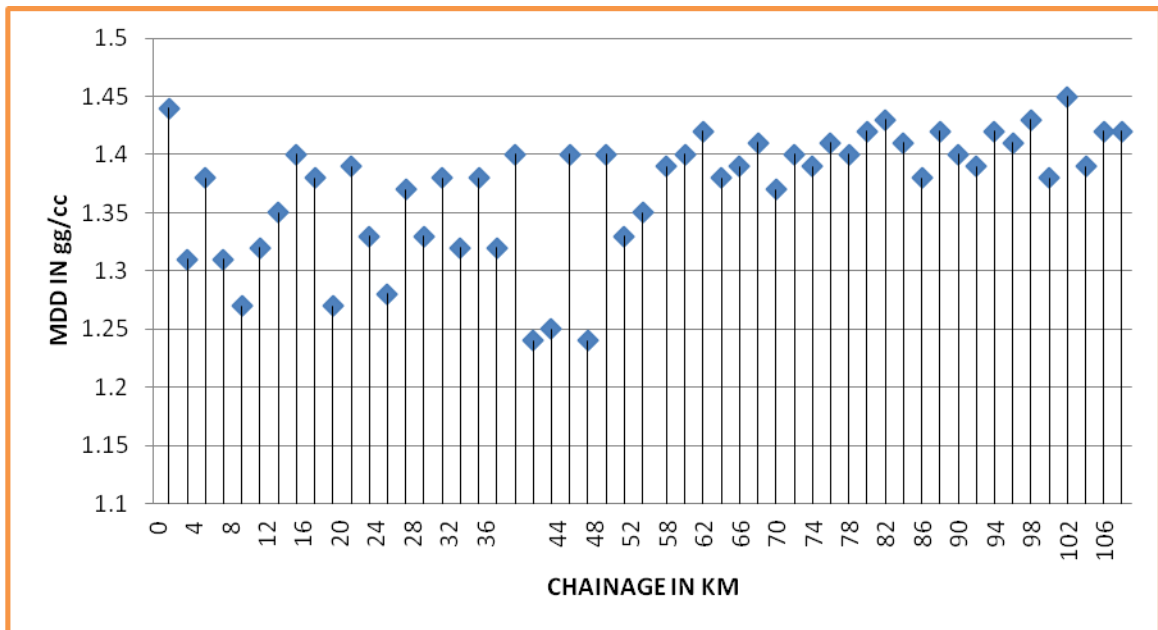


Fig. 4.6 Maximum Dry Density along the road alignment

4.5.4 Consolidation test for representative natural sub-grade soils

Laboratory determination of the consolidation characteristics of clay soils is usually carried out on saturated soil using an Oedometer. The main purpose of the consolidation test is to obtain information on the compression properties of a saturated soil for use in determining the magnitude and rate of settlement of structures (Arora, 1997).

In general, the consolidation test of a soil should furnish the following information:

- (i) the relationship between time and percent consolidation.
- (ii) the relationship between the increasing or decreasing load on the soil and the change in the void ratio of the soil data on permeability of the soil as a function of that particular load

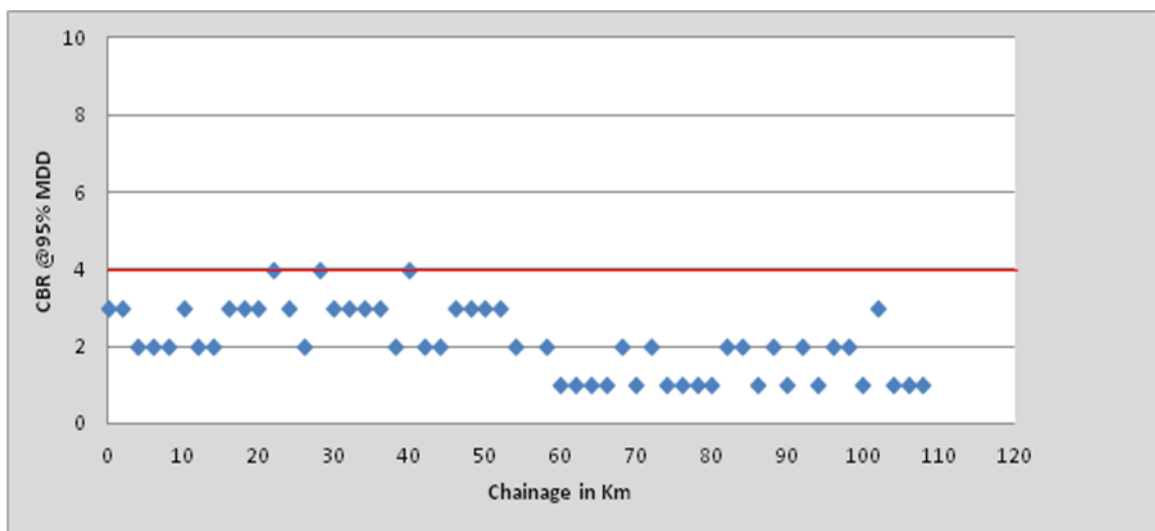


Fig. 4.7 CBR at 95% MDD along the Railway Corridor

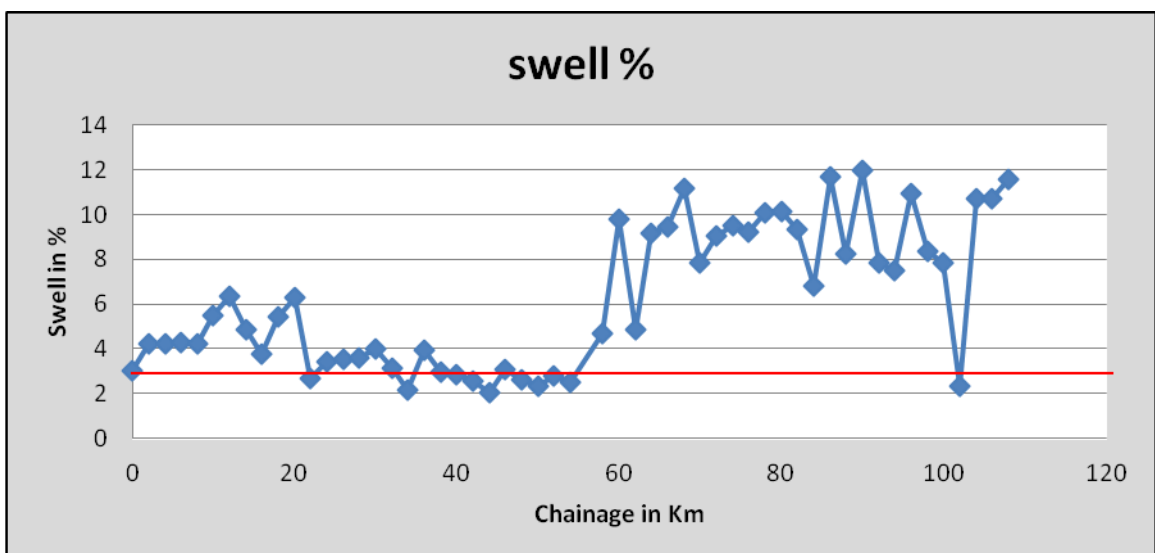


Fig. 4.8 Swell % along the railway alignment

To work out total settlement and rate of settlement of structure, this test is required. The test results presented here are only the typical ones which can better explain the overall results. The test results of all the other samples are presented in Annexure -five. The following characteristics are obtained from the consolidation test.

4.5.4.1 Compression and swelling indices

The compression index (cc) is equal to the slope of the linear portion of the ‘void ratio’ versus ‘logσ’ plot (Fig. 4.9).

Thus;

$$C_c = - \Delta e / (\text{Log}(\sigma_o - \Delta\sigma / \sigma_o)) \quad \dots\dots \text{Eq. 4.2}$$

Where; σ_o is initial effective stress, $\Delta\sigma$ is the change in effective stress and Δe is the change in void ratio.

The compression index is useful for the determination of the settlement in the field. The C_c values for reddish brown, black and yellowish to grey silty clay soil are 0.111, 0.165 and 0.224, respectively.

Swelling index (c_e) is the slope of the e -logσ plot obtained from rebound curves (Fig. 4.10).

$$\text{i.e. } (e_1 - e_2) / (\ln P_2 - \ln P_1) \quad \dots\dots \text{Eq. 4.3}$$

The C_e values for black cotton soils are 0.0006, for yellowish-grey and reddish brown soils it is the same and it's about 0.000425.

4.5.4.2 Pre-consolidation pressure (P_C)

A soil may pre-consolidate during the geologic past by the weight of an ice which has melted away, or by other geologic overburden or and structural loads which no longer exist. For example, thick layers of overburden soil may have been eroded or excavated away or heavy structures may have been torn down.

In general it is the maximum past pressure to which the sample has been consolidated. It is determined from the curves of void ratio versus log pressure curves using an empirical

graphical method proposed by Casagrande (Berry, 1987). A typical curve is presented in Fig. 4.9 which demonstrates empirical graphical method.

The P_c values for black and reddish brown clay soil is 80 and 85 KN/m^2 respectively whereas for yellowish grey soil it is 210 KN/m^2 . From the results obtained, the soils can still be consolidated in future in its natural state.

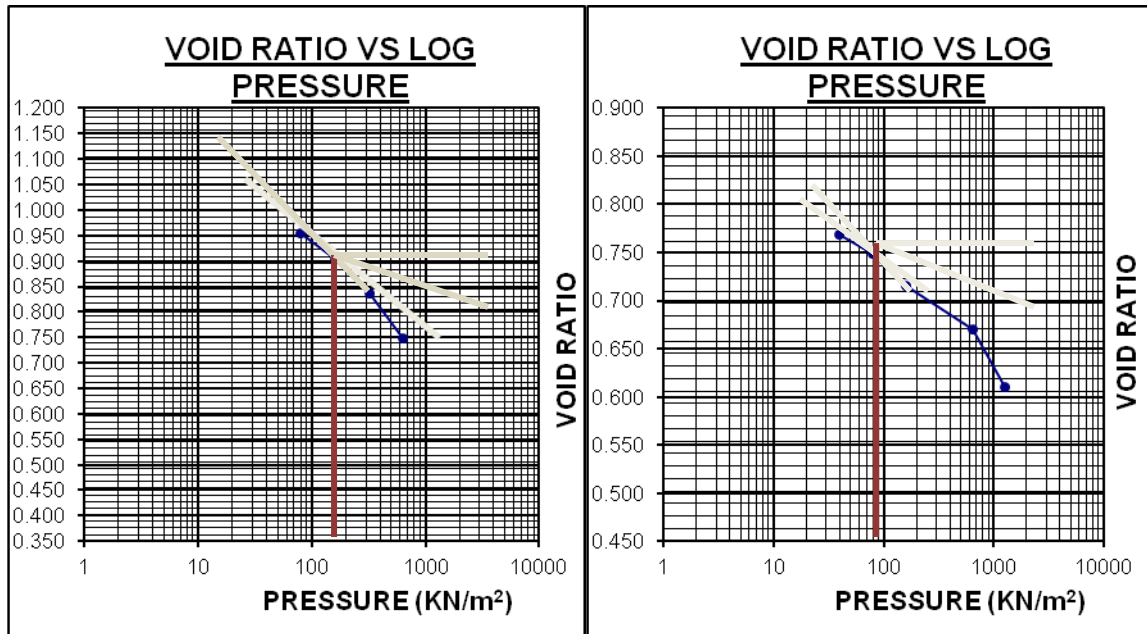


Fig. 4.9 Void ratio VS log P curves for yellowish brown/grey and reddish brown silty clay soil samples

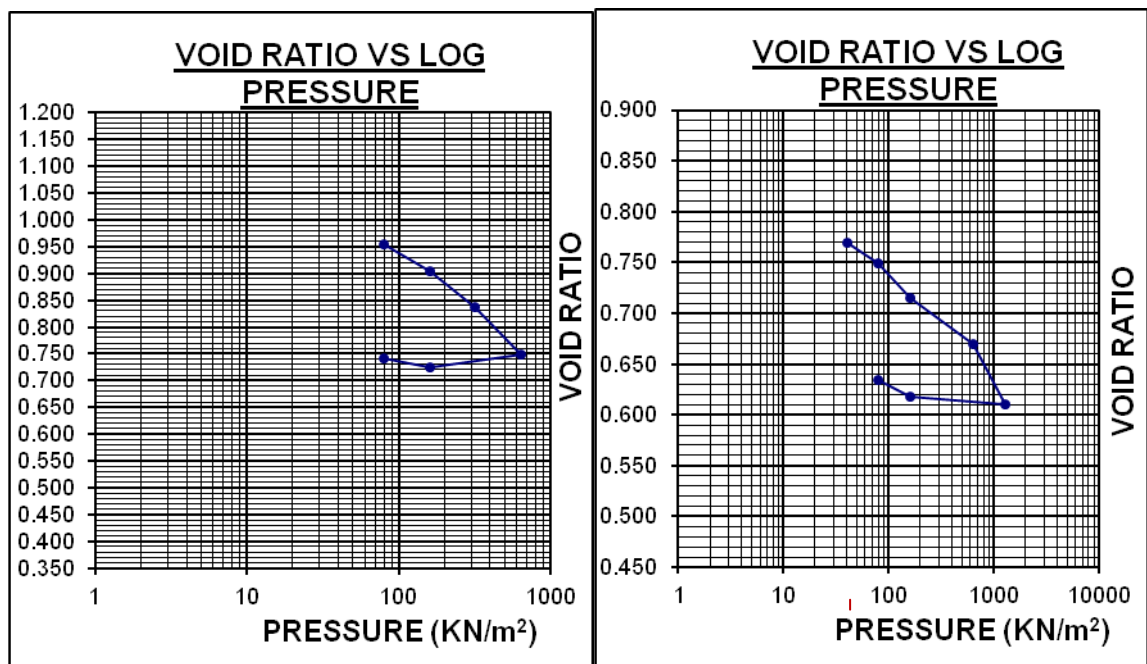


Fig 4.10 Void ratio VS log P rebound curves for yellowish brown/grey and reddish brown clay soil samples

4.5.4.3 Coefficient of compressibility (a_v)

Coefficient of compressibility (a_v) is calculated from the slope of e Vs σ graphs (Fig. 4.11) drawn from test results and is given by Eq. 4.4.

$$a_v = -\Delta e / \Delta \sigma \dots \dots \dots \text{Eq.4.4}$$

Where; 'e' is the void ratio and 'σ' is the applied pressure.

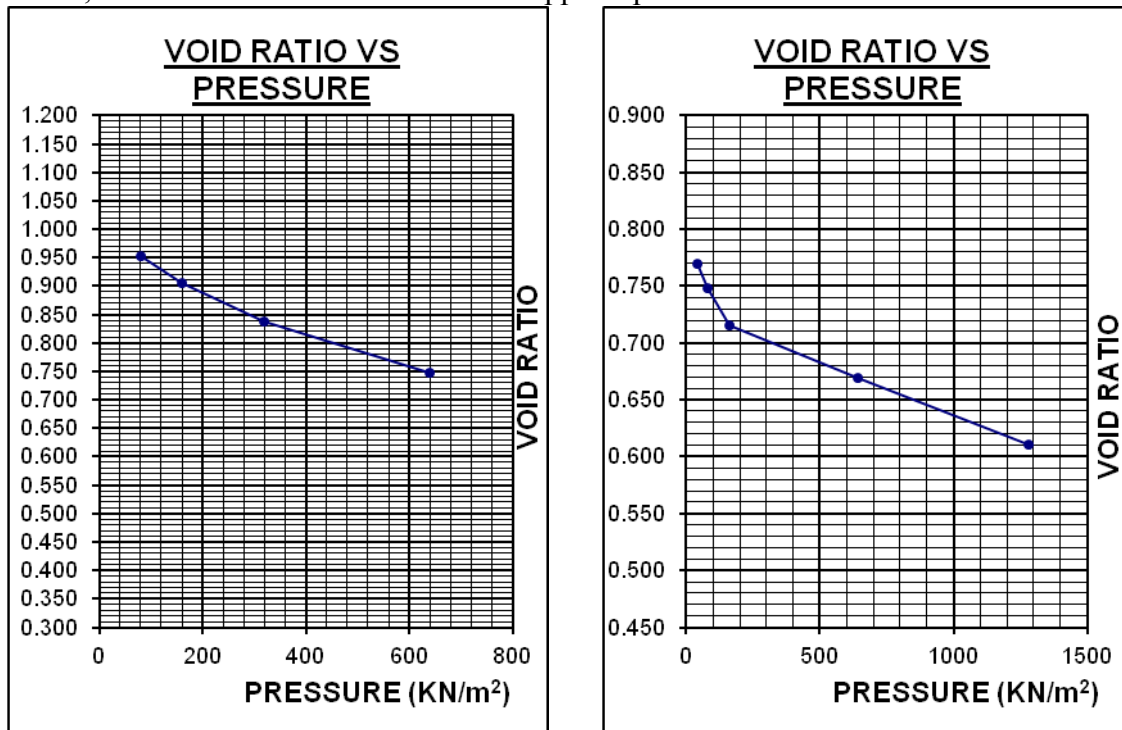


Fig . 4.11 Void ratio VS pressure curve for yellowish brown/grey and reddish brown silty clay soils

The calculated ' a_v ' average value for black silty clay is 0.0005965, for yellowish to grey silty clay is 0.000445 and for reddish brown silty clay is 0.00030285.

The coefficient of volume change (m_v) is given by the Eq. 4.5.

$$M_v = a_v / (1+e) \dots \dots \dots \text{Eq.4.5}$$

The average ' m_v ' value of soil samples are 0.0003054m²/KN, 0.000172m²/KN, 0.0002323m²/kN for black, reddish brown and yellowish grey silty clay soils, respectively.

4.5.4.4 Coefficient of consolidation, c_v

The coefficient of consolidation relates how long will it take for a given degree of consolidation to take place. There are two popular methods that can be used to estimate coefficient of consolidation, c_v . Taylor (1942) proposed one method called the square root of

time method. Casagrande and Fadum (1940) proposed the other method called the log time method (Lambe, 1969). The log time method makes use of the early (primary consolidation) and later time response (secondary compression), while the square root time method only utilizes the early time response, which is expected to be straight line. In theory, the square root time method should give good results except when non linearity's arising from secondary compression which causes substantial deviations from the expected straight lines (Lambe, 1969). The square root of time method is used in the present research work.

As proposed by Taylor (1942), the dial readings on the ordinate axis to a natural scale and then the corresponding values on the abscissa as the square root of time are plotted. A typical graphical plot is shown in Fig. 4.12. The analyzed results are given at Annexure - five.

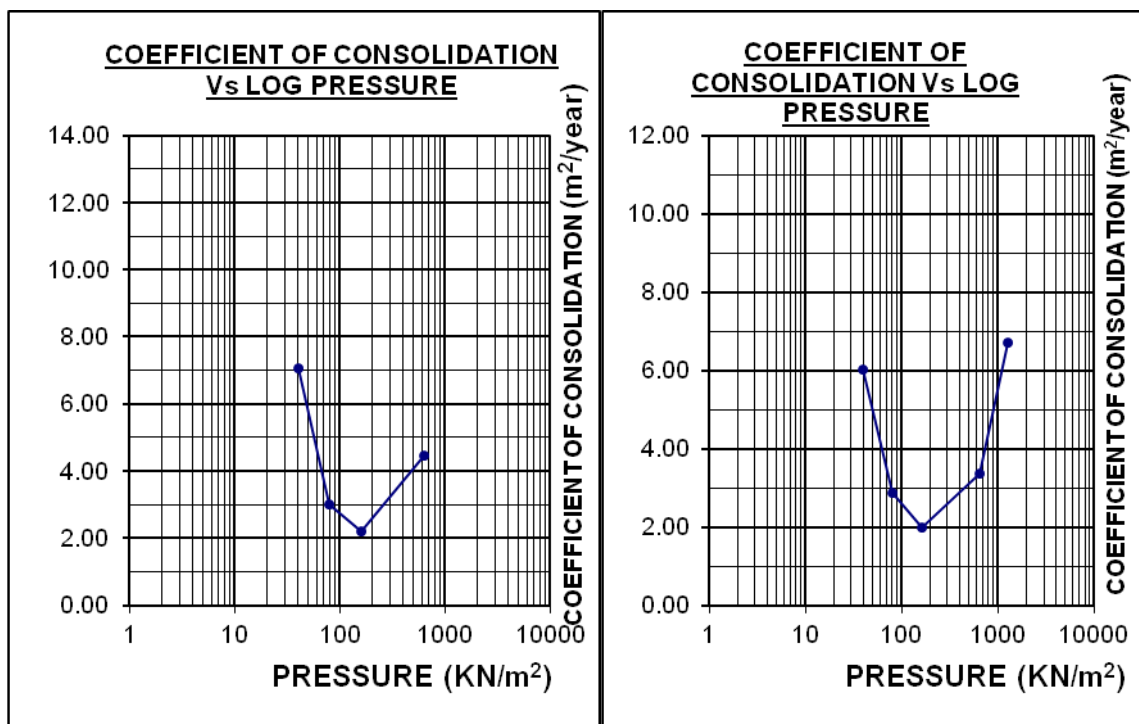


Fig.4.12 Coefficient of consolidation Vs log pressure curve for yellowish grey and reddish brown soil

4.6 Interpretation of test results

Based on the various laboratory tests conducted on soil samples collected from the field an attempt has been made to analyze the results during the present study. For this both secondary and primary data was considered. A systematic interpretation has been made in the following paragraphs.

4.6.1 Classification using index and engineering properties

Many soil properties and engineering behavior characteristics have been correlated with the plasticity index. By using the Cassagrande chart (Fig. 4.13) it has been attempted to plot the liquid limit and the plasticity index values of the soil of the railroad alignment. The swelling potential of the soil can be identified by looking at where the soil samples fall in the plasticity chart (Casagrande, 1948). For the present study, the samples tested for plasticity index were plotted on Cassagrande plasticity chart as shown in Fig. 4.13.

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). Even though most of the points are under “A” line they are following it in a closest preferred alignment. According to ASTM classification, most of the soil can be categorized as inorganic highly plastic silt (OH or MH) that can be compared to highly plastic clay (CH) having very high potential of swelling and shrinking on moisture fluctuation and A-7-5 and A-7-6 according to AASHTO.

By adopting the chart (Fig.4.13) proposed by Holtz and Kovacs (1981) it can be logically differentiated which clay mineral type is dominant in the soils of the railroad alignment. According to the chart the sub-grade soil is dominantly composed of illite and in some parts of the soil it is composed by montmorillonite.

As can be seen from Fig. 4.13, most of the points are under A-line. Being under A-line of the Casagrande plasticity chart underestimation of the degree of expansiveness and the expected swelling potential may be made. However, AASHTO plasticity chart (Fig. 4.15) and Seed et al. (1960) activity chart (Fig. 4.17) clearly indicates that natural sub-grade soil possess high to very high expansive behavior. The high expansiveness of natural sub-grade soil is also confirmed with the tested PI values (20 - 52) when compared with the published classification of degree of swelling potential (Table 4.1 and Table 4.2).

Although the above findings of the present research are not new as far as expansive soils of Ethiopia is concerned. Similar findings were also highlighted by Amiro Ambachew (2009) who carried out research on Engineering geological characterization and ground improvement of expansive soil, A case study of Metema –Abderafie Rural Road, North-western Ethiopia. Beniam Alemu (2010), Abnet Gebremedhin (2006), Amiro Ambachew

(2009) and the present study have find that the chart produced by Cassagrande under estimated the result while others specification (AASHTO plasticity chart, Seed et al. (1960) and activity chart) clearly indicates that natural sub-grade soil possess high to very high expansive behavior. Therefore, there is a need to redefine the Cassagrande chart to account for tropical expansive soil.

In order to classify the natural sub-grade soil based on its degree of expansion, the swelling potential of the expansive soils must be measured. The term swelling potential refers to the relative capacity for expansion of the different soils. The amount of swelling that may be realized in the field is a function of environmental condition. This will include whether the soil is left in place or re-compacted. It is important therefore to take into account the fact that two soils may have the same swelling potential, according to their classification, but exhibits very different amounts of swell (Seed et al., 1962 (b)).

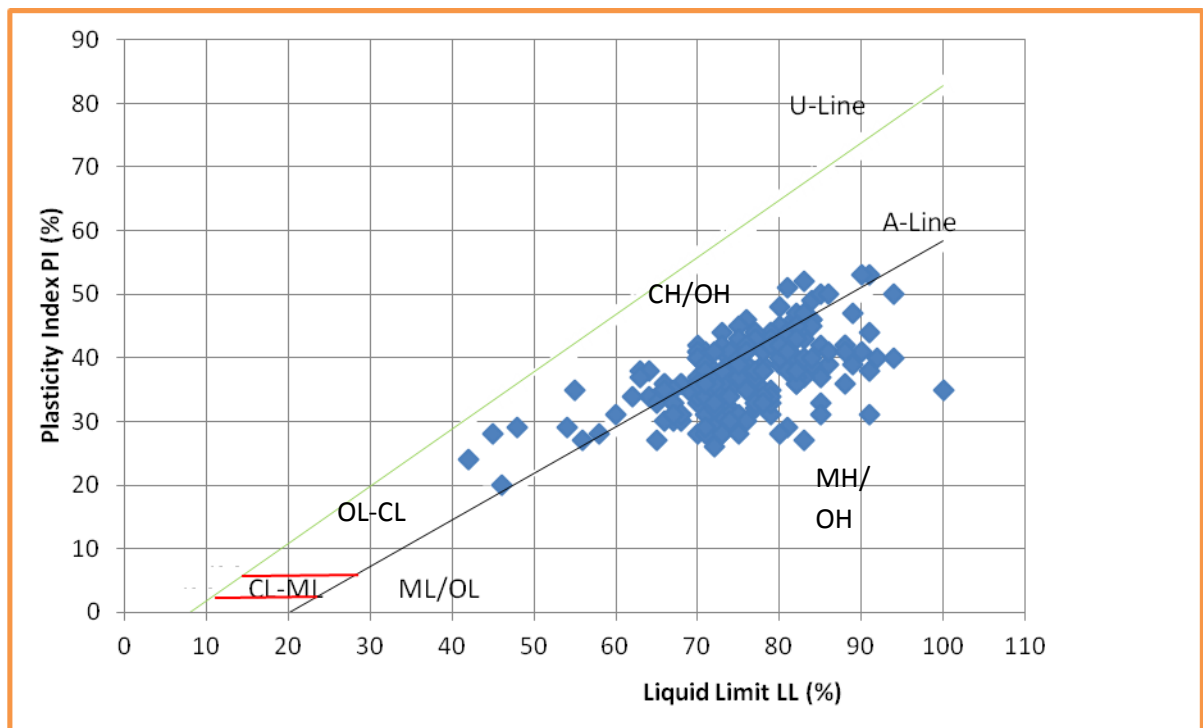


Fig.4.13 The natural sub-grade soil properties on the Cassagrande plasticity chart

Different methods have evolved to identify expansive soils based on the percentage of clay content (CC), plasticity index; liquid limit and shrinkage limit (Table 4.1, 4.2 and 4.3) (Muntohar, 2006). Soil that is susceptible for swell and long dry - wet periods are the two factors responsible for the occurrence of swell and shrinkage. The value of the shrinkage limit is a measure for the swell potential of the soil. Many classification schemes provide an

expansion rating to provide a qualitative assessment of the degree of probable expansion. Expansion rating may be something such as “high”, “Medium” and “Low,” (Ladde and Lambe, 1963). Holtz and Gibbs (1956) also demonstrated that plasticity index and liquid limit are useful indices for determining the swelling characteristics of most of the clays.

