JIMMA UNIVERSITY
SCHOOL OF GRADUATE STUDIES
JIMMA INSTITUTE OF TECHNOLOGY
FACULTY OF CIVIL AND ENVIROMENTAL ENGINEERING
HIGHWAY ENGINEERING STREAM

EVALUATION OF THE PERFORMANCE OF GRAVEL ROAD: A CASE STUDY ON MECHARE TO ARSSEMA ROAD, AMHARA REGION, ETHIOPIA

A final thesis submitted to the Faculty of Graduate Studies of Jimma University in Partial Fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering (Highway Engineering).

By:

Ashenafi Alemu

December, 2019

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December, 2019

Jimma, Ethiopia

Jimma University

Jimma Institute of Technology

School of Post Graduate Studies

Faculty of Civil and Environmental Engineering

Highway Engineering Stream

EVALUATION OF THE PERFORMANCE OF GRAVEL ROAD (A CASE STUDY ON MECHARE TO ARSSEMA ROAD SEGMENT)

By

Ashenafi Alemu

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DECLERATION

I, the undersigned, declare that this thesis entitled "Evaluations of the performance of gravel road." is my original work, and has not been presented by any other person for an award of a degree in this or any other university, and all sources of material used for theses have been duly acknowledged.

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As Master Research Advisors, we hereby certify that we have read and evaluated this MSc research prepared under our guidance, by **Ashenafi Alemu** entitled: Evaluations of the performance of gravel road.

We recommend that it can be submitted as fulfilling the MSc Thesis requirements.

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ACKNOWLEDGEMENTS

First, I would like to gratefully acknowledge the almighty God and his holly mother for their guidance through.

Secondly, my deepest gratitude goes to my advisor **Prof.-Dr.-Ing. Esayas Alemayehu** and my co-advisor **Engr. Burka Ibrahim** for all their limitless efforts in guiding my work and supports.

Thirdly, I gratefully acknowledge the support of my instructor's **Engr. Oluma Gudina** and **Engr. Teyba Wedajo** for all their limitless efforts in guiding my work and supports.

Mores so, my deepest appreciation goes to Woldia University soil and highway laboratory assistances to their limitless support guidance to conducting all the necessary laboratory tests.

Finally, my last but not least appreciation goes to Woldia university technology faculty help in laboratory test activities and Jimma University, School of Graduate Studies, Jimma Institute of Technology, Civil Engineering Department, and Highway Engineering Stream.

ABSTRACT

In Ethiopia, gravel road deterioration is becoming a common problem and great challenge, appearing even before the completion of a project in certain road projects. In most areas gravel road problems like rutting, fully erosion and corrugation and loose of materials are common before the design life and it require a lot of maintenance cost. In such case, the main objective of this research was to evaluate the performance of gravel road based on material quality, gravel loss, effects of poor drainage system and present serviceability rate of the road. The study was conduct in Mechare to Arssema existing road found in north wollo zone, Amhara region, Ethiopia.

The research designs were follow both experimental and analytical methods using qualitative and quantitative data. The study follows non-probability sampling techniques of purposive sampling methods. In order to examine the properties of subgrade soil and gravel material, disturbed soil samples were obtain from five different test pits in different locations at depth of 1.5-2.0m to remove organic matters for each subgrade soil and gravel materials on the existing route. The data processing and analyzing were conduct using both descriptive and analytical methods thus were Excel for laboratory analysis and Arc GIS for hydrological analysis.

In this study moisture content, Atterberg Limits testing, particle size distribution, soil classification, free swell index, specific gravity, compaction and CBR were determined for both subgrade soil and gravel material. Based on laboratory analysis the subgrade soil sample was high liquid limit, low CBR swell value. In gravel material property analysis all soil samples were classified in to A-2-7, the material type is sand with gravel, the material quality is zone B which is corrugated and raveling and the soil samples had 20.85%, 7.19%, 12.57%, 6.98% & 6.14% soaked CBR value, with 0.86%, 1.72%, 1.03%, 1.93% & 2.02% CBR swell respectively. In the case of present serviceability rate study the questionnaire were distributed and filled by road users which found under each section the present serviceability rate was 2.63 and the gravel loss determinations of the road using TRH20 gravel loss deterioration model the average gravel loss were 14.37mm. In condition survey, the existing road was evaluated using road condition rate. The road drainage system was evaluating field observations and peak discharge determination by using rational method to compare design discharge with their return periods. Catchment area of each watershed overall route corridor was delineate from DEM data and the sizes of each catchment area were determined using Arc GIS software.

Generally, the laboratory result shows both subgrade soil and gravel materials have poor material quality compared with standard specifications. The gravel loss of existing road does not consider as shown in the design profile, because the design thickness was different from the calculated thickness. During the condition survey evaluations the existing road is little or no roadway crown, moderate to severe wash boarding, severe loose aggregate, and moderate potholing. The design peak discharge calculated and the review peak discharge calculated of the return period is not equal. From this the road around this is damaged by over flooding of the water on the road because the calculated peak discharge for the design were less than the calculated peak discharge for the review at all stations.

Key Words: - *Gravel Road, performance evaluation, subgrade strength, gravel loss, road drainage.*

TABLE OF CONTENTS

Contents **pages**

LIST OF TABLES

LIST OF FIGURES

LIST OF ABBREVIATIONS AND ACRONYMS

AADT	Annual average daily traffic volume
AASHTO	American Association of State Highway and Transportation Officials
ADT	Average daily traffic
ARRA	Amhara rural road authority
ARRB	Australian Road Research Board
ASTM	American Society for Testing Materials standard
CBR	California Bearing Ratio test
CSA	Central Statistical Agency of Ethiopia
DDM	Drainage Design Manual
ERA	Ethiopian Road Authority
FSI	Free Swell Index
GL	Gravel Loss in mm
GM	Grading Modulus
HDM	Highway Development and Management
IDF	Intensity-Duration-Frequency
JIT	Jimma Institute of Technology
JU	Jimma University
LL	Liquid Limit
LS	Linear Shrinkage
LVR	Low Volume Road
PC	Pipe Culvert
PF	Plasticity Factor
PI	Plasticity Index
MMP	Mean Monthly Precipitation
PMDM	Pavement Material and Design Manual
PL	Plastic Limit
PSI	present serviceability Index
SCS	Soil Conservation Service
SP	Shrinkage Product
SSRW	Standard Specifications for Road Works
TRL	Transport Research Laboratory
TRH20	Technical Recommendations for Highways Manual (20)
URM	Unsealed Roads Manual

CHAPTER ONE

INTRODUCTION

1.1 Background

The importance of road in the development of any nation can hardly be over-emphasized, as it plays a strategically important role in the transportation of good and services. This commonly achieved through the vast network of roads that connect the rural and urban centers. Efforts at achieving the construction of more roads is delayed by the high cost of building new roads, which is attributable to non-availability of sound quality road building materials within the environment of many road project (Joel & Edeh., 2015).

In Ethiopia, the transport sector is a large working environment in which the road transport, air transport, marine transport and rail transport are functional. However, the dominant mode of transport is road transport, having a share of 90% in transporting passenger and cargo transports across the country. The Federal Road Authority is responsible in constructing and maintaining of the roads that connect region to region and sometimes it may manage the roads that have higher volume, which starts and ends with in one region. The regional road authorities only have low volume road, which are gravel roads that connect Zones and Wereda's in the region country (FDRE Ministry of Transport, 2014).

According to the 2011 data of Ethiopian Road Authority the country has 8,295 asphalt road networks 14,136 Gravel road networks and 30,712 rural road networks which sum up to total of 53,143km road networks, out of this, 44,848 which is about 84.4% of the country road network is unsealed, with road density of 0.65 per 1000 population in kilometer.

Unsealed gravel roads are vital first link in local economy as the Ethiopian economy based on agriculture and the agricultural products are transport on these unpaved gravel surfaced roads (ERA, 2011).

The repetitive traffic loading that the road experiences service life combined with environmental factors topography, climate change and other forms of deterioration, which ultimately degrade the serviceability, and durability of pavement structures.

More than 75% percent of the road network in sub- Sahara Africa countries regardless their traffic volume are unsealed roads, mostly surfaced by gravel materials or natural earth materials (Overby & Pinard, 2007).

Gravel road performance is one of the most important measures for pavement layers performance condition. These unpaved roads are located in the agricultural, forest areas, in cities, town and villages, although they are low-volume and low load bearing roads (Simeneh M., 2012).

Gravel roads are vital first link in the local economy as the Ethiopian economy is based on agriculture and the agricultural products are transported on these unpaved gravel surfaced roads. The existing road network has deteriorated a lot to the extent that only 11% of the paved roads and 19% of the gravel roads are presently in good condition and performing well persuade to the Ethiopian Road Authority (2011) data. For most developing and emerging economies, the road maintenance challenge is dominated by the maintenance of unsealed roads. Over the years, engineers have become knowledgeable at optimizing resources for maintaining unsealed roads. Various manuals and guidelines have produced by authorized organizations like ERA and Addis Ababa City Road Authority (AACRA), with extensive unsealed road networks.

Gravel loss is defined as the change in thickness of gravel roads surfacing over a period. The rate of gravel loss is recorded as the vertical loss in mm of material from the road surface. The loss of material for different types of material is not the same (Giummarra and D. C. Roux, 2008). Using the above definition at hand, gravel roads are designed having two kinds of layer. The first one, which is directly in contact with traffic, is designed considering the loss of surface material due to traffic, rainfall or precipitation and others. The second layer is designed to protect the subgrade from excessive compressive strain. Therefore, the thickness of the gravel road is the combination of or assumption of consideration of gravel loss and protection of the subgrade from damage (ERA, 2002).

Drainage system is a process of removing & controlling excess surface water with in right of way. Drainage is an important feature in determining the ability of given pavement to with stand the effects of traffic and environment. (Adequate drainage is very essential in the design of highways since it affects the highway's serviceability and usable life. If ponding on the traveled way occurs, hydroplaning becomes an important safety concern. Drainage design involves providing facilities that collect, transport and remove storm water from the highway (O'Flaherty, C.A., 2002).

Generally, many literatures, researches, and books agree that gravel road deterioration are the major problems of constructed road and it has a great negative impact on the development of road construction sectors in different countries like Ethiopia.

Studying the performance of these gravel pavement structure under traffic, climatic condition that they undergo after construction is essential. Even though the cause for the deterioration of this low volume road is too many, studying the material loss on the environmental and structural strength of the material with the traffic influence causes is desirable. Heavy vehicles load on the pavements subjects to high stresses causing damage. The road performance, serviceability and durability of the unsealed roads, which is one important way to minimize the pavement failure problem (Paige-Green 1989).

1.2 Statement of the Problem

In the world, different factors affect the overall performance of roads whether they are asphalt surfaced or gravel surfaced. A road is designed for parameters like traffic, surrounding atmospheric condition, and material property based on certain design principles and the standard for the intended use of the road.

In Ethiopia gravel, road deterioration is becoming a common problem and great challenge, consuming a lot of money, in some cases failure is appearing even before the completion of a project in certain road projects (Simeneh, M., 2012).

In most areas gravel road deterioration as loose material, rutting, erosion, and corrugation are common before the design life and the deteriorations are require a lot of maintenance cost.

From the above-mentioned problems, it is clear that gravel road deterioration and damages are the major problems in road construction industry in developing countries like Ethiopia, which needs special and organized consideration to overcome the failure of road.

These research focuses on factors affecting the performance of gravel road and determinations of gravel loss specific in Mechare to Arssema road and to identify common factors and notify recommendations to handling gravel road deteriorations. In addition, the research was attempts on the common factors affecting of gravel road, such as subgrade soil quality and gravel material quality, environmental condition and improper drainage facility on the existing route.

1.3 Research Question

The research was to answer the following research questions:

- What are the effects of subgrade soil and gravel materials on the performance of existing gravel road.
- How to evaluate the present serviceability rate and gravel loss on the study route?
- What is the hydrological condition of the existing road drainage system of the study road?

1.4 Objectives of the study

1.4.1 General objective

The main objective of this research study was to evaluate the performance of gravel road in Mechare to Arssema existing road.

1.4.2 Specific Objectives

The specific Objective of the research study includes the following:

- ✓ To evaluate the effect of subgrade soil and gravel material quality on the performance of existing gravel road.
- ✓ To evaluate the present serviceability rate and gravel loss on the existing gravel road.
- ✓ To evaluate the hydrological condition of the existing road drainage system in the study route.

1.5 Significance of the Study

The aim of this research was identify common factors and great challenges on gravel road performance. A good road structure helps in reducing the number of accidents and cost of road maintenance.

The gravel road deterioration is consuming a lot of money for maintenance and rehabilitation activities. Therefore; this study were minimize maintenance expense and find out reasons for inadequate provision of gravel road. The study was identifying the gap between standard specification and construction material properties along the existing route.

A gravel loss determination is a distress specific performance model that used to predict gravelling materials performance. The prediction of the expected material loss from a gravel road`s wearing course is greatest importance for gravel road design, addressing construction short falls and maintenance planning (Paige-Green, 1989).

This research also aimed at coming up with findings on the effects of poor road drainage system and poor material qualities in existing roads to minimize maintenance expenses proper protection and management of these road assets.

Finally, the research was identifying factors are more affecting the performance of gravel road and tries to propose remedial measurements for the problems.

1.6 Scope of the Study

This study was supported by different types of literatures, field observations, road user assessment, condition survey and series of laboratory experiments. However, the findings of the research were limited to the major cause of the road at different sections generally affected by subgrade soil and gravel material problems, and the hydrological situation of the existing road drainage system. Moreover, the study limited to evaluate the performance of gravel road when subjected to poor drainage system and unsuitable (weak) materials.

The relevant laboratory tests were conducted grain size analysis, Atterberg limit, Compaction; CBR and Free swell index tests both subgrade soil and gravel materials. The drainage problems were conduct based on hydrological study, which is determining peak discharge and culvert capacity for nine existing drainage system and; compared calculated discharge and design discharge. For this study, traffic volume counts were conduct in the main roads of existing road to determine ADT used for gravel loss. Then the material properties were comparing the results with ERA, AASHTO and ASTM specification likewise a recommendation was drawn and forwarded.

CHAPTER TWO

LITERATURE REVIEW

2.1. Introduction

2.1.1 Gravel Roads

According to William B. A. et.al (2001), the term gravel road is generally used to refer to all unpaved roadways. A true gravel road is a roadway whose surface layer is construct of mineral aggregate materials (such as sand, gravel, small rock or crushed stone) that are generally obtain from gravel pits and quarries. In line to this definition, Gravel roads are built and designed to certain engineering principles, including the supply, where warranted, of gravel wearing surface. Construction of these roads also involves a defined cross section, drainage and structures (bridges, culverts).

Good gravel road is construct of three different layers. The subgrade or roadbed is the bottom layer made up of the natural material (clay, silt or sand) found along the roadway alignment or fill to level a depression. The aggregate base is place on top of the subgrade and is ideally 45cm to 60cm depth. It should constructed from free draining and easily compactable gravel material (crushed stone) that produces a strong and stable layer. Such aggregate base materials should contain a minimal amount of fines (materials with a very small particle size such as clay or silt) since they tend to inhibit the free drainage of water, which could reduce the strength of the aggregate base (Austroads, 2003).

The surface layer (uniformly graded gravel or crushed stone) is placed on top of the aggregate base and it is at least 20cm in depth. Gravel roads can also be known as unpaved roads. This definition is clearly defined in the book, which says, “An unpaved road is a road with a soil or gravel surface” (Paige, Green 2000).

2.1.2 Factors affecting gravel road performance

According to Jones (1984), Dierks (1992), TRL, the rate of gravel road deterioration depends on traffic characteristics, intensity and duration of rainfall, wind forces, gradient, alignment, surface cross-fall , road width, natural weathering (mechanical and chemical) of gravel materials, subgrade material quality and characteristics, compaction achieved on respective layers of road the road structure and maintenance practices. Gravel roads, due to their nature of construction, are prone to deterioration by different factors. These factors are traffic (speed, volume and axle loads), environmental factors especially climate (temperature and precipitation), surfacing material (type and nature) and geometrical design of gravel roads. Due to these factors, gravel roads deteriorated early than anticipated by their design.

2.1.2.1 Environmental Factors

The environment in which a road is built has a greater influence on its life and performance than is generally realized. This environment has to be accepted as it is and the design and method of construction must be suitably adapted. The environment of a road is the sum of all those external conditions, inorganic as well as organic, to which a road is exposed and the more the road is in harmony with this environment, the better its performance. The environment characterized by topography, the climatic conditions (moisture and temperature) under which the road will function, and the underlying sub grade conditions (Jones, 1984).

2.1.2.2 Geometric road requirements

A. Design requirements

Most unsealed roads have developed over the years from routes that may have originally been built for the horse and cart, with little or no attention given to applying appropriate geometric designs to suit current motor vehicle requirements. As a result, there are many geometric design deficiencies on existing roads relating to narrow road widths, tight curves, poor drainage provisions and limited sight distances that can lead to higher gravel deterioration, increased maintenance costs and poor safety (Dierks, 1992).

B. cross-section

In the majority of cases, unsealed roads are either one lane two-way or two lane two-way. The main deciding factor as to whether a road is one or two lanes depends on the average daily traffic (ADT) carried and vehicle types. Changeover point is when a projected ADT exceeds 150 vehicles. Suggested minimum desirable road cross-section. If road widths fall between these values, then a road will exhibit a 'three wheel' effect, causing higher road maintenance and greater gravel loss due to the road crown having double the wear (Austroads, 2003).

C. Road cross falls

For unsealed roads it is critical that the road surface has a cross fall of between 4–6% in order to quickly shed water from the surface. If the cross fall is allowed to go flat (<4%), water is likely to remain on the surface, and the resulting ponding will lead to a weakening of the pavement structure and the rapid formation of potholes.

Cross-falls higher than 6% would have higher cross scour erosion and safety risks. Maintaining roads with the required cross fall will ensure better ride quality, lessen the risk of break-up of the road surface and considerably reduce routine maintenance operations (ERA, 2011).

D. Horizontal alignment

Poor road alignment–in terms of tight curves, insufficient super-elevation and high vehicle speeds–means that vehicles exert greater sideways force around a curve and cause the road surface to break up.

This creates loose gravel, which is more prone to twisting, erosion, dust emission and gravel loss through whip-off, wind and rain action.

E. Vertical alignment

Steep vertical grades (>8%) should be avoided on unsealed roads, as the road surface does not have the binding properties to withstand the acceleration and deceleration forces exerted by heavy vehicle drive axles. Steep grades can also lead to drainage channels being form down a road, causing the washing away of loose gravel. Various techniques are available to minimize the amount of scouring caused by water that can occur on a steep grade. The reader referred to the unsealed roads manual: Guidelines to good practice (ARRB, 2000).

2.1.2.3 Drainage structure

One method for increasing the life of a pavement structure is to make sure that drainage patterns exist that can quickly and effectively remove surface moisture.

Proper pavement cross slope and adequate roadside ditches are very effective in quickly getting surface runoff away from the pavement. However, even good surface runoff drainage will not entirely keep water out of the pavement. Water can enter the pavement by several routes through pavement joints and cracks, from adjacent grassed and landscaped areas, from high water tables, and via moisture vapor. An effective method of removing water that has entered the pavement system is the use of a subsurface drainage system (underdrains). Underdrains can consist of longitudinal drains along the edges of a pavement or lateral drains placed at the low points along the pavement's vertical profile (ARRB, 2000).

a) Surface drainage

Surface drainage consists of those elements that collect and remove water from the surface of the road and areas adjacent to the roadway. It includes culverts and any other drainage systems designed to intercept, collect and dispose of surface water flowing towards and onto the road surface from adjacent areas.

The importance of providing adequate cross fall to allow surface water to run off the pavement is paramount for unsealed roads. It is highly desirable that, in all relatively flat or very gently undulating country except perhaps in arid areas, raised formations should be used. However, in areas of negligible slope, which are prone to flooding, a raised formation may act as a dam for floodwaters. In such cases, the alignment should be along any slightly higher elevated sections of the ground surface (ARRB, 2000).

b) Subsurface drainage

Subsurface drainage systems drain water that has infiltrated through the pavement and the inner lope but also groundwater. Subsurface drainage systems are directly linked surface drainage systems (O'Flaherty, C.A., 2002).

According to the SRA handbook, culverts are road constructions with a theoretical span of ≤ 2.0 m. Culverts have an open inlet and outlet and conduct water underneath a road. The need for subsurface drains as alternatives to open drains depends on site conditions. Subsurface drainage consists of three basic elements. A permeable base, which is required to provide for rapid removal of water, which enters the road structure, a method of conveying the removed water away from the road structure and this, may consist of a base, sloped towards a drainage ditch. At the most, this may consist of a pipe collector system and a filter layer to prevent the migration of fines into the permeable base from the sub grade, sub base or shoulder base material (Wyatt & Macari, 2000).