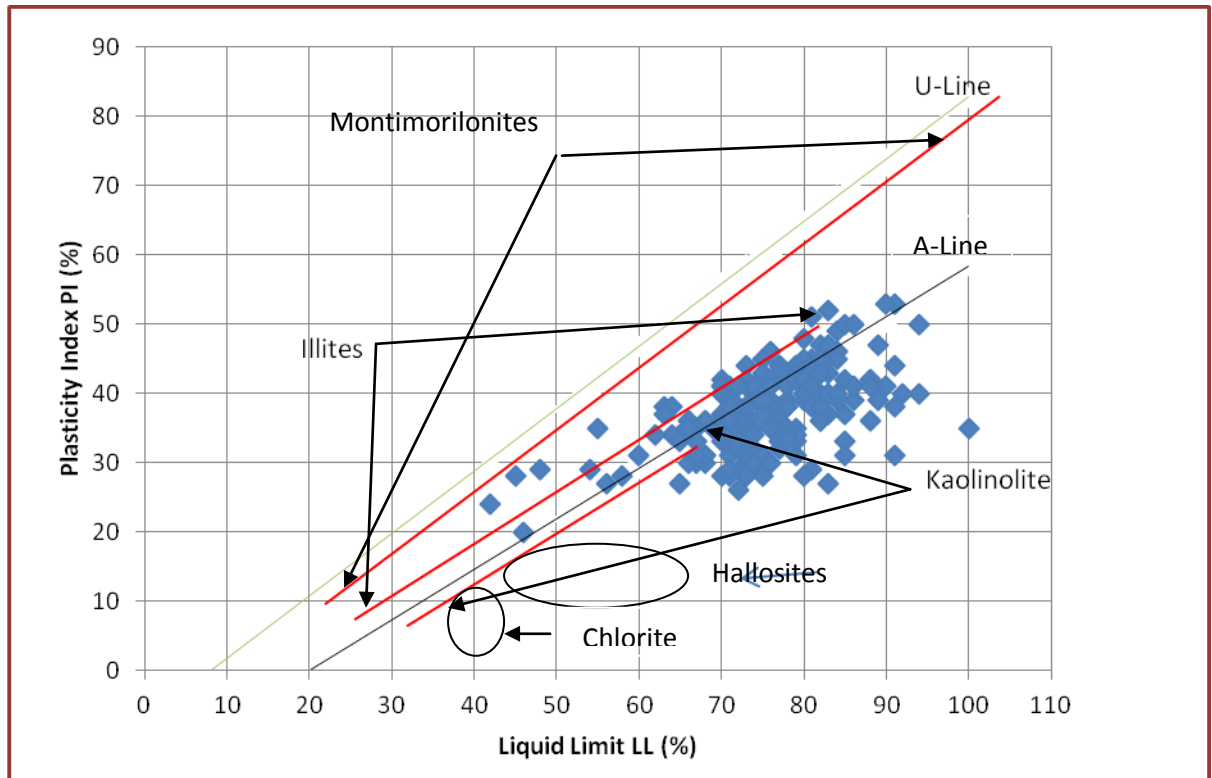


Fig.4.14 The natural sub-grade soil properties on the Cassagrande plasticity chart

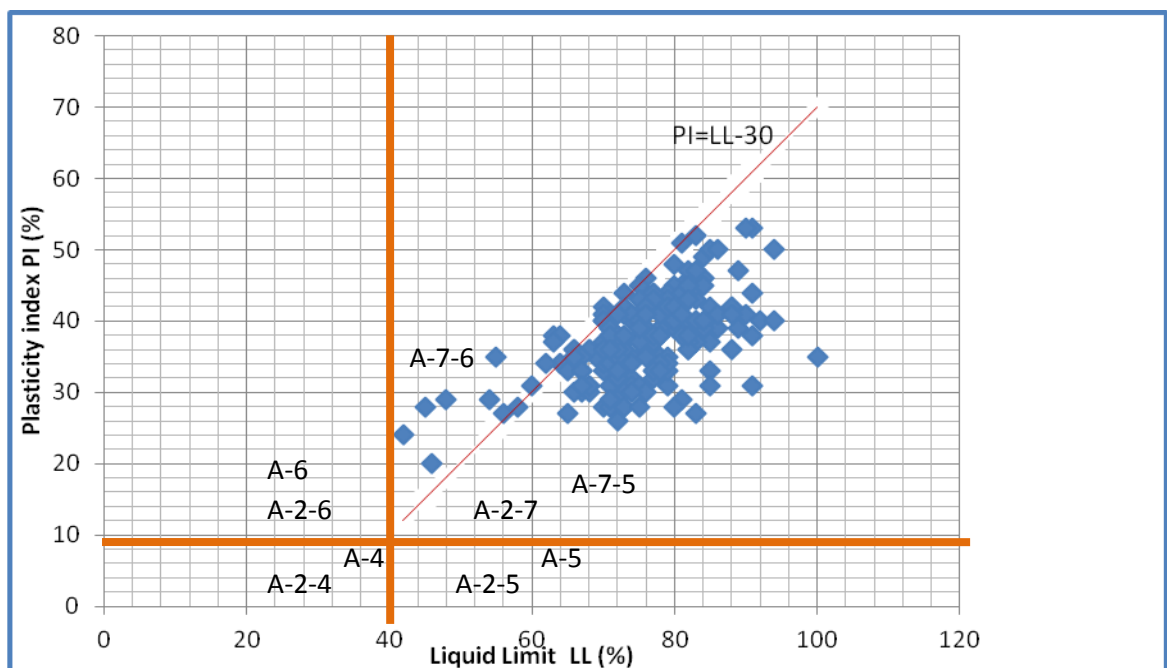


Fig. 4.15 The natural sub-grade soil properties on the AASHTO plasticity chart

Accordingly in the present study area in most of the case all the natural sub-grade soils possess high to very high degree of expansion with some exceptional value.

Table 4.1 Classification for degree of swelling potential (Source: Muntohar, 2006)

Degree of expansion	Chen (1983)	Seed et al. (1962)	Daksanamurthy and Raman (1973)	USBR (Holtz and Gibbs, 1956)
Very high	LL > 60	PI > 35	LL > 70	CC > 28
High	40 < LL ≤ 60	20 < PI ≤ 35	50 < LL ≤ 70	20 < CC ≤ 31
Medium	30 ≤ LL ≤ 40	10 ≤ PI ≤ 20	35 < LL ≤ 50	13 ≤ CC ≤ 23
Low	LL < 30	< 10	20 ≤ LL ≤ 35	CC < 13

Table 4.2 Swelling potential swell based on plasticity (Source: Holtz and Gibbs, 1956)

Swell Potential	LL (%)	PI (%)	SL (%)
low	20-35	<18	>15
medium	35-50	15-28	10-15
high	50-70	25-41	7-12
Very high	>70	>35	<11

Table 4.3 Degree of expansion base on liquid limit (Source: Ladde and Lambe, 1963)

Degree of expansion	Liquid limit
Low	20-35
Medium	35-50
High	50-70
Very high	70-90
Extra high	>90

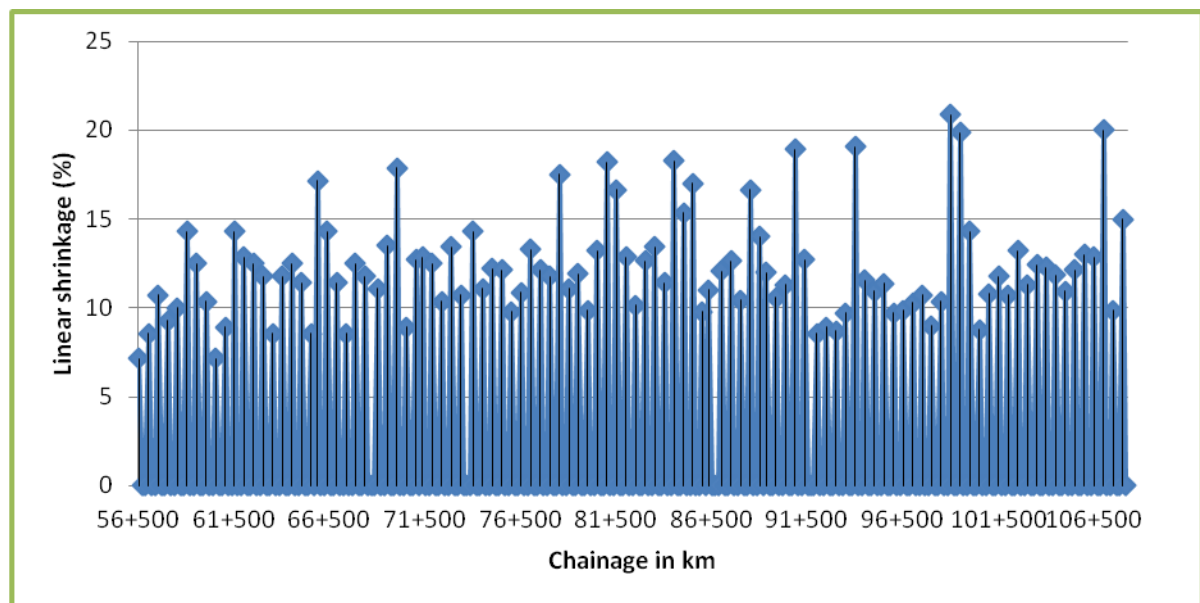


Fig. 4.16 Linear shrinkage across the Alignment

A preliminary classification based on percentage of clay fraction, soil particles less than 2 μm and plasticity index can also be used to categorize probable severity. Seed, et al. (1960), Van

der Merwe, (1964) and Skempton (1953) established useful empirical relationships between expansion potential and Physical properties of soils such as clay contents and soil suction.

Seed et al. (1962) proposed empirical relation that could enable to identify the swelling potential of soils and proposed a classification chart. The activity of clays is defined using Eq.4.6:

$$A_c = \frac{PI}{C} \quad \dots\dots \text{Eq. 4.6}$$

Where; ‘PI’ is plasticity index and ‘C’ is percentage of clay content finer than 0.002 mm by weight.

Soil with activity less than 0.75 is inactive, that with activity between 0.75 and 1.0 is active, and above 1.0 is very active. From the evaluation of the natural sub-grade soil through the activity chart of Seed, et al. (1960) it was found that the soils along the proposed alignment mostly falls into high to very high degree of potential expansiveness (Fig.4.16, Fig. 4.17).

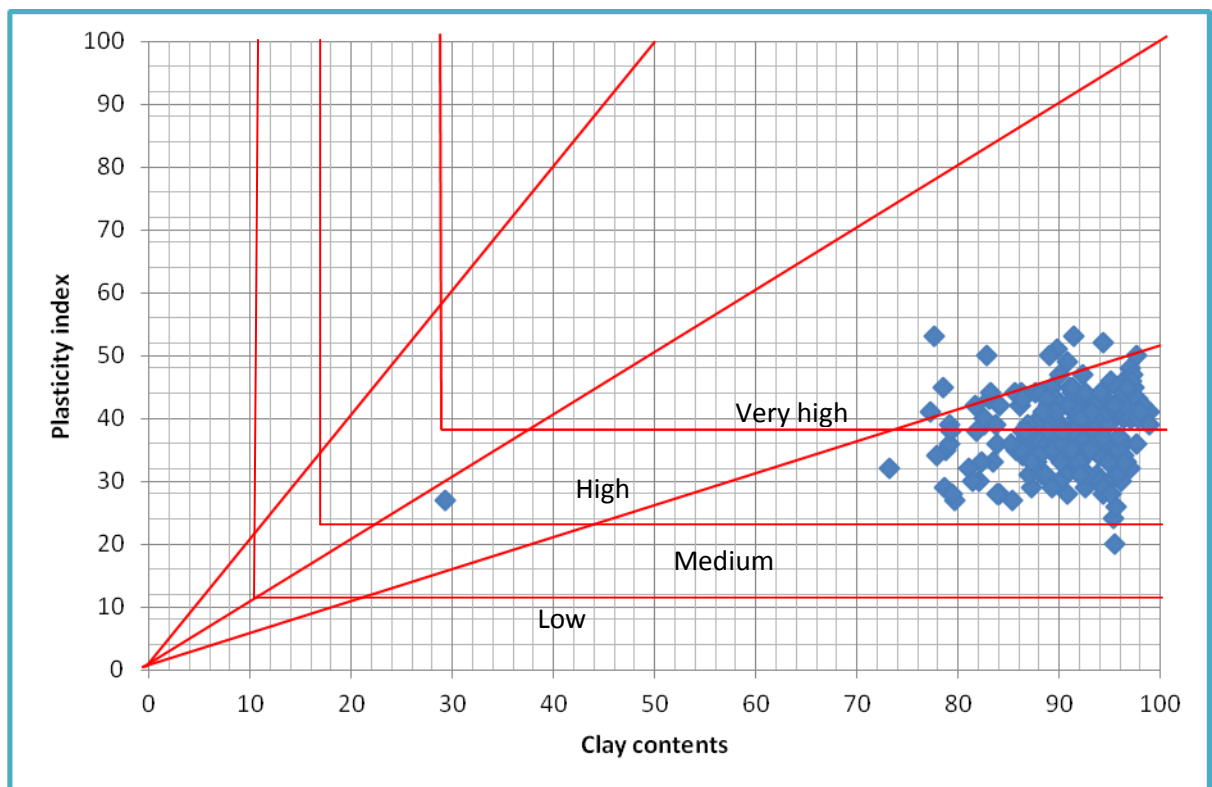


Fig. 4.17 Activity Chart for evaluation of potential expansiveness (Seed, et al., 1960).

Indirectly, the swelling potential is also estimated by using model proposed by Seed et al. (1962) which uses the plasticity index parameter only. The model is given by Eq. 4.7 and the category for swelling potential is shown in Table 4.4.

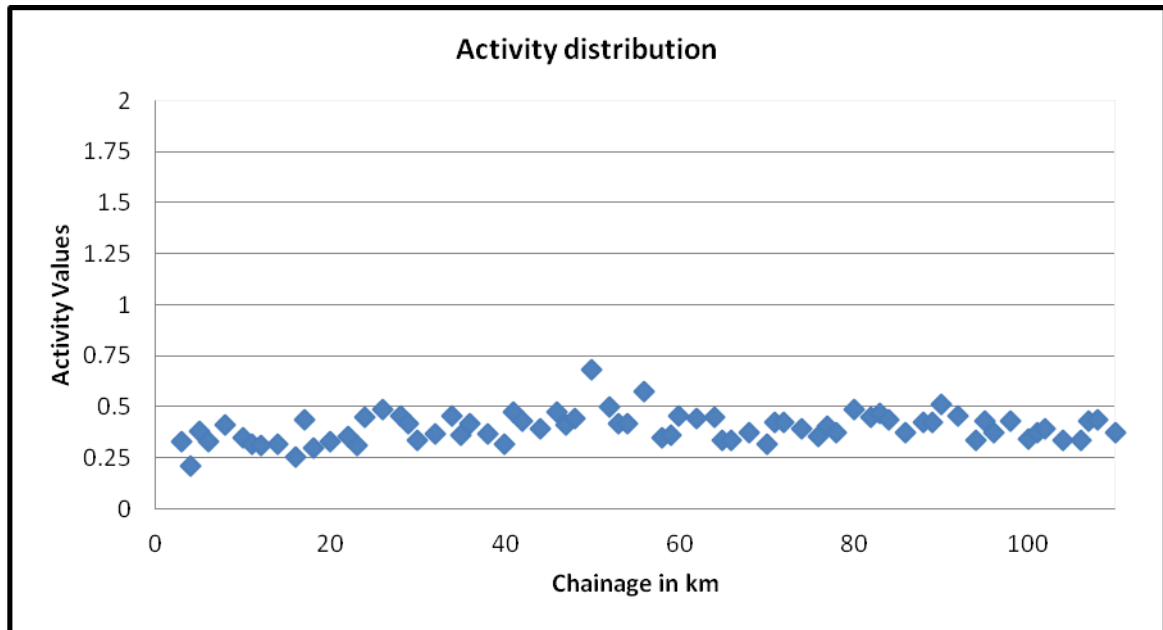


Fig. 4.18 Distribution of activity values (%) Versus chainage (Km)

$$SP = 60K(PI)^{2.44} \quad \dots\dots Eq.4.7$$

Where; ‘SP’ is swelling potential, ‘PI’ is plasticity index and ‘K’ is a constant which is equal to 3.6×10^{-5} .

Table 4.4 Category for Swelling Potential Classification

USBR, 1968 and (Seed et al., 1962)	Expansiveness
SP < 1.5	Low
1.5 ≤ SP < 5	Medium
5 ≤ SP ≤ 15	High
SP > 25	Very High

The estimated swelling potential for natural sub-grade soils of the proposed alignment according to model proposed by Seed et al. (1962) (Table 4.4) in general falls into high to very high range of expansiveness, as the natural sub-grade soils swelling potential ranges from 3 to 35 (Table 4.5, Swelling Potential as determined by Seed et al, (1962). Thus, it is imperative to conclude that the natural sub-grade soils along the proposed alignment are not suitable with respect to its swelling potential.

4.6.2 Strength characteristics of natural Sub-grade using UCS test

The allowable bearing capacity (q_{all}) of the natural sub-grade soils is calculated based on the unconfined compressive strength (UCS) tests on remolded samples. The bearing capacity is computed by applying Terzaghi bearing capacity equation (Eq. 4.8).

Table 4.5 Swelling potential of the natural sub-grade soil

Chainage (km)	Swell. Pot.	Chainage (km)	Swell. Pot.	Chainage (km)	Swell. Pot.
0+000	10.16	36+500	15.46	75+000	25.96
0+500	3.228	37+000	7.993	75+500	19.73
1+000	13.55	37+500	6.123	76+000	23.35
1+500	9.405	38+000	20.9	76+500	17.52
2+000	15.46	38+500	12.65	77+000	20.9
2+500	10.96	39+000	16.47	77+500	17.52
3+000	8.682	39+500	13.55	78+000	14.48
3+500	7.337	40+000	7.993	78+500	12.65
4+000	8.682	40+500	7.337	79+000	19.73
4+500	5.037	41+000	11.78	79+500	16.47
5+000	15.46	41+500	17.52	80+000	13.55
5+500	7.337	42+000	11.78	80+500	30.2
6+000	7.993	42+500	10.96	81+000	15.46
6+500	10.16	43+000	15.46	81+500	10.96
7+000	7.993	43+500	10.96	82+000	20.9
7+500	18.61	44+000	7.337	82+500	18.61
8+000	19.73	44+500	7.337	83+000	14.48
8+500	20.9	45+000	11.78	83+500	22.1
9+000	16.47	45+500	11.78	84+000	9.405
9+500	7.337	46+000	10.16	84+500	11.78
10+000	8.682	46+500	22.1	85+000	10.96
10+500	17.52	47+000	13.55	85+500	14.48
11+000	11.78	47+500	19.73	86+000	19.73
11+500	16.47	48+000	16.47	86+500	17.52
12+000	12.65	48+500	8.682	87+000	22.1
12+500	6.714	49+000	18.61	87+500	22.1
13+000	19.73	49+500	14.48	88+000	15.46
13+500	17.52	50+000	10.96	88+500	16.47
14+000	14.48	50+500	16.47	89+000	12.65
14+500	16.47	51+000	11.78	89+500	16.47
15+000	15.46	51+500	12.65	90+000	19.73
15+500	19.73	52+000	15.46	90+500	23.35
16+000	34.81	52+500	18.61	91+000	15.46
16+500	22.1	53+000	13.55	91+500	13.55
17+000	14.48	53+500	22.1	92+000	12.65
17+500	13.55	54+000	11.78	92+500	20.9
18+000	23.35	54+500	8.682	93+000	15.46
18+500	9.405	55+000	7.993	93+500	19.73
19+000	10.96	55+500	17.52	94+000	13.55
19+500	20.9	56+500	25.96	94+500	18.61
20+000	12.65	57+000	12.65	95+000	19.73
20+500	17.52	57+500	19.73	95+500	10.96
21+000	7.337	58+000	14.48	96+000	33.23
21+500	9.405	58+500	19.73	96+500	23.35
22+000	10.96	59+000	16.47	97+000	17.52
22+500	8.682	59+500	22.1	97+500	14.48
23+000	14.48	60+000	24.64	98+000	18.61
23+500	16.47	60+500	8.682	98+500	15.46
24+000	10.16	61+000	24.64	99+000	12.65
24+500	9.405	61+500	28.74	99+500	15.46
25+000	14.48	62+000	22.1	100+000	10.96
25+500	12.65	62+500	30.2	100+500	9.405
26+000	17.52	63+000	9.405	101+000	15.46
26+500	19.73	63+500	17.52	101+500	14.48
27+000	20.9	64+000	13.55	102+000	6.714
27+500	17.52	64+500	18.61	102+500	9.405
28+000	11.78	65+000	23.35	103+000	10.96
28+500	14.48	65+500	18.61	103+500	20.9
29+000	15.46	66+000	9.405	104+000	34.81
29+500	19.73	66+500	19.73	104+500	18.61
30+000	13.55	67+000	23.35	105+000	15.46
30+500	9.405	67+500	11.78	105+500	16.47
31+000	15.46	68+000	23.35	106+000	30.2
31+500	12.65	68+500	27.33	106+500	15.46
32+000	16.47	69+000	22.1	107+000	15.46
32+500	9.405	69+500	25.96	107+500	18.61
33+000	12.65	70+000	18.61	108+000	13.55
33+500	13.55	70+500	17.52		
34+000	6.714	71+000	24.64		
34+500	7.337	71+500	31.69		
35+000	13.55	72+000	13.55		
34+000	6.714	72+500	18.61		
34+500	7.337	73+000	17.52		
35+000	13.55	73+500	18.61		
35+500	17.52	74+000	19.73		
36+000	11.78	74+500	17.52		

$$Q_{ult} = CN_c + \gamma D \quad \dots\dots\dots Eq.4.8$$

Where; ‘C’ is the Cohesion of the soil ($=1/2q_u$), ‘N_c’ is the Bearing capacity factor, ‘ γ ’ is the Unit weight of the soil and ‘D’ is the Depth of foundation (average excavation level). The average depth of excavation to be made to the sub-grade soils is assumed to be 0.5m. The value of bearing capacity factor, N_c, is taken to be 5.14 (a value for saturated cohesive soils where $\phi=0$) (Terzaghi, 1943).

As stated above, the unconfined compressive strength (q_u) tests were conducted on remolded samples. Hence, a higher factor of safety (SF=5) is used to determine the allowable bearing capacities (Eq.4.9) of the natural sub-grade soils (Terzaghi, 1943).

$$Q_{all} = Q_{ult}/SF \quad \dots\dots\dots Eq.4.9$$

Perusal of Table 4.6 clearly indicates that almost all of the bearing pressures of the natural sub-grade soils are below the minimum value (138 kpas or 20 psi) as specified by AREMA (2010). From the overall sampled and tested data, only three samples from Station 8, 28 and 100 (in bold) satisfy the minimum requirement set by AREMA (2010).

4.6.3 California Bearing Ration for natural sub-grade soil

For the present study, the findings of laboratory investigation, as discussed in the previous section, revealed that the natural sub-grade soil is generally classified as unsuitable to be used as a railroad bed material due to its high potential for expansion. Besides, the strength property of the natural sub-grade is also unsuitable as per standard specifications. According to the laboratory three point CBR results (Fig. 4.7), the natural sub-grade material cannot full fill the requirements, as the values are less than 4%, which is the minimum requirement as specified by UIC (2005) for sub-grade material. Therefore, in addition to its expansion potential, the sub-grade material can also be not accepted with regards to CBR values.

4.6.4 Consolidation of natural Sub-grade soil

The consolidation tests have been conducted on very few representative soil samples collected from black, reddish brown and yellowish brown/grey expansive soils. The general observations made on consolidation of natural sub-grade soil, based on the available data is described in the following paragraphs.

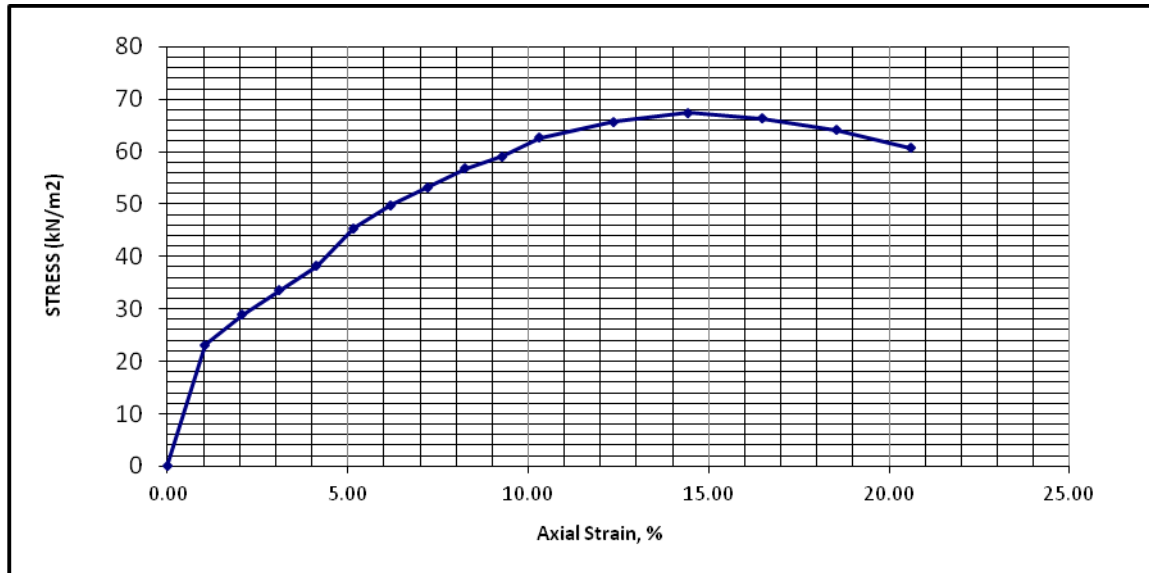


Fig.4.19 Representative UCS test graph at 50+000

Table 4.6 UCS tests and bearing capacity determination for natural Sub-grade Soils

Station (km)	Moisture Content (%)	Bulk Unit Weight (g/cm ³)	qu (KN/m ²)	C = (qu/2) (KN/m ²)	Strain at Failure (%)	Qult (kpass)	Qall (Kpas)
2	35.6	1.93	53.24	26.62	28.57	146.29	29.26
4	61.96	2.03	205.42	102.71	66.67	537.89	107.6
6	39.38	1.95	66.87	33.43	24.49	181.39	36.28
8	26.2	1.87	340.09	170.05	89.80	883.23	176.6
10	39.6	1.95	162.33	81.17	53.06	426.78	85.36
12	35	1.80	38.55	19.27	61.22	107.88	21.58
14	23.7	1.84	139.23	69.61	36.73	366.82	73.36
16	40	1.64	69.07	34.53	65.31	185.53	37.11
20	44.6	1.93	128.59	64.30	69.39	339.97	67.99
24	43.07	2.01	173.42	86.71	57.14	455.55	91.11
28	33	1.81	278.70	139.35	61.22	725.14	145
30	28.54	2.00	54.93	27.47	25.00	151.01	30.2
32	49.38	1.81	48.97	24.48	14.29	134.71	26.94
34	49.77	1.81	80.67	40.33	73.47	216.17	43.23
36+500	47	1.82	22.67	11.34	24.49	67.215	13.44
38	47	1.82	36.59	18.29	18.56	102.94	20.59
40	54.91	1.69	27.44	13.72	53.06	78.81	15.76
42	36.01	1.91	10.51	5.25	25.00	36.354	7.271
44	47.38	2.05	26.75	13.37	25.00	78.777	15.76
48	46.63	1.90	67.35	33.67	37.50	182.38	36.48
50	47.38	1.83	67.43	33.72	14.43	182.3	36.46
53	46.63	1.90	67.35	33.67	37.50	182.38	36.48
58	48.41	1.81	42.50	21.25	20.41	118.1	23.62
64	53.48	1.94	33.82	16.91	20.41	96.433	19.29
66	44.22	2.02	35.74	17.87	20.83	101.76	20.35
68	33.45	1.90	228.67	114.33	14.58	596.98	119.4
70	34.73	1.84	23.32	11.66	18.75	68.958	13.79
74	50.93	1.63	26.09	13.05	12.50	75.072	15.01
80	39.16	1.78	90.31	45.16	5.00	240.85	48.17
100	57.79	1.76	340.62	170.31	24.00	884.03	176.8

The consolidation test results revealed that compressive index (C_c) for reddish brown, black and yellowish grey expansive soils are 0.111, 0.165 and 0.224, respectively. Thus, it implies that the compression response of grey expansive soil is more than the other two materials.

Higher compressibility of the materials is associated with high settlement which may lead to differential settlement that ultimately results into development of cracks. The coefficient of volume compressibility results also revealed similar findings i.e. associated with high settlement.

The rate of settlement as observed from coefficient of consolidation results and from coefficient of consolidation versus applied pressure plots is high for both soils immediately after load application. The rate of settlement for these soils decreases with increasing load for some times. However, the rate of consolidation for the soils after a while is increased as the load application increase. Hence, the results do not show defined pattern for both soil samples. This may be due to the swelling properties of the soils under applied load.

4.7 Overall Characterization of natural Sub-grade

According to the laboratory results the sub-grade soil in general is characterized as, to fall under A-7-5 class and some A-7-6 of ASHTO soil classification; such soils are plastic clay and possess potential for high volume change under moisture variation. Hence, these soils may be rated as poor natural sub-grade soils. The engineering behavior in terms of its bearing strength and its potential for volume change under varied saturation may render major problems if not treated appropriately. The activity values of the natural sub-grade soil are below 0.75 and thus be classified as inactive clay. Though, through plasticity index the sub grade soil showed high to very high plasticity which makes this soil susceptible to volumetric change when subjected to moisture variations. Further, the CBR values for the natural sub-grade soil are found to be below 5%, which indicates very poor bearing capacity. Also, the settlement potential of the in situ sub-grade was analyzed and it was found to be normally consolidated materials. The allowable bearing capacity (q_{all}) of the natural sub-grade soils was also calculated based on the unconfined compressive strength (UCS) tests on remolded samples. The results obtained for allowable bearing capacity (q_{all}) are below the minimum allowable limit (138kpa) set by AREMA, (2010). Thus, the sub-grade soils that have bearing values less than the recommended value should be treated with special considerations. Table 4.7 presents the summary of results for overall natural sub-grade characterization and the standard specification applied to assess its suitability for intended engineering use.

In general, the above properties and their dominant values explain about the unsuitability of the natural sub-grade material to be used for railroad bed. The general characterization and all the empirical relation of the natural sub-grade soils along the proposed rail alignment

indicates that the in situ sub-grade soil needs to be improved before it is used for intended purpose.

Table 4.7 Summarized sub grade laboratory test results

Engineering properties	Ranges of values as determined during present study	Parameter Limits as specified by Standard Manuals	Manuals/ standards
PI	20-53	Max 12	AREMA, 2009
LL	42-100	Max 30	AREMA, 2009
Compacted density	-	95% of ASTM D698T	AREMA, 2009
Allowable bearing capacity	7-120kpa, only 3 samples are >138kpa	Minimum 20psi (138kpa)	AREMA, 2010
Swell	1.5-12	Max 2	UIC, 2005
Grain size	75-98, only one sample is 29%	Not more than 40% fines	UIC, 2005
CBR	1-3 but 3 samples = 4	Greater than or equal to 4	UIC, 2005

Therefore, taking in to consideration all the above mentioned engineering properties the contractor and consultants of this railway line should utilize the appropriate remedial measures. The client- Ethiopian Railway Corporation; ECR should also take the necessary measures to reduce the problems. This effort should also be accompanied by strict and professionally ethical quality assurance and supervision of construction.

CHAPTER V - CHARACTERIZATION OF EMBANKMENT FILL MATERIAL

5.1 Introduction

The presence of unsuitable natural sub-grade material may greatly influence the project budget and the design type. As far as the suitability of the natural sub-grade soil for present railway alignment is concerned it has been found that in its present state the natural sub-grade is not suitable, as discussed in detail in Chapter 4.

Based on more than 10 years of metrological rain fall data it is worth mentioning that the present study area is located in seasonally wet tropical area where annual rain fall is more than 500 mm. Such plastic natural sub-grade soils are particularly susceptible to considerable volume changes in response to moisture content fluctuations following seasonal climatic variations. Thus, this property can cause severe damages to any light civil structures unless proper measures are taken in their design. In the present study area few embankment materials borrow pits, for replacement of unsuitable soil as well as to raise the existing ground surface, when required to achieve the desired level, are distributed at different stations. The investigations thus, required were not only to find quality material of embankment fill but also need to be located borrow sites within economic haul distance. Therefore, to be economic, feasible borrow sites should be found along the route or offset of few kilometers only. Note that in the present chapter now on wards the embankment fill can also be called as sub-grade (general fill) or prepared sub-grade (structural fill).

It has already been mentioned that the proposed route alignment will be traversing along a flat terrain that is susceptible to flooding. Hence, earthwork structures such as; embankments provide the foundations for both roads and railways, to economically tolerate the magnitude of anticipated settlement and avoid adverse affects of flooding. These can be full filled by selecting suitable soil or rock to be used for construction, by controlling the placement of these materials as to location and compaction in the fill, and by designing the embankment to compensate for anticipated settlement and to full fill the required filter criteria. Therefore, the soil embankment fill (excluding slope protection) is used throughout the embankment and the slopes must be moderately flat for stability and ease of maintenance (AREMA 2009).

The embankment material is selected based on existing geological conditions of the surrounding project area and the availability of usable construction material at the site or within the economic haul distance from the project area. Thus, to assess the general quality of

the embankment material appropriate laboratory tests have to be performed on the representative samples. In order to identify locally available construction material, proper investigation and systematic study of engineering properties of the construction material for both, existing and anticipated adverse conditions is essential to assess its suitability for the desired purpose. Moreover, the site conditions also have significant effect on the performance of the selected material. Therefore, it is important to discuss the anticipated conditions in relation to the engineering properties of the construction material. Further, the dynamic behavior/ property of the material also need consideration in seismically active areas.

5.2 Construction material for proposed railway alignment

Based on the feasibility study conducted by the Classic Consulting Engineers P.L.C (CCE) for Sebeta-Ambo Railway Project, a total of 16 major potential sites (Fig.5.1) were identified for various types of construction materials required for the project (CCE, 2011).

The construction material available includes; materials for ballast, sub-ballast and for embankments fill material. From these identified sites, five sites may provide material for ballast aggregate (Q1 to Q5) and nine sites (BP1 to BP9) may provide sufficient quantity of material for sub-ballast. Besides, only two sites (BF1 and BF2) may provide material for granular embankment and fill materials. However, according to CCE (2011), since large parts of sub-grade soil of the proposed railway corridor is expansive, two borrow sites are not expected to be sufficient source for fill material. Thus, some of the selected sub-ballast quarries can also be utilized simultaneously as a source for fill material. Some of these selected materials, therefore, are characterized here as embankments fill materials. Besides, during the present research study while conducting the field data verification, additional fill granular (PBP10, PBP11, PBP12) and clay borrow (PBC1&PBC2) sites were also identified.

5.3 Embankment-fill material investigation

Borrow sources for embankment construction and replacement of weak sub-grade are required, as most sections of the present rail alignment need to be raised and the expansive soils are to be replaced. In light of this, during the geological and construction material investigation by Classics Consulting Engineers PLC (CCE, 2011/12), an investigation of embankments fill material was made in selected borrow sites to determine suitability and its available quantity as potential sources for general embankments fill material. Only two borrow area around Asgori and Ambo towns were selected (Fig.5.1) and their available

quantities were also estimated for general fill material. Based on this investigation report, the dominant soil material around the selected borrow area belongs to reddish, cone forming and highly to moderately weathered scoria.

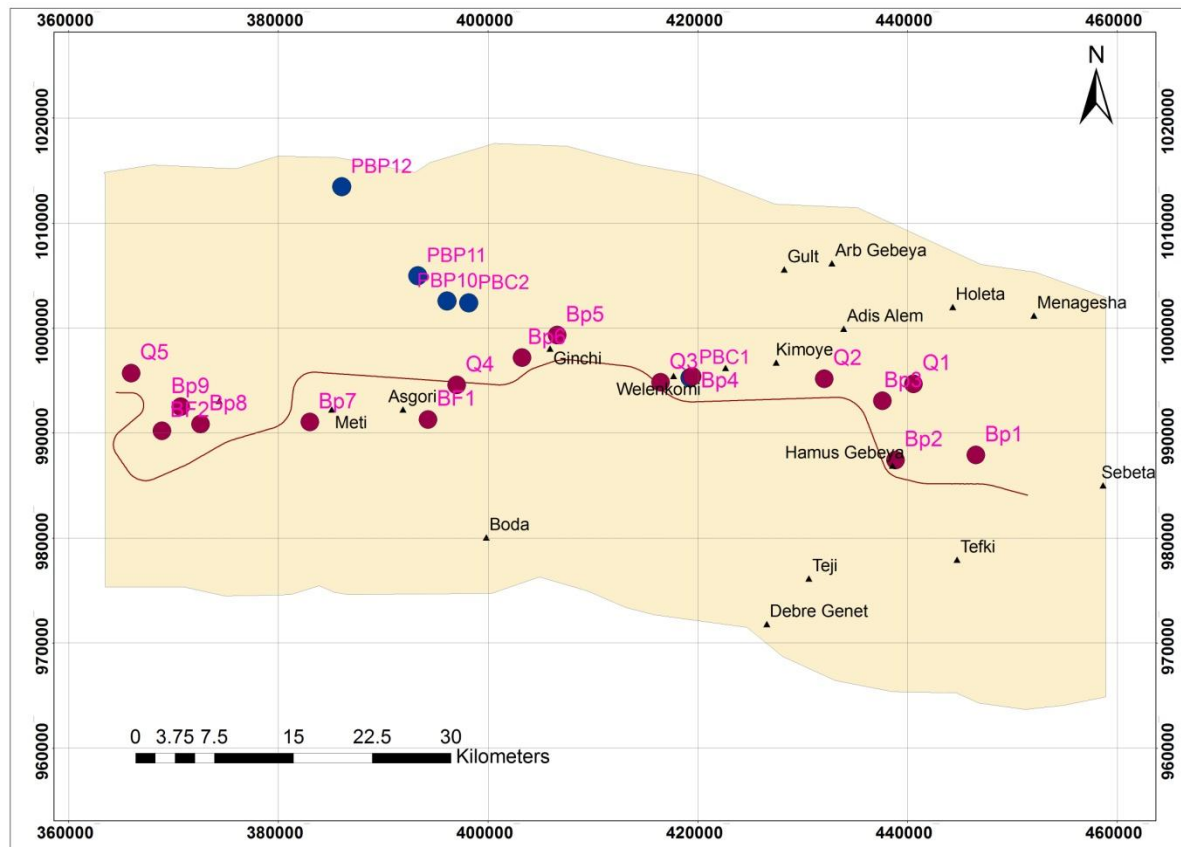


Fig 5.1 Relative location map of construction material with respect to alignment

Availability of construction material at economic haulage distance from the project area is one of the primary factors which affect the economic suitability of a project. However, according to CCE (2011/12), since most parts of natural sub-grade soil of the proposed railway corridor is expansive in nature, 2 borrow sites are not sufficient to be used as source for fill material and replacement of problematic soils. Thus, some of the selected sub-ballast quarries can also be utilized simultaneously as the source of fill material. Therefore, additional attempts were also made in the present study to characterize some of the highly weathered and fragmented sub-ballast rock unit for its suitability to be used as general fill material. Further, available quantities of fill material with relevant soil properties are the other suitability factors which need proper investigation and assessment.

During the present study, based on the laboratory test results on samples collected during the present study and the test results on soil and rock samples from previous study;

characterization and suitability of embankment-fill material has been made. For this particular attention was given to assess the general suitability of the embankments-fill materials for both general fill and structural fill. Due to the limitations on resources and time it was not possible to investigate entire corridor of the Sebeta-Ambo railway project. However, investigations were carried out along alignment from Weloncomi to Asgori (about 25 km in length). In general, the following consideration shall be given during final embankment design stage;

- (i) Quality of the material, so as to comply with the specification for intended use.
- (ii) Locations of the proposed borrow areas so as to minimize haul and obtain the most economic use of material.
- (iii) Ease of working (land acquisition, clearance of the site, access, over burden, thickness of exploited horizon, etc.

5.4 Field Investigation

During the present research study attempts were made to locate the possible source for granular fill and clay material for embankment fill (Fig.5.2). For this in total five test pits were excavated for visual observation and sampling for laboratory testing in the representative area. Two test pits were sampled from clay borrow area and three pits were made for granular fill material. During the present study, for embankment fill material investigation, in total nine samples were collected for granular fill and two samples were collected for clay material from stock pile. Various tests pertaining to classification and strength of the material were conducted and summary of results is presented at Annexure-3.

5.5 Laboratory Analysis

Laboratory procedure in analysis of embankment material is similar as that of the natural sub-grade procedure which is described in detail in Chapter - 4, therefore the laboratory procedure followed for embankment material are similar. However, for the classification purpose both AASHTO and USC (Unified Soil Classification) systems, as modified by Arora (1997) were used for the granular embankment material.

5.5.1 Laboratory Results Analysis

In order to establish the suitability of embankment fill material, the required properties which must be studied are; soil classification, density-moisture relation, one and three point CBR

with swell value and linear shrinkage.

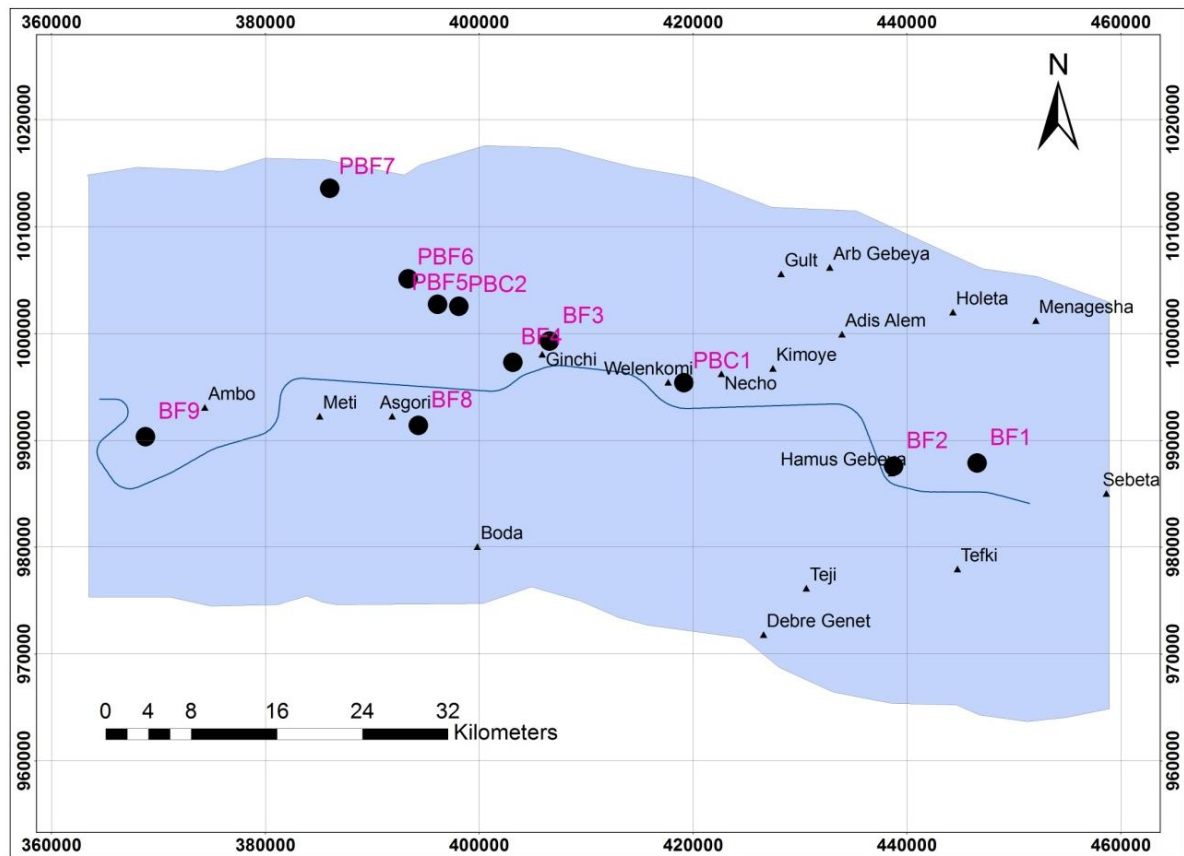


Fig 5.2 Potential sites of embankments fill material for rail railway line.

Index properties

Grain Size Analysis

For the soil classification purpose eleven representative samples were collected from the potential embankment fill material borrow sites. The results indicate that the fill material from borrow pits (PBC1, PBC2 & PBF7) is mostly composed of fine material. This material can be blended in proper proportion with other granular fill material from other borrow site which significantly lacks in fine material proportion. Thus, by proper blending of these two materials desired compaction of the embankment material can be achieved. However, further tests on various blending proportion of granular fill and fine material would be required to assess the suitability of fill material. These tests were not performed during the present study due to limitation on time and other resources.

A perusal of Table 5.2 shows the soil classification results for samples taken from different borrow area sites for embankment fill material.

Table 5.1 Potential sites for fill material – location and other details

Quarry No	Station	Offset	GPS Reading	Material Type	Overburden (m)	Est. Quantity (x 1000 m ³)	Accessibility Condition	Vegetation	Remark
Bp1	10+0	RHS 5.32km	E 446519 N 987911	Highly fragmented ignimbrite rock with welded pyroclastic deposit	0.5-1	> 400	Existing road	No vegetation	New quarry as the source of sub-ballast/ or fill
Bp2	13+0	RHS 1.7km	E 438858 N 987443	Highly welded and fragmented Pyroclastic deposit with highly fragmented ignimbrite rock	Nil	> 90	Existing dry season Earth road	No vegetation	New quarry for both fill and sub-ballast source
Bp3	19 +0	RHS 2km	E 437593 N 993079	Since it is located at the contact of Wechecha trachyte and quaternary sediment units it has the mixture of slightly welded pyroclastic material and fragmented basalt material at the foot of the ridge	1-1.5	>3,000	It need to construct access road	No vegetation	New quarry as the source of sub-ballast while the highly weathered superficial part is use as source of fill material
Bp4	54+0	RHS 1.5km	E 406571 N 999330	Highly weathered and fragmented basalt with flat top surface	Nil - 1	1,350	Existing road	Less vegetated grass land	Existing quarry, the top scoriaceous material can be use for fill while the fragmented basalt use for sub-ballast
Bp5	50+50	RHS 2.57km	E 403219 N 997180	Highly weathered and fragmented basalt with flat top surface	1-2	> 150	At the adjacent of Ginchi-Ambo asphalt road	No vegetation (farm land)	Existing quarry that can be use for sub-ballast source/or fill
BF1	64+0	LHS 3.8km	E 394265 N 991299	Cone formed moderately weathered scoria with superficial soil	Nil-0.5	4,500	It need to construct new access road	Less to moderate vegetated	New quarry that use as the source of fill material
BF2	103+70	RHS 2.65km	E 368886 N 990222	Cone forming and moderately weathered scoria material	Nil	1,800	Existing dry season earth road	No vegetation (grass land)	New quarry that can be use as the source of fill material
PBF5	55+00	RHS, 13km	E 396087 N1002586	Highly weathered, decomposed and intensively basalt.	Nil	>80	Existing gravel road	No vegetation	Existing quarry that can be used as sub-ballast/or fill materials
PBF6	55+00	RHS, 15km	E393300 N1004989	Highly weathered, decomposed and intensively basalt	Nil-0.60	>200	Existing gravel road	Less vegetated and patches of farm lands	Existing quarry that can be used as sub-ballast /or fill materials
PBF7	55+00	RHS, 20km	E386038 N1013472	Completely weathered and decomposed volcanic material (basaltic)	Nil-0.80	>320	Partially needs to construct access road	Surrounded by farm lands so need to be excluded	Existing quarry that can be used as fill materials
PCB1	38+00	RHS, 1km	E419246 N995287	Red silty clay	Nil-0.20	-	Partially need to construct access road	Less vegetated	Partially quarried that can be used as fine fill materials

Quarry No	Station	Offset	GPS Reading	Material Type	Overburden (m)	Est. Quantity (x 1000 m ³)	Accessibility Condition	Vegetation	Remark
PCB2	55+00	RHS, 10km	E398158 N100240	Red silty clay	Nil-0.50	-	Existing gravel road	Surrounded by farm lands	Partially quarried that can be used as fine fill materials

Perusal of Table 5.2 clearly shows that most of the granular embankment material (BF1,2,3,4,8,9) has been grouped into A-2-7(0) and A-2-5(0) as per AASHTO soil classification system, which represents clayey and silty granular soil, respectively. The USCS soil classification equivalence for A-2-7 is GM, GC, SM, SC and A-2-5 has equivalence GM and SM, respectively (Arora, 1997). Besides, few granular embankment material are classified as A-1-a (0); A-7-5(13) as per AASHTO system which represents gravely and sandy clay with gravely soils. These soils in USC system have equivalence with GW, GP and SC, respectively.

Further, all the fine clay borrow (BC) materials fall into A-7-5 group of AASHTO system which are mainly clayey soils. In USC system, the possible equivalence to these soils is OH, MH, CL and OL groups (Arora, 1997).

Further, the results (Table 5.2) also indicate that Group Index (GI) for most of the granular fill material is zero (value shown in parenthesis along with AASHTO class in Table 5.2). It implies presence of fine proportion within granular fill material with suitable strength property, as lower the GI value better the fill material (Arora, 1997). However, a low GI value does not always necessarily mean a suitable fill material. Therefore, analysis of other soil properties is important to understand the overall suitability of the embankment fill material.

A perusal of Table 5.2 also indicates the consistency limit results for the soil samples collected from various embankments fill borrow sites. The test results indicate that the liquid limit for all the samples is greater than 40 and the plasticity index is less than 26. Therefore, according to Ethiopian Road Manual (ERA, 2002) most of the sampled soil is suitable for prepared sub-grade material. However, according to Indian Standard, IS-2720, the soils are poor or unsuitable for the intended purpose even if some of the samples full filled the PI specification.

Table 5.2 Soil classifications results for embankment fill material

S.No.	Station	Atterberg limit			Wet sieve analysis (mm)									AASHTO CLASS (GI)	USC class
		LL	PL	PI	63	50	37.5	25	9.5	4.75	2.00	0.425	0.075		
PBC1	38+000,2km RHS	58	36	22						100	97	94	73	A-7-5(18)	OH,MH, CL,OL
PBC2	55+000,13km,RHS	59	37	22							100	97	94	A-7-5(17)	OH,MH, CL,OL
BF1	10+000,5.5kmRHS	45	37	8	100	100	72.97	57.48	33.15	24	14.7 8	8.03	6.2	A-2-5(0)	GM,SM
BF2	13+000,1.7km, RHS	42	31	11	100	100	85.22	69.32	37.04	30	17.5 9	8.67	4.2	A-2-7(0)	GM,GC,SM,SC
BF3	50+500,1.5km,RHS	45	36	9	100	100	70.98	57.28	32.28	25	18.7 2	8.52	4.6	A-2-5(0)	GM,SM
BF4	54+000,2.6km,RHS	43	31	12	100	100	80.16	64.01	37.94	28	12.2 3	6.92	4.8	A-2-7(0)	GM,GC,SM,SC
PBF5	55+000,13km,RHS	Np	Np	Np	100	98	50	45	34	28	24	8	2	A-1-a(0)	GW,GP
PBF6	58+000,15km,RHS	Np	Np	Np	100	86	68	52	28	18	15	7	2	A-1-a (0)	GW,GP
PBF7	70+000,20km,RHS	48	36	12				100	95	88	85	84	80	A-7-5(13)	OH,MH, CL,OL
BF8	64+000,3.8km,RHS	66	40	26							26.8 8	10.09	1.6	A-2-7(0)	GM,GC, SM,SC
BF9	103+700,2.7km, RHS	66	41	25							28.4 8	12.21	2.1	A-2-7(0)	GM,GC, SM,SC

Fig.5.3 and Fig. 5.4 shows the relation between liquid limit and plasticity index on a Casagrande LL-PI charts for soil samples collected from various embankment fill borrow sites. In Casagrande chart, A-line separates the clay and silts, if the fine grained soil is plotted above the A-line, it is designated as clay whereas, if it is plotted below the A-line, the soil is silt. Thus, depending on the position in the chart the silt and clay are further classified based on plasticity and mineralogical composition (Arora, 1997).

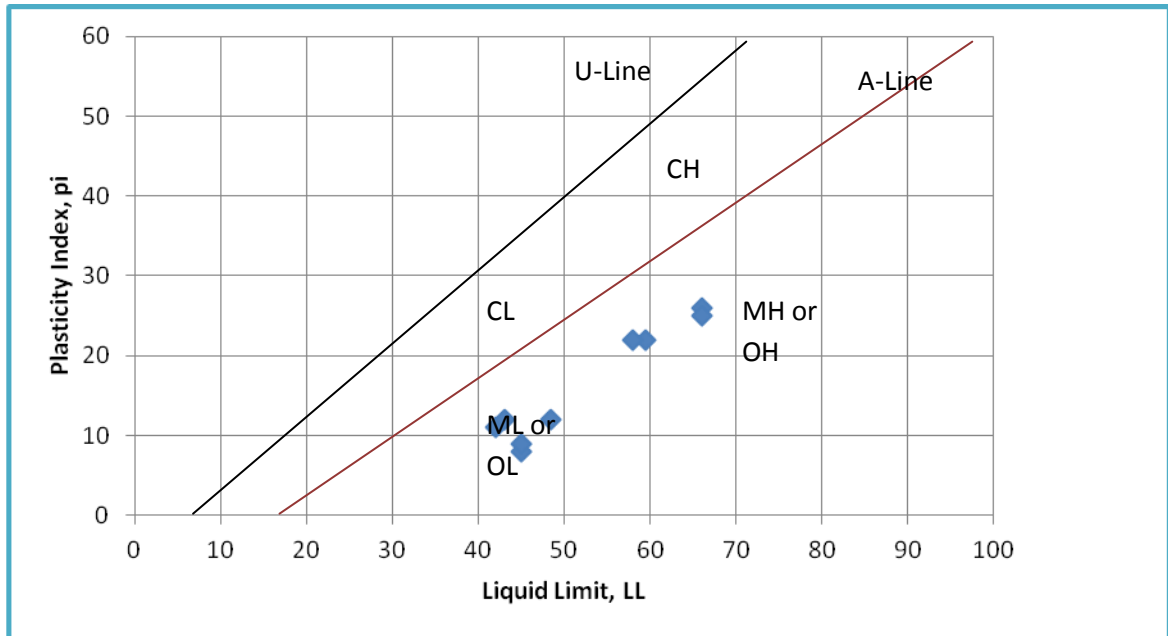


Fig 5.3 LL-PI Cassagrande chart showing plot for embankment fill material

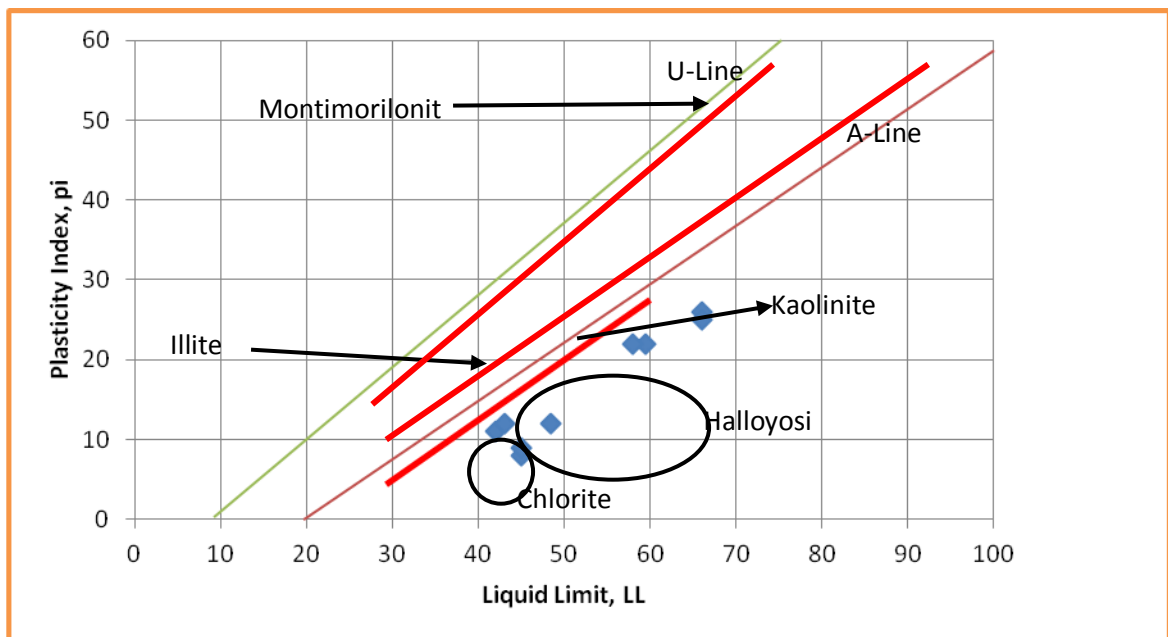


Fig 5.4 LL-PI Cassagrande chart showing plot for embankment fill material

A perusal of Fig. 5.3 clearly shows that all the soil samples are below A-line. Thus, all the soil samples can be classified as moderately plastic silt to low plastic silt.

Further, the results obtained from LL-PI charts were also compared with soil classification results. According to AASHTO soil classification the fine proportion of soil, which determines the plasticity, is silt fraction for most of the soil samples. Since much of the soil samples were retained on sieve No.10 (2.0mm size), these soil samples have been classified as silty gravel for granular fill material and silty clay for fine fill material.

As per USC System the soil groups are MH, OH, CL and OL for fine fill material. However, there is a range of alternatives for granular fill material between gravel and sandy soil class with silt and clay fines so that the soils are grouped as GM, GC, SM, SC. Further, more than 50% of granular fill soil samples are retained on 4.75mm sieve, therefore the coarse fraction of the soil is gravel. Besides, thorough evaluation of the LL-PI chart (Fig. 5.3) confirms the fine fraction for most of the soil samples is found below A-line and silt is the major fine fraction. Accordingly, silty gravel (GM) is the most reasonable granular fill soil. Further, the Cassagrande LL-PI chart (Fig.5.4) also indicates that most probable clay minerals are chlorites and halloysite in most of the soils for embankment fill.

A recap of classification of embankment fill soils indicates that according to AASHTO classification the soil samples are grouped as A-2-7, A-2-5, A-1-a and A-7-5 which in general terms are silty/or clayey gravel sand, highly fragmented gravel and clayey soil with gravel, respectively. However, AASHTO plasticity chart (Fig.5.5) shows that most of the embankment fill fine fraction is silt; therefore it is classified as silty gravel. Moreover, according to USC system all of the soil samples constitute silt fine fraction. Besides, silty gravel is also the dominant soil class as per USC system. Thus, based on the evaluation of the two classification system it may be concluded that both are providing almost similar results.

Further, based on distribution of particle size (gradation), coefficient of uniformity (C_u) was also computed for most of the granular soil samples by using equation 5.1

$$C_u = D_{60} / D_{10} \dots \dots \dots \text{Eqn 5.1}$$

Accordingly most of the samples satisfy the requirement set by IS-2720, $C_u > 7$, as shown in Table 5.3.

Table 5.3 Coefficient of uniformity (C_u) as determined for different granular soil material

Quarry No.	station	$C_{u=D60/D10}$
BF1	10+000,5.5km,RHS	46
BF2	13+000,1.7km, RHS	33
BF3	50+500,1.5km,RHS	35
BF4	54+000,2.6km,RHS	18
PBF5	55+000,13km,RHS	76
PBF6	58+000,15km,RHS	38
PBF7	70+000, 20km, RHS	-

Strength test - Compaction related properties

In order to determine the compaction characteristic of the embankment fill material representative samples from borrow area were 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 5.4 and Fig. 5.6, 5.7.