2.1.2.4 Failures of road drainage system

The roadway shall not obstruct the general flow of surface water or stream water in any unreasonable manner to cause an unnecessary accumulation either of water flooding or water saturated uplands, or an unreasonable accumulation and discharge of surface water flooding or water saturated lowlands.

The failure of road occurred on Mechare to Arssema road due to inadequate capacity of the drainage. If the failure is sudden and catastrophic, it can result in injury or loss of life and property (O'Flaherty, C.A., 2002).

2.1.3 Traffic requirement for gravel roads

Gravel roads are low volume roads in rural environment having traffic volume less than 300 vehicles per day (Behrens, 1999).

The Ethiopian Low Volume Road Manuals defines gravel road pavements as the road designs, which have Annual Average Daily Traffic (AADT) less than 300 at the time of construction. In addition, Harral and Faiz (1988) suggested that, gravel road can provide a good service under traffic volume ranging from 150 to 300vehicles per day. On other hand, traffic volume regardless the type of traffic and loading are used as the criteria for decision making in the management of roads. As supported by Shuler (2007), traffic volume can be used as the simplest criteria for decision-making.

Roads are designed and constructed to provide smooth or comfortable riding surface for vehicles at the same time to sustain the loads and traction effects caused by vehicles. If the volume of vehicles and its effects exceeds the capacity of the existing road structure, then problems can be revealed which pose to consideration of other type of road surface, which can sustain the effects. For cost-effective decisions on construction and economical maintenance strategies of gravel roads, there is a great need for road engineers and planners in a specific climate zone to understand the relationship between traffic volumes; construction standards, deterioration rate and maintenance level (Ellis, 1979).

2.1.3.1 Traffic volume studies

Traffic volume is defined as the number of vehicles that pass a point along a roadway or traffic lane per unit of time (Wright, Dixon and Meyer, 2004). Traffic volume studies are carried out to collect data on the number of vehicles that pass on a particular point on a highway facility during a specified time. Traffic volume studies are usually conducted when certain volume characteristics are needed (ERA LVM, 2011).

2.2 Engineering property of pavement material

According to layered pavement design method (1965), the pavement materials are an important component of pavement design, the selection of appropriate quality of materials for selected, sub grade, sub base, road base courses determine the capital, and whole life costs of the road, which primarily determines the performance of the road. In the selection of pavement materials guideline principles used for the material performance indication are California bearing ratio (CBR) strength, gradation, atterberg limits (liquid limit and plasticity limit) and plasticity indexes.

Major components of a pavement structures are:

1. Surface coarse
2. Base coarse
3. Sub base
4. Compacted sub grade
5. Natural sub grade

Bases and sub bases are usually granular materials or aggregates. The sub base which is lower in the structures does not require as high quality material as the base as loads are reduced considerably. The compacted sub grade may be the surface layer of the sub grade, compacted in cut areas, or the embankment materials in fill zones. The main function of a pavements is to reduce the high unit stress imposed by the vehicle on the surface to stress on the sub grade that are low enough to be carried without failure due to rutting, excessive settlement or other type of distress. The magnitude of stress reduction is mainly is the function of the thickness of the pavement structure. Therefore, the main variable in the design of pavement structure is the thickness.

The major factors involved in the design of pavement thickness are:

1. The magnitude of imposed loads
2. The strength of sub grade soil

Base courses in either pavement structures are composed of solely granular materials (aggregate), or soil or granular materials stabilized by an additive. Granular base courses are mainly aggregates from sand or gravel deposits or from quarries. The properties required in the materials vary with the type of pavement and the depth of the material in the pavement structure.

2.2.1 Subgrade soils

The type of subgrade soil is largely determined by the location of the road. However, where the soils within the possible corridor for the road vary significantly in strength from place to place, it is clearly desirable to locate the pavement on the stronger soils if this does not conflict with other constraints. For this reason, the pavement engineer should be involved in the route corridor selection process when choices made in this regard influence the pavement structure and the construction costs (ERA-PDM, 2002).

The strength of the road subgrade for flexible pavements is commonly assessed in terms of the California Bearing Ratio (CBR) and this is dependent on the type of soil, its density, and its moisture content. Direct assessment of the likely strength or CBR of the subgrade soil under the completed road pavement is often difficult to make. Its value, however, can inferred from an estimate of the density and equilibrium (or ultimate) moisture content of the subgrade together with knowledge of the relationship between strength, density and moisture content for the soil in question. This relationship must be determined in the laboratory. The density of the subgrade soil can controlled within limits by compaction at a suitable moisture content at the time of construction. The local climate and the depth of the water table on the road surface (ERA-PDM, 2002) govern the moisture content of subgrade soil.

According to ERA-PDM, 2002 volume 1 (Flexible pavements and gravel roads) chapter three explains details concerning subgrade materials. According to the manual, the type of soil, its density and moisture content assesses the strength of the Subgrade soil.

According to ERA, manual, 2013 subgrades are classified from S1 to S6 based on the California bearing ratio (CBR), and are illustrated in table below.

Table 2.1 CBR range subgrade class (ERA, 2013 Pavement design manual volume 1)

No.	Class	CBR Range (%)
1	S_1	<3
2	S_2	3,4
3	S_3	5,6,7
4	S_4	8 – 14
5	S_5	15 – 30

According to the soil and materials investigation report, sections of the route with CBR>3.5% and swells about 2% can be used for Embankment construction which needs to be covered with blanketing material. From Bowls, (1992) CBR values and the quality of subgrades in pavement design are explained below.

Table 2.2 CBR value range Subgrade quality (Bowls, 1992).

Serial Number	CBR (%) Range	Subgrade Quality
1	0-3	Very poor subgrade
2	3-7	Poor to fair subgrade
3	7-20	Fair subgrade
4	20-50	Good subgrade
5	50+	Excellent subgrade

2.2.1.1 Dealing with poor subgrade soils

The cost of a road is integrally linked with subgrade conditions. The poorer and more problematic the conditions, the greater the thickness required to support the design load. Sometimes certain special problems may arise in the subgrade below the material depth, which requires individual treatment. Some of the common problems, which need to considered, include:

- The excessive volume changes that occur in some soils as a result of moisture change (i.e. expansive soils and soils with a collapsible structure);
- The non-uniform support that results from wide variations in soil types over the road length;
- The presence of soluble salts which, under unfavorable conditions, may migrate upwards leading to several problems, including cracking of the surfacing;
- The excessive deflection rebound of highly resilient soils during and after the passage of aload (e.g.micaceous soils).

2.2.1.2 Improved subgrade layers

There are many advantages to improving the CBR strength of the in-situ subgrade to a minimum of 15% (Subgrade Class S5) by constructing one or more improved layers where necessary. In principle, where a sufficient thickness of improved subgrade is placed, the overall subgrade bearing strength is increased to that of a higher class and the sub-base thickness may be reduced accordingly. This is often an economic advantage as sub-base quality materials are generally more expensive than fill materials.

The use of improved subgrade layers also provides a number of other advantages, including:

- Provision of uniform subgrade strength;
- Protection of underlying earthworks;
- Improved compaction of layers above subgrade level;
- Provision of a more balanced pavement structure;
- Provision of a running surface for the traffic during construction;
- Provision of a gravel wearing course in the case of stage construction for future upgrading to a paved road;
- More economical use of pavement materials (thinner layers).

An improved subgrade placed on soils of any particular class must obviously be made of a material of a higher class (up to Class S5, since Class S6 is of sub-base quality).

The decision whether or not to consider the use of an improved subgrade layer(s) will generally depend on the respective costs of sub-base and improved subgrade materials.

2.2.2 Sub base materials properties

The engineering property of sub base materials used for the wearing course of gravel road are determined by their components or ingredients of the material, generally the sub-base materials consists of granular material ,gravel, crushed stone, reclaimed(blended) material or a combination of these materials. The material used for gravel road is the natural selected material, which fulfills the specification, listed under Pavement design manual of ERA volume I (2002) and) ERA LVR manuals (2011 in our country since these materials are used as pavement and pavement is the portion of the highway, which is most obvious to the motorist.

2.2.3 Gravel wearing course materials

The performance of the gravel surface depends on material quality, the location of the road and the traffic volume using the road. Gravel roads passing through populated areas in particular require materials that do not generate excess dust in dry weather. Steep gradients places particular demands for gravel wearing course materials that do not became slippery in wet weather or erode easily.

The specified strength of the design period shall be depends on the type of pavement (Naidoo, K., 2001). Consideration should give to the type of gravel wearing course material to be used in particular locations such as towns or steep sections.

Gravel loss rates of about 25-30mm thickness a year per 100 vehicles per day is expected, depending on rainfall and materials properties particularly plasticity (ERA 2002).

The materials of gravel wearing course should satisfy the following requirements that are often somewhat conflicting:

a) The materials should have sufficient cohesion to prevent raveling and corrugating (especially in dry conditions)
b) The amount of fines (particularly plastic fines) should be limited to avoid a slippery surface under wet conditions.

2.3 Performance-related specifications

Performance related specifications for wearing course materials have been developed for southern Africa based on extensive sampling, testing and monitoring of a large number of test sections (Paige-Green, 1989).

These specifications have successfully implemented in a number of African countries and are considered generally applicable to the Ethiopian environment.

The specifications identify the most suitable materials in terms of two basic soil parameters–Shrinkage Product and Grading Coefficient – which are determined from particle size distribution and linear shrinkage tests as shown in Figure 2.1.

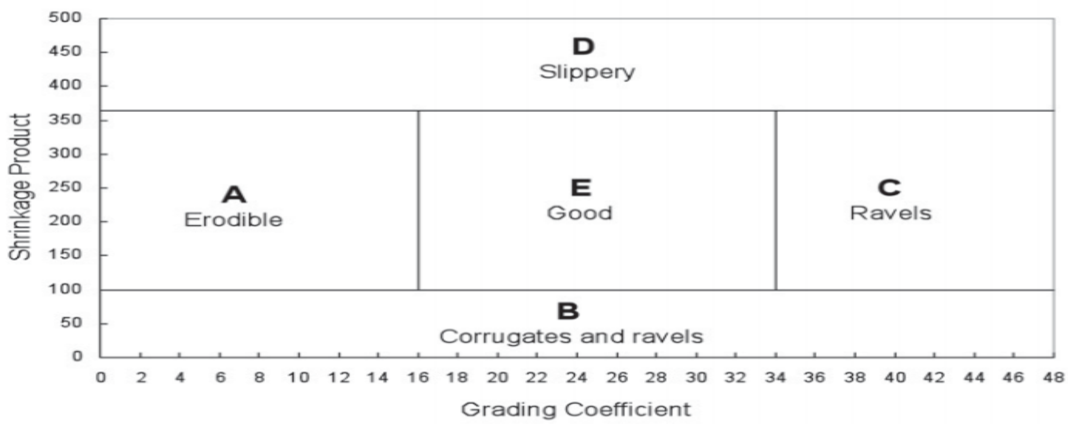

Figure 2.1 Material quality zones (Source: ERA LVR section D)

The material quality zones define material quality in relation to their anticipated in-service performance. The combination of grading coefficient and shrinkage product of each material determines which material quality zone it falls. The characteristics of materials in each zone are as follows:

A. Materials in this area generally perform satisfactorily but are finely graded and particularly prone to erosion. They should avoided if possible, especially on steep grades and sections with steep cross-falls and super-elevations. Roads constructed from these materials require frequent periodic labour intensive maintenance over short lengths and have high gravel losses due to erosion.
B. These materials generally lack cohesion and are highly susceptible to the formation of loose material (raveling) and corrugations. Regular maintenance is necessary if these materials are used and the road roughness is to be restricted to reasonable levels.
C. Materials in this zone generally comprise fine, gap-graded gravels lacking adequate cohesion, resulting in raveling and the production of loose material.
D. Materials with a shrinkage product in excess of 365 tend to be slippery when wet.
E. Materials in this zone perform well in general, provided the oversize material is restricted to the recommended limits.

2.3.1 Typical Distresses in Gravel-Surfaced Roads

There are seven primary distress types in gravel-surfaced roads (Eaton and Beaucham, 1992). These seven distress types are as follows:

a) **Corrugations:** Corrugations (also known as wash boarding) are closely spaced ridges and valleys at regular intervals. The ridges are perpendicular to the traffic direction. This type of distress is usually caused by traffic and loose aggregate, especially in prolonged dry periods. These ridges usually form on hills, on curves, in areas of acceleration or deceleration, or in areas where the road is soft or potholed.

b) **Dust:** The wear and tear of traffic on gravel roads will eventually loosen the larger particles from the soil binder. As traffic passes, dust clouds create a danger to trailing or passing vehicles and cause significant environmental problems.

c) **Improper cross section**: An unsurfaced road should have a crown with enough slope from the centerline to the shoulder to drain all water from the road's surface. No crown is use on curves, because they usually banked. The cross section is improper when the road surface not shaped or maintained to carry water to the ditches.

d) **Improper roadside drainage:** Poor drainage causes water to pond. Drainage becomes a problem when ditches and culverts are not in good enough condition to direct and carry runoff water because of improper shape or maintenance.

e) **Loose aggregate:** The wear and tear of traffic on gravel roads will eventually loosen the larger aggregate particles from the soil binder. This leads to loose aggregate on the road surface or shoulder. Traffic moves loose aggregate particles away from the normal wheel path and forms berms in the center of the roadway or along the shoulder.

f) **Potholes:** Potholes are bowl-shaped depressions in the roadway surface. They are usually less than 1 m (3ft.) in diameter. Potholes are produced when traffic wears away small pieces of the road surface. They grow faster when water collects inside the hole. The road then continues to break down because of loosening surface material or weak spots in the underlying soils.

g) **Ruts:** A rut is a surface depression in the wheel path that is parallel to the roadway centerline. Ruts are caused by permanent deformation in any of the road layers or subgrade. They can result from repeated vehicle passes, especially when the road is soft. Significant rutting can destroy a road.

The deteriorations of gravel road is governed by the behavior of the road materials, the drainage capacity under the combined action of traffic and climate, and the absence of sufficient maintenance activities (Dobson E., and Postill L., 1983).

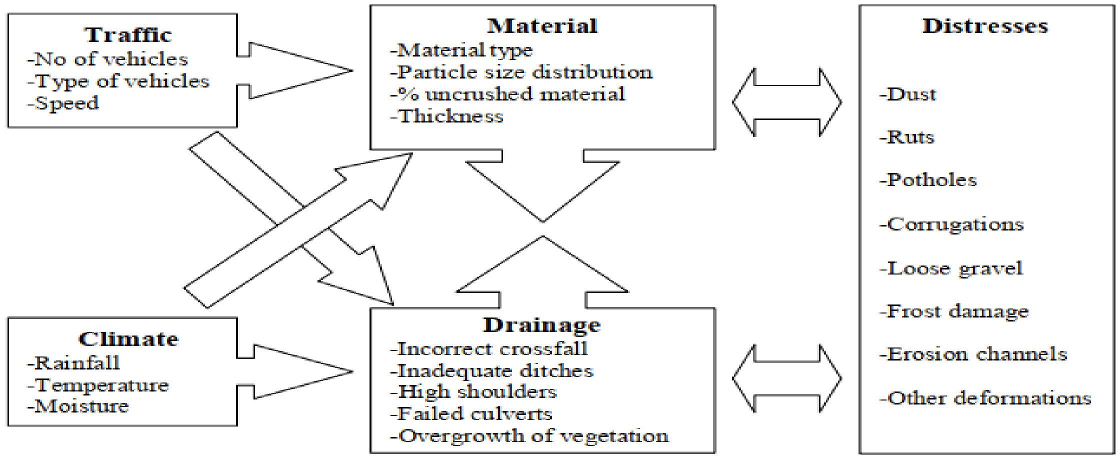

Figure 2.2 Schematic of the deterioration process on gravel roads (**source:** Transportation research records, 898).

2.3.2. Gravel-Surfaced Roadway Rating

According to rural road condition survey guide 1995 the road condition rates are Consider the following guidelines when rating the condition of gravel roads. They address the most common forms of distresses in gravel-surfaced roads.

a) **Rating = 100 to 81:** Roadway surface is in excellent condition with very good ride ability. The roadway has a good gravel thickness and excellent drainage. The only distress that is typically present is dusting in dry conditions.

b) **Rating = 80 to 61:** The roadway has adequate gravel thickness, a good pavement crown, and good drainage characteristics. Distresses that may be present include medium-severity loose aggregate and low-severity wash boarding. Some slight rutting (<25 mm) may exist in some areas during wet weather.

c) **Rating = 60 to 41:** The pavement has a good crown (75 to 150 mm). Primary ditches are present on more than 50 percent of the roadway. Secondary ditches are evident along the shoulder line, and some culvert cleaning is necessary. The gravel layer is adequate, but additional aggregate needed in isolated areas. Moderate wash boarding (25 to 50 mm deep) exists over 10 to 25 percent of the area, and moderate rutting (25 to 50 mm) occurs in wet weather. Occasional small potholes (< 50 mm deep) and some loose aggregate are present.

d) **Rating = 40 to 21:** Travel at slow speed is required. There is little or no roadway crown, moderate to severe wash boarding, severe loose aggregate and moderate potholing. Up to 25 percent of the roadway has little or no aggregate. More than 50 percent of the ditches are inadequate, secondary ditches exist along most of the roadway, and the culverts are partially filled with debris.

e) **Rating = 20 to 0:** Travel on the roadway is very difficult. There is either no roadway crown or the roadway is bowl-shaped with extensive ponding. Severe ruts and potholes exist over more than 25 percent of the roadway, and many areas (more than 25 percent) have little or no aggregate. There are few if any primary ditches, and secondary ditches are evident along most of the roadway. Culverts are either damaged or filled with debris.

2.3.3 Gravel loss

Gravel loss defined as a time-dependent reduction of the thickness of a gravel layer by the mechanical removal of gravel material from the road prism to the immediate surroundings of the road. It is a change in average gravel thickness over a period of time (Paterson 1991; Visser and Queiroz, 1979).

The rate of gravel loss depends on the intensity duration of rainfall, wind forces and traffic characteristics; also on gradient alignment, natural weathering (mechanical and chemical) of gravel materials, surface cross-fall, road width, material quality and characteristics, compaction achieved on respective layers of the road structure and maintenance practices (TRL, and Intech Associates 2002; Dierks, 1992 Jones, 1984).

A. Deterioration models for gravel loss prediction

Six international deterioration models for gravel loss prediction were investigate. These models were commonly use as part of Unpaved Road Management Systems (URMS). The gravel loss prediction model most commonly used in South Africa are those in the draft Technical Recommendations for Highways Manual 20 (TRH20) document and the World Bank Highway Development and Management (HDM-4) model. The different models that were used as part of this study these are:

- HDM-4 gravel loss deterioration model
- TRH20 gravel loss deterioration model
- Australian Road Research Board (ARRB) gravel loss deterioration model
- The Brazilian gravel loss deterioration model and
- The Kenya maintenance study

Based on research work carried out in Ethiopia (TRL, 2008), standardized gravel losses (gravel loss in mm/ year/100vpd) were determined in relation to the quality of the gravel wearing course.

Table 2.3 Typical standardized gravel loss (source: Design-Manual LVR Part-B).

Material Quality Zone[1]	Material Quality	Typical gravel loss (mm/yr/100vpd)
Zone A	Satisfactory	20
Zone B	Poor	45
Zone C	Poor	45
Zone D	Marginal	30
Zone E	Good	10

The rates of gravel loss increase significantly on gradients greater than about 6% and in areas of high and intense rainfall. On some gradients, the increase could be greater than 50% depending on the steepness of the gradient and material quality. Spot improvements should considered on these sections.

Re-gravelling should take place before the sub-base is exposed. The re-gravelling frequency, R, is typically in the range 5 - 8 years. This decreases considerably if poor quality gravels have to use. For example, if the gravel quality is in zones B or C, the loss rate will be 45mm per year per 100vpd. Therefore, a class DC4 gravel road carrying 200vpd will lose 90mm per year and require re-gravelling every two years (Design-Manual-for-Low-Volume-Roads-Part-B).

The wearing course thickness = R x GL……………………………………………………………Eq.2.1

R = re-gravelling frequency in years

GL = annual gravel loss.