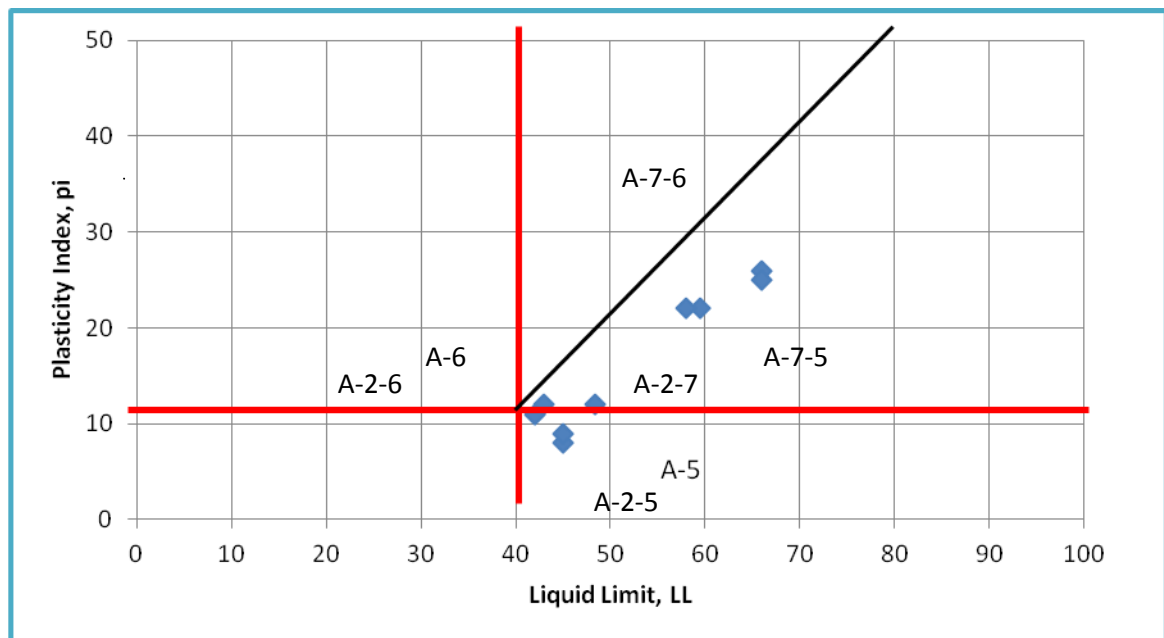


Fig 5.5 AASHTO plasticity chart showing plot for embankment fill material

A perusal of Table 5.4 indicates that in general OMC ranges from 4.6 to 32% and the dry density ranges from 1.34 to 2.2gm/cc. The material which have higher dry density at a given compaction effort generally is of high quality. According to Indian standards, IS 2720, material greater than 1.8gg/cc is suitable for prepared sub-grade soil. Thus, most of the fill material in the present case can satisfy the general suitability requirements. Also, the field dry density of the material should not be less than 95% at MDD of AASHTO T 180 D (AREMA, 2009).

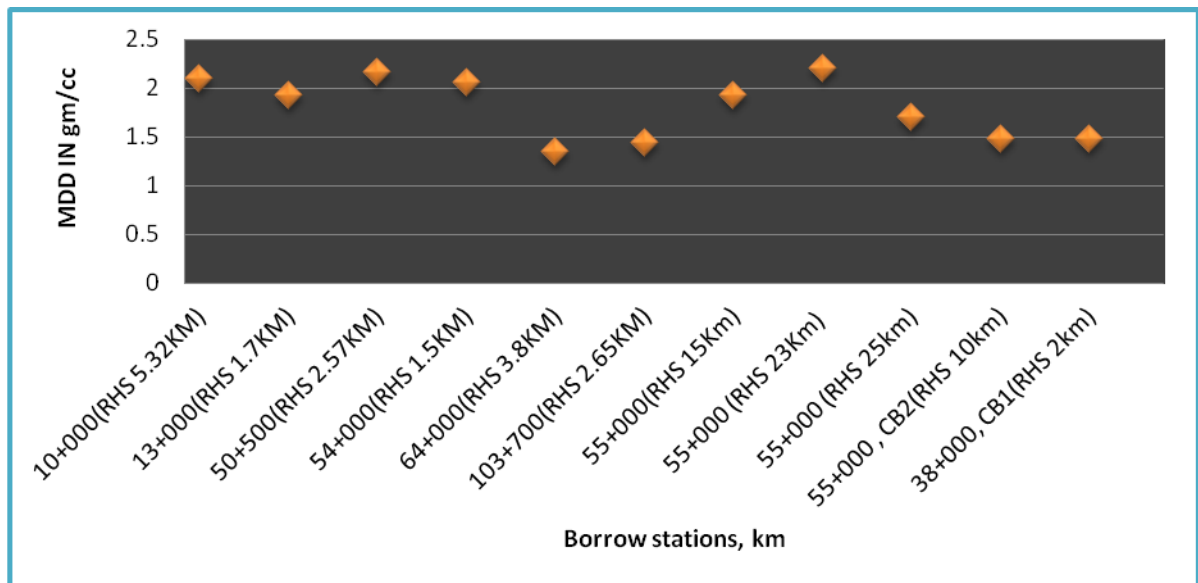


Fig 5.6 MDD Vs borrow stations

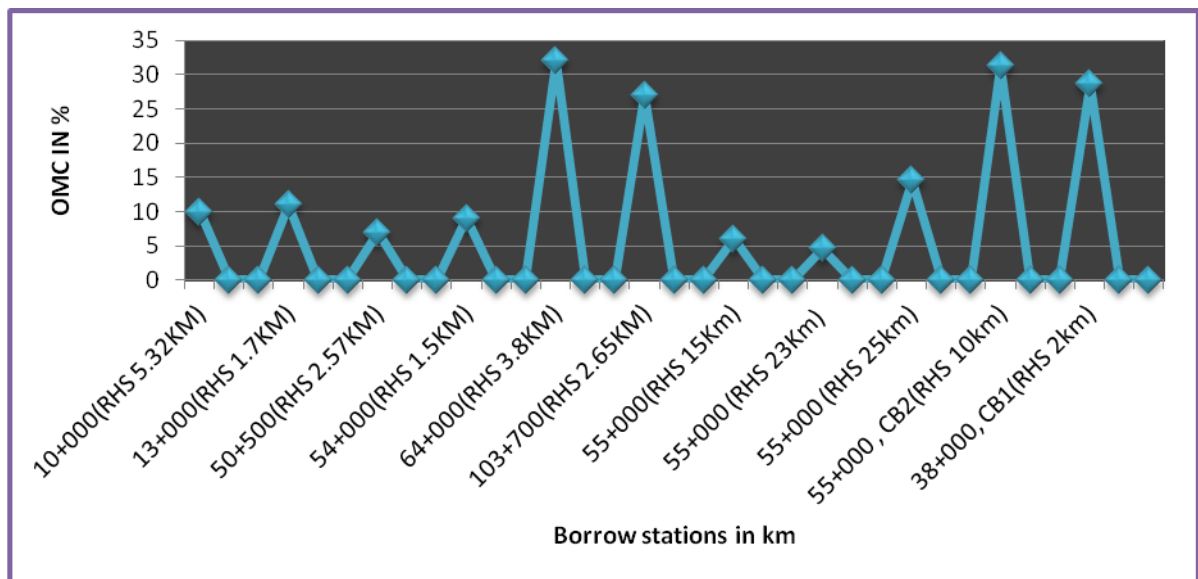


Fig 5.7 OMC vs borrow stations

The moisture content of fill material, other than rock fill, at the time of compaction shall be the optimum for the equipment being used and not more than 2% above the optimum for the material determined in accordance with the requirements of AASHTO T-180 (ERA, 2002).

CBR and Swell Potential

According to the laboratory three point CBR results (Table 5.4), the embankment material full fill the requirements, as the values are greater than 4 %, especially for granular fill material which is the minimum requirement of the UIC/ERA specification for fill material (ERA, 2002; UIC,2005).

Therefore, the material is well accepted as far as the CBR values are concerned. Moreover, the swell properties is affected by the fine fraction but it is not that much critical.

In general, the strength properties of all fill borrow sites are qualified according to both ERA (2002) and UIC, (2005).

Table 5.4 Summary of Proctor, CBR results and its manual standards

S.No	Stations	AASHTO 180 compaction				Eng. Properties	Values of Parameter	Manual Standards
		MDD	OMC	CBR	Swell			
PBC1	38+000,2km RHS	1.47	28.6	4.22	3.2	Particles finer than 75 micron	Not more than 50/40%	IS-2720/UIC
PBC2	55+000,13km, RHS	1.47	31.2	15	2.1			
BF1	10+000,5.5km RHS	2.1	10	41	0.3	Embankment s fill compaction	Not less than 95% at MDD	IS-2720/AREM A
BF2	13+000,1.7km, RHS	1.92	11	47	0.27			
BF3	50+500,2.6km, RHS	2.16	7	23	0.38	LL	Less than 35/55-60	IS-2720/ERA
BF4	54+000,1.5km, RHS	2.06	9	46	0.34			
PBF5	55+000,13km, RHS	1.92	6	56	0.08	PI	Less than 15/25-20	IS-2720/ERA
PBF6	58+000,15km, RHS	2.2	4.6	110	0.01			
PBF7	70+000,20km, RHS	1.7	14.6	25	0.16	MDD, gm/cc	Greater than 1.8gm/cc	IS-2720
BF8	64+000,3.8km RHS	1.34	32	39	0.2	CBR	Greater than 4/5	UIC/ERA
BF9	103+700,2.7km , RHS	1.43	27	25	0.23	Swell	Less than 3%	ERA

Dynamic property of embankment materials

In earthquake prone area the embankment must have sufficient resistance to overcome the effect of the dynamic load. It has to be free from susceptibility to the liquefaction potential under dynamic conditions.

The vulnerability of any cohesion less soil to liquefaction during an earthquake depends on the magnitude and number of cycles of stresses or strains induced in it by the earthquake shaking. These in turn are related to the intensity, predominant frequency, and duration of ground shaking. Seed and De-alba (1986) propose the representative number of cycles loading imposed verse earthquake magnitude as shown in Table 5.5.

Earth quakes of certain magnitude of destructiveness which had occurred in the near past, within a given area, will be expected to occur in the near future (Day, 2006). The maximum

magnitude of the earthquake that has occurred in the nearby city like Addis Ababa (less than 150 km from the project area) is 6.75mb (Samuel Kinde, 2002).

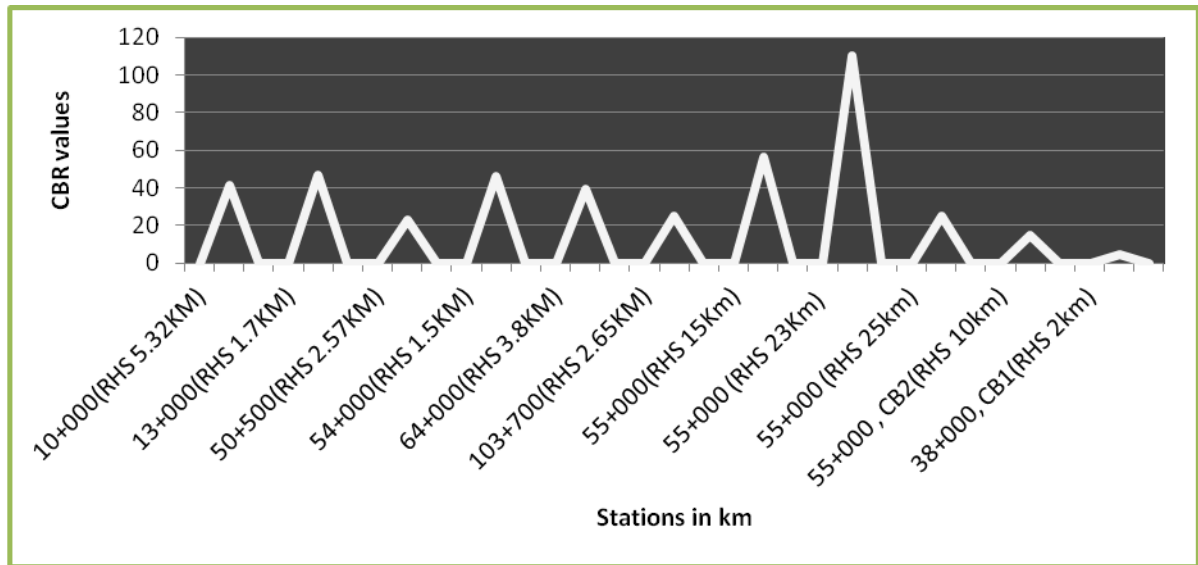


Fig 5.8 CBR Vs stations

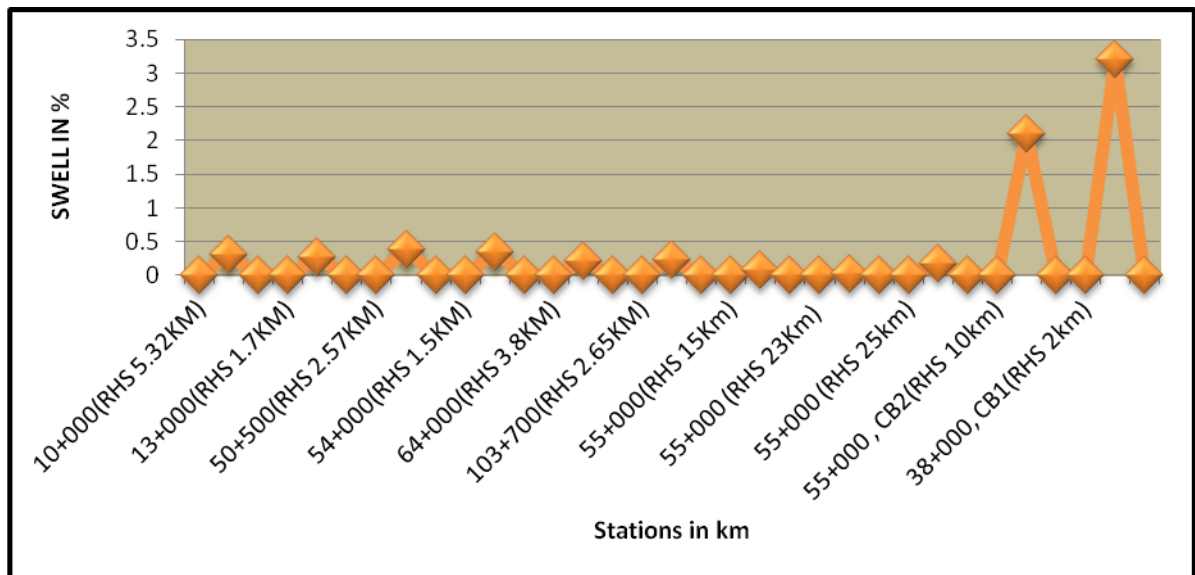


Fig 5.9 Swell Vs stations

Therefore, the corresponding cycle of loading expected to occur is 10 (Table 5.5). For conservative design the number of cyclic loading shall be increased up to 15 cycles and should be adopted for laboratory dynamic test (cyclic direct shear, cyclic triaxial, etc).

Table 5.5 Earthquake magnitude vs number of representative cyclic loading

Earthquake magnitude (mb)	Number of representative cycles	Volumetric strain ratio
8.50	26	1.25
7.5	15	1.00
6.75	10	0.85
6.00	5-6	0.60
5.25	2-3	0.40

Tsuchida (1970) summarized the results of sieve analyses performed on a number of alluvial soils that were known to have liquefied or not to have liquefied during earthquakes. Soils with a higher percentage of gravels tend to mobilize higher strength during shearing, and to dissipate excess pore pressures more rapidly than sands. However, there are case histories indicating that liquefaction has occurred in loose gravelly soils (Seed, 1968; Ishihara, 1985; Andrus, et al., 1991) during severe ground shaking or when the gravel layer is confined by an impervious layer. Accordingly, almost all of the granular embankment material is having a higher percentage passing of 2mm grain size.

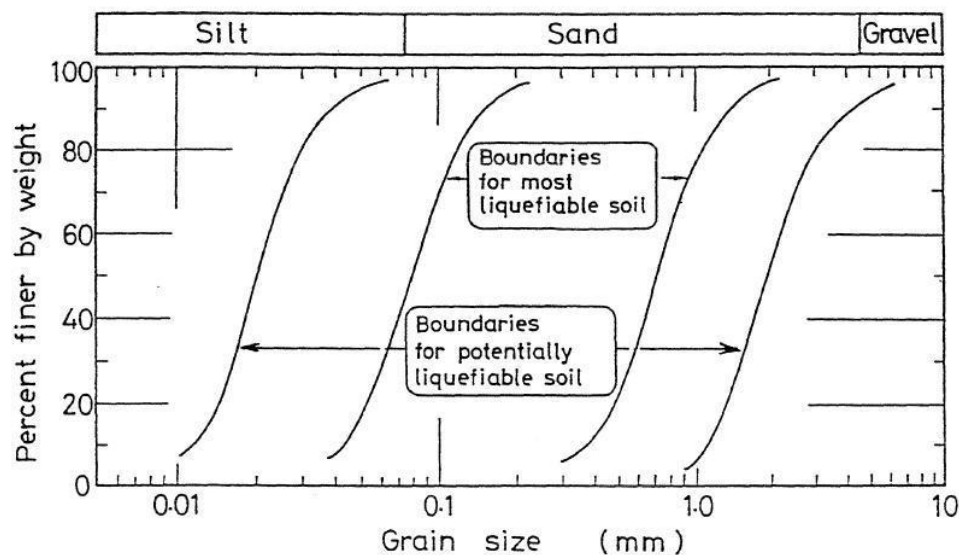


Fig 5.10 Limits in the gradation curves separating liquefiable and non liquefiable soils (Tsuchida, 1970)

Additional tests

Loss Angeles Abrasion (LAA)

Gravels 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. 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 aggregate. In accordance with the requirements of AASHTO T-96, 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.

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 5.2

$$LAA = \frac{\text{Total weight of the material tested} - \text{material retained on 1.7mm sieve size after testing}}{\text{Total weight of the material tested}} * 100$$

... Eq. 5.2

For the present study attempts were made to identify the material which may have low value of LAA by conducting specified test in the laboratory. Accordingly, the results obtained for different materials type are shown in Table 5.5. Hence, based on the acceptance limits set by the Los Angeles Abrasion Test AASHTO T-96, all granular fill material qualifies the requirement.

Table 5.6 Shows the different test results of the borrow sites

Sampling stations	Material description	Abrasion (LAA)
10+00 (offset 5.32km RHS)	Highly weathered & fragmented Ignimbrite	28
13+000 (offset 1.7km RHS)	Unwelded pyroclastic rock	34
50+500 (offset 2.57km RHS)	Highly weathered and fragmented Basalt	26
54+000 (offset 1.5km RHS)	Highly weathered and fragmented Basalt	32
64+00 (offset 3.8km RHS)	Highly weathered reddish scoria	42
103+700 (offset 2.65km RHS)	Highly weathered fine grained reddish scoria	40
55+00 (off set 13km RHS)	Highly weathered and fragmented basalt gravel	
58+00 (off set 15KM RHS)	Highly weathered and fragmented basalt gravels	
70+00 (off set 20km RHS)	Highly weathered and decomposed basalt	

5.6 Overall Characterization of Embankment Fill Material

For the present study the laboratory results of specified representative samples are mostly characterized based on soil classification and soil strength test. Thus, the general granular fill material based on AASHTO confirms that most of the soil falls into A-2-7, A-2-5, A-1-a and A-7-5 class which in general terms are silty/or clayey granular soils, highly fragmented gravel and silty clay with gravel, respectively whereas, as per the USC system equivalence includes [GM, GC, SM, SC], [GM, SM], [GW, GP] and [SC], respectively.

Moreover, analysis was also carried out using a combination of Cassagrande LL-PI chart of the material which indicates that the fine grained soil is dominantly composed of silt fine fraction. Therefore, the most reasonable soil class for granular fill is silty gravel (GM). Besides, few soil samples are classified as clayey gravel (GC) and fine fill soil as clayey silt/silty clay (MH).

The fill material has shown GI value of zero for most of the samples and the plasticity index is small, less than 15/25 (IS-2720; ERA, 2002). Therefore, the material is appropriate to be used as embankment fill and also for replacing problematic sub-grade soils. The only problem which can be anticipated with this type of material is its consolidation under the influence of dynamic loads as, the material lacks fine proportion. However, this problem can be addressed by blending of fine soil in appropriate proportion with the fill material. This perhaps require more detailed investigation to workout blending proportion

In addition, the CBR value for all fill materials is found to be greater than or equal to 4 (UIC, 2005) which indicates the good quality of the material. Almost all of the granular materials except scoria have appropriate MDD value; more than 1.8 gm/cc (IS-2720). Thus, the fill material is best to be used as embankment fill as well as replacing problematic sections.

However, it is worth mentioning that the present study area, according to metrological data obtained from the agency, is located in wet tropical zone having annual rain fall of more than 600 mm. It is also better to mention that the natural sub-grade soil material is poor. Hence, a layer of coarser grained blanket/ capping layer helps to act as separation layer between ballast and embankment fill / or sub-grade. Such a blanket layer also helps to prevent percolation of water into soil and mud pumping which reduces the variation of moisture content in embankment fills / or sub-grade soil below. Thus, it will improve the bearing capacity by modifying the stiffness and achieving a better distribution of transmitted loads on the sub-grade soil, which prevent ballast penetration into the formation (RDSO, GE-93, 2003).

Table 5.7 Summary of granular fill laboratory result

Engineering properties	Value of the parameters	Specifications	Manuals
Particles finer than 75 micron	1.61-6.2	Max 50/40%	IS-2720/UIC 2005
LL	42-66	Max 35/ 60	IS-2720/ERA 2002
PI	8-25	Max 15/25	IS-2720/ERA 2002
MDD	1.34-2.2	> 1.8gm/cc	IS-2720
CBR	4.2-56	> 4/5	UIC 2005/ ERA-2002
C _u	15-28	Above 7	IS-2720
Swell	0.2-3.2	max 3%	ERA-2002

The summary in Table 5.6 shows that the fill materials full fill most of the technical specification as per different manuals. However, due to the reason mentioned above the embankment fill as well as the poor natural sub-grade have to be capped with blanketing material. The next chapter, therefore, will discuss methods in characterizing blanket material.

CHAPTER VI - CHARACTERIZATION OF BLANKET MATERIAL

6.1 Introduction

Railway formation may develop instability 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. 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. Among various methods tried in past to improve the formation, lying of blanket has been the most successful one (RDSO GE-039, 2003).

Blanket is a layer of coarse grained material between ballast and embankment fill/ or sub-grade, spread over entire width. Blanket may be required over the formation where the soil is of poor quality, rainfall is heavy and traffic density is high, as the absence of blanket in such cases can lead to problems in services, such as swelling or heaving of formation (RDSO, GE-011, 2007).

Blanket material of desired properties, therefore, is to be selected carefully, after a thorough survey of the project area. The chosen material could be natural (from hill or river quarries), or manufactured. It could be by-product from a crusher, or bottom ash of a thermal power plant. Blending of either natural or manufactured materials could also produce the blanket material. When it is not possible to locate a reasonably economic source of blanket material, trials for either crushing sound rock material or have a blend of two or more crushed or natural materials are to be made (RDSO, GE-93, 2003).

Owing to the importance and necessity of the blanket for poor soil material and heavy rain fall area it was necessary to study and characterize the blanket material available in the study area. The present chapter therefore, addresses characterization of natural blanketing material available in the study area. The chapter has been systematically organized and presents all major steps followed during characterization of the blanket material. The borrow pits, already describe in Chapter-5 for coarse embankment fill material, are also characterized for its suitability for blanketing material.

In the present context it is worth mentioning that the present study area can be classified as wet tropical zone, since it is located where annual rain fall is more than 600 mm (according to

metrological data obtain from the Ethiopian Metrological Agency). Also, as discussed in Chapter-4 the natural sub-grade soil material, in general, has been characterized as poor. Therefore, above mentioned points makes it necessary to provide a layer of coarse grained blanket material to act as separation between ballast and embankment fill / or sub-grade material. This will help to prevent percolation of water down into the sub-grade soil and will also reduce the mud pumping. Further, this will also reduce the variation of moisture content in embankment fills / or sub-grade soil. Thus, it will help to improve the bearing capacity by modifying the stiffness and achieving a better distribution of transmitted loads on the sub-grade soil. It will also prevent the ballast penetration into the formation (RDSO, GE-93, 2003).

6.2 Blanket material investigation

For blanket material it is essential that suitable borrow pits are located, the materials are adequately tested for compliance with the specifications given and the suitable borrow areas are carefully delineated in the field (Paige, 1998).

The borrow areas for coarse embankment fill material, as describe in Chapter - 5 can also be utilized to provide blanketing material (Table 6.1). Therefore, the borrow sources are already located and adequately tested and analyzed. Hence, location of the borrow pits, material field investigation and laboratory testing procedures are similar to as described for embankment fill material.

The detailed description on all these has already been given in Chapter – 5. However, some of the properties from the standard specification which need explanation are describe and discussed in the following section.

6.3 Laboratory result analysis

Soil gradation / Grain size analysis

For the present study wet sieve analysis was carried out for the blanket material particle size distribution. This represents the distribution of particle size in the soil mass. The results thus obtained are presented in Table 6.2.

Based on the test results, almost all the material, except one, can satisfy the acceptable optimum limit of finest (3-10 micro), fines passing 75micro (RDSO, GE-0014, 2009).

Besides, the particle size distributions of the materials were also deduced from the gradation curve corresponding to the specification by RDSO, GE-0014 (2009) as shown in Table 6.3.

Table- 6.1 Sample location, designation and material types

Qry No	Stat.	Offset	GPS	Material types
Bp1	10+00	RHS 5.32km	E 446519 N 987911	Highly fragmented ignimbrite rock with welded pyroclastic deposit
Bp2	13+00	RHS 1.7km	E 438858 N 987443	Highly welded and fragmented Pyroclastic deposit with highly fragmented ignimbrite rock
Bp3	50+500	RHS 2.57km	E 403219 N 997180	Highly weathered and fragmented basalt with flat top surface
Bp4	54+00	RHS 1.5km	E 406571 N 999330	Highly weathered and fragmented basalt with flat top surface
BF1	64+00	LHS 3.8km	E 394265 N 991299	Cone formed moderately weathered scoria with superficial soil
BF2	103+700	RHS 2.65km	E 368886 N 990222	Cone forming and moderately weathered scoria material
PBF5	55+000	RHS, 13km	E 396087 N1002586	Highly weathered, decomposed and intensively basalt.
PBF6	55+000	RHS, 15km	E393300 N1004989	Highly weathered, decomposed and intensively basalt
PBF7	55+000	RHS,20km	E386038 N1013472	Completely weathered and decomposed volcanic material (basaltic)

Table 6.2 Soil classifications for blanketing material from various borrow sites

Station.	Wet sieve analysis (mm)									AASHTO Class (GI)	USC class
	63	50	37.5	25	9.5	4.75	2.00	0.425	0.075		
10+000,5.5 km,RHS (BF1)	100	100	72.97	57.48	33.15	24	14.78	8.03	6.2	A-2-5(0)	GM,SM
13+000,1.7 km, RHS (BF2)	100	100	85.22	69.32	37.04	30	17.59	8.67	4.2	A-2-7(0)	GM,GC SM,SC
50+500,1.5 km,RHS (BF3)	100	100	80.16	64.01	37.94	28	18.72	8.52	4.6	A-2-5(0)	GM,SM
54+000,2.6 km,RHS (BF4)	100	100	70.98	57.28	32.28	25	12.23	6.92	4.8	A-2-7(0)	GM,GC, SM,SC
55+000,13 km,RHS (PBF5)	100	98	50	45	34	28	24	8	2	A-1-a(0)	GW,GP
58+000,15 km,RHS (PBF6)	100	86	68	52	28	18	15	7	2	A-1-a (0)	GW,GP
70+000,20 km,RHS (PBF7)				100	95	88	85	84	80	A-7-5(13)	OH,MH, CL,OL
64+000,3.8 km,RHS (BF8)							26.88	10.09	1.6	A-2-7(0)	GM,GC, SM,SC
103+700,2.7km , RHS (BF9)							28.48	12.21	2.1	A-2-7(0)	GM,GC, SM,SC

According to the results shown in Table 6.3, much of the samples sieved cannot satisfy, for most sieve sizes, the permissible percentage pass set by RDSO GE-0014 (2009). This indicates that the materials need finer material to satisfy the required percentage pass as well as the intended compaction level. However, almost all samples except one can satisfy the percentage pass for 0.075mm sieve size.

Table 6.3 Grading Percentage of Blanket Material

S. No	Sieve size (mm)	% pass of blanketing materials for-							% pass (by weight), Cu & Cc set by manual (RDSO GE-0014, 2009)
		BF1	BF2	BF3	BF4	PBF5	PBF6	PBF7	
1	40	85	90	85	75	60	72	100	100
2	20	45	60	52	55	38	42	99	80-100
3	10	35	37	38	32	34	35	98	63-85
4	4.75	22	25	21	20	28	18	88	42-68
5	2.00	14.78	17.59	18.72	12.23	24	15	85	27-52
6	0.600	10	10	10	8	12	9	84.5	13-35
7	0.425	8.03	8.67	8.52	6.92	8	7	84	10-32
8	0.212	7	5	8	5	3	6	82	6-22
9	0.75	6.2	4.17	4.55	4.78	2	5	80	3-10
10	Cu	46.7	33.33	35.4	17.7	76	37.5	-	>7
11	Cc	3.8	4.1	3.2	2.7	1.9	5.0	-	1-3

The gradation curve can also be used to calculate C_u and C_c of the material. The following formula is used to find the coefficient of uniformity (C_u) and coefficient of curvature (C_c).

$$C_u = D_{60} / D_{10} \dots \dots \dots \text{Eqn. 6.1}$$

$$C_c = (D_{30})^2 / (D_{60} * D_{10}) \dots \dots \dots \text{Eqn. 6.2}$$

Where - D_{60} is the particle size such that 60% of the soil is finer than this size, D_{10} is the particle size such that 10% of the soil is finer than this size; D_{30} is the particle size such that 30% of the soil is finer than this size.

As shown in Table 6.3, serial no. 10 & 11, all C_u values except one can satisfy the requirement but half of the C_c values are over the limit. Hence, half of the material is well graded and all are coarse except one.

Filters

The objective of filters as seepage control measures for embankments is to efficiently control the movement of water within and about the embankment. In order to meet this objective, filters must allow relatively free movement of water, and have sufficient discharge capacity.

Filters are the most important component part of the inhomogeneous embankment section to protect piping failure. However, in homogeneous fill, the slopes can contribute to the seepage control by descending the velocity of the percolating water. This can be achieved by making the slopes of the embankment flattened (Cendergren, 1977).

According to the criteria set by RDSO-GE 0014 (2009), filter criteria is optional and can be adopted for different types of soils with blanket. Since the material used to characterize for blanketing in the present study is similar to that of material below blanketing, applying filter criteria is not as such necessary. However, flattening of the embankment as stated by UIC (2005) (1:1.5, 1:2) and correct cross-fall towards the drainage system can contribute for seepage control and erosion. Besides, according to UIC (2005), for materials having high permeable index, drainage is not as such necessary.

Material strength

Material strength is an indication of the capacity of the material to support the wheel loads of the traffic using the road. The traditional method for determining this property is the soaked California Bearing Ratio (CBR) test (Paige, 1998).

The test was conducted for all samples collected from the proposed quarry sites. A perusal of Table 6.4 clearly indicates that the CBR value for almost all the tested samples falls above the recommended value, greater than or equal to 25 (RDSO, 2009).

Table 6.4 Summary of CBR and LAA results

Sampling stations	Material description	CBR value	Abrasion (LAA)
10+00 (offset 5.32km RHS)	Highly weathered & fragmented Ignimbrite	46	28
13+000(offset 1.7km RHS)	Unwelded pyroclastic rock	39	34
50+500(offset 2.57km RHS)	Highly weathered and fragmented Basalt	47	26
54+000(offset 1.5km RHS)	Highly weathered and fragmented Basalt	41	32
64+00(offset 3.8km RHS)	Highly weathered reddish scoria	23	42
103+700(offset 2.65km RHS)	Highly weathered fine grained reddish scoria	25	40
55+00 (off set 13km RHS)	Highly weathered and fragmented basalt gravel	56	
58+00 (off set 15KM RHS)	Highly weathered and fragmented basalt gravels	110	
70+00 (off set 20km RHS)	Highly weathered and decomposed basalt	25	

Loss Angeles Abrasion (LAA)

The granular material must be tough and abrasion resistant to prevent crushing, degradation, and disintegration when stockpiled and wet, placed with a paver, compacted with rollers, and subjected to traffic loadings (Wu, 1998).

Table 6.4 shows the results of Loss Angeles Abrasion (LAA) for samples from different borrow areas.

The LAA test was conducted on 6 material samples collected from the quarry site. Based on the test results (Table 6.4), the Loss Angeles Abrasion (LAA) for all the tested samples falls within the range of 26 to 42. According to RDSO (2009) the materials having LAA less than 40 are acceptable for blanketing material. Therefore, blanketing material from all borrow areas is suitable, as far as LAA values are concerned.

6.4 Standard Specifications for Blanket Material

The specifications and thickness of various formation layers are in line with practices of UIC, AAR and other World Railways (RDSO, 2009). Hence, the general recommended specifications of blanket material (RDSO, GE-0014, 2009) are as follows:-

Blanket material should generally conform to following specifications:

- It should be coarse, granular and well graded.
- Skip (gap) graded material is not permitted.
- Uniformity Coefficient $C_u = > 4$ (preferably > 7)
- Coefficient of Curvature CC should be between 1 and 3.
- Fines (passing 75 microns) should be 3% to 10%.
- Los Angeles Abrasion value $< 35\%$, in exceptional case LAA value may be relaxed up to 40%.
- Minimum required Soaked CBR value 25 of the blanket material compacted at 100% of MDD
- The material for upper blanket layer, required for heavier axle load, shall be well- graded sandy gravel or crushed rock
- Filter Criteria should be satisfied with prepared sub-grade/sub-grade layer just below blanket layer, as given below:
 - Criteria-1: $D_{15}(\text{blanket}) < 5 \times D_{85}(\text{sub-grade})$
 - Criteria-2: $D_{15}(\text{blanket}) > 4 \text{ to } 5 D_{15}(\text{sub-grade})$
 - Criteria-3: $D_{50}(\text{blanket}) < 25 \times D_{50}(\text{sub-grade})$
- Filter Criteria is optional. This can be adopted with the experienced gained of its compliance for different types of soils with blanket.

Table- 6.5 Grading Percentage of Blanket Material

S. No	IS Sieve size (IS-2720)	Percent passing (by weight)
1	40 mm	100
2	20 mm	80-100
3	10 mm	63-85
4	4.75 mm	42-68
5	2.00 mm	27-52
6	600 micron	13-35
7	425 micron	10-32
8	212 micron	6-22
9	75 micron	3-10

6.5 Discussion on Laboratory Results

Based on the laboratory results the blanketing material has been characterized following soil classification (using particles size distribution, uniformity and gradation of soil size), filter criteria, soil strength and abrasion. The soil gradation of blanketing material indicates that, much of the samples sieved cannot satisfy the permissible percentage pass set by RDSO GE-0014, (2009). This indicates that the materials need finer fraction to satisfy the required percentage pass as well as for intended compaction level. However, almost all samples except one can satisfy the percentage pass for 0.075mm sieve size. Besides, all Cu but one and half of Cc values of the material satisfy pre-requisite set by the specification. Hence, much of the materials are coarse and well graded. The material strength also indicates that the CBR value for almost all the tested samples falls above the recommended value of greater than 25 (RDSO 2009). In addition, the abrasion test was conducted for most of the materials and all the tested samples falls within the range of 26 to 42 which attain the acceptable limit as suggested by RDSO (2009). Based on the present finding, having filter material for the proposed corridor is not necessarily rather flattening of the embankment and provision of drainage system is appropriate option to control erosion.

In general, the blanket material should be coarse, granular and well graded and should confirm the following specification.

Table 6.6 Summary of blanketing material laboratory result

Engineering properties	Value of the parameters	RDSO (2009)
Particles finer than 0.075mm	2.0-6.20%	3-10%
Particles finer than 0.425mm	6.92-12.21%	10-32%
Particles finer than 2mm	12.23-28.48%	27-52%
Cu	33-76	>7
Cc	2-5	1-3
LAA	26-42	<40%
CBR	23-47	>=25

The summary in Table 6.6 clearly shows that much of the blanket material as available from various borrow sites in the study area full fill most of the technical specification as per the standard.

CHAPTER VII - SUITABILITY ANALYSIS AND REMEDIAL MEASURES FOR THE LIKELY PROBLEMS ALONG THE ALIGNMENT

7.0 Preamble

The Sebeta - Ambo railway, single track railway, is our national Class I route, which begins at Sebeta station of Sebeta Province, via Welonkomi, Ginchi and finally reaches Ambo, specifically Guder town. The alignment extends to Kombolcha-Guraferda-Dima including Kombolcha-Hara Gebeya (direct to Boma with further extension to south Sudan) railway section that connects Addis Ababa with the Western part of Ethiopia.

The proposed railway between Sebeta and Ambo in western Oromia province has a length of approximately 108 km. With a west to east orientation, the line starts at the edge of Sebeta town, west station of Sebeta- Ambo Railway, passes along the Becho plain, and ends at Ambo specifically Guder town, following a gradient of 0-5%, 1200m and 5000m in minimum and maximum curve radius.

As the terrain is flat, it has the potential for higher average speeds. Freight trains can therefore maintain higher speeds and routes can be straighter than in hilly terrain.

The suitability of soil for a particular use depends on its response to that use. Suitability usually depends on one or more engineering properties of a soil. These properties are determined through the use of the physical characteristics and their interrelationships. The performance of engineering works will depend on the correct assessment of engineering properties to determine suitability and to predict performance of a soil for its intended use.

In the present chapter, a systematic suitability analysis of different materials, the physiographic environment and seismicity issues that may influence the proposed alignment are presented. Besides, it will also provide drainage and topographic characterization along the proposed railway alignment.

Finally, it provides the safest structural formation thickness in order to minimize the problem of sub-grade soil and the site condition (area rain fall regime). At the same time, main engineering geological problems associated with the materials encountered and the railway alignment in general are evaluated and its possible remedial measures are provided.

7.1 The Geological Environment

Geologic strata of the corridor include the Becho plain segment which is characterized by Quaternary deposit. This segment is mainly underlain by expansive silty clay soil of quaternary system that over lay quaternary basalt. Even though the route continues marching across expansive soil throughout the corridor, it also encounters mainly volcanic rocks and pyroclastic deposits in the nearby mountainous areas.

In the present study area, the geological structures such as; numbers of lineaments and a big fault was observed in the North of the corridor, which trends nearly EW. The said fault has been mapped on the geological map of the study area. Fault-related earthquakes usually have their most significant effect on foundations. These foundations can be engineered to be safe even in earthquake-prone areas. It has been discovered that, in the present study area the fault and proposed railway corridor are almost aligned side by side. The two are almost parallel within some kilometers away from the proposed corridor. The presence of this fault possibly will not produce too many route design effects. As previously recorded earthquake data shows, local earthquake effects associated with this fault have been minor in recorded history. Any movement in lifetime at the Addis Ababa-Ambo fault would likely be measured in inches, but even that should be an important consideration for the safety of the proposed railway line. The assessment of possible seismic effect due to this fault on the proposed rail alignment is beyond the scope of the present study. Therefore, it is strongly recommended to undertake additional investigations on that account in the later studies.

7.2 Physiographic Environment

The segment from Sebeta to Ginchi is known as the Becho plain section. This route is about 50-60 km in length and can be categorized as traversing lower hills, i.e. with the characteristics of low curve alignment and occupying a flat section. The section at the north of the plain, however, is bounded by steep hillsides, rounded-shaped hills, and exposed bedrock, with elevations of 2000-3300 m, and bordering hillsides of 5%-10% gradient (Fig.7.1). Generally, the proposed route in the section passes through flat and gentle slopes lands that have been reclaimed to agriculture.

Western part of the route corridor can be categorized as eroded middle-hill valley terrain, where the route gradually deviates from the Awash basin to Abay basin. This section is characterized by high raised mountain and deep valley terrain, with great topographic relief.

The route in general tries to adhere to the flat low lands which are affected by the geological structural lineament, with surrounding elevation variations of 2500-3378m and natural hill slopes of 5%-30% gradient. All in all, the topography is high in the western part of the corridor and low in the eastern as a whole, with precipitous surface and great topographic relief. In general, according to AREMA (2009), the entire proposed route passes through flat lands and slopes, less than 5% gradient. Hence, it is anticipated that the Freight trains can therefore maintain high speeds and routes can be straighter than in hilly terrain.

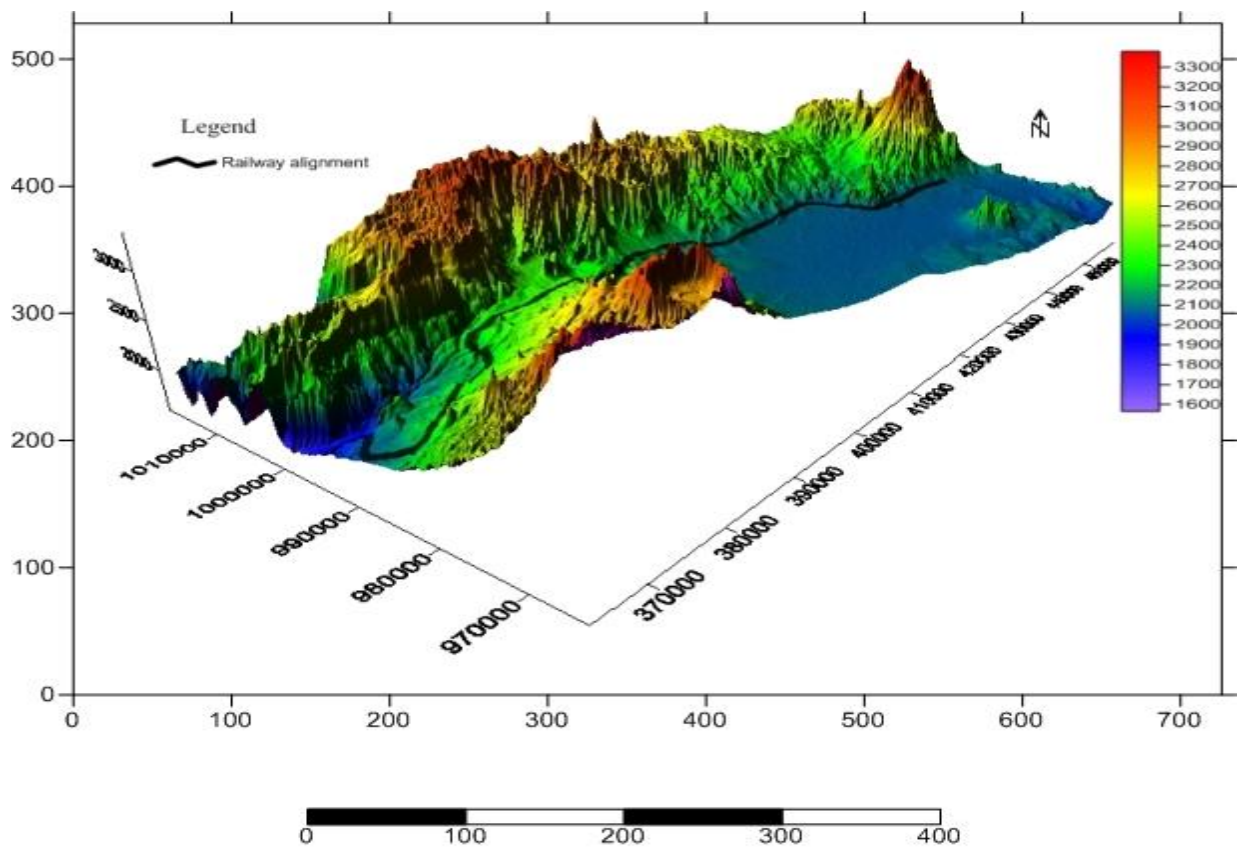


Fig 7.1 3D map showing general topography and proposed railway alignment

Even though the proposed alignment passes through flat to gentle slope areas (Fig.7.2), the railway track stretched through its own designed platform following the flat pattern. According to the designed cross section made by Classic Consulting Engineers, CCE (2010/211), the proposed rail route needs a filling of 0.2 m to 50 m and cutting of 0.1m to 48m.

These high fills and deep cuts may require special techniques and precaution. Hence, it requires special techniques to prevent the development of excessive pore pressure and ensure the stability of such fills and cuts during and after construction. Besides, due attention should

also be given for the availability of embankment material for such huge fill and the ground water potential of the area for such deep cuts. Considering the above mentioned problems the following recommendations are being forwarded through the present study.

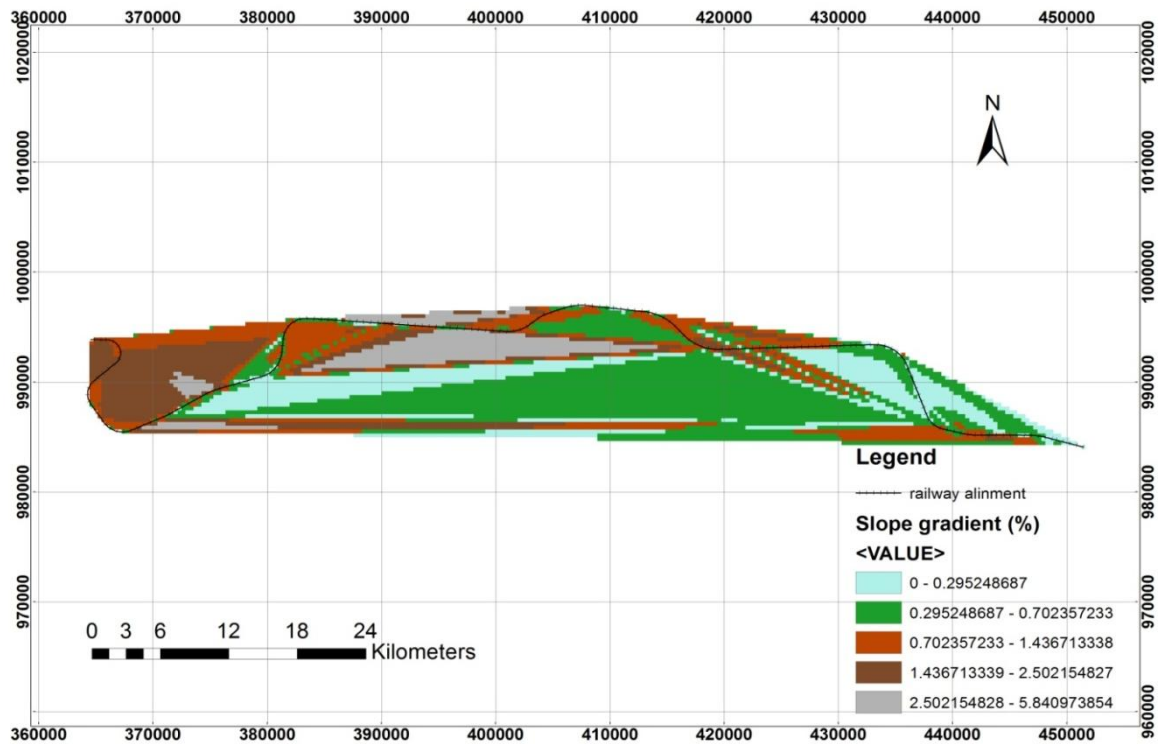


Fig. 7.2 Slope gradient along the proposed railway line alignment

Realignment: this is one of the approaches to implement; the solution is to realign the railroad to avoid such huge fills and deep cut areas. The route realignment is normally done during the reconnaissance and preliminary design phases. However, such a route realignment of some stretch in certain circumstances may result in changing huge portion of railroad design. This is because, realignment simultaneously change the general geometric design (gradient and radius of curvature) of the stretch. Hence, this could not be practical during the construction phase. Besides, the rugged topography in some portions of the areas may not favor such realignment.

Bridge over embankment (Bridge approach embankment): Whenever it is possible, it is the best solution to avoid construction of huge fill and to minimize the shortage of embankment materials for such huge fill. Since the embankment is used/act as foundation, the entire embankment section must be compacted to 95 % of maximum dry density. According to WSDOT (2003) Standard Specifications, because foundation stresses are transferred outward as well as downward into the bearing soils, the limits of the foundation

material should extend horizontally outward from each edge of the footing, a distance equal to the thickness of the fill below the foundation.

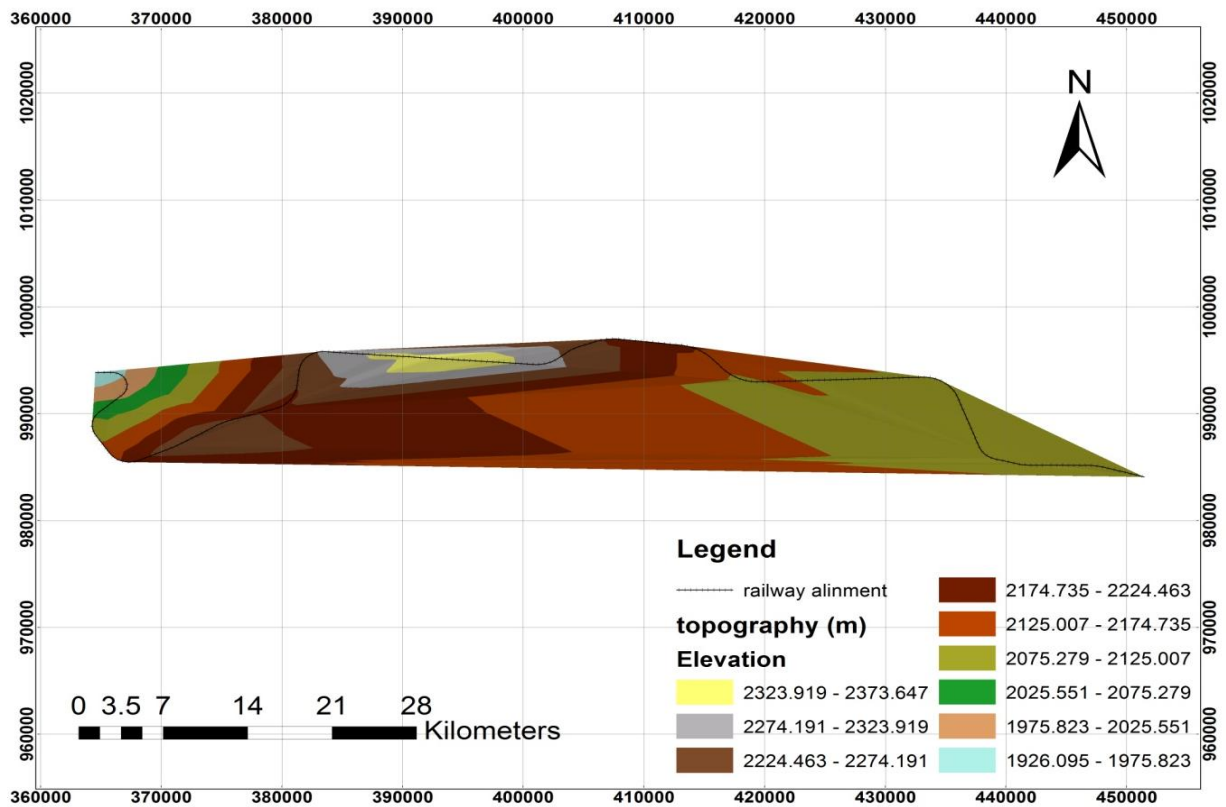


Fig. 7.3 Elevation map along the proposed railway line alignment

Tunneling: One of the options to avoid deep cutting is to provide a tunnel along the alignment. However, this is uneconomical as well as the technology is pre-mature with least experience for railway tunneling in the country.

7.3 Hydrological and hydro geological environment

The proposed rail line alignment falls primarily within the Awash River basin and partially within the Abay River basin which, in general, receives an average of 30 to 45 inches of rainfall annually, according to the data provided by the Ethiopia Metrological Agency. Each of the river systems has its own tributaries that flow through the project area. In general, the main surface water flows across the project corridor mainly towards southeast and northwest direction in the Awash and Abay River basin, respectively. Most of the streams in the study area have radial pattern however, some streams which flows from northern ridges have parallel to sub-parallel drainage patterns. The proposed project lies north of, but does not cross, the Omo River Basin and Dendi Lake. The proposed corridor will also be crossing many perennial as well as intermittent rivers which need proper over pass. A minimum of ten

stream crossings needs construction of bridges to pass the railroad and the rest may be provided with culverts.

According to the report by Water Work Design and Supervision Enterprise (WWDSE, 2007), evaluation of water resources of the Ada'a and Becho ground water basin for irrigation development project, the major groundwater aquifers underlying the project area are the lower basalt aquifer and the shallow alluvium aquifer. Lower Basalt aquifer is composed of tertiary Tarmaber basalt composed of dominantly scoraceous basalt and Amba Aiba basalt, both formation have high transmissivity value and hydraulically interconnected. The alluvial at Abay plateau, Becho plain have no significance from hydro-geological point of view. They have small extent, shallow depth and composed of silty clay sediment of low permeability. In general, according to UIC (2005), depending on local conditions, the ground water table should be lowered to a minimum distance between 0.8 m and 2.5 m below rail top.

7.4 Suitability of natural sub-grade soil

The overall suitability of the in situ sub-grade soil which has been determined from laboratory tests and field investigations indicates that the natural sub-grade soil is not suitable as far as its engineering behavior is concerned. The poor engineering properties of the natural sub-grade soil exhibits unsuitable condition for railroad construction. The engineering behavior in terms of its bearing strength, its settlement potential and its potential for volume change under varied saturation may render major problems, if not treated appropriately. The dominant soil type in the study area is silty clay for which PI value ranges from 20 to 53% (high to very high plasticity) susceptible to volumetric change when subjected to moisture variations. Based on the AASHTO classification most of the natural sub-grade soil falls under A-7-5 with some soils falling in to A-7-6 and hence rated as poor in situ sub-grade soils. The activity of the sub-grade soil ranges from 0.33 to 0.69 which is below 0.75 and thus, classified as inactive clay. Though, through plasticity index the sub-grade soil showed high to very high plasticity which makes this soil susceptible to volumetric change when subjected to moisture variations. The CBR values for the natural sub-grade soil are found to be below 5%, which indicates very poor bearing capacity. The in-situ material also shows a high settlement potential. Hence, the poor engineering properties of the natural sub-grade soil exhibit unsuitable condition for railroad construction. Therefore, such natural sub-grade soil requires appropriate soil treatment methods (AREMA 2010). Thus, attempts were made and appropriate natural sub-grade improvement has been suggested, which are discussed in detail

in the following paragraph. In addition to this the appropriate structural formation thickness based on the different country railroad formation practices is also covered below.

7.5 Suitability of embankment materials

During the present study various tests on the embankment materials were conducted and analyzed. The tests that have been conducted during previous and present study are; classification, Atterberg limits, Proctor compaction, California Bearing Ratio (CBR), CBR-swell and Loss Angeles Abrasion (LAA). The results in general indicates that the soil material collected for granular embankment fill includes GW, GM and GM, GC, SM, SC and fine fills mainly fall within OH, MH, CL and OL as per Unified Soil Classification System. Except MDD test, these materials (85-100%) fall within permissible limit as per different standard specifications as mentioned in the Chapters-6. However, according to IS-2720 the permissible limit for MDD is greater than 1.89 gm/cc, hence the MDD value of the material ranges from 1.34-2.20 gm/cc, in which only 55% is greater than 1.89 gm/cc. In either case the compressibility of the soil used is an important consideration as higher compressibility of material is associated with high settlement potential which may lead to differential settlement that ultimately results into development of cracks. For the present study, based on the liquid limit and plasticity index value, attempts were also made to determine compressibility of fill material.

According to Sherard (1995) the compressibility of material with plasticity index between 10 and 20 is medium and for those with the value having more than 20 is high to very high. The test result of granular soil, except scoria, indicates that the plasticity index value of the materials ranges from 0 - 12. Thus, compressibility of these materials is low. However, the test result of fine fill soils and the scoracious materials of the study area showed that the liquid limit of the samples are greater than 50 and their plasticity index is above 20. Therefore, according to Sherard (1995) the compressibility of the soil is high hence these materials need proper control before construction.

The dynamic property of the fill material of the proposed rail route is affected by gradation and grain shape of the gravelly material, besides the design slope. Angularity of grains is the main factor for interlocking within the materials. Both grain shape and grading will influence the ranges of void space that are possible in this zone. According to Tsuchida (1970) soils with a higher percentage of gravels tend to mobilize higher strength during shearing, and to dissipate excess pore pressures more rapidly than fine soils. Hence, the probability of

liquefaction, for the proposed embankment materials, to occur is very low. Further, improving density also plays significant role in controlling dynamic property of the material. In general, the selected fine fill materials from borrow pits can be blended in proper proportion with granular fill material which significantly lack in fine material proportion. Thus, by proper blending of these two materials (maximum of 40-50% fines) desired compaction of the embankment material can be achieved. However, further tests on various blending proportion of granular fill and fine material would be required to assess the suitability of fill material. These tests were not performed during the present study due to limitation on time and other resources.

7.6 Suitability analysis of blanketing materials

According to the report furnished by RDSO GE-39 (2003), lying of blanketing materials on problematic site condition has been the most successful way of overcoming the problems. Hence, during the present study attempts were made to characterize the capping layer using the granular fill materials. Accordingly, the result indicates that the soil gradation satisfies the acceptable optimum limit of finest, fines passing 75micro. However, the soil gradation results could not satisfy the optimum limit for 2 & 0.425 mm sieve. The blanketing material also satisfied the criteria set by RDSO GE-0014 (2009) with respect to strength and abrasion tests. However, all C_u values for blanket material except one can satisfy the standard specifications but half of the C_c values are over the limit. Hence, half of the material is well graded and all are coarse except one. According to the criteria set by RDSO-GE 0014 (2009), filter criteria is optional and can be adopted for different types of soils with blanket. Since the material used to characterize for blanketing in the present study is similar to that of material below blanketing, applying filter criteria is not as such necessary. However, flattening of the embankment as stated by UIC (2005) (1:1.5, 1:2) and correct cross-fall towards the drainage system can contribute for seepage control and erosion. Besides, according to UIC (2005), for materials having high permeable index, drainage is not as such necessary.

7.7 Ground improvement for embankment on soft and expansive soils

This sub topic briefly outlines the problems associated with expansive in situ sub-grade soils and relevant remedial measures which may be considered. The natural sub-grade, in this context, is the in situ material upon which the embankment material will be placed. Problem soils are considered to bring undesirable property during the life service of the railroad, if encountered along the route. The main problems of soils encountered in the present study

area include soils having much lower strength, soils that possess a CBR value less than 4%, expansive soils which exhibit a considerable amount of volume change upon variation in moisture content and also soils that possess settlement potential.