According to unsealed roads manual (URM), an estimate of the annual gravel loss has given by the following equation:

GL =f $\left(\frac{T^2}{T^2+50}\right) * (4.2 + 0.92T + 3.50R^2 + 1.88V$……………………………………………….Eq.2.2

Where

GL = the annual gravel loss (mm)

T = the total traffic volume in the first year in both directions (thousands of vehicles)

R = the average annual rainfall (m)

V = gradient (%) for uniform road length

f = constant for gravel materials (0.94 to 1.29 for lateritic gravels, 1.1 to 1.51 for quartizitic gravels, 0.7 to 0.96 for volcanic gravels, 1.5 for coral gravels and 1.38 for sandstone gravels).

According to Technical Recommendations for Highways Manual 20 (TRH20), the annual gravel loss (AGL expressed in mm) can predicted by the following:

A_{GL} = 3.65(ADT (0.059+0.0027N-0.0006P26.5)-0.367N-0.0014PF+0.0474P26.5)....................Eq. 2.3

Where

GL is the average gravel thickness loss (mm).

ADT is the average daily traffic in both directions.

N represents climate in terms of Weinert N-value.

Based on design manual for low volume roads the N-value in an arid, semi-arid or dry climate (the Bereha and Kolla regions of Ethiopia where the Value is greater than 4) and the N-value in a seasonally tropical or wet climate (e.g. the Weina Dega, Dega and Wurch regions in Ethiopia, where the Value is less than 4). The study area climate condition is Weina Dega then N-value is 4.

P26.5 is the percentage of gravel materials passing through a 26.5 mm sieve.

Plasticity factor (PF) is the product of plastic limit and the percentage passing through a 0.075 mm sieve.

ERA 2002 recommended that gravel loss rates of about 25-30 mm thickness a year per 100 vehicles per day is expects depending on rainfall and materials properties.

B. Determination of wearing course thickness

According to the (LVR part D), the design thicknesses required increases considerably if the gravel is weak hence stronger gravels should generally be used if they are available at reasonable cost. On relatively weak subgrades (S2 and S3), the use of strong gravels (G45) should be avoided because of the poor “balance” of such pavements. Instead, the use of an improved subgrade layer should considered for the advantages provided. Where the available gravel is not homogeneous, it will be necessary to substitute a particular class of gravel with one or more different classes of gravel of appropriate thickness

(Emery 1985).

The wearing course of a new gravel road shall have a thickness D calculated from

$D = D_1 + N.\ GL$……………………………………………………………………………Eq.2.4

Where D_1 is the minimum thickness from Figure 2.2

N is the period between re-gravelling operations in years

GL is the annual gravel loss

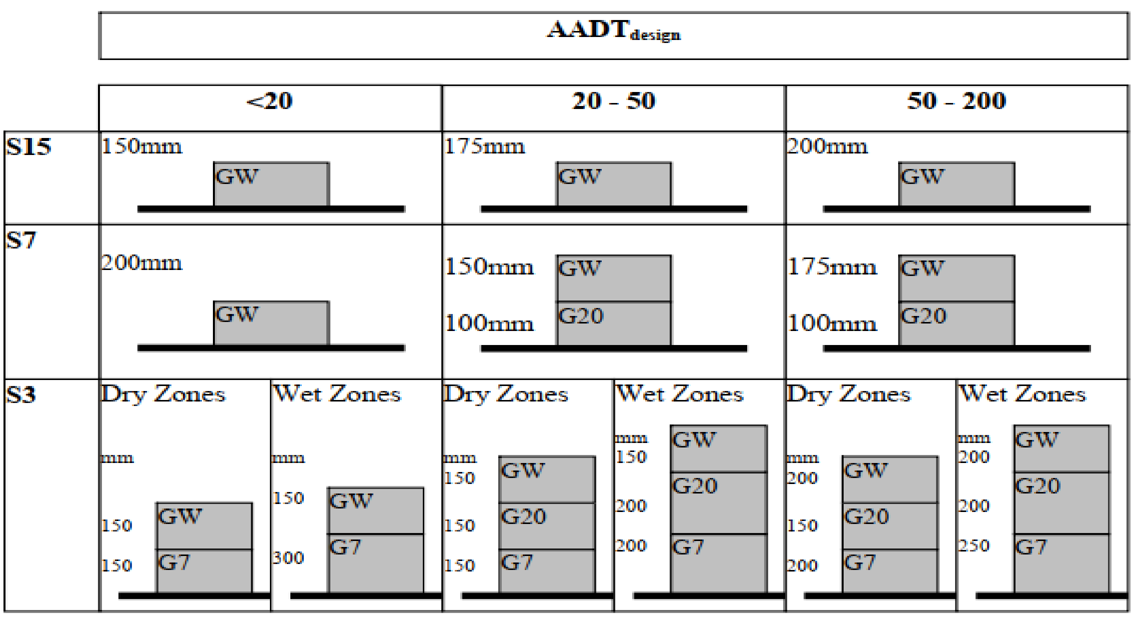

Figure 2.3 Pavement and Improved Subgrade for Gravel Roads for AADTs <200 (Source: ERA Geometric Design Manual -2002).

2.4 Material requirements for gravel roads in rural areas

A wide range of materials including lateritic, calcareous and quartzite gravels, river gravels and other transported and residual gravels, or granular materials resulting from weathering of rocks can used successfully as road base materials. The behavior of lateritic materials in pavement structures depends mainly on their particle size characteristics, the nature and strength of the gravel-sized particles, the degree of compaction as well as traffic and environmental conditions (ERA LVR, 2016).

The most important requirements for a laterite to show good field performance are that the material is well graded with a high content of hard, or quartz particles with adequate fines content. The specifications of gravel materials for rural areas are recommended in the following table.

Table 2.4 Recommended material specifications for unsealed rural roads (Source: ERA LVR)

Maximum size (mm)	37.5
Oversize index (I_o)[a]	≤ 5 %
Shrinkage product (S_p)[b (2)]	100 - 365 (max. of 240 preferable)
Grading coefficient (G_c)[c (2)]	16 - 34
Soaked CBR (at 95 per cent Mod AASHTO)	≥ 15 %
Treton impact value (%) [(4)]	20 – 65

a I_o = Oversize index (percent retained on 37.5 mm sieve)
b S_p = Linear shrinkage x percent passing 0.425 mm sieve
c G_c = (Percentage passing 26.5 mm - percentage passing 2.0 mm) x percentage passing 4.75 mm)/100

2. 5 Gravel road structures

Good performance of gravel road depends not only on traffic volume requirements but also on other factors such as the proper design of road cross section, quality of gravel materials, appropriate and proper use of equipment, proper application of material and skilled personnel (Skorseth, Selim et al., 2000). The road cross section of gravel roads includes the crown (camber), shoulder and the drainage structures (Skorseth, Selim et al., 2000).

The following is the brief explanation of these elements as shown on Figure 2.1

- **Crown or Normal cross fall:** For rapid flow of water from the road surface the road carriage surface, cross fall or crown is required to properly designed and maintained throughout the service life of the road. To achieve this, function the design standards for low volume roads specified the cross fall of 4-6% to be adequate for gravel road cross section. If the gravel road has no crown many problems will occur such as potholes and rutting. During the rainy season, water will not flow out of the road; the running water on the road surface will soften the gravel-wearing course and cause rutting.

- **Road Shoulder:** On gravel road cross section, shoulder is the part which connects the road carriage way and the side ditch. Road shoulder is the supporting edge of the carriageway of the road. Other functions of the road shoulder are to provide safety space for drivers to gain control when forced out of the road and to drain water from the carriageway to the side ditch (Skorseth, Selim et al. 2000).

  Another problem of high shoulder is that the running water erodes the surface gravel sometimes even the subgrade can be eroded which may lead to serious safety hazard (Skorseth, Selim et al., 2000).

  Road Ditch: This is the main and common drainage structure for roads. Road ditches should maintained to a good standard, gentle slope, free of debris and good shape and size to facilitate efficient flow of rainwater and avoiding drainage problems on road surface. This can easily achieved using proper equipment and skilled operators during the dry season or periods of low rain (Skorseth, Selim et al., 2000).

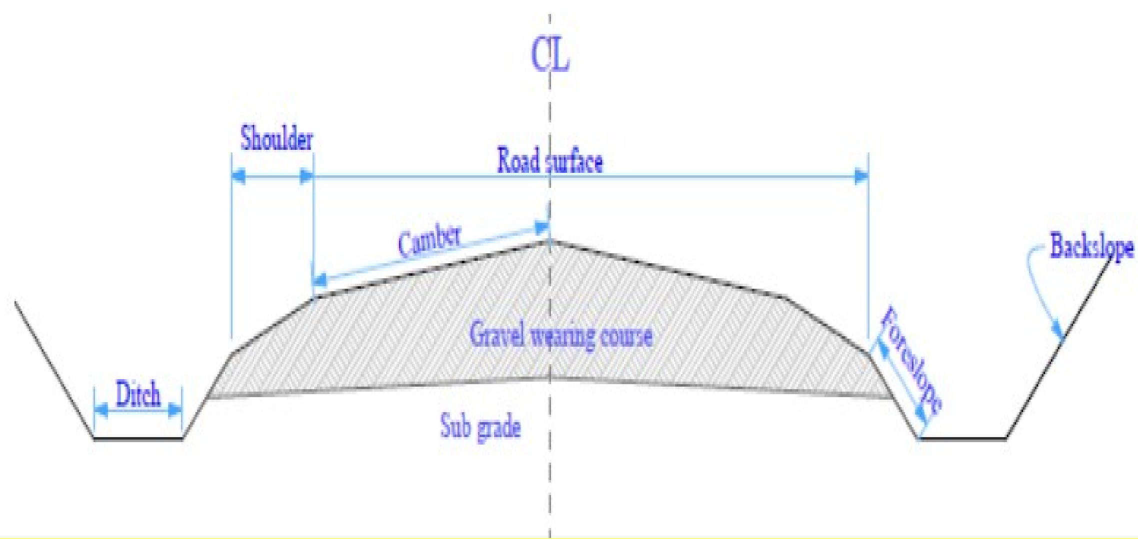

Figure 2.4 Elements of gravel roadway cross section (**source:** ERA LVR section D)

CHAPTER THREE

METHODOLOGY AND MATERIALS

3.1 Research methods

The methodologies adopted to achieve the objectives were outline as follows:

1) Review applicable practices, research findings and other relevant information in material quality and drainage system of gravel road.
2) Relevant literatures on current gravel road performance and main factors have reviewed.
3) The laboratory analysis was find out information about the quality of materials and also compare design standard specifications and construction requirements.
4) The road user assessment was evaluate the present serviceability rate of existing road.
5) The Hydrological investigations describe the existing condition of the road and storm water drainage facilities.

3.2 Descriptions of Study area

The road project is found in North Wollo zones, Habru and Gubalafto woreda of the Amhara Regional State. It connects woldia town, Gubalafto woreda and Habru woreda. The road starts at Mechare and passesing through Wetek Teklehaymanot and geographically located at 1308194.2m north & 565356.4m east, an average elevation of 2112 meters above mean sea level. The climate along the project route is categorized as Woina Degga. According to the Central Statistical Agency of Ethiopia (CSA), this Zone has a total population of 1,500,303 and a road density of 69.7kilometers. The research conducted on the already existed road that are completed and substantially constructed during the different road sector development stages having remarkable road defects at different locations in study area.

Figure 3.1 Location map of study area (**Source:** Google Earth2019)

3.3 Research design

The research study was follow both experimental and analytical method using qualitative and quantitative data. The data gathered from the study area were categorized and interpreted.

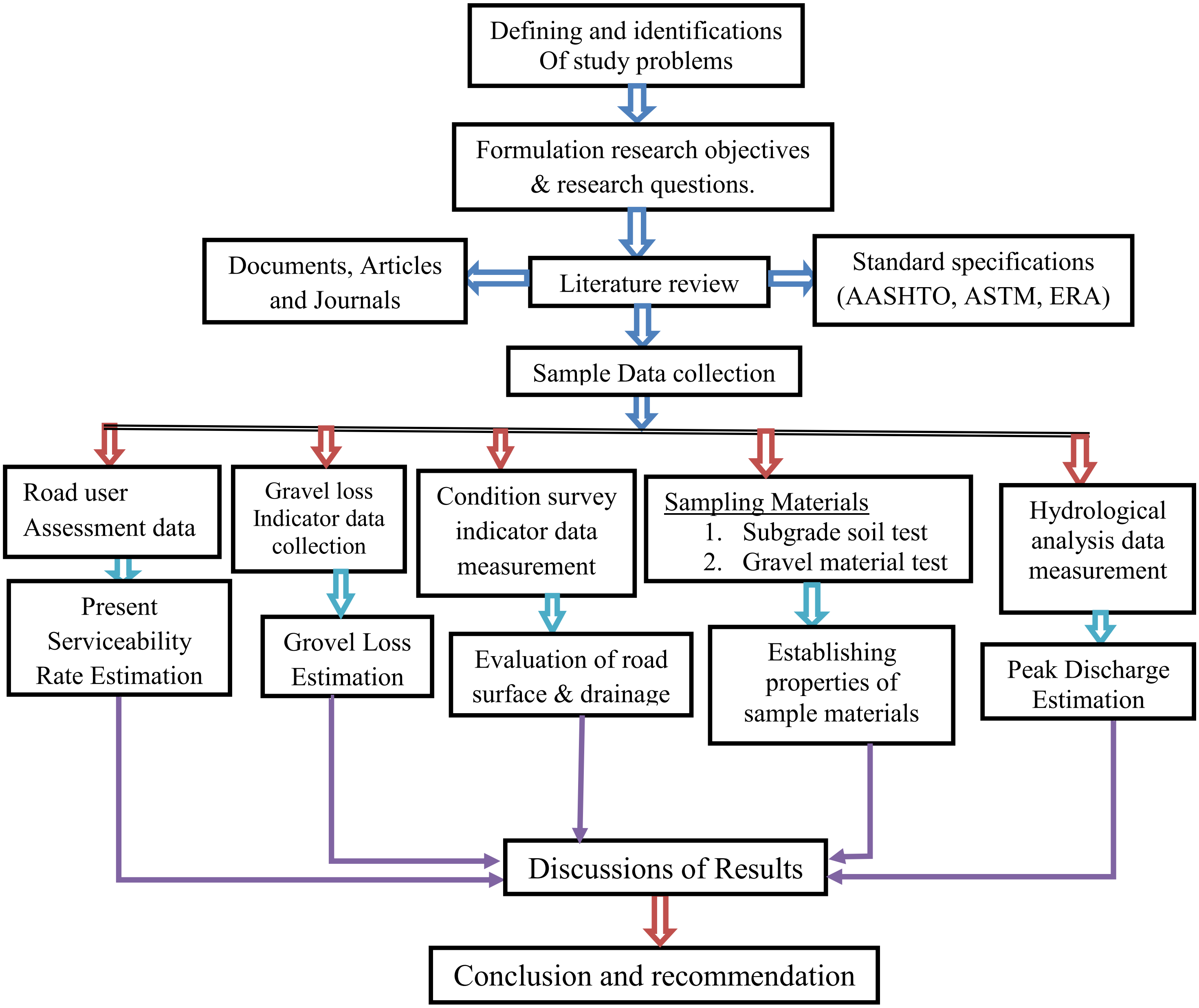

Figure 3.2 Study Design (Research Frame)

3.4 Study Population

The Study populations were contain subgrade soil and gravel material, road drainage system and road users. The total number of populations that considered in the study is only the population existing within the range of the study area, which covers 52km lengths of existing gravel road section.

3.5 Sample size and sampling procedures

Non -Probability-sampling techniques of purposive sampling were selected, because it is known to be representative of the total population, or it is known that it was produce well matched groups. This method was appropriate when the study places special emphasis upon the control of certain specific variables.

3.5.1 Sample size

In this study, the road length cover 52km, samples were taken at five stations for each subgrade soil and gravel material based on the availability of open space beside damaged road edges to digging samples. Gravel loss data were estimated using the soil sample laboratory results (material properties) and traffic volumes. The traffic volume were collected both direction traffic count for seven day traffic. Condition survey conducted the existing gravel road based on roadway rating. The excavation was made manually using the shovel by selected damaged areas in the existing road. The road drainage system coordinates were taken at nine drainage structure inlet points using handheld GPS.

3.5.2 Sampling Techniques

The sampling technique used for this research was a purposive sampling, which is non-probability method, because the experimental investigations in this research were executed particularly on the subgrade soil and gravel materials. The condition survey was conduct based on measuring road problems occur on study area. The catchment area of each watershed on the whole route corridor was delineate from DEM data. The sizes of each catchment area were determined using Arc GIS software. Since these, study pick out the sample in relation to some principle, which are considered important for the particular study. These sampling techniques were proposed based on the target to perform study area observation and laboratory test on the selected part of study to investigate the performance of gravel roads.

3.6 Study Variables

The study variables evaluated in this research were both dependent and independent variables. Which display the factors affecting the performance of gravel roads.

3.6.1 Dependent variables

The dependent variable in these studies is performance of Gravel road.

3.6.2 Independent variables

The independent variables that are to be measured and manipulated to determine its relationship to observed phenomena are selected and listed below.

- ✓ Grain Size Analysis

- ✓ Atterberg limits
- ✓ Free Swell Index
- ✓ Specific gravity
- ✓ Compaction
- ✓ Peak discharge
- ✓ Gravel loss
- ✓ Present serviceability rate

3.7 Data collection process

In this study, both primary and secondary data was used:

Primary data source: Primary data for this study were a laboratory experiment output, drainage structure locations (coordinates) and traffic volume count.

Site visit / observations: site visit was carried out to determine current conditions of existing road in comparison with the acceptable standards. The research employed use a physical observation checklist, which was fill through observations and a digital camera was use to take photographs of the status of the road surface and road drainage system.

Field survey measurement was done using surveying equipment's such as Tape, engineering level, and handheld GPS.

Questionnaire is a research instrument consisting of a series of questions and other prompts for gathering information from respondents. Was asking the drivers and people living around the study area. The study area information that was gathered from the road user.

Questionnaire type one

Type one questionnaire would be structured to be filled by government bodies in charge of construction and maintenance of gravel roads, in particularly Amhara Rural road Authority and their consultants.

The main objective of this questionnaire is to know the responsibilities, and challenges experienced by the bodies mandated to construct and maintain the road. In addition, it sought to understand the role of the consultant in the gravel road provision.

Questionnaire type two

This type of questionnaire was structured to be filled by road users and the people who live adjacent to the Mechare to Arssema road. The road users referred here includes people who travel through that road frequently, both the public service transport providers and those using private Vehicles and pedestrians. It was intended to know what factors affecting gravel road performance and how activities have changed because of road segment problems. This were help in understanding the present serviceability rate of existing road, what major factors has affected the road and to obtain their views on the way forward.

Secondary data source: the data from different written documents, areal map, published and unpublished data, internets etc.

Photography: - Photograph is an indirect way of data collection. It was majorly used to capture the current status of the drainage system and gravel surface in Mechare to Arssema road. It was meant to give a visual understanding of the research topic to the readers of this research project, the rate of deterioration, maintenance and the state of the gravel road surface.

3.8 Data Processing and Analysis

Both descriptive and inferential statistics were used in the data analysis to obtain the significant results and establish relative importance of controlling factors affecting the performance of gravel roads investigations of subgrade soil, gravel material and the hydrological study analyzed using descriptive and exploratory, such as rational method and GIS software were used. Qualitative and quantitative methods used and Microsoft word and Excel of analysis used for data that are collected through laboratory output and questionnaires.

Laboratory techniques used determine the soil classification and strength evaluation of subgrade soil and gravel material properties of soils those are:-

- ✓ Gradation analysis
- ✓ Atterberg limit test
- ✓ Proctor compaction
- ✓ CBR test

The result of laboratory tests were going to analyzed using excels to draw different kind of graphs and charts. Comparison of test results with standard specification on ERA, ASTEM and AASHTO design manual was important aspect of analysis. Condition survey was evaluate measuring roadway problems and site observations. The present serviceability rate was determine road user assessment evaluations and gravel loss also determined using TRH20 deterioration model then compare with the design thickness of the surface course materials. In road drainage, system the data’s was analysis using Arc GIS software for EDM delineation and rational methods for estimating catchment discharge.

3.9 Ethical Considerations

This study was conducted in a manner that is consistent with ethical issues that need to be considered in conducting a thesis. Accordingly, letter from the Jimma University Institute of technology department of civil engineering is written for the concerned bodies. Hence, most individuals, the researcher visited for interview, accepted and cooperated with the researcher thesis. Moreover, prior consent of the participants is requested before conducting the questionnaires.

3.10 Data quality assurance

Before data collection all the source populations availability has checked and respondents daily work schedule has respected. All the questions that are put in simple and clear ways, willingness of the respondents to answer the questions and collaborates with the study is test out, all necessary schedule are worked out needed to administrate the questionnaires to conduct observations group to measurements.

The assurances of those data are highly recognized and those data are true. However, in order to obtain quality of data is going to be assured by giving attention to the following points.

- Pre –test of the available instrument done before the main data collection period begin and the data's were collected after gaining awareness on how to collect relevant data by principal investigators.
- Samples were collected from appropriate locations and at appropriate depth.
- Standard formats used for recording test results to prevent loss of data.
- Checklists used for condition survey data collections to prevent loss of results.

3.11 Materials

3.11.1. Subgrade soil

Subgrade of a pavement should be strong enough to give adequate support to the pavement and for supporting and distributing the wheel loads. The design and behavior of a flexible pavement depends mainly on the stability of the subgrade soil, which can increased by compacting the soil at optimum moisture content thus achieving maximum dry density.

The subgrade soil sample used for this research work was collect from existing route segment on different stations. The soil is Dark gray and black in color. The samples were disturbed collected at a depth of about 1.5m to 2.0m. Full properties the soil is address in the methodology section.

3.11.2. Gravel materials

The gravel materials used for this research work were collect from existing route segment on different stations. The visual soil description is Dark gray silt gravel, Darkish Quartzite Silty gravel and Dark Grayish Volcanic Silt gravel in color. The samples were disturbed collected at a depth of about 1.5m to 2.0m. Full properties the soil is address in the methodology section.

Figure 3.3 Photos of test pit taking for subgrade soil and gravel material at different stations

3.12 Laboratory Testing and Analysis

Disturbed soil sample collected from different stations along the read segments was used in the experimental work. Series of test like sieve analysis; specific gravity test, Atterberg's limit test and free swell index test were conducted in the laboratory to determine the index properties of the soil. These are indicative tests that are usually used for identifying wither the soil is expansive or not. Soil was classified as per AASHTO and unified Soil Classification System (USCS) based on the index properties of the soil.

The conducted tests however included hydrometer analysis, Atterberg limits, sieve analysis, specific gravity, moisture density relation, free swell, CBR and percent swell of CBR to fully characterize and attain the objectives of the research.

3.12.1 Subgrade soil requirements

3.12.1.1 Sample preparation and test requirements

The soil samples were first prepare based on their test pit stations and the sample were air dried, then respective of each test procedures preparing uniform samples for sieve analysis, Atterberg Limit and Free Swell Index tests, compaction and Californian bearing ratio tests.

A. Moisture content

The moisture content of the soil which is defined as the ratio between mass of water to mass of soil solid was determine immediately after the sample was taken from the site.

The samples were kept in plastic bag to prevent moisture loss during transportation from site to laboratory. The method employed for determining the moisture content was oven-drying method. The measured amount of wet soil was put in an oven of thermostatically controlled oven at 110 ± 5°C, kept for 24hours, and examined for weight loss. The result of moisture content determination is attached in appendix A.

B. Specific Gravity

Specific gravity, which is the measure of heaviness of the soil particles were determined by the method of small pycnometer method using a soil sample passing 2mm sieve and oven dried at 110±5degrees centigrade. Specific gravity is the ratio of the mass of the unit volume of soil at a stated temperature to the mass of the same volume of gas-free distilled water at a stated temperature. The specific gravity of the samples was determined using ASTM D 854-83, and the result of the test is as tabulated at Table 4.4.

C. Atterberg Limit test

This lab is perform to determine the plastic and liquid limits of a fine-grained soil. The liquid limit (LL) is arbitrarily defined as the water content, in percent, at which a part of soil in a standard cup and cut by a groove of standard dimensions will flow together at the base of the groove for a distance of 13 mm (1/2in.) when subjected to 25 shocks from the cup being dropped 10mm in a standard liquid limit apparatus operated at a rate of two per second. The liquid limit of a soil highly depends upon the clay mineral present. The conventional liquid limit test is carried out in accordance of test procedures of AASHTO T89. A soil containing high water content is in the liquid state and it offers no shearing resistance. The plastic limit (PL) is the water content, in percent, at which a soil can no longer be deformed by rolling into 3.2mm (1/8in.) diameter threads without crumbling. The conventional plastic limit test is carried out as per the procedure of AASHTO T90. The soil in the plastic state can be remolded into different shapes. When the water content is reduced the plasticity of the soil decreases changing into semisolid state and it cracks when remolded. The results of Atterberg limit test computed for the collected samples were written in Table 4.2.

D. Free Swell Index

The free swell index is also one of the most commonly used simple tests to estimate the swelling potential of expansive clay soil. The procedure involves in taking two oven dried soil samples passing through the 425µm sieve, 10g each was placed separately in two 100ml graduated soil sample. Distilled water was filled with one cylinder and kerosene in the other cylinder up to 100ml mark. The final volume of soil is computed after 24 hours to calculate the free swell index.

$$FSI = (V_w - V_k)/ V_k * 100 \qquad (3.1)$$

Where FSI = Free Swell Index

V_w = Final volume in water

V_k = Final volume in kerosene

The free swell Index of the study area soil was presented on appendix A.

E. Compaction

In this study, Standard Proctor compaction tests were conduct on the soil to determine the relationship between the moisture content and dry density for specific compaction effort according to AASHTO T99-94. The soil was compacted with different moisture content in three layers each suffering 25 blows. After obtaining the density and moisture of each compacted soil sample, the relationships for dry density and moisture content are, obtain in the figure 4.6.

F. California Bearing Ratio (CBR)

AASHTO T193-93 conducts the CBR and CBR-swell tests for the natural subgrade soils. The CBR is express by force exerted by the plunger and the depth of its penetration into the specimen; it is aim at determining the relationship between load and penetration. The samples are compacted in five layers with 56 blows from the Automatic compactor. The compacted soil samples of the CBR mold are soaked for 96 hours in a water bath to get the soaked CBR value of the soil. The test consisted of causing a cylindrical plunger of 50mm diameter to penetrate a pavement component material at speed of 1.25mm/minute. The loads for 2.5mm and 5.0mm were recorded. The greatest value calculated for penetrations at 2.5mm and 5.0mm was having been recorded as the CBR value. However, if the greater recorded value was obtained first for penetration at 5.0mm the laboratory test, was repeated again and result were taken as it is for the next penetration result. The equation to be computing the CBR value is as follows.

CBR (%) =100*(x/y………………………………………………………………………………. (3.2)

Where: 'X' = material resistance or corrected unit load value on the piston (pressure) for 2.5 or 5.0mm of penetration, y = standard unit load for well-graded crushed stone. For 2.5mm Penetration = 13.2KN and for 5.0mm penetration =20KN used standard loads.

The determined laboratory results are tabulated at table 4.6

G. CBR swell of the soil

The CBR swell of the soil is measured by placing the tripod with the dial indicator on the top of soaked CBR mold. The compacted soil samples of the CBR mold are soaked for 96hours in a water bath to get the CBR swell of the soil. The initial dial reading of the soil of the dial indicator on the soaked CBR of mold is taken just after soaking the sample. At the end of 96hours the final dial reading of the dial indicator is taken hence, the swell percentage of the initial sample length is 116.43mm, see Table 4.6. Then CBR swell is given by:

Percent Swell = (Change in Length in mm during soaking /116.43) *100%…………………………. (3.3)

3.12.2 Gravel material test and requirements

The Quality of gravel materials is another factor, which should be carefully considered for good performance gravel roads. Gravel is a combination of three types of materials of different sizes, which are stone, sand, and fines. Good gravel requires a certain percentage of stone to carry loads of vehicles especially during wet season, a percentage of sand sized particles to fill the voids between stones and give stability, and a percent of plastic fines to combine the stones and sand to make firm gravel surface to shade water.

Proper gradation of gravel materials leads to good quality gravel. As ascertained factors, which contribute to a good gravel, are particle size distribution and cohesion, which can have a range from very fine particle sizes up to about 37.5mm. Too high percentage of large particles (stones) will result in poor riding quality and the make the maintenance work difficult. On other side, the fine particles should have good plasticity to provide bond with larger particles when dry but high plasticity is not recommended because will make the road slippery and impassable during wet season. In order to determine the quality of gravel, soil materials tests should be conducted in accordance with the locally available specification manuals.

A. Soil Particle Size Analysis

This is the measurement of size distribution of individual particles of given soil sample. The relative proportions of different grain sizes of soil sample can be determined by soil particle size analysis. However, soil behavior can be affected to some degree by factors such as particle size and shape. In general, it is expected that soil materials, which have same particle size distribution curve, will have similar engineering physical properties.

The presentation of particle size distribution data can be done in two formats, table and graph format. The table format is the format in which the total percentage of particles of a sample that passes a given sieve size are recorded while in the graphical format the relationship of the sieve or particle size versus the percentage passing the given sieve are plotted. In this study, the results for soil particle size distribution were used for soil visual classifications and as parameters to assess the performance relationship of gravel materials used for road construction and maintenance. This was based on the specification requirement given by Standard Specifications for Road Works (SSRW) and Pavement Materials and Design Manual (PMDM). Therefore, Table format for data presentation was used. In addition, Table 4.7 shows sieve analysis results of gravel materials passing sieves 50mm, 37.5mm, 25mm, 19mm, 2mm, 0.425mm and 0.075mm.

Grading Coefficient (GC): Is among of the parameters derived from particle size distribution results. GC is used as an indicator for the performance characteristic for gravel materials. As specified on the ERA LVR the recommended GC value for marginal gravel materials can range from **16- 34** inclusive.

GC= (Percentage passing 26.5mm - percentage passing 2.0mm) x percentage passing 4.75mm)/100………………………………………………………………………………………… (3.4)

Grading Modulus (GM): Is a simple method for the assessment of properties of the soil and gravel materials. According to (SABS, 1996) the value for grading modulus of gravel materials with low fine fraction will be greater than **1,** while for gravel materials with higher fine fraction will have grading modulus value less than **0.8.**

In addition, materials with Grading Modulus greater than 2 denotes coarsely graded gravel with relatively good quality and materials with GM less than 2 indicates fine size gravel with poor quality for road construction uses (SNRA, 2009).

GM = (300-%pass 2mm-%pass 0.425mm -%pass 0.075mm)/100……………………………… (3.5)

B. Atterberg Limits Test

The most used Atterberg limits for soil and road engineers and for this study are Liquid limit (LL), Plastic Limit (PL) and Shrinkage Limit (SL). From the plastic limit and liquid limit results, the value for plasticity index can therefore computed. The range of moisture content over which soil material is at plastic state is defined as the Plasticity Index (PI), or Numerically PI is the arithmetic difference between Liquid and Plastic Limits. PI results can be used to predict the strength of gravel material. Furthermore, gravel materials with PI less than 6 are not suitable to be used as wearing material for gravel roads because of insufficient plasticity for that purpose (Naidoo and Purchase, 2001).

LL and PI can be the best option in prediction of the quality of gravel material by excluding granular materials to a certain extent (O'Flaherty, 2002).

Shrinkage Product (SP): Is another parameter for Atterberg limits, which can be defined as the product of the linear shrinkage and the percentage of materials passing sieve, size 0.425mm. As ERA LVR recommended that SP for road surfacing gravel material ranges from 120- 365, (max. of 240 preferable) to reduce the dust problems in built up areas. Materials of high SP can be slippery during operation of the road and materials with low SP can lead to ravel and corrugation problems.

C. Strength Tests

Strength Tests conducted for strength were Compaction tests to determine the Maximum Dry Density (MDD), Optimum Moisture Content (OMC) and California Bearing Ratio (CBR). Compaction test are conducted to study the relationship between the grading characteristics, degree of compaction, the maximum dry density and the optimum moisture content of soil materials. Therefore, materials, which are poorly, graded gives low density and a well-graded material gives high density (Gidigasu, 1983).

3.12.3 Evaluation of gravel loss using the material test result and traffic volume

Gravel loss is the single most important reason why gravel roads are expensive in whole life cost terms and often unsustainable, especially when traffic levels increase. Reducing gravel loss by selecting better quality gravels or modifying the properties of poorer quality materials is one way of reducing long-term costs.

It refers to the amount of gravel wearing course that has been swept away which needs to be replaced in order to restore the original designed thickness. The thickness of a gravel-wearing course varies between 150-250mm depending on the strength characteristics of the roadbed. This implies that gravel loss should be measured in average millimeters reduction of gravel layer thickness.

Most gravel roads are constructed with wearing course of about 150 mm thick of compacted gravel materials. The rate of gravel loss depends on the intensity and duration of rainfall, wind forces and traffic characteristics; also on gradient alignment, natural weathering (mechanical and chemical) of gravel materials, surface cross-fall, road width, material quality and characteristics, compaction achieved on respective layers of the road structure and maintenance practices (TRL).

A. Traffic volume studies

Traffic volume studies are carried out to collect data on the number of vehicles that pass on a particular point on a highway facility during a specified time.

The other type of data collected in this study in order to assess the performance of gravel road was traffic volume count. For the case of gravel roads, the traffic data are required for the determination and specification of the road layer thickness and the decision making to set limitation of different road technology due to the volume level. In order to predict the performance of gravel material, the knowledge of the interaction between traffic and the gravel-wearing course is important (Mwaipungu, 2015).

AADTs are used in several traffic and transportation analyses for estimation of highway user revenues, establishment of traffic volume trends, evaluation of the economic feasibility of highway projects, development of freeway and major arterial street systems, and development of improvement and maintenance programs (ERA LVR manual). ADT is the average of 24-hour counts collected over a number of days greater than one but less than a year. It may be measured for six months, a season, a month, a week, or as little as two days. An ADT is a valid number only for the period over which it was measured. ADT may be used for planning of highways activities, measurement of current demand, and evaluation of existing traffic flow. The basic form of manual count involves a person recording each vehicle by making calculation marks on a field sheet. Appendix C shows a typical field sheet form, which was used to collect, classified traffic volume data.

3.12.4 Hydrological Study and Analysis

The hydrological study undertaken for the project road was aimed at the determination of design discharges for a given set of return periods that were consequently utilized for checking of existing and design of new drainage structures. In undertaking the hydrological study and analysis, the following operations are undertaken:

- Data completion
- Catchment area delineation
- Basin characteristics determination
- Selection of hydrologic procedure
- Return period adoption
- Peak flow or discharge estimation

A. Study of Watershed Characteristics

To obtain information and data on relief, geomorphology soil type, land cover and catchments parameter of the streams along the route - topographic maps, land use and land cover maps, soil and geomorphology maps, national atlas of Ethiopia as well as site visit inspection and assessment information were used. To study of the watershed characteristics extensive study has been done using satellite data. Data regarding catchments areas, i.e. watershed size and shape, stream slope, stream length and land slope were determined from satellite data DEM 30mx30m resolution.

Catchment area delineation: The catchment area of each watershed on the whole route corridor was delineated from DEM data. The sizes of each catchment area were determined using Arc GIS.

Hydrologic soil grouping: Soil properties influence the relationship between rainfall and runoff by affecting the rate of infiltration. The SCS has divided soils into four hydrologic soil groups based on infiltration rates (Groups A, B, C, and D).

The Hydrologic soil grouping for each catchment is identified from examination of available soil maps and physical assessment done on site. The type of soil on the study area is Lithosols (FAO, 1998) that covers almost 80% of the soil is Clay loam, silty clay loam, sandy clay, silty clay or clay of hydrologic soil group D.

Vegetation cover, land use and land cover: Land use and land cover for each catchment is identified from examination of available land use and land cover maps, land use shape files and physical assessment done on site. Based on the obtained data’s the road passes through predominantly cultivated land and some parts were covered with grass pasture and bushes.

Rainfall and temperature: The climate along the project route is categorized as Weyna Dega in which it is characterized by low temperature and high relative humidity. The mean annual rainfall of the area ranges between 400mm-799mm, The major rainy seasons of project area is during Kiremet (June - September) in which its amount ranges between 200-399mm and during Belg times(February –may) and between times(October-January) it ranges b/n 50-99mm.

According to ERA (2013), Drainage Manual the project road is located in Rainfall Regime C rainfall intensity has taken from the manual shown in appendix D figure 1.1.

B. Hydrological Design Criteria

The hydrological design criteria consider the following parameters based on the road drainage design characteristics.

1. Catchment Size

The following methods have adopted from ERA drainage design manual (ERA DDM**)** to calculate the peak discharge depending on the size of the catchment area:

- ✓ For catchment area < $0.5km^2$ →Rational Method
- ✓ For catchment area > $0.5km^2$ → The United States Soil Conservation Service method

From the above recommendation, this research was used rational method.

2. Return Periods (Design Frequency)

The frequency of the flood for the design of drainage structures depends on the risk likely to encounter during the anticipated service life of the road. Return period with which a given flood can expected to occur is the reciprocal of the probability or chance that the flood will be equaled or exceeded in a given year. The drainage facilities have been designed for recurrence interval as shown on the appendix D, table 1.1as per Drainage Design Manual of ERA recommendation.

3.12.4.1 Design methodology for rational method

The Rational Method is most accurate for estimating the design storm peak runoff for areas up to 50ha (0.5km2). The Method can be applied to small rural catchments if they do not exceed 0.5km2 as per ERA drainage design manual (DDM).

The consequences of applying the Rational Method to larger catchments is to produce an over estimate of discharge and a non-conservative design.

The rational formula is expressed as:

$Q = 0.278C\ I\ A$…………………………………………………………………………………Eq.3.1

Where:

Q = Maximum rate of runoff, m^3/s

C = Runoff coefficient representing a ratio of runoff to rainfall

I = Rainfall intensity for a duration equal to the time of concentration and for design return period, mm/hr.

A = catchment area tributary to the design location, km^2

Assumptions inherent in the rational formula are as follows:

- ✓ The peak flow occurs when the entire watershed is contributing to the flow
- ✓ The rainfall intensity is the same over the entire drainage area
- ✓ The rainfall intensity is uniform over a time duration equal to the time ofconcentration, Tc.
- ✓ The frequency of the computed peak flow is the same as that of the rainfall intensity, i.e., the 10-yr rainfall intensity is assumed to produce the 10-yr peak flow
- ✓ The coefficient of runoff is the same for all storms of all recurrence probabilities because of these inherent assumptions, the rational formula should only be applied to drainage areas smaller than 50 ha ($0.5km^2$).

In the rational method the following parameters used for computing peak discharge (Q).

1. **Runoff Coefficient**

The ground cover and a host of other hydrologic abstractions considerably affect the coefficient. The rational equation in general relates the estimated peak discharge to a theoretical maximum of 100% runoff. The Values of C vary from 0.05 for flat sandy areas to 0.95 for impervious urban surfaces, and considerable knowledge of the catchment has needed in order to estimate an acceptable value. The coefficient of runoff also varies for different storms on the same catchment, and thus, using an average value for C, gives only a rough estimate of discharge in small uniform urban areas. On top of this, the rational formula has used for many years as a basis for engineering design for small land drainage schemes and storm-water channels. In ERA drainage design manual (DDM) determination of C (for non-urban catchments), depending on terrain type and hydrologic soil grouping shown in appendix D.

2. **Rainfall Intensity**

Rainfall intensity, duration curve and frequency curves are necessary to use the rational method. ERA drainage design manual (DDM) divides the country into different rainfall region and for each provides Intensity -Duration - Frequency (IDF) curves, Project IDF curve has been used of ERA regional IDF curve and adopted for this particular project of region C shown in appendix D, figure 1.2.

3. **Time of Concentration**

The rainfall intensity used in the rational method is determined from the time ofconcentration (Tc). It was calculated using the following methods:

i. calculation of the time of concentration for overland flow

Overland flow is the type of flow that occurs in small, flat or in upper reaches of catchments, where there is no clearly defined watercourse.

Run-off is in the form of thin layers of water flowing slowly over the uneven ground surface.

The kerby formula is recommended for the calculation of Tc in this case.

$Tc = 0.604(rL/s^{0.5})^{0.467}$……………………………………………………………kerby formula.

Where:

Tc= time of concentration (hours)

r = roughness coefficient obtained from DDM

L=hydraulic length of catchment, measured along flow path from the catchment boundary to the point where the flood needs to be determined (km).

S=Slope of the catchment, $s = \frac{H}{1000*L}\ (m/m)$

H =height of most remote point above outlet of catchment (m).

ii. Calculation of time of concentration for defined watercourses

In a defined watercourse, channel flow occurs. The recommended empirical formula for calculating the time of concentration in natural channels was developed by the US Soil Conservation Service.

$$T_c = \left(\frac{0.87L^2}{1000S_{av}}\right)^{0.385}$$ ……………………………………………………………………………Eq.3.2

Where:

Tc = time of concentration (hours).

L = hydraulic length of catchments measured along flow path from the catchment boundary to the point where the flood needs to be determined (km).

S_{av} = average slope (m/m).