7.8 The expected problems associated with natural sub-grade

Railway formation may develop instability for reasons of poor bearing capacity of formation, excessive settlement, sub-grade attrition due to mud pumping and loss of soil from formation on account of erosion, ants, termites, burrowing animals etc. 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. Therefore, excessive and frequent disturbance to the levels at rail top may occur due to one or more of the following reasons:

7.8.1 Sub-grade attrition

Abrasion due to traffic and thermal forces on the top surface of sub-grade causes wear of top surface. Moreover, ballast puncture and fine particles from ballast and sub-grade create conditions of poor drainage and retention of water at the bearing surface. Ballast layer itself gets checked and track sleepers start moving up and down in grooves leading to suction of fines slurry into the grooves and pumping out of the same alternatively with the movement of wheel loads. This condition can be avoided only by deep screening and provision of a separator/blanket layer between ballast and sub-grade (RDSO 2003, GE-039). Hence, in the present study, considering the problems, as mentioned earlier, like poor drainage as a result of high rainfall in the area, poor quality of in situ sub-grade and expected high traffic density, a separator or blanketing layer has been proposed. However, in order to reduce the effect of loads transmitted to formation through ballast and to guard against the possibility of ballast penetration into the formation, it may be desirable to provide a sub-ballast of 15cm depth below the ballast layer (RDSO 2007; GE-0011). The sub-ballast may comprise of locally available coarse material so as to serve as an effective medium between the formation earths and ballast stone.

7.8.2 Low bearing capacity

The sub-grade must be able to support loads transmitted from the embankment or dynamic loading, repeated imposition of axle loads. This load bearing capacity is often affected by degree of compaction, moisture content, and soil type. This is a general problem for all the materials categorized as unsuitable, with bearing capacity of less than 138pci, regardless of

whether they are highly expansive or not. With regard to the sub-grade soil in the present study area, the bearing capacity for most of the samples tested was found to be less than 138kpa. Such soils are hence unsuitable for use as direct support of the embankment fill and axle load of the train due to their poor bearing capacity. Thus, before acceptance as foundation material, these soils may require special treatment that may include excavation and replacement or placement of embankment or improved sub-grade material of sufficient thickness on top of the unsuitable soil before placing the structure etc (AREMA-2010).

7.8.3 Consolidation settlement

Since the natural sub-grade is made up of cohesive soils, consolidation settlement is expected to take place under imposed embankment loads as shown by previous tests on settlement potential of the natural sub-grade in the present study area. Settlement predictions should show not only how fast construction should proceed but also demonstrate that any settlements, which occur after the line is opened, can be rectified by routine track maintenance (UIC, 2005). Hence, if due care has not been exercised during construction, there may be a major maintenance problem and one of the alternatives given below has to be implemented but provision of a confining layer will be the best solution. In general, since the project area is made up of cohesive soft expansive soils, the possible settlement which may be expected in the proposed alignment is both initial and primary consolidation settlement.

7.8.4 Volume change due to moisture content variation

Soils sensitive to moisture content variation exhibit a marked change in volume with minor changes in the moisture content. As such, they may not lack in shear strength and behave well when isolated from the effects of precipitation and evaporation. The distress to track is mostly experienced during dry season when shrinkage cracks of the soil become wide enough to consume large quantities of ballast or embankment fill material. The expansive potential of the sub-grade soil in the present study area can be classified as high to very high as discussed in Chapter 4. Further, plasticity of a soil is an indication of the expansive nature of the soil. Accordingly, the sub-grade soil of the present study area is classified as high to very high plastic. Thus, in turn the sub-grade soil needs due consideration for its expansiveness. Hence, the best remedy against the problem of volume change lies in confinement of these soils within a nearly impermeable layer on all sides or by adopting any other feasible improvement techniques. Draining of the logged water from the marshy stretch is also very important as there are such stretches in the study area. Hence, it is necessary to reduce the capillary effect

of the plastic clay layer. This can be achieved by using selected fill of granular nature material which has wide inter-granular opening and possess relatively less or no capillary rise potential.

7.8.5 Loss of soil from formation on account of erosion

Highly expansive clays, such as black cotton soils, when dry exhibit a sand like texture and are susceptible for erosion to a much greater extent than that normally anticipated from their plasticity and clay content (ERA, 2001). Regarding the present study area, it contains high to very high expansive nature clay soils. Besides the route is stretched along the flat terrains which is susceptible for erosion. Hence, appropriate remedial techniques have to be adopted to minimize the problem.

7.8.6 Progressive Shear Failure under dynamic loading

Stresses imposed on the sub-grade top by the axle load may be large enough to cause a progressive shear failure (general shear failure). This condition is most likely to occur on the top most part of the sub-grade where the effect of the imposed live loads is the greatest. Repeated imposition of axle loads (known as dynamic loading) could cause accumulation of strain in the soil. If the imposed stress is higher than a threshold limit, such accumulation is very rapid and the soil fails to carry the load safely.

During the research conducted over the last past years, it has been possible to establish a concept of threshold stress for cohesive soils, and a design theory based on this concept. Simply put, the design of embankment has to cater to limit the imposed stresses on top of the sub-grade to below the threshold stress of the soil (RDSO's TM -54).

For the present study, progressive shear failure under dynamic loading has not been studied due to limitations on resources however it is suggested to carry out studies for any possible shear failure in the sub-grade before making any final design considerations.

7.9 Ground Improvement Techniques/ Soft Ground Improvement Methods

Different literatures and design manuals including RDSO: 0014 (2007), JNR (1980), UIC (2005) proposes different alternative remedial techniques for tackling the problem associated with such soils during construction and/or service performance of road projects. These techniques can be adopted to improve the ground strength on which the embankment/fills are

constructed. The underlying soil for fill is required to satisfy the same basic requirements of a continuous spread foundation system (RDSO/GE: 0014, 2007).

The underlying soil should have the strength to support the proposed embankment and live loads with an adequate safety factor. In addition, the embankment/fill needs to be designed and constructed such that it can tolerate the projected degree of settlement. It is occasionally necessary to remove and replace portions of weak or highly compressible underlying soil or to improve their characteristics by using stabilization procedures or controlled construction techniques.

The appropriate treatment method to be considered shall be economically viable and commensurate with the specific railroad standard and expected traffic and level of service. Besides, local conditions such as topography and alignment, availability of materials, hydrology and drainage has to be considered. It is also revealed that there is scarcity of suitable fine fill material in the vicinity of the present project area which requires using material preferably from the alignment soil for embankment construction. Hence, for the present study these conditions will be taken into consideration while assessing and selecting better alternative remedial option. Therefore, controlled construction techniques could include one or combination of the following:

7.9.1 Removal and replacement of weak soil (Recommended option)

For localized areas with soft soils (CBR<3) of limited depth and thickness, removal of unsuitable material and replacement with suitable fill may be carried out. These unsuitable materials were encountered in valleys and low-lying areas and may be replaced with well-compacted suitable fill, preferably coarse-grained/sandy soils. Excavation and replacement could be carried out up to 2 m. A depth of 2m is reasonable as the general seasonal moisture variation zone is confined within this depth limit. The removal and replacement may be required to be carried out even in ‘cutting’ areas where the naturally occurring soils were found to be of low shear strength and which possess high moisture content variations. Sub-surface drainage may have to be introduced in most of such areas.

7.9.2 Stage constructions of the fill (Recommended option)

This technique can enhance the bearing capacity of the sub-soil and provide the site for required construction of embankment up to the design height in the phases/stages with a

designed strength of the soil and calculated waiting period for the next loading upon the previous loading.

Stage construction is employed mainly as a means of gradually increasing the shear strength of soft clay which would otherwise be inadequate to carry the intended embankment load without failure. In stage construction, advantage of increase in shear strength of sub-soil strata due to consolidation by surcharge of embankment loading is taken into account.

The gain in shear strength is a function of angle of shearing resistance improved in terms of effective stress parameters and degree of consolidation but it is time consuming.

Construction of embankment together with appropriate design and construction measures can also be used to minimize the moisture changes and hence, associated deformations (swelling and shrinkage) of the soils. By means of this method, the moisture fluctuation is regulated by confining the expansive clay under improved sub-grade and by providing protective blankets. According to RDSO GE-0014 (2007), in case of all new construction, minimum height of embankment should not be less than one meter to ensure proper drainage, effective stress dispersal, and uniform riding qualities. Hence, this will be the best option to overcome the problem related with swelling and shrinkage especially in area where thick expansive soil is observed.

7.9.3 Preloading and surcharging

The preloading technique is a simple one and is an economical method for accelerating consolidation as compared with other methods of improving ground support. However, adequate instrumentations for monitoring the settlement, development and dissipation of pore water pressures are essential for the success of this technique. Preloading is particularly economical technique in the construction of railway fills on soft clays, since; the material can stay in place and need not be relocated.

For low embankment over soft compressible soil where the poor ground is of limited thickness (short drainage path) or is capable of compressing rapidly under load of excess preloaded fill due to presence of sand lenses, preloading may be resorted. Preloading of soft soils is based on the consolidation concepts, whereby pore water is squeezed from the voids until the water content and the volume of the soil are in equilibrium under the loading stresses imposed by the surcharge. This is usually accompanied by gain in shear strength of soil. To a

certain extent, the primary consolidation under final loading can be achieved during construction and hence post construction settlement reduces (RDSO, 2009). Since this method is simple and economical, it may prove to be feasible for present study area. However, availability of adequate instrumentation for success of the techniques and the rate of compressibility of the in situ soil under load of excess preload fill will also determine its applicability.

7.9.4 Installation of vertical drainage system

Vertical drains are used where preloading alone shall not be efficient. Vertical drains in soft clay accelerate the primary consolidation of clay since they bring about rapid dissipation of excess pore water pressure. Vertical drains have no direct effect on the rate of secondary compression but the early completion of primary consolidation brings about the earlier onset of secondary settlement. Therefore, the structures or embankments can be put to use earlier than it would be possible otherwise.

The accelerated rate of gain in shear strength of clay enables the loads to be applied more rapidly than would otherwise be possible. Steep side slopes and avoidance of berms in case of embankments may be possible when sand drains are used. The effectiveness of vertical drains depends mainly on the engineering properties of soils, namely, soil permeability and coefficient of consolidation and their variations in space and time. Vertical drains can be successful in accelerating the rate of consolidation of soft fine-grained soils. They are, however, ineffective in organic soils and highly stratified soils. Generally, the effectiveness of the drains are depending upon the engineering properties of soils, the site Conditions, availability of equipment and technology of installation. Hence, the economical cost of the equipment and advance technology of installation will makes it difficult for implementing for the intended project.

7.10 Determination of the thickness of the track bed layers to give the desired bearing capacity

From the stress analysis, it is evident that most of the stresses from heavy axle load (up to 32.5 T load (25t/32.5t)) are dissipated up to a depth of 1.5 m below bottom of ballast, thereafter the stresses are within tolerable limit of stresses including reasonable factor of safety for soils. This region is to be provided with blanket layer which or in lower layers supplemented / replaced by prepared sub-grade particularly in bottom portion. Also, below the blanket layer, the layer of prepared/ good imported soil with minimum prescribed CBR

value is essential and has been recommended as prepared sub-grade layer up to a depth of about 1.5 m below top of formation (RDSO, GE-0014, 2007).

Ethiopian Railways is increasing its network to cater demand of growing traffic. Heavy axle load has been introduced on many identified routes to increase freight transportation. Furthermore, Dedicated Freight Corridors are to be constructed very soon which is being planned for higher axle load of 25 t - 32.5 t. The proposed Sebeta-Ambo railway is one of the freight corridors which will be constructed with higher design axle load of 25T.

Owing this higher axle load, formation with provision of blanket layer on top and underlying sub-grade layers of adequate strength is essential to be provided with a rationale criterion for safe operations of the loads. In the present study guide line proposed by RDSO has been adopted. This Guideline outlines the key components on sub-grade and formation design issues and reviews the design methodologies adopted on various World Railway Systems, e.g. American Railways, European Railways, and Australian Railways. Also, it covers in brief a few design methods available with calculations done for different axle load and soil conditions.

This guideline is based on strength criteria considering CBR value of sub-grade layer. It also recommends specifications and thickness of two alternative systems of formation layers, viz. (i) a conventional single blanket layer system over embankment fill, (ii) two layer systems comprising of blanket and prepared sub-grade layer over the normal fill layers. Both the alternate systems have been specified for 25T, 30T & 32.5T axle load cases.

In this part of the study, it has been attempted to deal with the principles and application of various country design manuals in connection to the formation structural design of the project alignment. The formation structural design involves determining the thickness of capping layer, prepared and improved sub-grade layer. It also involves establishing specifications of construction materials and assessment of their conformity together with the materials in hands.

7.10.1 Design methods based on practice in world railway systems

Various design methods of formation, particularly for blanket thickness, are in use in different railway systems. These are based on different properties of soil used in embankment construction which governs the behavior of soil viz. percentage of fines (size less than 75

microns) present in the soil, CBR value of soil, un-drained shear strength C_u of soil etc. Hence, RDSO guide line considers various design methods of formation.

Considering the relevant good features of Indian Highway system and foreign railway practices, RDSO guideline and specification uses strength based design system. Minimum CBR values of sub-grade/prepared sub-grade have been prescribed for selection of soil/material for the sub-grade. The specifications and thickness of various formation layers has been specified, therefore, these are in line with practices of UIC, AAR and other World Railways.

7.10.1.1 Design of formation for 25 T axle load on the basis of RDSO

Keeping in view the other relevant factors, in addition to axle load (25T), like Speed (120-160 kmph) and GMT (≥ 25) suitable for Ethiopians Railways, detailed calculations of blanket thickness for various soils have been carried out. This is because Ethiopia Railway discipline specific Performance Specifications (ERPS) for all supply items and installation arrangements are to be fully compliant with UIC or similar international railway standards.

Calculated Thickness of blanket material, prepared sub-grade and type of sub-grade along with soil types as per IS classification belonging to various ‘Soil Quality SQ (Table 7.1)’ have been given in Table-7.2 and Table 7.3. The description of soil quality class is given in Table 7.1.

In the proposed study area, the prepared sub-grade material (general fill) and improved prepared sub-grade (the structural fill), for most of the samples exhibited similar soils containing fines less than 12% which both commensurate with QS3 soil quality class.

Table 7.1 Description of soil quality class

Soil Quality	Description W.R.T fine particles (size less than 75micron)	Soils as per IS 1498-1970 Classification Conforming to Referred Soil Quality	CBR Values
QS1	Soils containing fines >50%	CL, ML, CL-ML, CI, MI, CH, MH	Minimum CBR values of 4% should be insured for soil to be used as sub-grade materials
QS2	Soils containing fines from 12% to 50%	GM, GC, SM, SC	
QS3	Soils containing fines < 12%	GW, GP, SW, SP, GW-GM, GW-GC, SW-SM, GP-GM, GP-GC, SP-SM, SP-SC	

Tables 7.1 and Table 7.2 therefore provides the formation design for two different systems for blanket, improved prepared sub-grade (structural fill) and prepared sub-grade (general

embankment fill) which are mandatory provisions to be adopted for the proposed alignment, as follows:

- (i) Conventional blanket layer over formation sub-grade
- (ii) Blanket layer over prepared sub-grade layer (good/imported soil)

Any of the two systems may be considered for adoption in the field based on good soil availability and material cost economics.

Table 7.2 Single Blanket Layer over Embankment Fill and Layers Thickness (RDSO)

Layers	Axle load 25T
Blanket, Well Graded Sand Gravel Layer	60 cm for SQ3, 75 cm for SQ2, 100cm for SQ1 Blanket Compacted up to 98 % MDD, Min. EV2 – 100 MPa
Embankment fill (CBR = 7 – 5, Minimum EV2 = 45 MPa) SQ1 is better to be avoided, otherwise. Top Layer of Thickness (T)	T = 50 cm, SQ2/ SQ3 soils CBR > = 6 generally, but not < 5 in isolated cases Compacted up to 97% of MDD
Ground soil/sub-soil strata	Min. Ev2 >= 20 MPa or Allowable Bearing Capacity >= 138kpa (20Psi) however the ground soil has below the stated values hence Ground improvement is required in area where no cutting is there.

Where E_{v2} = modulus of elasticity (also deformation) at the second step loading during the plate load test on compacted layers per km on field.

Note :

- (i) For axle load 25 T and higher, blanket & Embankment fill thickness can be reduced with determination of soil strength parameters CBR & Threshold stress determined based on repeated Triaxial tests.
- (ii) Uniform total thickness of formation layers of 2 m should be provided including blanket, top layer of embankment fill etc.

Table 7.3 Two Layer System (Blanket & Prepared Sub-grade on Embankment Fill)

Layers	Axle Load of 25T
Blanket, Well Graded Sand Gravel Layer	30 cm for SQ3, 45 cm for SQ2 Compacted up to 100% MDD Min. EV2 – 100 MPa
Layer 1 : Prepared Sub-grade (CBR >= 6 - 8 (of compacted soil up to 97%) SQ2/SQ3 & Limit fines 12 – 50%, SQ1 is better to be avoided. Plasticity Index <= 12) (Good/Imported Soil)	100 cm CBR > = 7 generally, but not < 6 in isolated cases compaction: 98% MDD, minimum :Ev2 45 MPa
Embankment fill (CBR > = 4 – 5)	CBR > = 5 generally, but not < 4 in isolated cases minimum Ev2 :30 MPa compaction :97% MDD
Ground soil/sub-soil strata	Min. Ev2 = 20 MPa or Allowable Bearing Capacity >= 138kpa (20Psi) however the ground soil has below the stated values hence Ground improvement is required in area where no cutting is there.
Total Thickness (Blanket + Layer 1)	130-145 cm

Note : Uniform total thickness of formation layers of 2 m should be provided including blanket, prepared sub-grade & top layer of embankment fill etc.

7.10.1.2 Design of formation for 25 T axle load on the basis of UIC method

Keeping in view the other relevant factors, Speed, GMT etc. similar with the above, detailed calculations of blanket thickness for various soils, based on UIC practices have also been carried out. Hence, according to this method 150 mm of blanket material is required as both the general and prepared sub-grade materials are similar and belongs to SQ3 soil category class.

Table 7.4 UIC Based Two Layers System of Blanketing On Track Formation

Soil Quality Category in Sub-grade	Top Soil of Formation (prepared Sub-grade)		Recommended Thickness (mm) of Blanket for Axle Loads		
	Quality	Thickness(mm)	25T	30T	32.5T
SQ1	SQ2	500	250	450	600
SQ1	SQ3	500	150	350	500
SQ2	SQ2	-	250	450	600
SQ2	SQ3	350	150	350	500
SQ3	SQ3	-	150	350	500

(Ref : Calculations based on UIC practices in terms of UIC Code 719R-1994)

Note-

- (i) Thickness of blanket material has been worked out with the provision of 300mm ballast.
- (ii) Recommended blanket thickness is suitable for $GMT \geq 25$ & Speed < 160 kmph.
- (iii) Geo-textile should be provided below blanket layer, if prepared sub-grade is of SQ2 soil.

From the perusal of practices adopted in other leading railway systems, it is evident that multi-layer formation system– layers with adequate strength and stiffness has to be adopted. Layers comprises of blanket layer, prepared sub-grade/top layer of formation (adopted by RDSO) etc. is preferred in place of single layer blanket system if sufficient construction material is available around the corridor. After saying all these it is also suggested that natural sub-grade improvement should be done by cutting and replacing it with suitable material. Also, stage construction of the embankment fills together with designing the formation layer as per RDSO Guidelines and Specifications for Design of Formation for Heavy Axle Load is required. During the present study specification and thickness of formation layers for Heavier Axle Loads has been explored (Annexure -one).

CHAPTER VIII - CONCLUSION AND RECOMMENDATION

8.1 Conclusion

The Sebeta- Ambo Railway Corridor, that connects Addis Ababa with the western part of Ethiopia, located and extended 25 to 120 km from Addis Ababa along the main road of Addis Ababa - Ambo has 108 km extension. It is bounded by UTM coordinates of 451435.7 m to 364520.3 m E and 984085.7 to 993860.7 m N. For the success of this project, identification of the likely problems along the alignment, related to topography, ground soil, embankment material, hydrology and hydrogeology in general have substantial impact on the long term stability and economic feasibility of the project. Moreover, identification of suitable material for general and structural fill and possible remedial measures for the problems plays great role in design and construction of the project. In the present study the ground in situ soil and embankment fill has been characterized in terms of their; index and engineering properties, dynamic behavior/ properties and its relative response to the site conditions. The general suitability based on standard specifications has also been analyzed for; topography, hydrology-hydrogeology of the area. In order to characterize the material, systematic methodology has been adopted, which includes; detailed literature and previous works review, and various field and laboratory tests. In general, the present study is focused on characterization and suitability analysis of ground soil, embankment fill materials and site condition in general. Moreover, possible remedial measures for the likely engineering geological problems along the corridor are also worked out. Further, attempts were also made to estimate the sub-structure thickness by considering experiences from different countries.

The proposed corridor contains both flat and gentle slopy with minor rolling land features with slope gradient of 0 - 5 %. In order to attain design gradient level, construction of large fills, bridges and cutting is required. The alignment also passes through well known river basin Awash so that the proposed alignment will be crossing through many perennial and intermittent rivers. Hence, provision of number of river crossing arrangements is also expected in the area. As the area is extensively used for irrigation, ground water in certain reaches is expected at shallow depths. Besides, in low lying areas along the proposed alignment marshy conditions may prevail. 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.

Based on the laboratory test results of soil samples generated during the previous and present study; the natural sub-grade soil is unsuitable to be used as structural formation. The soils in general fall into A-7-5 class and some of the soil samples fall into A-7-6 soil class as per ASSHTO classification. Further, the sub-grade soil also possess high to very high degree of expansiveness with PI value ranges from 20 – 52 % which makes the soil susceptible to volumetric change when subjected to moisture variations. Both the CBR and the allowable bearing capacity values are below the allowable limit set by the standards. Besides, the settlement potential of the in situ sub-grade soil is related to normally consolidated materials. Thus, based on the above said properties it may safely be concluded that the sub-grade soils, in general, possess unsuitable engineering characteristics and are not suitable to be used in its present state to provide sound and stable foundation for proposed railway structure. Besides other weaknesses, the major unsuitability of sub-grade soil came from its potential for higher volume change with varying moisture content. As long as the sub-grade soil is found to be unsuitable, economically and practically feasible remedial measures should be selected for the proposed structure in order to improve the soil engineering behavior. Hence, removal and replacement with suitable material or stage construction of the embankment fill may be the most feasible remedial measures for the proposed alignment. Besides, provision of coarser well graded blanketing material on top will also increase the bearing capacity of the structural formation in general.

During the present study, characterization of embankment material for both general and structural fill was attempted by using index, engineering and dynamic property of the materials. During the present study, number of laboratory tests was performed to characterize and analyze the suitability of embankment fill material. Based on the laboratory test results generated from primary and secondary data, the granular fill materials were classified as A-2-7, A-2-5, A-1-a and A-7-5 class as per ASSHTO classification System whereas as per USC soil classification system the soil equivalence classes are [GM, GC, SM, SC], [GM, SM], [GW, GP] and [SC], respectively. However, analysis made through Cassagrande LL-PI chart indicates that the fine soil within granular fill material is dominantly silt fraction. Hence, the most probable soil class for most of the granular fill is silty gravel (GM). Further, entire fine clay borrow (BC) material fall into A-7-5 group of AASHTO system which are mainly clayey soils. In USC system, the possible equivalence to these soils is OH, MH, CL and OL group. The analysis for fill material properties indicates GI value of zero, low PI value, (less than 15/25), CBR value greater than 4 and appropriate MDD values for most of the samples.

Thus, according to most of the standard manuals, the fill material is best to be used as embankment fill, for both general and structural fill, as well as replacing problematic sections. The only problem which can be anticipated with this type of material is its consolidation under the influence of dynamic loads as, the material lacks fine proportion. However, this problem can be addressed by blending it with fine soil in appropriate proportion with the fill material. This perhaps requires more detailed investigation to workout blending proportion.

The project area is located in wet tropical zone and has poor natural sub soil therefore through the present study provision of blanket material has been suggested. The blanket material may help to substantially increase the bearing capacity of the sub-grade and may also help to prevent percolation of water in to the sub-soil. Thus, characterization of blanketing/capping layer was needed to minimize the specified problems. Hence, the similar material what was proposed for fill was also characterized for the blanketing material. Therefore, based on the laboratory results and the specification set for blanket material, the material was characterized following soil properties such as; particles size distribution, coefficient of uniformity, coefficient of curvature, filter criteria, soil strength and abrasion. The material coefficient of uniformity, materials strength and abrasion tests was conducted for most of the blanket material samples and the results falls above the recommended values. Even though almost all samples except one can satisfy the percentage pass for 0.075mm sieve size, the soil gradation curves for other sieve size also indicate, much of the samples cannot satisfy the permissible percentage pass set by the standards. Besides, all C_u except one and half of C_c values of the material can satisfy the standard requirements. Hence, much of the material are coarse and well graded, as a result they are suitable to be used as capping layer. However, the material needs finer fraction to satisfy the required percentage pass as well as to meet the required compaction level. Based on the present findings, applying filter criteria for the blanket layer is fictitious. Flattening of the embankment and provision of drainage system will be more appropriate option to control any possible erosion.

8.2 Recommendations

After detailed discussion and interpretation of the engineering geological characterization of the overall alignment and material plus ground improvement methods for sub-grade soils in general and for the subject corridor in particular, the above mentioned conclusions have been made. However, as the present study shows most of the identified materials for the

embankment have a major drawback even if they are available within economical distance. Therefore, before construction it is necessary to conduct detailed study on the properties of the identified material. However, based on the test results of primary, secondary data and site condition in general, the following recommendations are forwarded;

- The present study is conducted by taking very limited parameters for consolidation test. Besides, no mineralogical tests for the natural sub-grade were undertaken. Therefore, it is recommended that more additional parameters of settlement potential and mineralogical tests should be performed to have more realistic results.
- It is recommended that removal and replacement of weak ground soil with suitable material may be the best feasible method for shallow weak soil. However, for those sections where large embankment fill may be required, stage construction of the fill may be more appropriate. By means of this method, the moisture fluctuation is regulated by confining the expansive clay under improved sub-grade and by providing protective blankets.
- As investigated from the present study, the dominant fill material, either for replacement of unsuitable ground soil or for embankment construction, is mainly granular soil with limited fine grained soils. Therefore, it is necessary either to identify another borrow area with suitable fine material within reasonable distance or to blend the available material with fine fraction soils with an appropriate ratio. However, blending of the material in appropriate ration is the best option as the area has shortage of fine materials which can full fill the requirements.
- The cross section along the proposed alignment suggests large fill and deep cuts to maintain the design ground level. Hence, to avoid large fills, through this study it is suggested to provide partial fill with bridge over it where the valley span is large and simple single span girder bridge where the span is relatively less. Provision of partial fill with bridge over it will ensure stability of fill by minimizing the height of fill with gentler slopes. Also, this will check the development of pore water pressure within the embankment. Besides, non availability of appropriate fill material within the economic distance from the proposed alignment also suggests avoidance of large fills. However, more detailed techno-economic feasibility studies would be required to make the final choice for the selection of an appropriate option.

- Deep cuts require special techniques and precaution. The stability of cut slopes, its design and maintenance may be of concern. Considering the ground water, it's recommended that depending on local conditions, the ground water table should be lowered to a minimum distance between 0.80m and 2.50m below rail top.
- The excavated materials from cutting that can fulfill the standard requirement can also be used as embankment fill materials.
- From the perusal of practices adopted in other leading railway systems, it is evident that multi-layer formation system– layers with adequate strength and stiffness has to be adopted i.e. Blanket layer over improved prepared sub-grade layer (good/imported soil) has to be recommended.

Table 8.1 Recommended thickness, two Layer Systems

Layers	Axle Load of 25T
Blanket, Well Graded Sand Gravel Layer	30 cm for SQ3, 45 cm for SQ2 Compacted up to 100% MDD Min. EV2 – 100 MPa
Layer 1 : improved Prepared Sub-grade (CBR \geq 6 - 8 (of compacted soil up to 97%) SQ2/SQ3 & Limit fines 12 – 50%, SQ1 is better to be avoided. Plasticity Index \leq 12) (Good/Imported Soil)	100 cm CBR $>$ = 7 generally, but not $<$ 6 in isolated cases compaction: 98% MDD minimum :Ev2 45 MPa
Embankment fill (CBR $>$ = 4 – 5)	CBR $>$ = 5 generally, but not $<$ 4 in isolated cases minimum Ev2 :30 MPa compaction :97% MDD
Ground soil/sub-soil strata	Min. Ev2 = 20 MPa or Allowable Bearing Capacity \geq 138kpa (20Psi) however the ground soil has below the stated values hence Ground improvement is required in area where no cutting is there.
Total Thickness (Blanket + Layer 1)	130-145

It should be noted that the thickness (Table 8.1) is determined based on the standard set by RDSO, GE-0014 (2007) where in area of problematic soil is observed and in case of new construction, minimum height of embankment should not be less than one meter to ensure proper drainage, effective stress dispersal, and uniform riding qualities.

- Taking in to consideration all the above mentioned poor engineering properties of the sub-grade and the fill/ blanket material contractor and consultants of the proposed railway line should utilize the appropriate remedial measures. The client- Ethiopian Railway Corporation; ECR should take the necessary measures to reduce the problems. This effort should also be accompanied by strict and professionally ethical quality assurance and

supervision of construction. Besides, for the proper functioning of the proposed project during its design life, proper routine maintenance activities should be carried out.

- The field survey assessments for construction material are based on observations of surface features and/or at shallow depths only and should be taken as indicative and not definitive. The samples obtained are therefore only representative of surface material and should not necessarily be considered representative of deeper soils and rock material. Further, quality assurance tests shall be carried out during the construction stage by the contractor while locating suitable construction material sources.

All efforts were made to conduct the present study in a systematic manner, well supported by actual test results and scientific observations. The findings and recommendations made through this study should be considered as indicative only as the study was performed under limitations of time and resources. Thus, the quality of results may be affected to certain degree of inaccuracy. Therefore, it is strongly recommended to conduct more elaborate scientific study on various aspects before adopting the findings of the present study for implementation. However, the present study provides a general methodology to conduct similar studies on engineering suitability of rail way project.

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ANNEXURE 1 - SPECIFICATION AND THICKNESS OF SUB-GRADE LAYERS FOR HEAVIER AXLE LOADS (RDSO)

1. Single Blanket Layer over Embankment Fill- Specification and Layer Thickness

Layers	Specification	Axle Load 25T	Axle Load 30 T	Axle Load 32.5T
Blanket – Well Graded Sand Gravel Layer	i) $C_u > 7$ and C_c between 1-3. ii) Fines (passing 75 microns) :3% to 10% iii) Los Angeles Abrasion value < 40% (iv) Minimum CBR value 25 of the blanket material compacted at 100% of MDD (v) Size gradation within specified range or enveloping curves. (vi) Filter Criteria should be satisfied with layer, as given below : Criteria–1: $D_{15}(\text{blanket}) < 5 \times D_{85}(\text{sub-grade})$ Criteria–2: $D_{15}(\text{blanket}) > 4 \times D_{15}(\text{sub-grade})$ Criteria–3: $D_{50}(\text{blanket}) < 25 \times D_{50}(\text{sub-grade})$ Minimum EV2 – determined from 2nd step Plate Load Test on top of compacted blanket layer (Ref : German Code : DIN 18134 – 2001)	60 cm for SQ3, 75 cm for SQ2 100 cm for SQ1 (SQ1 only with dispensation of PCE/ CAO) SQ2/SQ3 are preferred soil below blanket. Blanket Compacted upto 98 % MDD Min. EV2 – 100 MPa	75 cm for SQ3, 80 cm for SQ2 SQ2/SQ3 are preferred soil below blanket. Blanket Compacted upto 100% MDD For new line construction, Min. EV2 – 120 MPa As in-situ Assurance Test	80 cm for SQ3, 100 cm for SQ2 SQ2/SQ3 are preferred soil below blanket. Blanket Compacted upto 100% MDD For new line construction, Min. EV2 – 120 MPa As in-situ Assurance Test
Embankment Fill Top Layer of Thickness (T)	CBR = 7 - 5 (of compacted soil at 98% of MDD) (Organic soils to be avoided) (SQ1 to be avoided) Minimum EV2 = 45 MPa Compaction :	T = 50 cm, SQ2/SQ3 soils CBR > = 6 generally, but not < 5 in isolated cases (For SQ1 soil, CBR > = 4 generally, but not < 3 in top 1 m. & for Lower Fill Layers CBR > = 3) Compacted up to 97% MDD	T = 75 cm SQ2/SQ3 soils CBR > = 6 generally, but not < 5 in isolated cases Compacted upto 98% of MDD	T = 75 cm SQ2/SQ3 soils CBR > = 7 generally, but not < 6 in isolated cases Compacted upto 98% of MDD
Ground Soil/Sub-soil Strata	Minimum Undrained Cohesion of soil, $C_u = 25$ KPa or Minimum Ev2 = 20 MPa Ground Improvement is required, if $C_u < 25$ kPa or Ev2 < 20 MPa	Min. Ev2 = 20 MPa	Min. Ev2 = 20 MPa	Min. Ev2 = 20 MPa

Note : 1. SQ1 (fines > 50%), SQ2 (fines : 12 to 50 %), SQ3 (fines <12%)

2. For axle load 25 T and higher, blanket & Embankment fill thickness can be reduced with determination of soil strength parameters CBR & Threshold stress determined based on repeated Triaxial tests.

3. Uniform total thickness of formation layers of 2 m should be provided including blanket, top layer of embankment fill etc.

2. Two Layer Systems (Blanket & Prepared Sub-grade on Embankment Fill)

Layers	Specification	Axle Load 25T	Axle Load 30 T	Axle Load 32.5T
Blanket – Well Graded Sand Gravel Layer	i) $C_u > 7$ and C_c between 1-3. ii) Fines (passing 75 microns) :3% to 10% iii) Los Angeles Abrasion value $< 40\%$ (iv) Minimum CBR value 25 of the blanket material compacted at 100% of MDD (v) Size gradation – within specified range or enveloping curves (vi) Filter Criteria should be satisfied with subgrade layer, as given below : Criteria–1: $D_{15}(\text{blanket}) < 5 \times D_{85}(\text{sub-grade})$ Criteria–2: $D_{15}(\text{blanket}) > 4 \times D_{15}(\text{sub-grade})$ Criteria–3: $D_{50}(\text{blanket}) < 25 \times D_{50}(\text{sub-grade})$ Minimum EV2 - determined from 2nd step Plate Load Test on top of compacted blanket layer (Ref : German Code : DIN 18134 – 2001)	30 cm for SQ3, 45 cm for SQ2 SQ2/SQ3 are soil below blanket. Compacted upto 100% MDD Min. EV2 – 100 MPa	45 cm for SQ3, 60 cm for SQ2 SQ2/SQ3 are soil below blanket. Compacted upto 100% MDD For new line construction, Min. EV2 – 120 MPa As in-situ Assurance test	45 cm for SQ3, 60 cm for SQ2 SQ2/SQ3 are soil below blanket. Compacted upto 100% MDD For new line construction, Min. EV2 – 120 MPa As in-situ Assurance Test
Layer 1 : Prepared Subgrade (Good/Imported Soil)	CBR $\geq 6 - 8$ (of compacted soil upto 97%) SQ2/SQ3 & Limit fines 12 – 50% (SQ1 to be avoided) Plasticity Index ≤ 12 Compaction : Minimum EV2 :	100 cm CBR ≥ 7 generally, but not < 6 in isolated cases 98% MDD 45 MPa	100 cm CBR ≥ 7 generally, but not < 6 in isolated cases 98% MDD 60 MPa	100 cm CBR ≥ 8 generally, but not < 7 in isolated cases 98% MDD 60 MPa
Embankment Fill	CBR $\geq 4 - 5$ (of compacted soil upto 97%) (Organic soils to be avoided) Minimum EV2 : Compaction :	CBR ≥ 5 generally, but not < 4 in isolated cases (For SQ1 soil, CBR ≥ 3 generally, but not < 2 in isolated cases) 30 MPa, 97% MDD	CBR ≥ 5 generally, but not < 4 in isolated cases 30 MPa 97% MDD	CBR ≥ 5 generally, but not < 4 in isolated cases 30 MPa 97% MDD
Ground Soil/Sub-soil Strata	Minimum Undrained Cohesion of soil, $C_u = 25$ KPa or Minimum EV2 = 20 MPa Ground Improvement is required, if $C_u < 25$ kPa, or Ev2 < 20 MPa	Min. Ev2 = 20 MPa	Min. Ev2 = 20 MPa	Min. Ev2 = 20 MPa
Total Thickness (Blanket + Layer 1)		130-145 cm	145-160 cm	145 – 160 cm

Note: Uniform total thickness of formation layers of 2 m should be provided including blanket, prepared subgrade & top layer of embankment fill etc.

ANNEXURE 2

TABLE 1- Sub-grade laboratory classification, proctor and CBR result description, (Sampling depth for all pits <=1.00m)

Station	Material description	% pass of wet sieve (mm)			Atterberg Limit			AASHTO (GI)	MD D T-180, g/cc	OMC %	3-Point CBR			Swell %
		2	0.425	0.075	LL%	PL %	PI %				No. of Blows	2.54mm	5.08mm	
0+000	black cotton soil	100	99.12	97	74	42	32	A-7-5 (41)	1.3	32	10	2	2	4
											30	3	2	3
											65	3	3	2
0+500	light brown clay	100	98.42	95.5	46	26	20	A-7-6 (22)						
1+000	black cotton soil	100	96.85	94.3	71	35	36	A-7-5 (42)						
1+500	black cotton soil	100	95.89	93.7	74	43	31	A-7-5(38)						
2+000	dark brown clay	100	96.55	91.8	78	40	38	A-7-5(44)	1.44	24	10	2	2	5
											30	2	2	4
											65	3	3	2.36
2+500	black cotton soil	100	98.05	94.9	85	52	33	A-7-5(44)						
3+000	Reddish brown clay	100	97.44	94	73	43	30	A-7-5(37)						
3+500	Dark brown clay	100	94.89	90.8	45	17	28	A-7-6(26)						
4+000	black cotton soil	100	96.31	94	67	37	30	A-7-5(36)	1.31	28	10	1	1	6.62
											30	2	2	4.24
											65	2	2	2.58
4+500	black cotton soil mixed with granular	100	98.75	95.4	42	18	24	A-7-6(24)						
5+000	black cotton soil	100	93.92	86.6	63	25	38	A-7-6(36)						
5+500	light reddish brown clay	100	96.25	94.3	73	45	28	A-7-5(36)						
6+000	black cotton soil	100	92.44	87.3	72	44	28	A-7-5(33)	1.38	27	10	1	1	5.76
											30	2	2	4.27
											65	2	2	2.64
6+500	black cotton soil	100	96.01	90.8	72	40	32	A-7-5(37)						
7+000	Grey to reddish clay	100	95.6	92.5	48	18	30	A-7-6(29)						

7+500	Reddish brown clay	100	94.5	91.6	90	48	42	A-7-5(49)						
8+000	Reddish brown clay	100	92.09	86.1	88	46	42	A-7-5(45)	1.31	30	10	2	1	5
											30	2	2	4.24
											65	3	3	2.36
8+500	Black cotton soil	100	89.92	94.6	79	36	43	A-7-5(50)						
9+000	Black cotton soil	100	96.05	93.8	82	43	39	A-7-5(47)						
9+500	Light red to grey clay	100	92.23	83.9	70	42	28	A-7-5(30)						
10+000	Black cotton soil	100	91.5	82	66	36	30	A-7-5(29)	1.27	32	10	1	1	6
											30	2	1	5.46
											65	3	2	2.5
10+500	Black cotton soil	100	95.41	88.1	92	51	41	A-7-6(46)						
11+000	Black cotton soil	100	96.86	95	70	35	35	A-7-5(40)						
11+500	Black cotton soil	100	96.6	93.5	86	47	39	A-7-5(48)						
12+000	Black cotton soil	100	98.9	95.5	78	43	35	A-7-5(44)	1.32	32	10	2	2	8.55
											30	2	2	6.33
											65	3	2	2
12+500	Light grey Sandy silty clay	100	93.41	85.4	56	29	27	A-7-6(26)						
13+000	Black cotton soil	100	93.44	88.9	85	43	42	A-7-6(47)						
13+500	Black cotton soil	100	98.1	93.5	75	35	40	A-7-5(45)						
14+000	Black cotton soil	100	98.23	94.3	85	48	37	A-7-5(47)	1.35	31	10	2	1	7.67
											30	2	2	4.86
											65	2	2	2.09
14+500	Black cotton soil	100	89.86	82.5	84	45	39	A-7-5(40)						
15+000	Dark grey silty clay	100	95.41	91.8	76	39	37	A-7-5(43)						
15+500	Black cotton soil	100	96.65	94.7	88	46	42	A-7-5(52)						
16+000	Black cotton soil	100	87.75	77.6	91	38	53	A-7-5(46)	1.4	23	10	2	2	4.64
											30	3	3	3.75
											65	4	3	1.3
16+500	Black cotton soil	100	95	87.6	91	48	43	A-7-5(49)						
17+000	Black cotton soil	100	95.6	89.2	83	46	37	A-7-5(42)						
17+500	Black cotton soil	100	93.9	86.8	82	46	36	A-7-5(40)						

18+000	reddish brown silty clay	100	87.96	78.5	81	36	45	A-7-5(40)	1.38	26	10	2	1	6.9
											30	2	2	5.45
											65	3	3	3.31
18+500	reddish brown silty clay	100	93.29	88.5	79	48	31	A-7-5(37)						
19+000	reddish brown silty clay	100	96.21	92.1	79	46	33	A-7-5(40)						
19+500	Black cotton soil	100	97.11	94.2	80	37	43	A-7-5(50)						
20+000	Black cotton soil	100	90.23	78.7	78	43	35	A-7-5(33)	1.27	30	10	2	1	7.69
											30	2	2	6.29
											65	3	2	3.62
20+500	Black cotton soil	100	93.86	89.5	82	42	40	A-7-5(45)						
21+000	reddish brown silty clay	100	93.11	84.1	71	43	28	A-7-5(30)						
21+500	Grayish silty clay	100	96.21	92.7	71	40	31	A-7-5(37)						
22+000	Black cotton soil	100	95.3	87.7	70	37	33	A-7-5(35)	1.39	24	10	2	2	4.98
											30	4	3	2.67
											65	4	3	1.88
22+500	Black cotton soil	100	98.04	94.9	74	44	30	A-7-5(38)						
23+000	Black cotton soil	100	94.56	87.2	82	45	37	A-7-5(41)						
23+500	reddish brown silty clay	100	95	92.1	76	37	39	A-7-5(44)						
24+000	Black cotton soil	100	92.52	81	77	45	32	A-7-5(32)	1.33	34	10	2	2	5.15
											30	3	2	3.44
											65	3	3	2.15
24+500	Black cotton soil	100	87	75	75	44	31	A-7-5(35)						
25+000	Black cotton soil	100	96.6	75	75	38	37	A-7-5(42)						
25+500	Black cotton soil	100	96.1	74	74	39	35	A-7-5(42)						
26+000	Black cotton soil	100	92.34	74	74	34	40	A-7-5(38)	1.28	36	10	2	1	4.78
											30	2	2	3.54
											65	2	2	2.54
26+500	Black cotton soil	100	97.89	93.7	79	37	42	A-7-5(48)						
27+000	Black cotton soil	100	97.67	92.4	82	39	43	A-7-5(49)						

27+500	Black cotton soil	100	96.65	92	83	43	40	A-7-5(47)						
28+000	Black cotton soil	100	94.9	91.4	77	43	34	A-7-5(40)	1.37	29	10	2	2	4.11
											30	3	2	3.61
											65	4	3	2.39
28+500	dark brown silty clay	100	92.54	87.7	73	36	37	A-7-5(39)						
29+000	Black cotton soil	100	93.78	89.9	71	33	38	A-7-5(40)						
29+500	Black cotton soil	100	92.31	81.7	70	28	42	A-7-6(38)						
30+000	Black cotton soil	100	87.9	79.1	66	30	36	A-7-5(31)	1.33	32	10	2	2	6.19
											30	2	2	3.97
											65	3	2	2.73
30+500	Black cotton soil	100	95.2	92.8	67	36	31	A-7-5(36)						
31+000	Black cotton soil	100	96.24	88.9	73	35	38	A-7-5(40)						
31+500	Black cotton soil	100	97.51	93.5	66	31	35	A-7-5(39)						
32+000	Black cotton soil	100	96.23	90.1	89	50	39	A-7-5(46)	1.38	38	10	2	2	3.84
											30	2	2	3.15
											65	3	3	1.56
32+500	Black cotton soil	100	94.11	90.6	91	60	31	A-7-6(41)						
33+000	dark brown gravely silty clay	100	96.9	94	100	65	35	A-7-5(49)						
33+500	Reddish to yellowish brown silty clay soil	100	95.9	92.2	88	52	36	A-7-5(45)						
34+000	Black cotton soil	100	91.2	79.7	83	56	27	A-7-5(30)	1.32	38	10	2	2	4.13
											30	3	2	2.16
											65	3	3	1.74
34+500	Yellowish grey silty clay	100	93.21	84	73	45	28	A-7-5(30)						
35+000	Black cotton soil	100	92.54	83.8	73	37	36	A-7-5(36)						
35+500	Black cotton soil	100	95.45	91.7	94	54	40	A-7-5(50)						
36+000	Black cotton soil	100	94.22	90.3	71	37	34	A-7-5(38)	1.38	31	10	2	2	4.95
											30	3	3	3.92
											65	4	3	1.46
36+500	Black cotton soil	100	92.01	81.9	91	53	38	A-7-5(40)						

37+000	reddish brown silty clay	100	98.21	94.3	54	25	29	A-7-69(31)						
37+500	reddish granular silty clay	100	97.5	95.6	72	46	26	A-7-5(35)						
38+000	Black cotton soil	100	90.54	83.1	83	40	43	A-7-5(42)	1.32	33	10	2	2	4.99
											30	2	2	2.97
											65	3	2	2.16
38+500	Black cotton soil	100	92.55	85.8	79	44	35	A-7-5(38)						
39+000	Black cotton soil	100	96.86	94.5	80	41	39	A-7-5(47)						
39+500	Black cotton soil	100	98.21	94.3	82	46	36	A-7-5(45)						
40+000	Black cotton soil	100	86.78	78.7	81	52	29	A-7-5(30)	1.4	25	10	3	2	3.43
											30	3	3	2.87
											65	4	4	1.93
40+500	reddish brown silty clay	100	93.21	83.9	75	47	28	A-7-5(31)						
41+000	Black cotton soil	100	96.74	92.6	79	46	34	A-7-5(41)						
41+500	Black cotton soil	100	97.59	94.4	75	35	40	A-7-5(46)						
42+000	Black cotton soil	100	92.55	86.4	70	36	34	A-7-5(35)	1.24	25	10	1	1	3.28
											30	2	2	2.57
											65	2	2	1.31
42+500	Black cotton soil	100	90.81	83.5	77	44	33	A-7-5(34)						
43+000	Black cotton soil	100	98.2	93.5	64	26	38	A-7-6(41)						
43+500	Reddish brown silty clay	100	95.6	91.9	72	39	33	A-7-5(38)						
44+000	Black cotton soil	100	88.18	79.4	58	30	28	A-7-5(24)	1.25	31	10	2	1	3.13
											30	2	2	2
											65	2	2	1.68
44+500	reddish brown silty clay	100	91.44	95.1	80	52	28	A-7-5(38)						
45+000	Black cotton soil	100	96.89	94.2	78	44	34	A-7-5(42)						
45+500	Black cotton soil	100	94.7	87.6	65	31	34	A-7-5(35)						
46+000	Black cotton soil	100	84.4	73.2	72	40	32	A-7-5(27)	1.4	29	10	3	2	4

											30	3	3	3.09			
											65	4	3	1.18			
46+500	reddish brown siltyclay	100	92.62	83.3	73	29	44	A-7-6(41)									
47+000	Grey gravely silty clay	100	95.3	88.8	76	40	36	A-7-5(40)									
47+500	Black cotton soil	100	96.1	89.3	76	34	42	A-7-5(44)									
48+000	Black cotton soil	100	89.99	83.7	71	32	39	A-7-5(37)	1.24	31	10	2	2	4.98			
					76	46	30				30	3	2	2.6			
											65	3	2	2.32			
48+500	Black cotton soil	100	93.5	90				A-7-5(36)									
49+000	Black cotton soil	100	96.56	94.5	71	30	41	A-7-5(46)									
49+500	Black cotton soil	100	92.31	96	63	26	37	A-7-5(41)									
50+000	reddish brown silty clay	100	92.72	82.3	77	44	33	A-7-5(34)	1.4	30	10	2	2	3.33			
					71	32	39				30	3	2	2.36			
											65	3	3	1.76			
50+500	Black cotton soil	100	90.56	79.2				A-7-5(34)									
51+000	Grey silty clay	100	96.7	93.9	64	30	34	A-7-5(38)									
51+500	Black cotton soil	100	95.44	89.1	55	20	35	A-7-5(33)									
52+000	Brownish gravely silty clay	100	91.71	79.3	75	37	38	A-7-5(35)	1.33	31	10	2	2	5.15			
														30	3	2	2.77
														65	3	3	1.53
52+500	Black cotton soil	100	90.47	77.3	70	29	41	A-7-6(34)									
53+000	Black cotton soil	100	98.01	93.4	66	30	36	A-7-5(40)									
53+500	Black cotton soil	100	96.82	92.3	77	33	44	A-7-5(48)									
54+000	Black cotton soil	100	87.66	77.9	74	40	34	A-7-5(31)	1.35	29	10	2	1	4.3			
														30	2	2	2.52
														65	3	3	1.84
54+500	Black cotton soil	100	92.12	81.4	74	44	30	A-7-5(30)									
55+000	reddish brown silty clay	100	93.5	89.3	71	42	29	A-7-5(33)									
55+500	Black cotton soil	100	97.1	95	83	43	40	A-7-5(49)									
56+500	Black cotton soil	100	94.3	90	82	35	47	A-7-5(50)									
57+000	Grey gravely silty clay	100	97	93.0	76	41	35	A-7-5(42)									

				7										
57+500	Black cotton soil	100	97.1	90.8 1	78	36	42	A-7-5(46)						
58+000	Grey gravely silty clay	100	91.6	89.9	77	40	37	A-7-5(41)	1.39	28	10	2	2	5.17
											30	2	2	4.68
											65	2	2	4.42
58+500	Black cotton soil	100	97.3	93.3 4	80	38	42	A-7-5(48)						
59+000	Grey gravely silty clay	100	95.59	89.9 5	71	32	39	A-7-5(41)						
59+500	Black cotton soil	100	97.32	86.2 3	79	35	44	A-7-5(45)						
60+000	Black cotton soil	100	98.3	96.3 2	84	38	46	A-7-5(55)	1.4	25	10	1	1	8.77
											30	1	1	9.79
											65	1	1	12.4
60+500	Grey gravely silty clay	100	98.93	96.0 5	68	38	30	A-7-5(37)						
61+000	Black cotton soil	100	97.58	95.1 5	76	30	46	A-7-5(52)						
61+500	Black cotton soil	100	97.66	90.7 5	84	35	49	A-7-5(53)						
62+000	Black cotton soil	100	95.77	92.5 8	81	37	44	A-7-5(50)	1.42	31	10	1	1	7
											30	1	1	4.85
											65	1	1	6.09
62+500	Black cotton soil	100	98.67	97.6 8	85	35	50	A-7-5(60)						
63+000	Grey gravely silty clay	100	96.79	92.1 1	68	37	31	A-7-5(36)						
63+500	Black cotton soil	100	98.25	96.3 6	80	40	40	A-7-5(49)						
64+000	Grey gravely silty clay	100	98.25	96.3 6	71	35	36	A-7-5(43)	1.38	25	10	1	1	8.83
											30	1	1	9.15

											65	2	1	6.28
64+500	Black cotton soil	100	95.54	91.9 8	73	32	41	A-7-5(45)						
65+000	Black cotton soil	100	98.32	97.4	83	38	45	A-7-5(55)						
65+500	Black cotton soil	100	97.54	95.1	86	45	41	A-7-5(51)						
66+000	Grey gravely silty clay	100	98.74	88.2 9	85	54	31	A-7-5(38)	1.39	33	10	1	1	7.3
											30	1	1	9.45
											65	1	1	8.81
66+500	Black cotton soil	100	88.58	86.1 4	76	34	42	A-7-5(42)						
67+000	Black cotton soil	100	97.79	95.7	75	30	45	A-7-5(51)						
67+500	Black cotton soil	100	89.97	87.9 5	71	37	34	A-7-5(36)						
68+000	Black cotton soil	100	92.77	89.2 1	80	35	45	A-7-5(48)	1.41	26	10	1	1	10.7
											30	2	2	11.2
											65	3	2	9.36
68+500	Black cotton soil	100	97.77	96.9 8	80	32	48	A-7-5(56)						
69+000	Black cotton soil	100	88.77	85.5 5	79	35	44	A-7-5(44)						
69+500	Black cotton soil	100	98.74	97.2 4	89	42	47	A-7-5(58)						
70+000	Black cotton soil	100	95.2	92	88	47	41	A-7-5(49)	1.37	32	10	1	1	8.75
											30	1	1	7.82
											65	1	1	6.71
70+500	Black cotton soil	100	98.44	92.1 9	84	44	40	A-7-5(47)						
71+000	Black cotton soil	100	97.67	97	84	38	46	A-7-5(56)						
71+500	Black cotton soil	100	94.45	89.8 3	81	30	51	A-7-5(53)						
72+000	Grey gravely silty clay	100	97.22	96.1 2	68	32	36	A-7-5(42)	1.4	31	10	2	2	9.66
											30	2	2	9

											65	2	2	8.57
72+500	Black cotton soil	100	99.93	98.8 5	76	35	41	A-7-5(50)						
73+000	Black cotton soil	100	98.74	97.1	70	30	40	A-7-5(46)						
73+500	Black cotton soil	100	97.13	96.3	70	29	41	A-7-6 (67)						
74+000	Black cotton soil	100	95.17	93.7 6	73	31	42	A-7-5(47)	1.39	27	10	1	1	10.1
											30	1	1	9.48
											65	1	1	8.68
74+500	Black cotton soil	100	98.59	97.0 2	71	31	40	A-7-5(47)						
75+000	Black cotton soil	100	95.58	92.3 2	83	36	47	A-7-5(52)						
75+500	Black cotton soil	100	97.48	96.4 2	76	34	42	A-7-5(49)						
76+000	Black cotton soil	100	94.64	91.0 8	84	39	45	A-7-5(50)	1.41	28	10	1	1	10.4
											30	1	1	9.24
											65	1	1	8.81
76+500	Black cotton soil	100	98.75	98.2	71	31	40	A-7-5(47)						
77+000	Black cotton soil	100	98.47	97.7	75	32	43	A-7-5(51)						
77+500	Black cotton soil	100	97.85	94.4 6	70	30	40	A-7-5(45)						
78+000	Black cotton soil	100	98.33	86.6 9	71	34	37	A-7-5(38)	1.4	29	10	1	1	10.1
											30	1	1	10.1
											65	1	1	8.68
78+500	Black cotton soil	100	95.8	89.3 5	72	37	35	A-7-5(38)						
79+000	Black cotton soil	100	98.33	98.2 7	79	37	42	A-7-5(52)						
79+500	Black cotton soil	100	96.79	94.8	78	39	39	A-7-5(46)						
80+000	Black cotton soil	100	97.92	97.6 1	72	36	36	A-7-5(44)	1.42	30	10	1	1	10.3
			30								1	1	10.1	
			65								1	1	9.67	

80+500	Black cotton soil	100	91.79	89	86	36	50	A-7-5(53)						
81+000	Brown silty clay	100	94.54	91.2 7	71	33	38	A-7-5(41)						
81+500	Grey gravely silty clay	100	97.15	96.6 6	65	32	33	A-7-5(39)						
82+000	Black cotton soil	100	90.15	89.6 6	79	36	43	A-7-5(46)	1.43	29	10	2	2	9.57
											30	2	2	9.31
											65	2	2	8.82
82+500	Black cotton soil	100	95.98	94.7 3	78	37	41	A-7-5(48)						
83+000	Black cotton soil	100	94.87	90.2 3	70	33	37	A-7-5(40)						
83+500	Black cotton soil	100	98.72	88.3	77	33	44	A-7-5(45)						
84+000	Light brown silty clay	100	97.8	95.7 8	60	29	31	A-7-6(35)	1.41	27	10	2	2	8.31
											30	2	2	6.83
											65	2	2	5.88
84+500	Light brown silty clay	100	97.71	96.4	62	28	34	A-7-6 (39)						
85+000	Light brown silty clay	100	94.85	90.1 7	67	34	33	A-7-5(36)						
85+500	Light brown silty clay	100	95.23	92	70	33	37	A-7-5(41)						
86+000	Black cotton soil	100	96.48	95	81	39	42	A-7-5(50)	1.38	30	10	1	1	13.2
											30	1	1	11.7
											65	1	1	11.7
86+500	Black cotton soil	100	91.28	89	70	30	40	A-7-5(41)						
87+000	Black cotton soil	100	95.28	94.0 6	81	37	44	A-7-5(51)						
87+500	Black cotton soil	100	97.51	96.3 3	81	37	44	A-7-5(53)						
88+000	Light brown silty clay	100	92.66	90.8	75	37	38	A-7-5(42)	1.42	26	10	1	1	8.92
											30	2	2	8.25
											65	2	2	7.70
88+500	Light brown silty clay	100	99.92	98.8	76	37	39	A-7-5(49)						