The formula for determining the slope according to the slope methods reads:

$$\mathbf{Sav} = \frac{\mathbf{H0.85L - H0.10L}}{\mathbf{(1000)(0.75L)}} \text{ Or, } \mathbf{Sav} = \frac{\mathbf{H}}{\mathbf{(1000)(0.75L)}}$$

Where:

S_{av} =average slope (m/m)

H 0.10L =elevation height at 10% of the length of the watercourse (m)

H 0.85L = elevation height at 85% of the length of the watercourse (m)

L = length of watercourse (km)

H = H 0.85L - H 0.10L (m)

4. Frequency Factor

Frequency analysis is hydrologic term used to describe the probability of occurrence of a particular hydrologic event (e.g. rainfall, flood, drought, etc.). Therefore, basic knowledge about probability (e.g. distribution functions) and statistics (e.g. measure of location, measure of spread, measure of skewness, etc.) is essential. Frequency analysis usually requires recorded hydrological data.

As per ERA drainage design manual (DDM), the frequency factor is used to magnify the less frequent storms, i.e. storms with recurrence interval greater than 10yr.

In this study the frequency factors (C_f) was select based on recurrence interval as shown in appendix D, table 1.3.

5. Velocity

It is the velocity of water in the flow path. This is because excessive flow velocities will cause scour (clean). The risk of scour depends on the gradient (slope) and geometry of the channel, the soil conditions and the vegetation cover. When the velocity of flow increases beyond a limit, the risk of scour will increase.

It is sound practice to calculate the average flow velocity (V = L/Tc) after determining Tc in order to ensure that it falls within realistic times. Typical value of the flow velocity ranges from **0.1 to 4m/s**, depending on the natural conditions. For short pasture and a low slope of 2%, the velocity is about **0.3** m/sec. The flow velocity depends on the catchment characteristics and slope of the watercourse. The velocity was estimated from appendix D, Figure 1.3

CHAPTER 4

RESULT AND DISCUSSION

This chapter presents the results of laboratory tests, condition survey, gravel loss estimation and drainage capacity determination from a simple rural catchment area and a discussion related to the results.

4.1 Property of natural subgrade soil

The test result conducted for identification and determinations the properties of natural subgrade soil were presented in table 4.1.

Table 4.1 Summary of Geotechnical properties of the natural subgrade soil for all stations soil sample

Parameters	Test results				
	0+540	**1+780**	**3+975**	**5+540**	**10+435**
Natural Moisture Content, %	18.69	26.16	23.57	23.95	24.84
% passing No.200 sieve, %	92.1	91.8	91.5	91.1	91
Liquid Limit, %	84.95	53.4	81.26	66.1	60.45
Plastic Limit, %	36.54	27.95	32.52	25.64	29.75
Plasticity index, %	48.41	25.45	48.74	40.46	30.7
AASHTO Soil Classification	A-7-5	A-7-5	A-7-5	A-7-6	A-7-6
USCS (Group Symbol)	CH	CH	CH	CH	CH
Soil Group Name	Fat Clay	Fat Clay	Fat Clay	Fat Clay	Fat Clay
Subgrade Rating	Fair to poor	Fair to poor	Fair to poor	Fair to poor	Fair to poor
Specific Gravity	2.71	2.65	2.7	2.75	2.74
Free Swell Index, %	79.3	71.5	80.1	75.7	78.5
Maximum Dry Density g/cm3,	1.32	1.46	1.22	1.44	1.27
Optimum Moisture Content, %	33.28	36.6	34.02	35.99	35.32
Soaked CBR value, %	2.2	2.95	2.25	2.91	2.56
CBR-Swell, %	4.12	3.25	3.96	4.05	4.16

Generally Liquid limit less than 35% is low plasticity and between 35% and 50% intermediate plasticity, between 50% and 70% high plasticity and between 70% and 90% very high plasticity (Whitlow, 1995). As a result, all soil samples of plastic limit have 84.95%, 53.4%, 81.26%, 66.1% and 60.45% these values indicate all soil samples were high plastic clay. Therefore, the subgrade soil shrink and swell easily and does not resist internal and external load. Finally, the structure make crack and easily demolished.

4.1.1 Particle size distribution

Sieve analysis and hydrometer analysis were determined according to AASHTO T87, and T88 standard test method. The detail tabular experimental results are presented in appendix A, and the soil for sample stations (0+540 to 10+435) are passing through No.200 sieve as shown in Figure 4.1 to 4.5

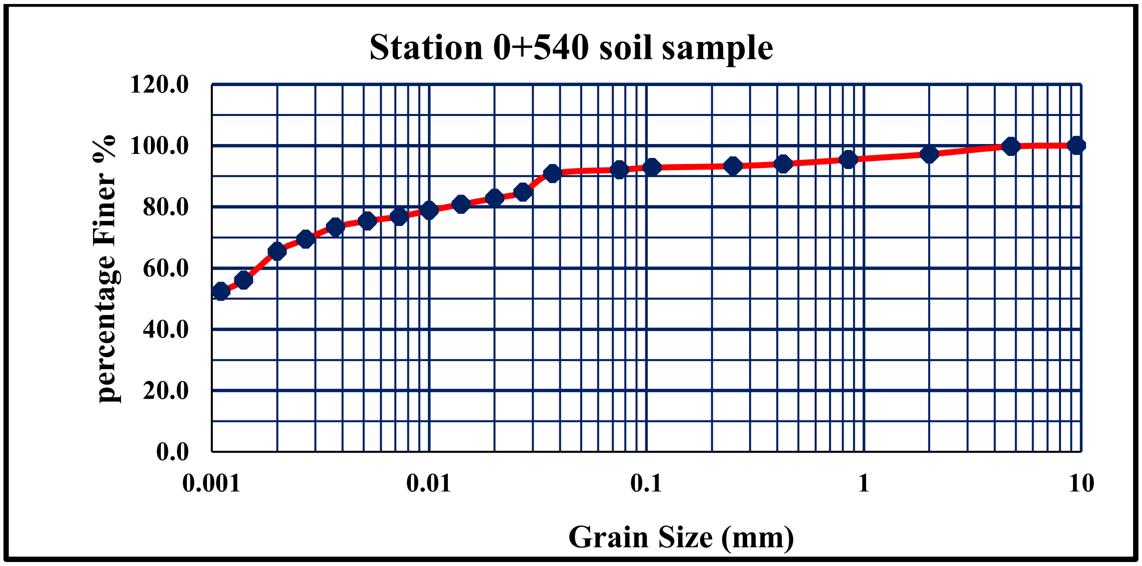

Figure 4.1 Grain size distribution curve of station km0+540 soil sample

According to AASHTO soil classification system the soils has 92.1% (which is greater than 35%) passing through sieve No200. Therefore, the soil is fine grain soil. Which is classified as silty-clay materials and the soil is categorized as poor clay subgrade soil.

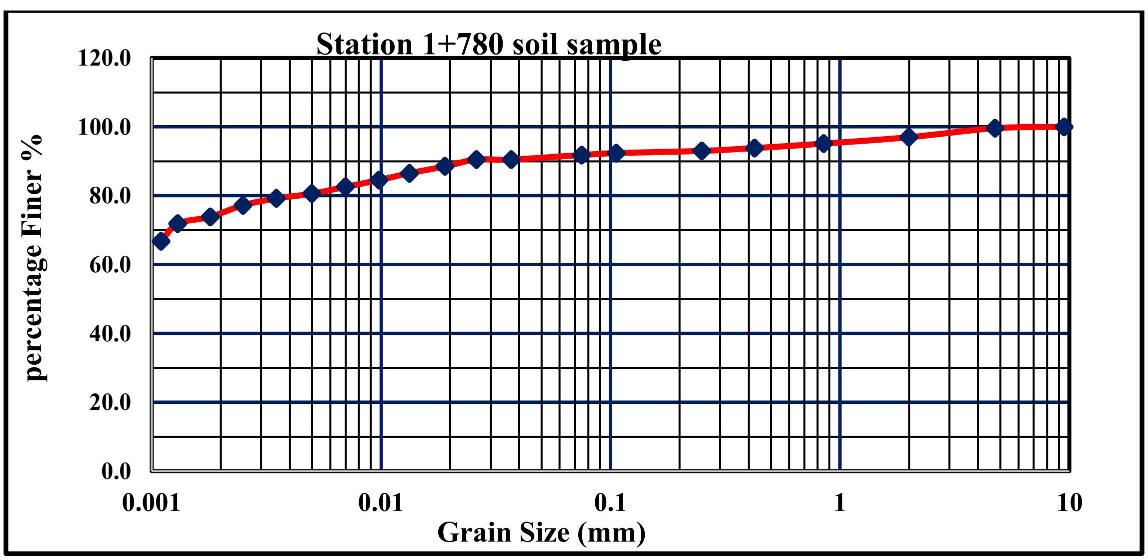

Figure 4.2 Grain size distribution curve of station km1+780 soil sample

The soil for sample station 1+780 is Black, and almost 91.8 % of the soil is passing through No.200 sieve as shown in figure 4.2. Almost the given soil sample were a fine clay soil.

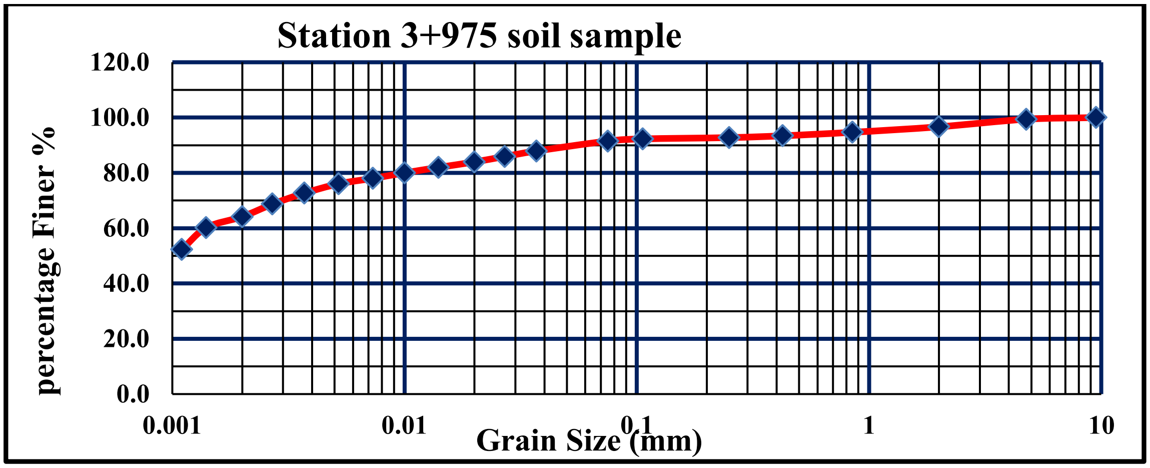

Figure 4.3 Grain size distribution curve of station km3+975 soil sample

The soil for sample station 3+975 is Black, and almost 91.5 % of the soil is passing through No.200 sieve as shown in figure 4.3. Almost the given soil sample were a fine clay soil.

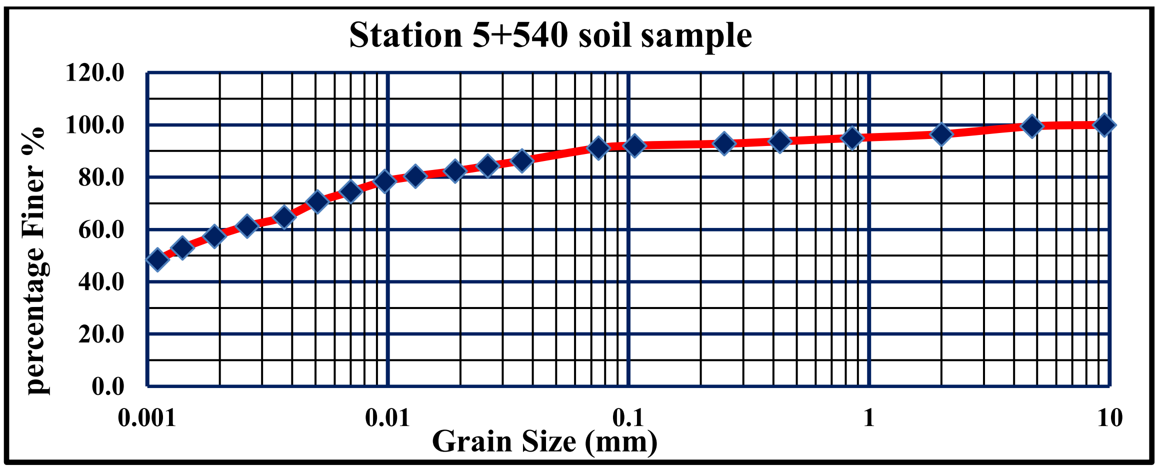

Figure 4.4 Grain size distribution curve of station km5+540 soil sample

The soil for sample station 5+54 is Black, and almost 91.1 % of the soil is passing through No.200 sieve as shown in figure 4.4. Almost the given soil sample were a fine clay soil.

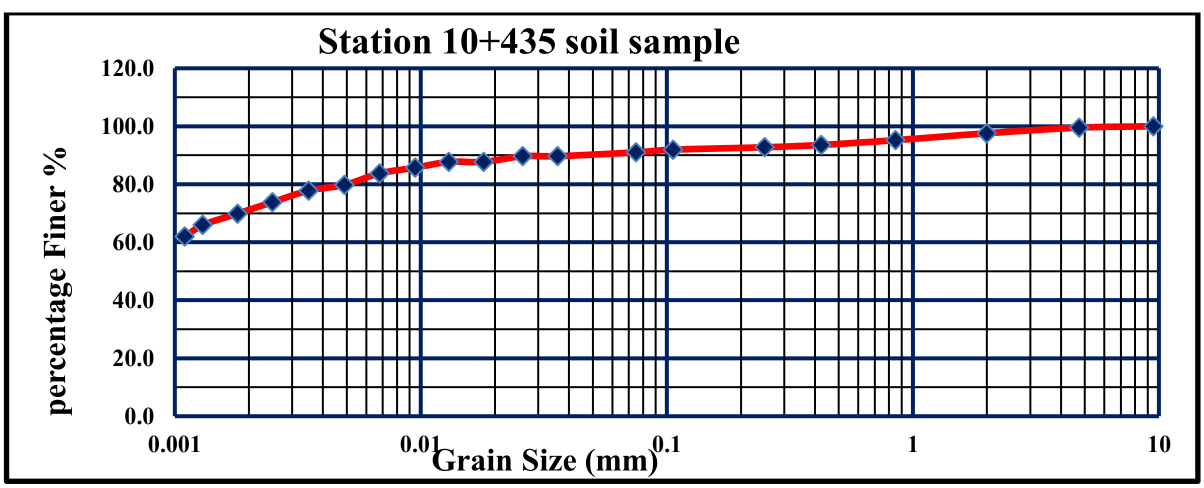

Figure 4.5 Grain size distribution curve of station km10+435 soil sample

The soil for sample station 10+435 is Black, and almost 91 % of the soil is passing through No.200 sieve as shown in figure 4.5. Almost the given soil sample were a fine clay soil.

4.1.2 Atterberg limit test on natural subgrade

Atterberg limits (liquid limit, plastic limit tests) were determined according to AASHTO T89, and T90 standard test method. The detailed tabular results of the Atterberg limits were shown in appendix A. Based on the Atterberg test results summary of both soil samples are tabulated below in table 4.2

Table 4.2 Summary of Atterberg limits for the natural subgrade soil for all soil sample stations

NATURAL SUBGRADE SOIL				
Sample station	Average Liquid Limit (%)	Plastic Limit (%)	Plasticity Index	Plasticity properties
0+540	84.95	36.56	48.41	Very high plasticity
1+780	53.4	27.95	25.45	High plasticity
3+975	81.26	33.52	48.74	Very high plasticity
5+540	66.1	25.64	40.46	High plasticity
10+435	60.45	29.75	30.7	High plasticity

According to Atterberg, limit test result as shown by Table 4. 2 The soil sample changed from liquid state to plastic state and got an average liquid limit of 84.95%, 53.4%, 81.26%, 66.1%, and 60.45% respectively. The given soil sample translate from plastic state to semisolid state and got an average plastic limit of 36.56%, 27.95%, 33.52%, 25.64%and 33.18% for all stations of soil sample respectively. The difference between the liquid lime and plastic limit is called Plastic Index. The soil sample also has Plasticity Index of 48.41%, 25.45%, 48.74%, 40.46% and 30.7% for all soil samples respectively. As result of Plastic Index indicates both the native subgrade soil samples have poor for sub grade material.

4.1.3. Engineering Classification of Soil

4.1.3.1 AASHTO Classification system

The AASHTO system uses similar techniques as that of USC but the dividing line has an equation of the form PI= LL-30. It generally classifies a soil broadly into granular material and silt-clay material. The granular material is further divide into three groups, which are A-1, A-2 and A-3. The silt-clay material is in turn divide into four groups namely, A-4, A-5, A-6 and A-7. As it can observed from AASHTO Classification, system plasticity chart is as Follows in Fig.4.6.

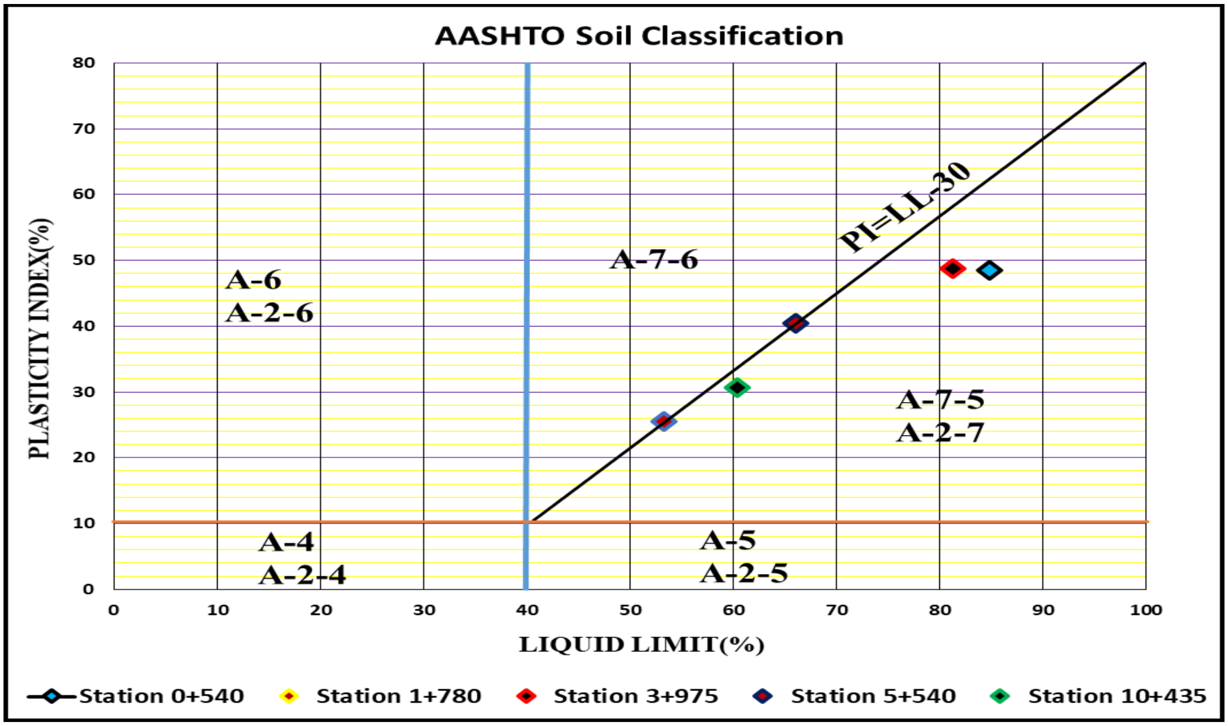

Figure 4.6 Liquid limit and Plasticity index for Nine AASHTO soil groups

As results of atterberg limit test result all samples has different Liquid limit and plastic Index, however according to AASHTO soil classification system all soil samples have classified under group A-7-5 and A-7-6. Which are three soil samples under soil group A-7-5 and two soil groups were under A-7-6.

The natural subgrade material is unsuitable for the constructions purpose compared with standard specifications.

Table 4.3 Summary of AASHTO soil classification and their group index value

Sample Name	Soil Group	Group Index	Subgrade Rating
0+540	A-7-5	54	Poor
1+780	A-7-5	27	Fair
3+975	A-7-5	53	Poor
5+540	A-7-6	42	Poor
10+435	A-7-6	33	Fair

4.1.3.2 Unified soil classification (USCS) system

This system describes a system for classifying minerals for engineering purposes based on laboratory determination of particle-size characteristics, liquid limit and plasticity index and shall be used when precise classification is required (ASTM). Expansive soil must have a significant clay content, probably falling within the unified symbols CL or CH.

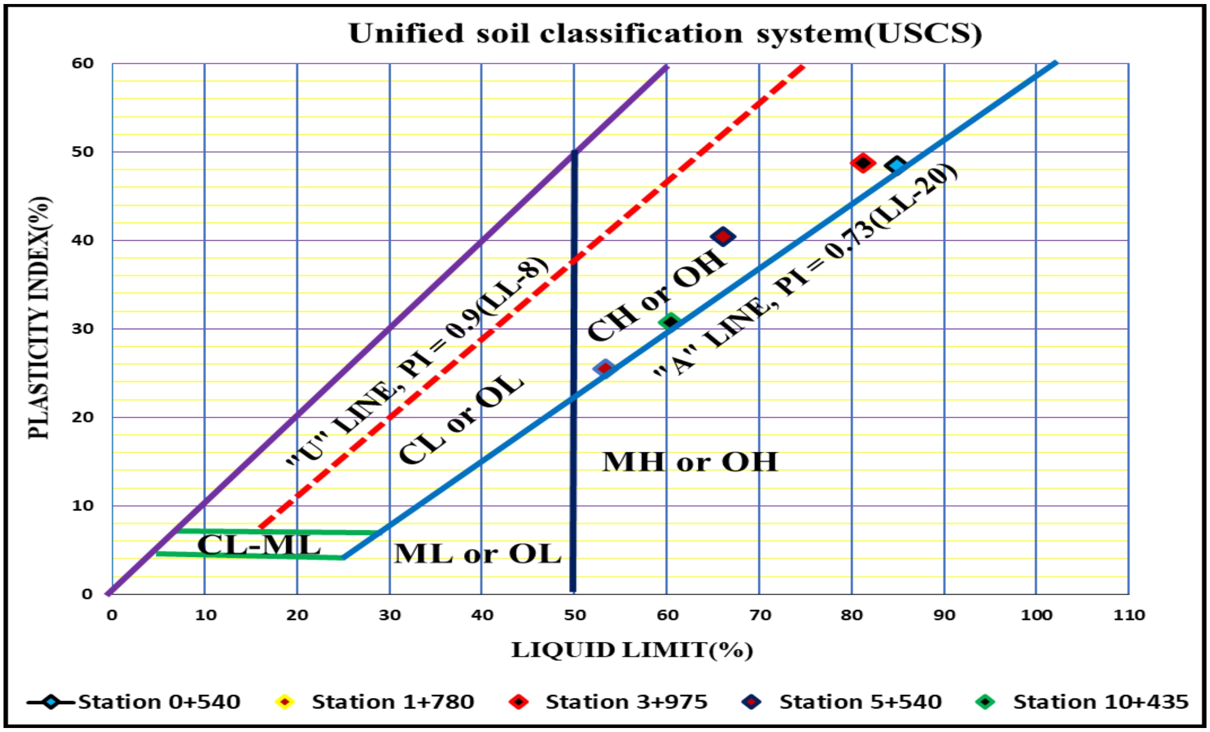

Figure 4.7 Soil classifications according to Unified Soil Classification System (USCS).

According to USCS, if the Liquid limit are greater or equal to 50% the soil can be clay, silt, or organic depends on whether the soil coordinates plot above or below the A line. Since both soil sample has liquid limit more than 50% and above A-Line, so they are classified under high to very high CH.

4.1.4 Specific Gravity of natural subgrade soil

This test was conduct on fined grained particles of materials used for the study and summary of the test results are tabulated as followed in Table 4.4.

Table 4.4 Specific Gravity of Natural subgrade soil Sample

Sample Stations	Specific Gravity (Gs)
0+540	2.74
1+780	2.65
3+975	2.70
5+540	2.75
10+435	2.71

As table 4.4 showed that all sample has an average specific gravity of 2.74, 2.65, 2.7, 2.75 and 2.71. The specific gravity of solid particles of most soils varies from 2.5 to 2.9. For most of the calculations, specific gravity (Gs) can assumed as 2.65 for Cohesion less soils and 2.70 for clay soils. This result indicated both samples are dived under clay soil.

4.1.5 Free swell index

The test tries to give a fair approximation of the degree of expansiveness of a given soil sample.

Table 4.5 Free swell index test result natural subgrade soil Sample

Sample Stations	Free swell index (%)
0+540	79.3
1+780	71.5
3+975	80.1
5+540	75.7
10+435	78.5

Free swell test value of all soil samples indicates, 79.3%, 71.5%, 80.1%, 75.7% and 78.5% respectively. Soils having a free swell value above 100 can cause damage whereas free swell as low as 100 percent can cause considerable damage to light loaded structures and soils having a free swell value below 50 percent seldom exhibits appreciable volume change even under light loads. Hence the free swell value of the soil under study exceeds 50%, and such soils undergo volumetric changes leading to pavement distortion, cracking and general unevenness due to seasonal wetting and drying (Ranjan & and Rao, 2002).

4.1.6 Compaction test results of natural subgrade soil

Standard Proctor compaction tests were conduct on the soil to determine the relationship between the moisture content and dry density for specific compaction effort according to AASHTO designation T99-94. All soil sample has optimum moisture contents of 33.28%, 36.6%, 34.02%, 35.99%, and 35.32%. In addition; the maximum dry density of all soil sample were1.32g/cm^3, 1.46 g/cm^3, 1.22 g/cm^3, 1.44 g/cm^3 and 1.27g/cm^3 as shown below in figure 4.6.

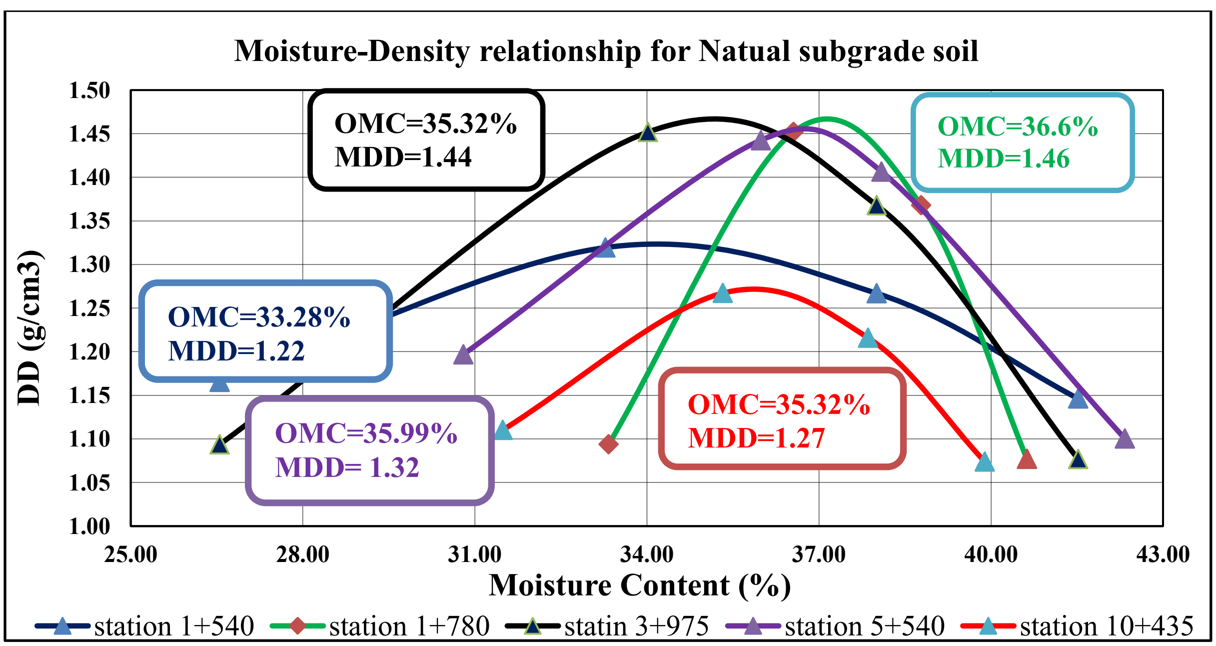

Figure 4.8 Moisture-density relation for the natural subgrade soil for all stations

4.1.7 CBR test result of natural subgrade soil

Strength of the soil has also been determined. A one point (56 blows) soaked CBR test was conducted according to AASHTO T193, summary of results as presented table 4.6 and Figure 4.7 blows.

Table 4.6 summary of CBR test result of natural subgrade soil for all soil sample stations

Sample Station	No of blow	Load (KN)				Calculated CBR (%)				Swell (%)
		Trial 1		Trial 2		Trial 1		Trial 2		
		@2.5	@5.0	@2.5	@5.0	@2.5	@5.0	@2.5	@5.0	
0+540	56	0.24	0.42	0.27	0.44	1.82	2.10	2.05	**2.20**	4.12
1+780	56	0.348	0.464	0.389	0.497	2.64	2.32	**2.95**	2.49	3.25
3+975	56	0.298	0.43	0.273	0.411	**2.26**	2.15	2.07	2.06	3.96
5+540	56	0.384	0.517	0.346	0.495	**2.91**	2.59	2.62	2.48	3.35
10+435	56	0.338	0.46	0.313	0.436	**2.56**	2.30	2.37	2.18	4.16

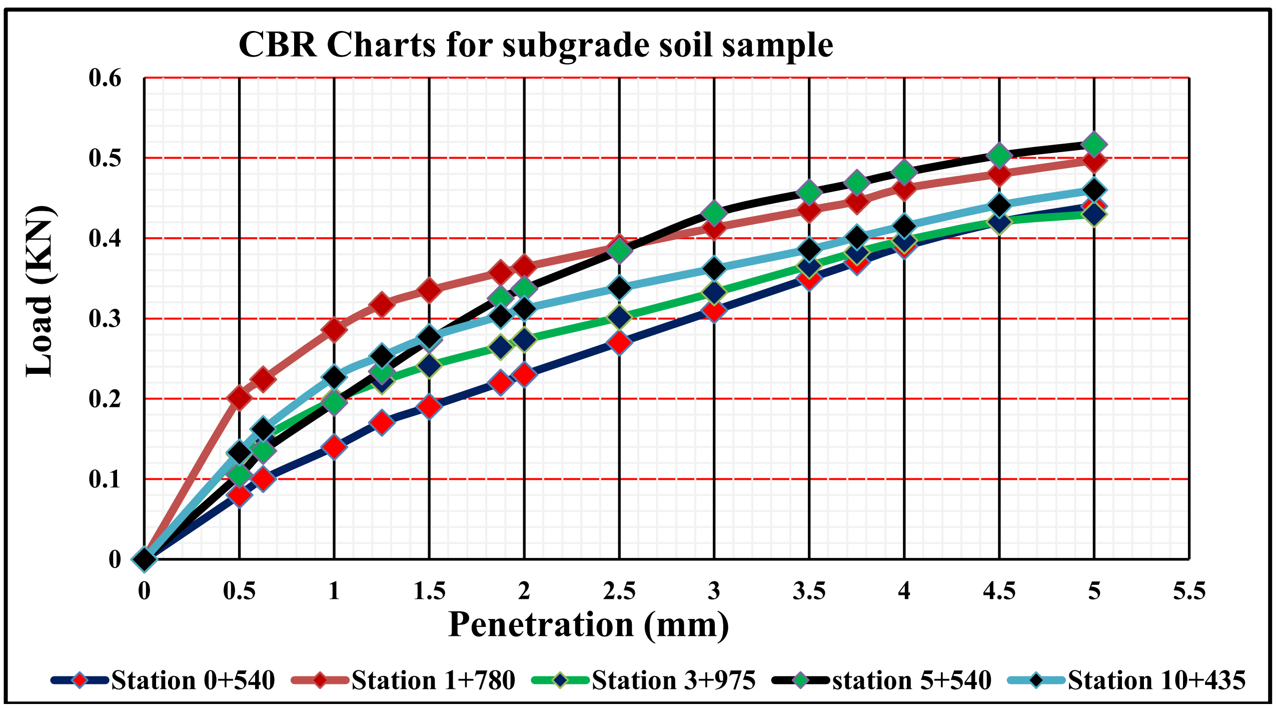

Figure 4.9 Charts of selected CBR results on natural subgrade soil for all soil samples stations

According to laboratory result as presented in table 4.6 and figure 4.7, all soil sample had 2.2%, 2.95%, 2.26%, 2.91% & 2.56%soaked CBR value, with 4.12%, 3.25%, 3.96%, 3.35% & 4.16% CBR swell respectively. From the soaked CBR test, it was found that the natural subgrade soil has low CBR value, as compared with ERA LVR standards, Soils with a soaked CBR of less than 3% are described as Low

Strength soils. All materials shall be brought to a strength of at least a minimum CBR of 7% for minor gravel roads and at least a minimum CBR 25% for major gravel roads recommended that (ERA, 2002).

Based on the above specification not all subgrade soil samples satisfy the minimum requirements as sub-grade material. In addition, CBR swell values are above the specified maximum value of 2%, hence this soil is expansive soil, such relatively poor soil should be excavated and replaced, or covered with an improved subgrade.

4.1.8 Overall Characterization of the natural subgrade soil

According to the laboratory test results of the natural subgrade soil sample obtained during the present study, the proportion of fines grained soil. It have high liquid limit, all soils samples are classified in to A-7-5 and A-7-6 as per the AASHTO soil classification and CH as per the Unified soil classification system.

As far as the engineering performance of soils of this class is concerned, such soils are expansive soils, which have high volume changing properties with variation in moisture content (Chen, 1988).

ERA Standard Specs 2013 recommended that Clay material having a Liquid Limit (LL) exceeding 60; or a Plasticity Index (PI) exceeding 30; or CBR value less than 3% at 95% of modified AASHTO compaction (AASHTO method T-180) after 4 days soaking. Swell value of more than 3% when determined in accordance with AASHTO T-193 at 95% of modified AASHTO compaction. Accordingly, all soil samples show excess values in each parameter except, station 1+780m and the soil in general thus had expansive property and poor quality.

The free swell index of 79.3%, 71.5%, 80.1%, 75.7% and 78.5%for all soil sample respectively, also revealed that the soils are expansive soil, since its free swell index is greater than 50%. Furthermore, the CBR value and percent swell of the soil samples indicates that the soils has a low load bearing capacity and high swelling potential when compared to ERA's specifications of CBR>3% and percent swell of CBR more than 3% which makes it unsuitable for construction without any suitable treatment measure. However, the comparisons above between ERA design manual and laboratory results of the soil shows that, it do not full fill the requirements as a sub-grade and are determined to be unsuitable for sub-grade in road construction. Therefore, the existing road was failed in the case of poor quality subgrade soil.

4.2 Property of gravel materials used in the study road

The results of the tests were conducted for identification and/or determination of properties of the gravel materials are presented in table 4.7.

Table 4.7 Summary of Engineering properties of the gravel material

Parameters	Test results				
	0+540	1+780	3+975	5+540	10+435
Natural Moisture Content, %	7.02	8.33	6.9	7.36	7.72
% passing No.200 sieve, %	1.1	3.2	0.8	1.2	2.6
Liquid Limit, %	40.59	44.27	41.08	43.99	44.02
Plastic Limit, %	28.91	30.21	27.80	28.89	28.78
Plasticity index, %	11.68	14.06	13.28	15.10	15.24
AASHTO Soil Classification	A-2-7	A-2-7	A-2-7	A-2-7	A-2-7
USCS (Group Symbol)	SW	SW	SW	SW	SW
Soil Group Name	Sand with Gravel	Sand with Gravel	Sand with Gravel	Sand with Gravel	Sand with Gravel
Maximum Dry Density g/cm3,	1.98	1.89	1.88	1.89	1.88
Optimum Moisture Content, %	15.49	18.28	16	17	19
Soaked CBR value, %	20.85	7.19	12.57	6.98	6.14
CBR-Swell, %	0.86	1.72	1.03	1.93	2.02

Generally Liquid limit less than 35% is low plasticity and between 35% and 50% intermediate plasticity, between 50% and 70% high plasticity and between 70% and 90% very high plasticity (Whitlow, 1995). As a result, these values indicate all soil samples are intermediate plasticity and the material type is sand with gravel based on sieve analysis and atterberg limit test.

4.2.1 Particle size analysis

A basic element of a soil classification system is the determination of the amount and distribution of the particle sizes in the soil. The presentation of particle size distribution data can be done in two formats, table and graph format. The table format is the format in which the total percentage of particles of a sample that passes a given sieve size are recorded while in the graphical format the relationship of the sieve or particle size versus the percentage passing the given sieve are plotted in appendix B.

In this study, the results for particle size distribution were assessing the performance relationship of gravel materials used for road construction. This was based on the specification requirement given by Pavement Materials and Design Manual (PMDM) and ERA manuals. Therefore, Table format for data presentation was used as shown Table 4.8 shows the sieve analysis results of materials passing sieves 37.5mm, 25mm, 19mm, 12.5mm, 9.5mm, 4.75mm, 2mm, 0.85mm, 0.425mm, 0.25mm and 0.075mm.

Based on the recommended range for GC, results show that sample station 3+975 is within the recommended range of GC that has 29.47 the other samples stations, which are 0+540, 1+780, 5+540 and 10+435 samples. The samples were out of the recommended limit or above the upper limit, no samples with GC values below the lower limit as shown on Table 4.7. Based on laboratory test results as presented on Table 4.8 show that all samples, which have GM, value greater two. Under the recommended specification given above all samples, have course-graded materials, which are relatively good quality for gravel road construction.

Table 4.8 Summary of Grading Modulus (GM) and Grading Coefficient (GC) for soil samples

Station	Particle Size Distribution (% passing)							GM	Gc
	37.5	25	19	4.75	2	0.425	0.075		
0+540	100	100	96.2	54.1	27.3	6.2	1.1	2.65	39.33
1+780	100	100	97	75	46.4	14.8	3.2	2.36	40.5
3+975	87.6	48.1	81.3	65.2	39.3	9.9	0.9	2.5	29.47
5+540	97.5	96.7	95.5	70.3	40.3	8.9	1.2	2.5	39.65
10+435	100	96.7	93.9	77.3	48.2	13.8	2.6	2.35	37.49

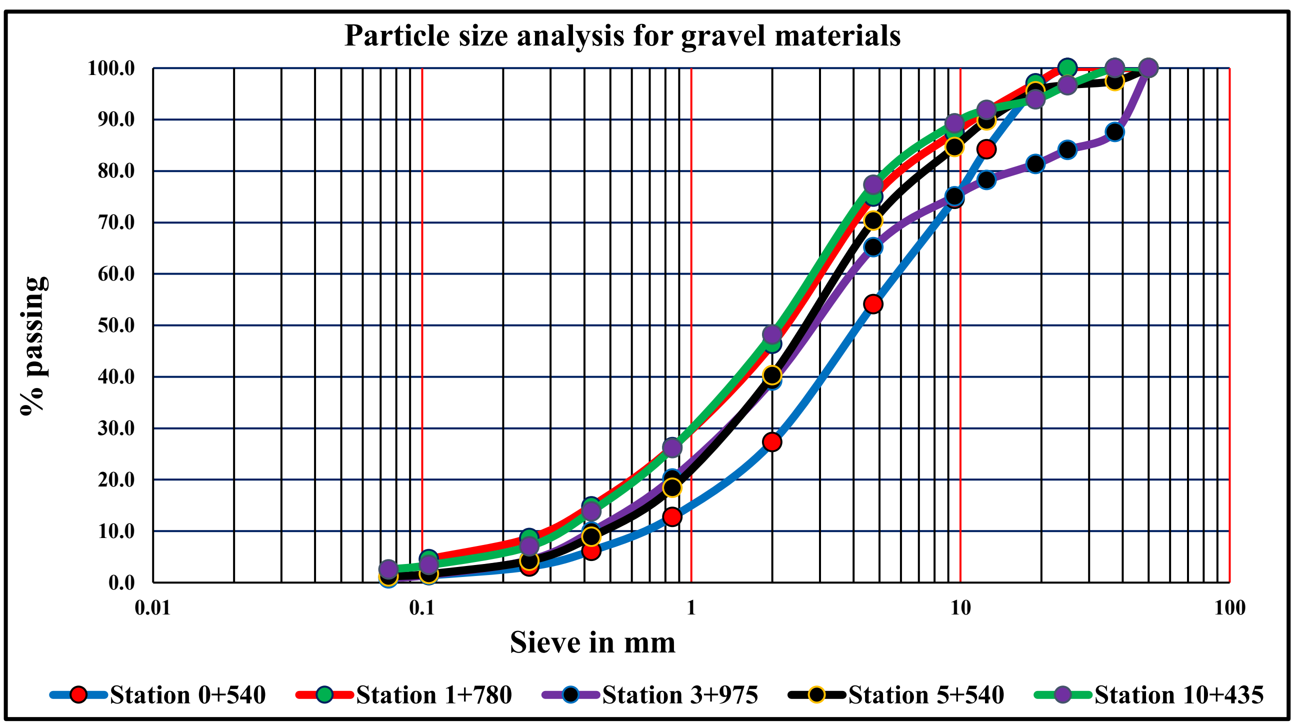

Figure 4.10 Grain-Size Distribution curve of all statins for gravel material.