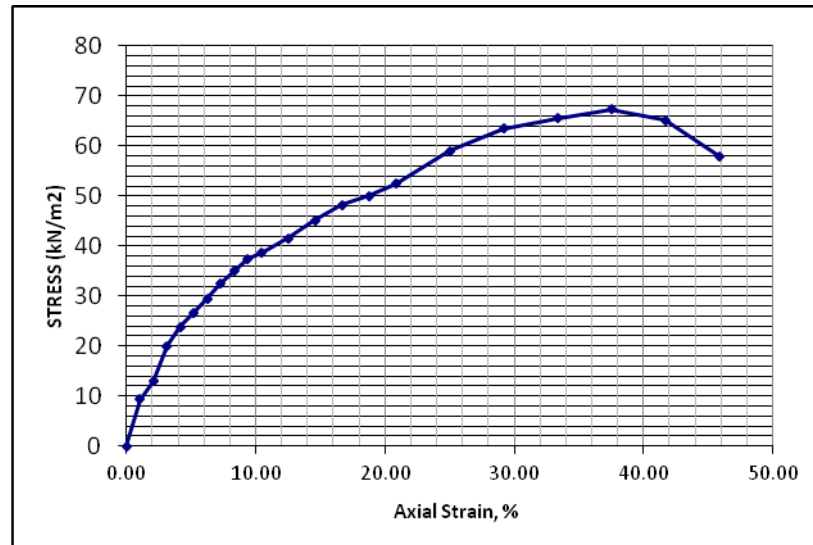
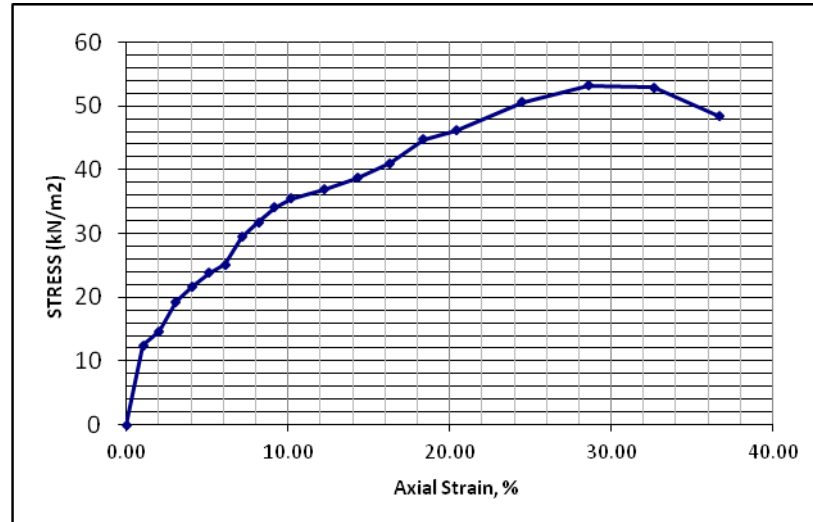
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89+000	Light brown silty clay	100	88.8	86.2 4	69	34	35	A-7-5(36)						
89+500	Light brown silty clay	100	89.8	87.7 4	71	32	39	A-7-5(40)						
90+000	Black cotton soil	100	97.7	95.5	79	37	42	A-7-5(50)	1.4	30	10	1	1	11.6
											30	1	1	12
											65	1	1	11.26
90+500	Black cotton soil	100	98.25	96.6 8	82	37	45	A-7-5(54)						
91+000	Light brown silty clay	100	97.11	90.7 9	72	34	38	A-7-5(41)						
91+500	Light brown silty clay	100	93.77	91.1 5	68	32	36	A-7-5(39)						
92+000	Black cotton soil	100	96.88	95.7 2	67	32	35	A-7-5(41)	1.39	27	10	1	1	8.75
											30	2	2	7.82
											65	2	2	6.71
92+500	Black cotton soil	100	93.44	88.7 1	77	34	43	A-7-5(45)						
93+000	Light brown silty clay	100	94.88	91.4 3	73	35	38	A-7-5(42)						
93+500	Black cotton soil	100	95.88	93.4 8	76	34	42	A-7-5(47)						
94+000	Light brown silty clay	100	91.77	90.3 4	70	34	36	A-7-5(39)	1.42	24	10	1	1	7.74
											30	1	1	7.51
											65	2	2	6.41
94+500	Black cotton soil	100	93.73	89.6 9	74	33	41	A-7-5(43)						
95+000	Black cotton soil	100	88.93	84	80	38	42	A-7-5(42)						
95+500	Light brown silty clay	100	98.54	87.2 1	67	34	33	A-7-5(34)						
96+000	Black cotton soil				83	31	52	A-7-5(58)	1.41	25	10	1	1	12

		100	95.72	94.2 9							30	1	1	11
											65	1	1	11.6
96+500	Black cotton soil	100	92.74	91.3 6	82	37	45	A-7-5(50)						
97+000	Black cotton soil	100	97.92	97.7 6	75	35	40	A-7-5(48)						
97+500	Light brown silty clay	100	96.67	94.3 8	77	40	37	A-7-5(44)						
98+000	Light brown silty clay	100	91.78	88.3 3	72	31	41	A-7-5(\$2)	1.43	25	10	2	2	8
											30	2	2	8.38
											65	2	2	8.13
98+500	Light brown silty clay	100	97.46	86.4 1	77	39	38	A-7-5(40)						
99+000	Light brown silty clay	100	97.19	91.1 6	66	31	35	A-7-5(38)						
99+500	Light brown silty clay	100	89.91	88.8 5	71	33	38	A-7-5(40)						
100+000	Black cotton soil	100	98	96.3 2	78	45	33	A-7-5(43)	1.38	29	10	1	1	8.75
											30	1	1	7.82
											65	1	1	6.71
100+500	Black cotton soil	100	95.3	92.1	68	37	31	A-7-5(36)						
101+000	Black cotton soil	100	92.87	91.3 8	78	45	33	A-7-5(43)						
101+500	Light brown silty clay	100	97.25	95.8	75	38	35	A-7-5(45)						
102+000	Light brown silty clay	100	32.5	29.2 6	65	35	27	A-2-7(1)	1.45	28	10	3	3	3.56
											30	3	3	2.36
											65	4	4	2
102+500	Light brown silty clay	100	91.84	88.6 4	67	36	31	A-7-5(33)						
103+000	Black cotton soil	100	97.54	94.8 2	72	39	33	A-7-5(40)						
103+500	Black cotton soil	100	91.23	86.3	82	39	43	A-7-5(45)						

				5										
104+000	Black cotton soil	100	93.45	91.4 4	90	37	53	A-7-5(58)	1.39	30	10	1	1	10.6
											30	1	1	10.7
											65	1	1	11.3
104+500	Black cotton soil	100	98.47	98	81	40	41	A-7-5(51)						
105+000	Black cotton soil	100	97.52	91.4	81	43	38	A-7-5(44)						
105+500	Black cotton soil	100	88.89	86.9 2	75	36	39	A-7-5(40)						
106+000	Black cotton soil	100	90.62	82.8 8	94	44	50	A-7-5(50)	1.42	31	10	1	1	8.76
											30	1	1	10.7
											65	1	1	11.7
106+500	Black cotton soil	100	94.75	90.2 3	73	35	38	A-7-5(41)						
107+000	Light brown silty clay	100	97.71	86.7 1	82	44	38	A-7-5(41)						
107+500	Black cotton soil	100	95.58	93.2 2	76	35	41	A-7-5(46)						
108+000	Light brown silty clay	100	89.78	85.2 7	71	35	36	A-7-5(36)	1.42	27	10	1	1	9.62
											30	1	1	11.6
											65	1	1	12.6

Table 2 Unconfined compressive strength laboratory test result on remolded natural sub-grade soil

Station (km)	Moisture Content (%)	Bulk Unit Weight (g/cm ³)	qu (KN/m ²)	C = (qu/2) (KN/m ²)	Strain at Failure (%)
2	35.6	1.93	53.24	26.62	28.57
4	61.96	2.03	205.42	102.71	66.67
6	39.38	1.95	66.87	33.43	24.49
8	26.2	1.87	340.09	170.05	89.80
10	39.6	1.95	162.33	81.17	53.06
12	35	1.80	38.55	19.27	61.22
14	23.7	1.84	139.23	69.61	36.73
16	40	1.64	69.07	34.53	65.31
20	44.6	1.93	128.59	64.30	69.39
24	43.07	2.01	173.42	86.71	57.14
28	33	1.81	278.70	139.35	61.22
30	28.54	2.00	54.93	27.47	25.00
32	49.38	1.81	48.97	24.48	14.29
34	49.77	1.81	80.67	40.33	73.47
36+500	47	1.82	22.67	11.34	24.49
38	47	1.82	36.59	18.29	18.56
40	54.91	1.69	27.44	13.72	53.06
42	36.01	1.91	10.51	5.25	25.00
44	47.38	2.05	26.75	13.37	25.00
48	46.63	1.90	67.35	33.67	37.50
50	47.38	1.83	67.43	33.72	14.43
53	46.63	1.90	67.35	33.67	37.50
58	48.41	1.81	42.50	21.25	20.41
64	53.48	1.94	33.82	16.91	20.41
66	44.22	2.02	35.74	17.87	20.83
68	33.45	1.90	228.67	114.33	14.58
70	34.73	1.84	23.32	11.66	18.75
74	50.93	1.63	26.09	13.05	12.50
80	39.16	1.78	90.31	45.16	5.00
100	57.79	1.76	340.62	170.31	24.00



ANNEXURE - 3

Summary of laboratory results For construction materials for embankment fill and blanketing

Table 1 Classification Test Results

Station	Material descriptions	Atterberg limit			Wet sieve analysis (mm)									AASHTO CLASS (GI)	USC class
		LL	PL	PI	63	50	37.5	25	9.5	4.75	2.00	0.425	0.075		
38+000,2km RHS	Reddish brown silty clay (fine fill)	58	36	22						100	97	94	73	A-7-5(18)	OH,MH, CL,OL
55+000,13km,R HS	Reddish brown silty clay (fine fill)	59	37	22							100	97	94	A-7-5(17)	OH,MH, CL,OL
10+000,5.5kmR HS	Highly weathered and fragmented Ignimbrite	45	37	8	100	100	72.97	57.48	33.15	24	14.78	8.03	6.2	A-2-5(0)	GM,SM
13+000,1.7km, RHS	Unwelded pyroclastic rock (rahyolitic)	42	31	11	100	100	85.22	69.32	37.04	30	17.59	8.67	4.2	A-2-7(0)	GM,GC, SM,SC
50+500,1.5km,R HS	Highly weathered and fragmented Basalt	45	36	9	100	100	70.98	57.28	32.28	25	18.72	8.52	4.6	A-2-5(0)	GM,SM
54+000,2.6km,R HS	Highly weathered and fragmented Basalt	43	31	12	100	100	80.16	64.01	37.94	28	12.23	6.92	4.8	A-2-7(0)	GM,GC,S M,SC
55+000,13km,R HS	Highly weathered and fragmented Basalt	Np	Np	Np	100	98	50	45	34	28	24	8	2	A-1-a(0)	GW,GP
58+000,15km,R HS	Highly weathered and fragmented Basalt	Np	Np	Np	100	86	68	52	28	18	15	7	2	A-1-a (0)	GW,GP
70+000,20km,R HS	Completely weathered and decomposed Basalt to soil	48	36	12				100	95	88	85	84	80	A-7-5(13)	OH,MH, CL,OL
64+000,3.8km,R HS	Highly weathered red scoria	66	40	26							26.88	10.09	1.6	A-2-7(0)	GM,GC, SM,SC
103+700,2.7km, RHS	Highly weathered ,fine grained reddish scoria	66	41	25							28.48	12.21	2.1	A-2-7(0)	GM,GC, SM,SC

Table 2 Summary of embankment fills Proctor and CBR Test Result

Station	Material descriptions	MDD T-180, g/cc	OMC %	CBR	SWELL	Linear shrinkage	LAA
38+000,2km RHS	Reddish brown silty clay (fine fill)	1.47	28.6	4.22	3.2	10.57	
55+000,13km,RHS	Reddish brown silty clay (fine fill)	1.47	31.2	15	2.1	8.46	
10+000,5.5kmRHS	Highly weathered and fragmented Ignimbrite	2.1	10	46	0.34	6.5	28
13+000,1.7km, RHS	Unwelded pyroclastic rock (rahyolitic)	1.92	11	39	0.31	13.43	34
50+500,1.5km,RHS	Highly weathered and fragmented Basalt	2.16	7	47	0.27	7.93	26
54+000,2.6km,RHS	Highly weathered and fragmented Basalt	2.06	9	41	0.3	10.71	32
55+000,13km,RHS	Highly weathered and fragmented Basalt	1.92	6	56	0.08	2.36	
58+000,15km,RHS	Highly weathered and fragmented Basalt	2.2	4.6	110	0.01	2.68	
70+000,20km,RHS	Completely weathered and decomposed Basalt to soil	1	14.6	25	0.16	4.77	
64+000,3.8km,RHS	Highly weathered red scoria	1.34	32	23	0.2	3.93	42
103+700,2.7km, RHS	Highly weathered ,fine grained reddish scoria	1.43	27	25	0.23	3.21	40

ANNEXURE 4 Metrological data

Element: Monthly Maximum Temperature in °c

Region: SHOA

Station: Addis Alem

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
2000	23.9	24.1	25.6	24.9	24.2	24.0	23.2	21.2	22.0	24.1	24.1	23.8
2001	24.1	25.5	24.7	23.6	20.0	23.3	21.9	21.7	23.5	24.4	24.3	24.4
2002	24.6	24.8	24.9	26.9	28.2	26.0	23.1	21.8	23.4	24.2	24.9	23.9
2003	24.8	24.6	26.2	26.1	27.8	23.3	20.7	20.8	22.3	25.6	26.1	25.7
2004	27.0	26.4	27.1	25.2	27.1	24.0	21.3	21.5	22.3	23.8	24.7	x
2005	24.7	27.9	26.5	25.3	24.5	23.7	21.6	22.0	22.8	24.4	24.0	25.4
2006	26.0	26.6	25.2	24.5	25.0	24.2	21.8	20.8	21.9	x	24.5	24.2
2007	24.8	25.1	26.2	24.0	25.4	22.9	21.2	20.4	22.5	24.4	24.8	24.4
2008	25.9	26.5		27.3	25.5	22.5	21.4	20.5	22.7	23.1	23.3	
2009	24.1	25.8	26.7	26.5	26.9		21.3	21.1	23.3			
2010	25.4	25.9	25.7	25.8	25.1	23.9	21.4	27.4	22.4	25.7	25.5	25.4

Element: Monthly Maximum Temperature °c

Region: SHOA

Station: Ambo J.C.A

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1997	x	x	x	x	x	x	x	25.2	26.2	26.4	26.7	27.8
1998	27.7	28.9	28.6	30.0	x	x	x	x	x	x	x	x
1999	28.6	29.6	28.3	28.7	28.0	24.9	21.4	21.9	23.6	24.1	25.3	25.8
2000	26.9	28.3	29.7	27.1	26.8	24.0	22.5	22.0	23.5	24.8	25.3	25.8
2001	26.7	28.5	26.6	28.1	26.6	23.0	22.3	22.3	24.2	26.2	26.2	26.0
2002	26.1	28.2	27.7	28.5	29.3	25.2	23.9	22.5	24.3	26.4	27.0	26.8
2003	27.4	28.7	28.3	27.4	28.9	24.8	22.3	22.3	23.3	25.7	26.3	26.1
2004	27.9	28.0	29.2	27.4	28.2	24.9	22.4	22.8	23.6	25.2	26.7	26.9
2005	27.0	30.1	29.2	28.4	27.0	25.3	22.4	23.3	24.3	24.9	26.3	26.7
2006	27.0	29.2	27.8	26.5	26.5	24.4	22.6	22.2	23.8	25.9	26.3	26.6
2007	27.4	27.6	28.9	27.9	26.9	23.9	22.0	22.0	23.4	25.7	26.3	26.5
2008	28.0	28.7	30.1	29.0	26.6	24.3	22.5	22.3	24.2	26.0	24.9	26.5
2009	27.2	28.5	29.8	28.7	29.2		22.9	23.1	25.5	26.2		

Element: Monthly Maximum Temperature °c
 Region: SHOA
 Station: Kimoye

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1993	25.4	25	27.9	25.1	25	24.2	22.6	21.5	22.3	24.1	24.8	25.6
1994	26.4	27.4		27.3	27.7	23.8	21.5	21.3	22.6	24.5	24.4	25
1995	26.3	27	27.4	25.4	26.4	25.5	21.6	21.7	22.8	25.1	26.2	25.5
1996	24.5	27.7	26.2	26	26	22.9	22	21.8	23.1	24.8	25.2	25
1997	25.4	27.5	28.3	26.7	28.5	25.9	22.9	22.6	24.9	25.2	25.6	26.5
1998	26.6	27.8	27.3	29.3	27.5	25.4	22.2	22	23.5	24.4	26.4	26.2
1999	26.5	28.9	28.1	29.1	27.8	25.6	21.4	22.1	23.2	23	24.7	25.6
2000	26.3	27.5	28.5	27	26.7	24.8	22.5	21.6	22.5	23.7	25.3	25.9
2001	26.3	27.8	25.3	27.3	26	23.5	22.7	22.1	23.2	25.4	26.1	26.1
2002	25.4	27.7	26.6	28	28.8							
2006							23.2	22.4	23.3	26	27	26
2007	27.0	x	x	27.0	26.9	24.5	22.4	22.8	x	x	x	x

Element: Monthly Minimum Temperature °c
 Region: Shoa
 Station: Ambo

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1997	x	x	x	x	x	x	x	11.1	10.1	10.6	11	10.3
1998	12.5	13.3	14.0	x	x	x	x	x	x	x	8.5	8.7
1999	9.6	10.9	12.4	12.0	10.8	11.0	12.0	11.3	10.3	10.8	7.7	9.8
2000	10.9	12.0	14.2	13.1	11.6	11.1	11.7	11.8	11.4	10.9	10.3	9.7
2001	10.3	11.9	12.9	13.0	12.3	12.0	12.0	12.7	9.8	10.3	9.9	10.8
2002	11.5	12.5	11.8	13.0	12.9	12.1	12.0	12.1	11.5	10.6	11.5	12.5
2003	11.3	12.7	13.4	13.0	11.2	11.8	12.8	12.9	11.2	9.4	11.2	10.4
2004	12.7	12.4	13.9	13.5	11.9	12.2	12.0	11.9	11.1	10.6	10.4	11.9
2005	11.0	12.6	12.9	14.1	12.6	11.3	11.8	11.6	11.5	9.4	9.0	9.0
2006	11.9	12.5	12.3	13.1	10.9	11.5	12.6	12.7	11.5	11.5	11.8	12.3
2007	12.6	11.9	13.0	13.5	13.5	12.8	12.6	12.5	12.3	10.1	11.4	10.3
2008	12.6	13.1	13.8	14.0	12.7	12.6	12.2	11.8	11.3	11.2	11.3	11.6
2009	12.3	13.3	13.9	14.4	13.0		12.9	13.1	10.8	11.5		

Element: Monthly Minimum Temperature °c

Region: Shoa

Station: Addis Alem

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1997	x	x	x	x	x	x	10.7	10.5	9.9	9.3	8.3	7.0
1998	9.6	11.0	11.2	11.9	11.5	10.9	11.0	11.8	10.3	8.6	4.4	2.7
1999	5.9	6.6	9.2	9.2	10.1	9.9	10.0	9.9	9.6	8.5	4.5	4.4
2000	4.5	5.2	7.3	7.5	7.5	9.4	9.5	9.5	9.7	7.8	5.7	4.7
2001	5.7	6.3	7.0	6.9	4.1	9.3	9.3	10.3	8.6	7.7	5.6	5.5
2002	7.0	7.3	7.3	8.6	9.6	9.2	9.9	9.5	7.6	6.4	6.1	8.7
2003	5.7	7.3	9.3	9.2	9.7	9.5	9.8	10.6	11.1	12.7	15.1	15.3
2004	13.5	12.6	14.1	12.8	14.9	12.0	9.8	10.5	12.8	12.4	13.0	x
2005	11.3	14.1	13.7	12.8	13.4	11.9	9.9	11.3	12.9	12.3	8.2	2.0
2006	5.8	10.3	10.7	11.7	10.6	10.6	11.2	11.5	10.3	x	7.7	7.8
2007	8.6	10.2	10.1	11.0	10.9	10.7	11.3	11.1	10.5	7.1	6.4	4.9
2008	7.6	7.8	8.8	10.6	11.4	11.2	10.1	10.8	10.3	8.8	7.3	
2009	8.4	9.2	10.2	10.4	11.6	x	11.0	11.0	10.2			
2010	7.8	10.6	10.1	10.7	12	10.5	11.3	11.2	10.4	7.7	6.7	

Element: Monthly Minimum Temperature °c

Region: Shoa

Station: KIMOYE

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1994	5.9	8.1	12.0	12.0	11.8	12.0	12.7	12.2	10.4	7.0	7.7	5.8
1995	7.1	10.5	11.5	13.3	11.8	10.9	12.1	12.3	10.5	7.8	7.2	8.7
1996	9.8	9.0	12.3	11.5	11.6	12.1	12.3	11.7	10.9	7.5	7.2	7.0
1997	10.4	7.3	11.9	12.1	11.7	12.4	12.1	12.1	11.0	10.5	10.1	7.7
1998	10.8	12.3	12.7	12.9	13.1	11.8	12.8	13.2	12.1	10.3	5.4	4.1
1999	7.2	7.4	11.5	11.4	11.8	11.5	12.0	11.5	10.4	9.9	5.2	6.1
2000	6.5	7.4	10.1	12.3	11.8	11.4	12.1	11.6	11.2	9.6	7.5	6.5
2001	8.0	8.6	11.9	11.5	12.3	11.5	12.3	12.5	10.2	9.0	6.7	7.3
2002	9.2	9.0	12.3	11.9	13.3	x	x	x	x	x	x	x
2006	x	x	x	x	x	x	12.1	11.9	10.8	9.3	7.5	9.8
2007	8.9	x	x	12.8	12.7	12.3	13.0	12.5	x	x	x	x

Element Monthly Rainfall In mm
 Region SHOA
 station SEBETA

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1999	0.0	23.3	48.2	14.0	56.8	98.1	218.0	220.8	77.2	102.1	1.1	1.3
2000	x	0.0	3.2	75.7	37.3	96.3	173.7	182.5	216.0	26.8	21.4	24.6
2001	0.0	x	158.0	11.1	156.4	118.7	273.3	163.6	65.6	20.5	0.0	0.0
2002	25.4	47.2	100.1	31.8	30.6	x	173.0	251.1	113.0	0.6	0.0	21.1
2003	3.0	5.8	52.1	76.4	13.6	122.7	274.9	327.4	118.2	1.2	0.0	39.2
2004	17.2	15.2	46.3	122.8	19.4	136.6	261.5	189.7	166.0	55.9	0.0	0.0
2005	x	24.5	61.3	135.3	193.2	91.5	157.2	161.6	69.6	3.2	9.3	0.0
2006	0.0	13.8	53.0	84.0	51.5	81.1	288.2	273.0	185.4	13.1	0.0	0.3
2007	23.6	28.2	31.7	100.1	68.4	125.4	83.6	242.5	128.2	3.6	0	0
2008	0.0	0.0	0.6	21.5	56.6	134.7	239.4	313.4	201.2	41.5	83.7	0.0
2009	30.1	0	4.2	65.3	24.4	62.8	287.5	246.9	115.3	20.2	1.5	47.4

Element: Monthly Rainfall in mm
 Region: SHOA
 Station: WELENCOMI

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1995	0.0	45.6	30.8	135.1	67.0	94.9	207.9	249.4	129.1	0.0	0.0	14.3
1996	68.9	4.1	165.4	67.0	158.0	167.6	293.0	210.6	129.0	7.4	3.5	1.9
1997	38.1	0.0	18.0	78.7	36.1	138.1	276.5	207.4	121.2	45.0	47.0	0.0
1998	50.1	39.8	36.8	51.4	87.2	221.7	241.2	220.3	138.1	70.0	2.4	0.0
1999	4.8	0.0	8.2	26.8	62.1	127.8	197.1	175.6	83.6	57.3	0.0	0.0
2000	x	x	x	x	x	x	185.7	202.1	145.9	55.0	26.5	9.4
2001	0.7	8.9	115.1	26.8	110.3	161.5	267.6	211.2	57.7	15.2	0.0	0.0
2002	38.0	16.8	43.8	32.2	34.5	195.5	224.2	175.9	45.2	2.0	0.0	97.7
2003	22.2	0.0	103.5	130.3	14.9	177.6	315.8	185.1	61.8	0.0	2.3	2.4
2004	49.4	2.4	22.8	171.1	22.4	x	235.1	198.7	164.4	10.0	0.0	0.0
2005	62.8	0.0	81.5	126.5	119.6	151.6	264.3	118.2	77.3	3.4	2.5	0.0
2006	0.0	25.4	87.2	70.0	67.6	x	x	x	x	x	x	x
2007	30.2	28.7	29.2	x	x	167.4	219.1	190.3	85.3	20.7	x	x
2008	0.0	13.7	0.0	21.6	113.5	x	x	271.9	x	x	x	x

Element: Monthly Rainfall in mm

Region: Shoa

Station: GINCHI

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1999	16.3	0.0	26.5	163.4	66.1	150.3	226.4	196.0	96.4	178.7	4.2	0.0
2000	0.0	0.0	8.3	129.4	85.2	159.4	141.6	236.2	185.9	43.7	37.0	
2001	3.5	47.3	118.4	33.8	112.4	270.7	238.0	151.0	96.5	31.3	2.5	0.0
2002	112.7	76.5	88.7	40.5	38.2	157.2	209.4	160.1	79.9	0.0	0.0	61.4
2003	19.6	61.2	107.0	162.5	15.4	141.6	227.8	207.0	117.7	0.0	2.1	14.9
2004	107.8	6.7	21.4	214.1	57.9	132.6	198.4	200.4	178.0	19.1	0.0	1.0
2005	65.6	2.5	119.5	45.6	85.5	116.4	260.5	241.6	185.8	12.8	12.5	0.0
2006	0.0	15.3	320.4	191.5	133.5	237.8	434.2	629.2	157.1	19.5	0.0	14.1
2007	19.9	24.1	37.3	31.6	157.4	248.1	232.3	182.6	122.1	35.3	0.0	0.0
2008	0.0	5.7	0.0	18.1	100.6	152.7	294.9	236.6	111.2	41.7	62.6	0.0

Element: Monthly Rainfall in mm

Region: Shoa

Station: Ambo J.C.A

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1997	28.7	0.0	31.0	83.1	59.7	132.0	92.1	183.4	59.2	87.5	22.1	2.2
1998	74.3	10.4	40.6	30.7	X	X	X	X	X	X	X	0.0
1999	8.4	1.0	39.5	20.1	99.2	108.4	195.9	132.9	95.9	119.9	1.3	X
2000	0.0	0.0	9.3	51.5	93.7	121.2	185.5	191.6	131.2	83.7	20.7	14.8
2001	0.0	10.2	52.5	70.4	148.3	148.5	180.8	243.1	109.9	41.8	5.4	11.0
2002	78.7	16.9	55.6	56.3	39.5	178.0	171.9	111.7	40.3	3.0	0.0	17.2
2003	41.7	100.0	54.8	106.0	9.4	209.0	133.9	142.2	71.2	9.3	1.0	10.4
2004	39.2	16.6	30.2	108.7	26.8	137.1	203.6	215.0	138.9	19.0	0.0	8.9
2005	25.2	0.0	86.8	47.5	58.1	166.3	158.0	187.1	98.4	19.0	10.2	0.0
2006	0.0	4.7	150.7	72.3	157.7	109.9	196.8	298.6	76.5	17.9	18.8	0.0
2007	49.3	54.3	40.2	38.9	131.6	275.2	232.6	218.3	111.7	11.0	0.0	0.0
2008	0.0	0.0	1.0	18.7	157.3	161.8	308.0	260.2	84.0	64.8	101.8	2.7
2009	26.9	23.1	28.7	61.2	47.5	x	285.7	271.9	65.3	56.1	x	x

Element: Monthly Rainfall in mm

Region: Shoa

Station: Addis Alem

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1998	61.1	86.8	107.4	67.3	101.1	208.1	289.5	251.0	154.0	53.9	21.9	0.0
1999	29.5	3.3	26.6	74.6	13.5	197.5	255.0	262.2	46.8	116.6	0.0	0.0
2000	0.0	0.0	12.5	108.1	135.4	168.1	266.6	238.2	186.8	33.6	17.4	29.2
2001	0.0	0.0	169.4	28.2	124.3	233.9	309.7	265.0	43.3	21.4	0.0	0.0
2002	0.0	84.1	59.2	0.0	0.0	145.9	139.9	103.2	38.4	0.0	0.0	62.4
2003	0.0	65.8	109.6	84.1	0.0	173.1	226.0	232.1	165.3	11.0	6.6	4.0
2004	17.5	27.6	70.5	144.6	69.9	143.2	190.4	212.5	172.5	17.9	1.3	x
2005	14.2	15.2	139.2	55.1	96.9	93.4	218.8	224.0	99.0	21.9	12.5	0.0
2006	0.0	83.5	101.0	61.1	83.2	148.9	334.3	265.4	135.5	x	0.0	9.8
2007	x	59.3	120.1	94.9	123.4	170.1	237.3	204.3	75.0	23.1	0.0	0.0
2008	1.9	12.5	14.6	36.9	125.6	145.1	291.5	268.1	85.6	46.9	32.6	x
2009	43.2	2.8	48.6	11.3	51.5	x	297.6	280.6	95.0	x	x	x
2010	0.4	87.2	99.0	112.8	73.7	156.1	340.1	250.4	233.4	8.8	0.0	x

Element: Monthly Rainfall in mm

Region: SHOA

Station: KIMOYE

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1997	23.4	0.0	20.4	37.2	24.7	89.6	248.1	149.8	59.1	40.5	47.5	1.9
1998	49.5	38.8	108.6	65.5	121.3	263.4	174.4	202.8	149.5	63.4	5.0	0.0
1999	16.1	0.0	15.7	28.5	45.5	113.7	211.3	196.8	56.9	85.1	0.0	0.0
2000	0.0	0.0	3.7	60.9	80.5	123.6	223.0	207.5	135.3	48.9	9.6	53.9
2001	11.3	0.0	161.6	24.5	151.8	137.8	247.8	213.7	49.7	14.1	1.9	0.0
2002	38.2	21.8	64.8	49.0	53.4	189.2	193.4	198.0	21.1	0.0	0.0	32.8
2003	24.3	99.3	37.8	104.9	14.2	154.8	184.1	165.8	121.6	1.6	4.7	1.9
2004	26.0	3.1	26.8	180.1	36.5	169.5	261.2	189.8	152.0	38.1	0.0	0.3
2005	34.7	8.4	115.9	92.9	114.0	112.7	162.1	176.9	134.7	32.4	5.7	0.0
2006	0.0	20.5	90.7	78.3	109.3	112.8	241.1	207.9	114.1	13.2	0.0	1.0
2007	17.7	x	x	94.2	141.2	172.8	246.7	218.6	x	x	x	x