4.2.2 Atterberg limit test

Atterberg limits (liquid limit, plastic limit and shrinkage limit tests) were determined according to AASHTO T89 and T90 standard test method. The detailed tabular results of the Atterberg limits of gravel material shown in appendix B.

Table 4.9 Summary of Atterberg Limit tests for gravel material samples in study road.

Station	Atterberg Limit Tests				
	Liquid Limit	Plastic Limit	Plasticity Index	Linear shrinkage	Shrinkage product
0+540	40.59	28.91	11.68	5	30
1+780	44.27	30.21	14.06	7	110
3+975	41.08	27.80	13.28	5	51
5+540	43.99	28.89	15.10	6	49
10+435	44.02	28.78	15.24	7	92

According to Atterberg, limit test result as shown by Table 4.9 the soil sample changed from liquid state to plastic state and got an average liquid limit of 40.59%, 44.27%, 41.08%, 43.99%, and 44.02% respectively. The given soil sample translate from plastic state to semisolid state and got an average plastic limit of 28.91%, 30.21%, 27.8%, 28.89%and 28.78% for all stations of soil sample respectively. The difference between the liquid limit and plastic limit is called Plastic Index the results were 11.68%, 14.06%, 13.28%, 15.10% and 15.24% respectively.

If the gravel wearing course material quality is good, the plasticity index should be not greater than 15 and not less than 8 for wet climatic zones and should be not greater than 20 and not less than 10 for dry climatic zones. The linear Shrinkage should be in a range of 3-10% (ERA, 2002).

The soil sample also has Plasticity Index of 11.68%, 14.06%, 13.28%, 15.10% and 15.24% for all soil samples respectively. As a result, these values indicate all the soil samples were intermediate plasticity index values.

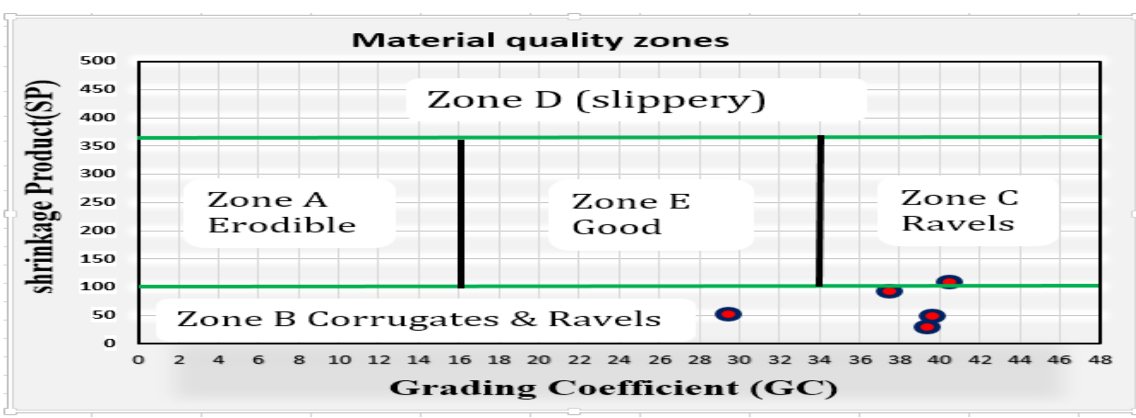

Figure 4.11 Material quality zone determinations using shrinkage product and grading coefficient.

As specified by ERA LVR shown on Figure 4.8 the performance of gravel material depends on the correlation between SP and GC. As indicated on the mentioned figure, these parameters depend on the particle size distribution whereby excessive coarse materials will lead to poor grading and shaping of the road during construction and maintenance practices. In addition, corrugations and raveling problems are the result of low value of SP less than 100 and slippery condition during wet season will be the result of SP value higher than the upper limit.

Based on the specifications all gravel material samples were generally at risk to the formation of loose material (raveling) and corrugations.

4.2.3 Compaction test results of gravel material

Standard Proctor compaction tests were conducted on the soil to determine the relationship between the moisture content and dry density for specific compaction effort according to AASHTO designation T99-94. All soil sample has optimum moisture contents of 15.49%, 18.28%, 16%, 17%, and 19%. And also; the maximum dry density of all soil sample were 1.98 g/cm^3, 1.89 g/cm^3, 1.88 g/cm^3, 1.89 g/cm^3 and 1.88g/cm^3 as shown below in Fig 4.12.

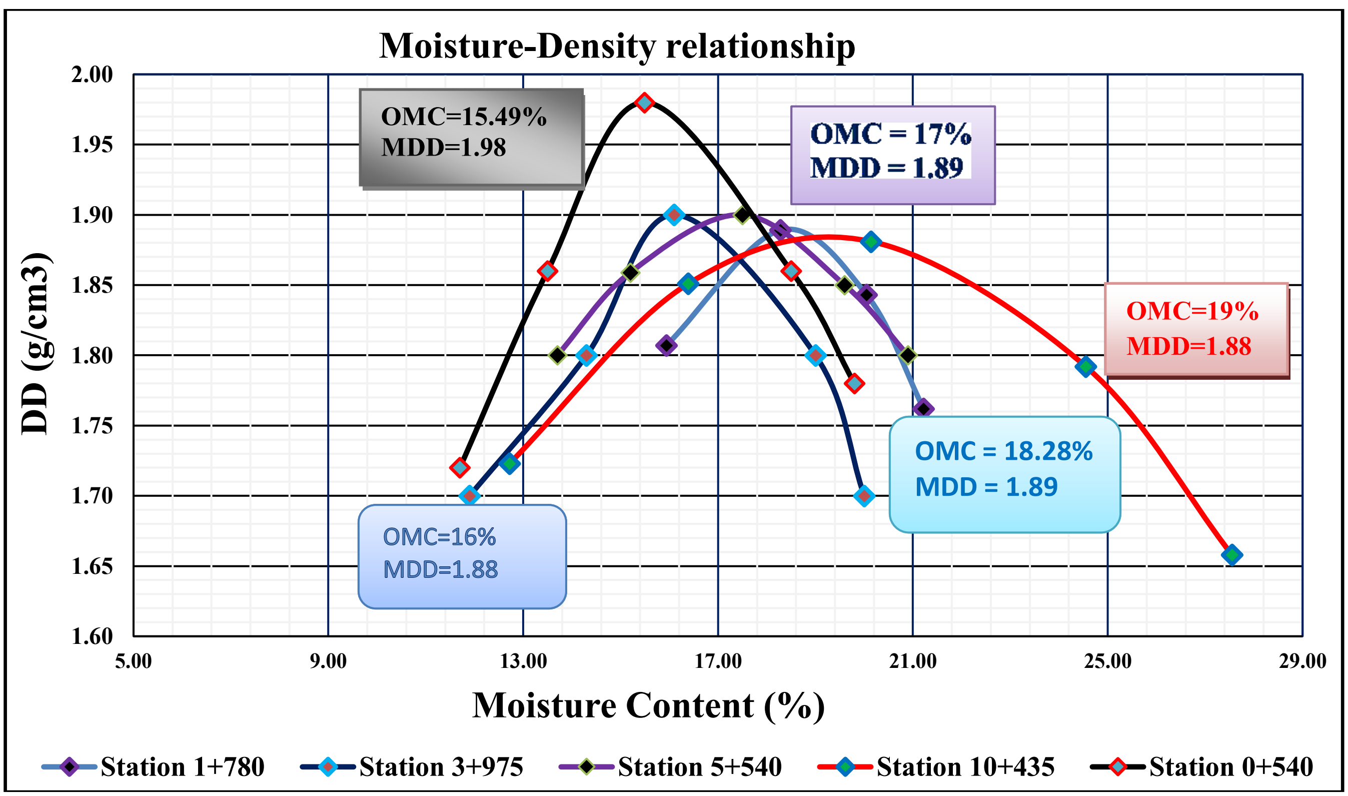

Figure 4.12 Moisture Density Relationship for Gravel Materials

4.2.4 CBR test result of gravel material

Strength of the soil has also been determined. A one point (56 blows) soaked CBR test was conducted according to AASHTO T193, summary of results as presented in table 4.10 and figure 4.11 blow.

Table 4.10 CBR test result of gravel material

Sample Station	Load (KN)				Calculated CBR (%)				Swel l (%)	Gravel class
	Trial 1		Trial 2		Trial 1A		Trial 2A			
	2.5	**5.0**	**2.5**	**5.0**	**2.5**	**5.0**	**2.5**	**5.0**		
0+540	2.375	3.908	2.654	4.17	17.99	19.54	20.11	**20.85**	0.86	G25
1+780	0.764	1.261	0.940	1.437	5.79	6.31	7.12	**7.19**	1.72	G7
3+975	1.489	2.514	1.132	2.013	11.28	**12.57**	8.58	10.07	1.03	G7
5+540	0.921	1.323	0.833	1.264	**6.98**	6.614	6.31	6.32	1.93	G7
10+435	0.796	1.227	0.742	1.173	6.03	**6.14**	5.62	5.87	2.02	G7

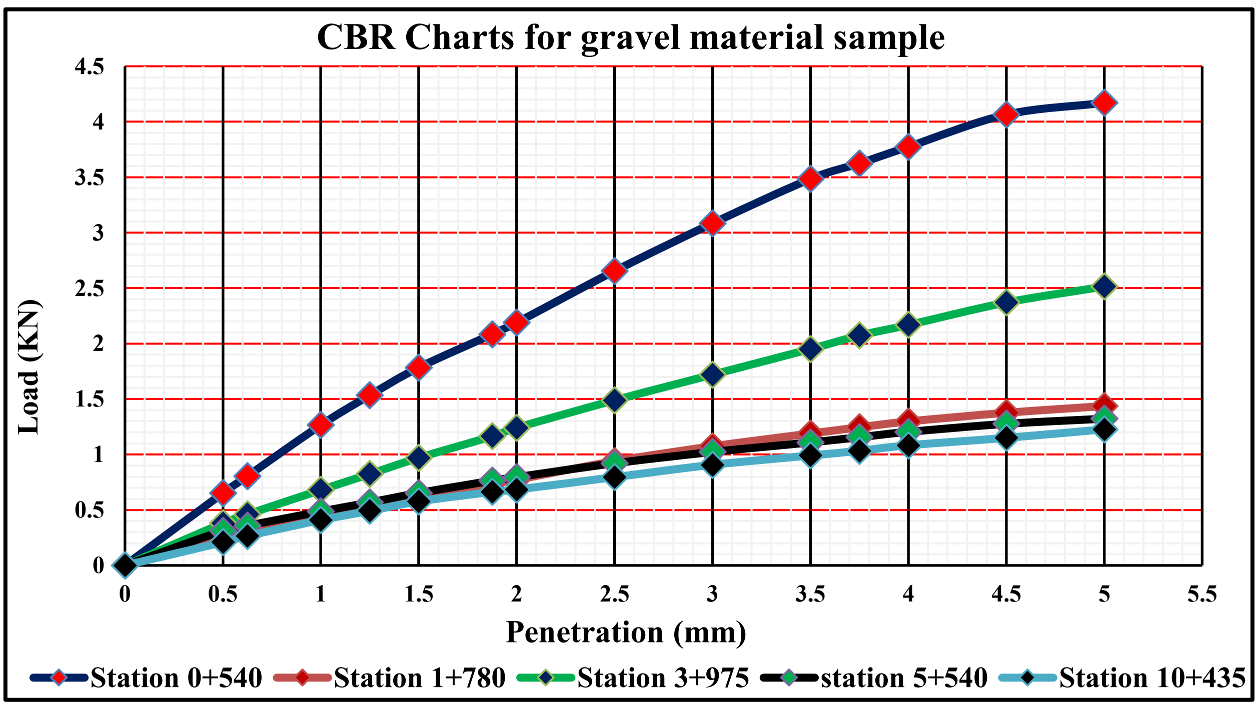

Figure 4. 13 Charts of CBR results on gravel materials for all soil sample stations.

According to laboratory result as presented in table 4.10, all soil sample had 20.85%, 7.19%, 12.57%, 6.98% &6.14% soaked CBR value, with 0.86%, 1.72%, 1.03%, 1.93% &2.02% CBR swell respectively. Except station 0+540 the soaked CBR test, was found that the gravel material has low CBR value, as compared with ERA LVR standards, materials with a soaked CBR of less than 15% are described as Low Strength material. Not all samples satisfy the minimum requirements as gravel material.

From the above result, it is observed that the selected material used for the wearing course has been lost the initial strength after it has been open to traffic. The reduction in the CBR value is due to the sub base material used for the wearing course is corrugated at certain station and crushed too fine in certain station as it is visualized during condition survey time sampling the project route this is due to traffic, weather and environment condition that the material suffering after it has been open to traffic. The correlation between lab result and performance is that the loss in material strength is loss in riding quality or comfort to drive, when the material strength is reduced

4.2.5 Overall Characterization of the gravel material

According to the laboratory test results of the gravel material obtained during the present study, the proportion of coarse grained soil. It have intermediate liquid limit, all soils samples are classified in to A-2-7 as per the AASHTO soil classification and SW as per the Unified soil classification system.

ERA LVR Standard Specs 2013 recommended that gravel material having a Liquid Limit (LL) less than 45; or a Plasticity Index (PI) less than 12; or CBR value greater than 15% at 95% of modified AASHTO compaction (AASHTO method T-180) after 4 days soaking; or a swell value of less than 2% when determined in accordance with AASHTO T-193 at 95% of modified AASHTO compaction. Accordingly, all samples show less value in each parameter except, station 0+540 and the material in general thus had weak property.

The performance of gravel material depends on the correlation between SP and GC. As indicated on the mentioned figure 4.8, these parameters depend on the particle size distribution whereby excessive coarse materials will lead to poor grading and shaping of the road during construction and maintenance practices. In addition, corrugations and raveling problems are the result of low value of SP less than 100 and slippery condition during wet season will be the result of SP value higher than the upper limit.

Based on the specifications all gravel material samples were generally at risk to the formation of loose material (raveling) and corrugations.

The CBR value and percent swell of the soil samples indicates that the material has a low load bearing capacity and low swelling potential when compared to ERA’s specifications except station one. However, the comparisons above between ERA LVR design manual and laboratory results of the material shows that, it do not full fill the requirements as a gravel material and are determined to be unsuitable for road construction. Therefore, the existing road is failed in the case of poor quality gravel material.

4.3 Evaluations present serviceability rate, road condition and gravel loss

4.3.1 Determination of present serviceability rating (PSR)

Present serviceability rating from very good to very poor as gathered from the field rater or the drivers, guideline used for the drivers were questionnaire requesting road performance form very good to very poor. The scale for the rating is described as-

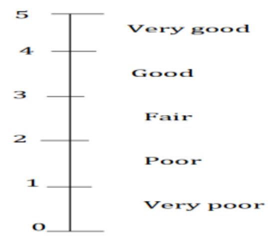

Figure 4.14 Scale for the rating of user assessment

By distributing questionnaire to the drivers on the study, site different rating values had been gathered from the selected section of Mechare – Arssema road segment. The summarized result from the rater is displayed below table 4.11under the user assessments.

User assessment is the road users feed back to the road pavement, in order to evaluate the performance rating of the pavement, experts who drove around the pavement give rate for the pavement, in this research for the study route of Mechare – Arssema road segment various sections were rated by road users and the value was assign as shown in the table 4.11 from very good to very poor and the assigned values of [4-5] and [0-1] respectively.

Table 4.11 Evaluated results for PSR in all selected sections using the road user assessment rating values.

Road Sections	Scales for the rating of user assessment				
	Very Good (4-5)	Good (3-4)	Fair (2-3)	Poor (1-2)	Very Poor (0-1)
Section 1 (0-5km)				1.25	
Section 2 (6-10km)			2.9		
Section 3 (11-15km)				2	
Section 4 (16-20km)			2.9		
Section 5 (21-25km)				1.5	
Section 6 (26-30km)				1.25	

The questionnaire is distributed and filled by the drivers, which found under each section, in the study route four sections were selected each having 5km length.

Performance rating value for the road segments as evaluated from the all rater is determined average present serviceability rate is 1.96.

Based on user assessment the selected route segment the present serviceability rate of the existing road was grouped under poor condition.

4.3.2 Evaluations gravel road using condition survey

During observations of the existing road, the travel on the roadway is very difficult. The roadway is bowl-shaped with extensive ponding. Severe ruts and potholes exist in the roadway, and many areas have little or no aggregate. There are few if any primary ditches, and secondary ditches are evident along most of the roadway. Culverts are either damaged or filled with debris.

Table 4.12 Relationship between numerical rating and subjective evaluation for existing gravel roads.

Road Sections	Numerical Rating	Subjective Evaluation	Level of repairs
Section 1 (0-5km)	40 to 21	Poor	Rehabilitation
Section 2 (6-10km)	60 to 41	Fair	Heavy maintenance
Section 3 (11-15km)	40 to 21	Poor	Rehabilitation
Section 4 (16-20km)	60 to 41	Fair	Heavy maintenance
Section 5 (21-25km)	40 to 21	Poor	Rehabilitation
Section 6 (26-30km)	40 to 21	Poor	Rehabilitation

Based on the condition survey the two sections, which are section 2 and section 4, were the pavement has a good crown, primary ditches are present on more than 50 percent of the roadway, secondary ditches are evident along the shoulder line, and some culvert cleaning is necessary. The gravel layer is adequate, but additional aggregate is needed in isolated areas. Moderate wash boarding exists over 10 to 25 percent of the area, and moderate rutting occurs in wet weather. Occasional small potholes and some loose aggregate are present. The road sections were shown in the following figure 4.15.

Figure 4.15 Photographs of gravel surfaced roadway in fair condition

In the rest sections of existing road, condition travel at slow speeds is required. There is little or no roadway crown, moderate to severe wash boarding, severe loose aggregate and moderate potholing. Up to 25% the roadway has little or no aggregate. More than 70% the ditches are inadequate, secondary ditches not exist along most of the roadway, and culverts are totally filled with debris as shown in the figure 4.16.

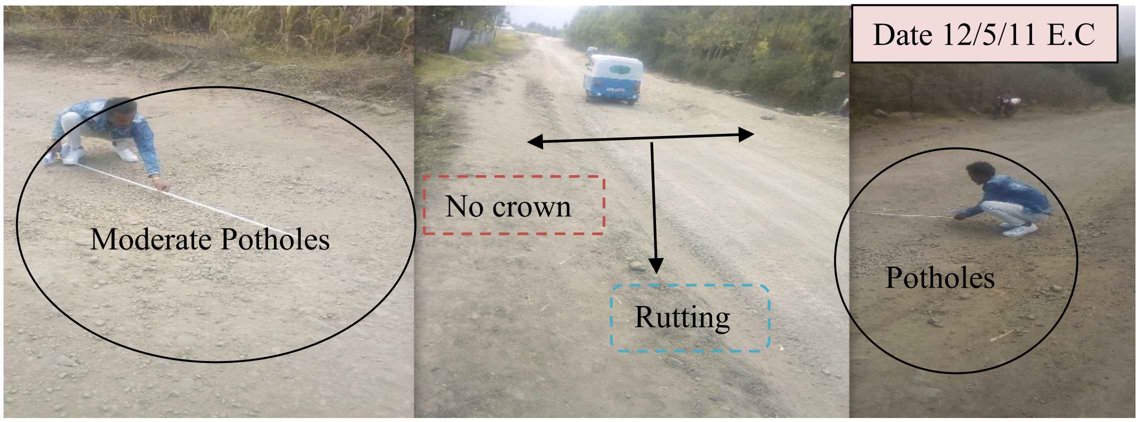

Figure 4.16 Photographs of gravel surfaced roadway in poor condition

As shown in the above photographs the study road section has different severities like potholes, rutting, inadequate drainage and culvert structures were totally filled with debris, inadequate cross section and the road totally needed rehabilitation level of repair.

4.3.3 Determinations of gravel loss for wearing course materials

In the study the deterioration model was selected TRH20 gravel loss deterioration model because; this deterioration model is need the following parameters the parameters were the traffic volumes, climatic conditions and material properties data. Therefore, all necessary data were determine based on laboratory analysis of gravel materials and determinations of traffic characteristics. These parameters are play major role in the prediction of annual gravel loss.

As show in the following table, the gravel loss is computed in five stations.

Table 4.13 Determining the gravel loss in terms of material characteristics and traffic volume.

Test Section	ADT	Weinert N-value	%passing 26.5mm sieve	%passing 0.075mm sieve	PL	PF	GL (mm)
0+540	76	3.7	100	1.1	28.91	31.801	14.68
1+780	76	3.7	100	3.2	30.21	96.672	14.34
3+975	76	3.7	84.1	0.8	27.8	22.24	14.62
5+540	76	3.7	96.7	1.2	28.89	34.668	14.64
10+435	76	3.7	96.7	2.6	28.78	74.828	14.43
Average gravel loss along the existing road							**14.54**

The wearing course thickness was determining using regrading interval and design thickness of the road. Based on determinations of wearing course thickness formula the calculated wearing course thickness of the study road was 272.7mm, the existing road was constructing using 200mm surface course thickness, the difference between calculated thickness and design thickness is 73mm, and the existing road does not consider predictions of gravel loss during design of the road.

4.4 The hydrological conditions of existing road drainage system

4.4.1 Results from observation and photography

This research project employed both observation and photography as tools for which data was collected. This involved observation and taking of photographs to show the current state of the drainage system in study road. From observation also; a brief description of what was observed would be given with the help of photographs.

From observation, the conditions of the existing road drainage system was under poor condition. Figure with definitions from the field survey, it was observed that the road surface moderate corrugation, rutting, road edge damages and accumulation of soil on a large area of road surface as shown below.

Figure 4.17 Side drain is blocked with debris, vegetation and solid waste.

The existing drain is located on one side of the road. It is open and earth ditch with 1m width and 0.5m to 0.8m depth. The condition of this drain is very bad. The side drain was blocked with debris, vegetation, and solid waste as shown in above figure.

Figure 4.18 Failures of road edges due to water ponding on surface of road.

Providing the road profile with a gentle longitudinal gradient improves the road surface drainage. This slope facilitates the discharge of water from sections of the road surface with limited cross slope. For roads with asphalt surface, the camber is normally 2% to 3%, because water would easily flow off on the surface

Figure 4.19 Formations of potholes and corrugation by lack of adequate crown

As shown on the above figure the road that lacks and inadequate crown, potholes and corrugations are also forming because of lack of crown to drain water from the road surface.

The road longitudinal and cross section slopes are almost flat with some deformations. The damage of the road was observed mainly in the middle portion of the road length, particularly in the edge lane of the carriageway. The majority of the damage was moderate to severe deformation and raveling. The severe raveling has run to potholes and depression areas where the top surface has delaminated from the road as clearly shown in above of the photographs.

Figure 4. 20 A poor cross section with no crown on the surface of the road

As shown on the above figure the major problem is a poor cross section with a crown on the surface and no ditches at the edge of the roadway to drain water off the surface and away from the road.

Figure 4. 21 Over flooding of water on the road by poor road drainage cross section

As we can see from the above photos, the water picture spreads along the longitudinal profile of the road as there is no crown, poor cross section and there is no side drain provided for the road. Therefore, water is over flooding of the road. According to field observation made, some of the side drain ditches were constructed for nothing, as there is no inlet or opening to collect storm water from the adjacent surrounding area or road.

Figure 4. 22 Lack of side ditch drainage affects the performance of the road

The damage was observed mainly in the middle portion of the road length, particularly in the center of carriageway. The majority of the damage was moderate to severe erosion and raveling. The severe raveling has led to potholes and rutting where the top surface has delaminated from the road pavement as clearly shown in above of the photographs.

The solutions to improve these problems, it is essential that adequate drainage systems provisions are made for road surface to ensure that a road pavement performs satisfactorily.

Figure 4.23 V- Ditch does not work well in highly erodible soils

According to see from this pictures the road structure and the shoulders are washed away by poor drainage conditions especially during rainy seasons, force the water to enter the pavement from the sides as well as from the top surface by lack of drainage of the surface of the road and a V-ditch does not work well in highly erodible soils as shown in the above photograph.

4.4.2 Results from Hydrological Analysis

4.4.2.1 Descriptions of Catchment Area

The catchment area of each watershed and stream length on the whole route corridor was delineated from DEM data. The sizes of each catchment area and stream lengths were determined using Arc GIS software. Descriptions of catchment area for existing drainage systems is tabulated in table 4.13 that are determined by digital elevation model (DEM).

Table 4.14 Catchment characteristics of existing drainage system.

Station	Length (m)	Area (km^2)	Land Coverage	Land Use	Soil type
0+330	1280	0.340	Agricultural land	Intensively Cultivated	Clay
1+320	1460	0.485	Agricultural land	Intensively Cultivated	Clay
2+850	470	0.264	Bush Forest land	Moderately Cultivated land	Rock
3+400	830	0.306	Bush Forest land	Moderately Cultivated land	Rock
4+650	350	0.125	Dense forest land	Moderately Cultivated land	Clay
5+020	240	0.114	Dense forest land	Moderately Cultivated land	Clay
5+440	464	0.280	Dense forest land	Moderately Cultivated land	Rock
6+890	840	0.350	Bush forest land	Moderately Cultivated land	Clay
9+780	252	0.140	Agricultural land	Moderately Cultivated land	Clay

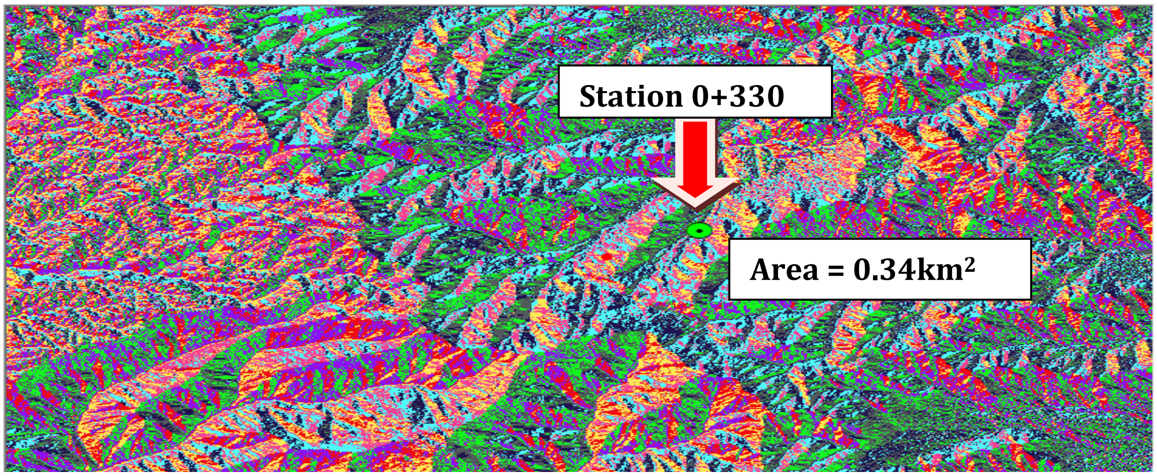

Figure 4. 24 Sample Catchment area for Drainage System at Station 0+330

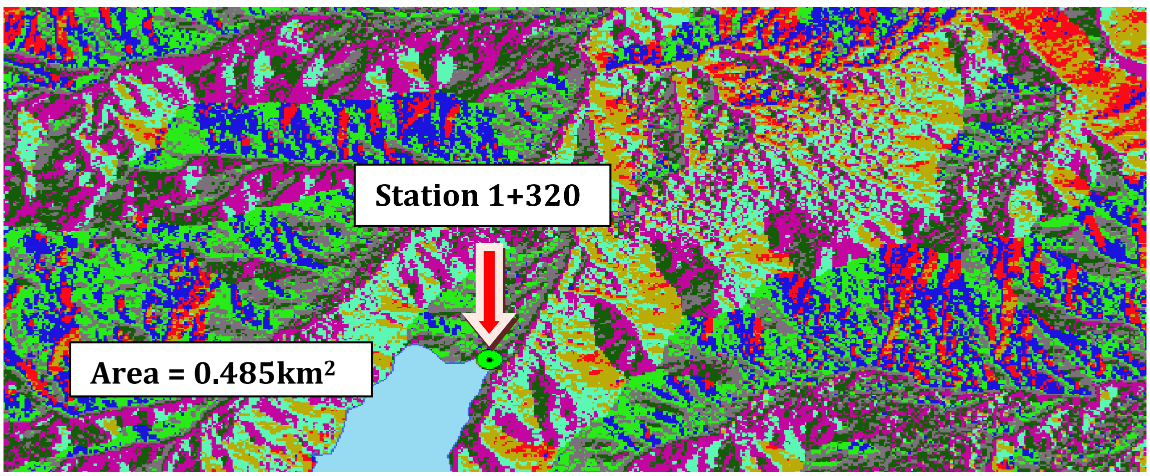

Figure 4. 25 Sample Catchment area for Drainage System at Station 1+320

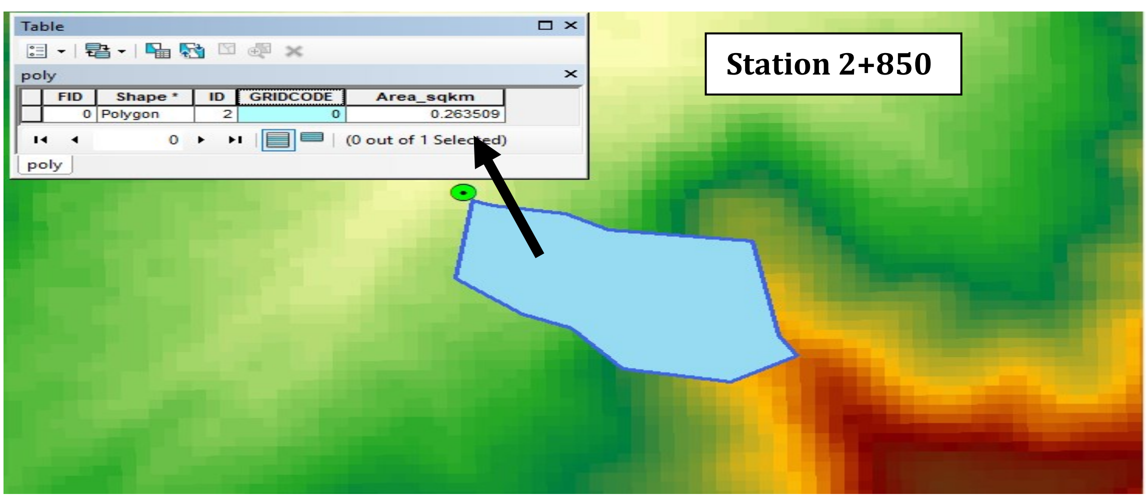

Figure 4. 26 Sample Catchment area for Drainage System at Station 2+850

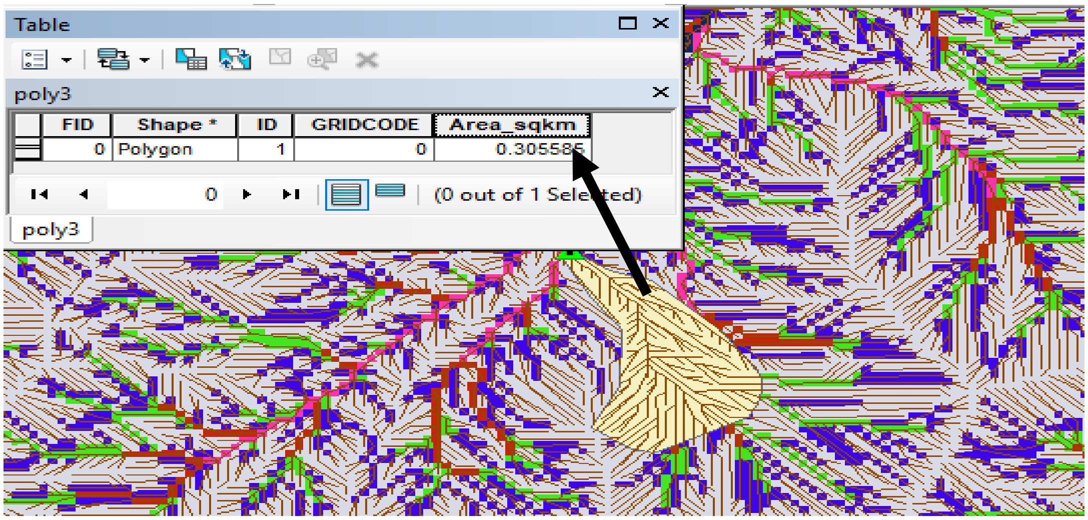

Figure 4.27 Sample catchment area for drainage system at station 3+400

- In the same way, the other catchment areas are delineated using the same procedure.

4.4.2.2 Sample calculation of catchment parameters for station 0+330

It describes the process to determine the peak runoff from a simple rural catchment area for station 0+330 based on the following.

i. Determine Catchment Area

The catchment area was determined using Arc GIS that shown on the above figures.

ii. Determine longest flow path and elevations

Table 4.15 Sample elevation, land cover, soil type rainfall region and hydrological soil group

Station	Catchment Area (Km^2)	Stream length (km)	Upstream Elevation (m)	Downstream Elevation (m)	Land use land Cover	Soil Type	Hydrologic Soil Group	Rainfall Region
0+330	0.340	1.280	2200	1920	Cultivated	Clay	D	C
1+320	0.485	1.460	2320	1820	Cultivated	Clay	D	C
2+850	0.264	0.47	2090	1950	Moderate Cultivated	Rock	D	C
3+400	0.306	0.83	2180	1900	Moderate Cultivated	Rock	D	C

iii. Calculate Time of concentration

The time of concentration in rural area, divide in to two sections as specified as shown below.

1) Time of concentration for over land flow

$Tc = 0.604(rL/S^{0.5})^{0.467}$ ……Eq. 4.3

r = roughness coefficient of land use intensively Cultivated =0.06 (remains cover ≤ 20%) from appendix D, Table 1.4

L = hydraulic length of catchment, measured along flow path from the catchment boundary to the point where the flood needs to be determined (km) =1.28km

H = height of most remote point above outlet of catchment or change in elevation (m).

S=Slope of the catchment, $S = \frac{(\Delta H)}{1000*L}$, $S = \frac{(2200-1920)}{1000*1.28} = 0.22(\mathrm{m/m})$

Tc = $0.604(rL/S^{0.5})^{0.467}$, Tc= $0.604(0.06*1.28/0.22^{0.5})^{0.467}$ = 0.26hr

2) Time of concentration for defined watercourse

$$\mathrm{Tc} = (\frac{0.87\mathrm{L}^2}{1000\mathrm{Sav}})^{0.385}, \mathrm{Tc} = (\frac{0.87*(1.28)^2}{1000*0.13})^{0.385} = \mathbf{0.29hr}$$

L = hydraulic length of catchments, measured along flow path from the catchment boundary to the point where the flood needs to be determined (km) = 1.28km

S_{av} = average slope (m/m), S_{av} = **Δ**H/ (1000*) (0.75*L) = 280/ (1000) (0.75*1.28) = **0.13m**

- Total time of concentration = 0.26hr+0.29hr = **0.55hr = 33min**

iv. Determine rainfall intensity

The catchment area were found in rainfall region C, use the IDF curve of rainfall region C (or use project specific IDF curve) and find the rainfall intensity for different return periods.

The following results were compute using IDF curve of rainfall region C and time of concentration (Tc) as shown in appendix D, Figure D.2.

I_2 = 49.5mm/hr. I_{10} = 70mm/hr.

I_5 = 65mm/hr. I_{25} = 78.5mm/hr.

v. Determine runoff coefficients

The runoff coefficient were determining depends on the average catchment slope, permeability of the soil and vegetation cover the parameters were selected based on catchment terrain, soil type, Cultivation and Grassland/scrub respectively as shown in appendix D table 1.2.

$C = C_s + C_p + C_v$……………………………………………………………………………….Eq.4.4

Cs= average catchment slope coefficient = 0.10

Cp= Soil permeability coefficient = 0.25

CV= Land cover coefficient = 0.20

C = Cs + Cp + Cv, C = 0.10+0.25+0.20 = 0.55

vii. Calculate the Peak flood

In this study the discharge of all drainages were determined using rational method which is the areas were less than 0.5km^2.

$Q = 0.278\ C\ C_f\ I\ A$…………………………………………………………………………….Eq.4.5

Q = maximum rate of runoff, m3/s

C = runoff coefficient

I = average rainfall intensity for a duration equal to the time of concentration, for a selected return period, mm/hr.

A = catchment area tributary to the design location, Km^2

Table 4.16 Computing sample peak flood (Q) for station 0+330 using rational method.

Return period	Year 5	Year 10	Year 25
I =mm/hr.	65	70	78.5
Frequency Factors (C_f)	1.0	1.0	1.1
Runoff coefficient(C)	0.55	0.55	0.55
Area (km^2)	0.34	0.34	0.34
Maximum rate of runoff(Q) =m3/s	**3.38**	**3.64**	**4.49**

Table 4.17 Compare design discharge and calculated discharge for sample station 0+330

Design discharge			Calculated discharge			The difference value		
Q_5	Q_{10}	Q_{25}	Q_5	Q_{10}	Q_{25}	Q_5	Q_{10}	Q_{25}
1.79	**2.15**	**2.76**	**3.38**	**3.64**	**4.49**	**1.59**	**1.49**	**1.73**

- By the same procedures, catchment parameters of the rest stations are determined.

Table 4.18 Summary of calculated discharge (Q) for all stations

Station	**Area**	**Tc**	**Intensity (mm/hr.) from IDF curve**			**Freq. Factor**			**Calculated Discharge (m^3/s)**			
km	Km^2	min	I_5	I_{10}	I_{25}	C_{f5}	C_{f10}	C_{f25}	C	Q_5	Q_{10}	Q_{25}
0+330	0.340	33	65	70	79	1	1	1.1	0.55	3.38	3.64	4.49
1+320	0.485	44	51	60	75	1	1	1.1	0.55	3.78	4.45	6.12
2+850	0.264	32	66	71	79	1	1	1.1	0.55	2.64	2.85	3.51
3+400	0.306	32	66	71	79	1	1	1.1	0.55	3.06	3.30	4.07
4+650	0.125	28	70	75	81	1	1	1.1	0.55	1.34	1.43	1.70
5+020	0.114	24	75	79	85	1	1	1.1	0.60	1.43	1.49	1.78
5+440	0.28	26	73	76	83	1	1	1.1	0.75	4.26	4.44	5.33
6+890	0.35	33	65	70	79	1	1	1.1	0.75	4.74	5.11	6.30
9+780	0.140	28	70	75	81	1	1	1.1	0.55	1.50	1.61	1.91

Table 4.19 Summary of design discharge (Q) for all stations (Source: ARRA)

Station	**Area**	**Tc**	**Intensity (mm/hr.) from IDF curve**			**Freq. Factor**			**Design Discharge (m^3/s)**			
km	Km^2	min	I_5	I_{10}	I_{25}	C_{f5}	C_{f10}	C_{f25}	C	Q_5	Q_{10}	Q_{25}
0+330	0.257	23	76	91	106	1	1	1.1	0.33	1.79	2.15	2.76
1+320	0.470	37	52	65	73	1	1	1.1	0.33	2.25	2.79	3.44
2+850	0.128	11	104	112	146	1	1	1.1	0.33	1.22	1.32	1.89
3+400	0.159	18	87	102	122	1	1	1.1	0.33	1.27	1.49	1.96
4+650	0.047	8	112	115	159	1	1	1.1	0.33	0.48	0.50	0.75
5+020	0.016	2	130	135	184	1	1	1.1	0.36	0.19	0.22	0.32
5+440	0.13	7	114	126	161	1	1	1.1	0.45	1.36	2.05	2.89
6+890	0.231	14	96	120	135	1	1	1.1	0.45	2.04	3.47	4.29
9+780	0.020	4	123	137	174	1	1	1.1	0.33	0.23	0.25	0.35

Based on the above tables the design peak discharge and calculated peak discharge of the return period is not equal. From this the road around this is damaged by over flooding of the water on the road because the design peak discharge was less than the calculated peak discharge for the review at all stations and the difference between them were high Therefore, before the culverts are constructing the design and the review data must be checked. Because based on these values more volume of runoff is very high on the culvert stations. Initially, one rows of pipe culvert was constructed. The one rows of pipe culvert could not accommodate the peak discharge during the rainy season after constructed because the active channel width is greater than the span of the culvert.

Table 4.20 Summary of proposed structure dimensions for all stations using discharge.

St. No	station	Cat. Area	Selected Return Period	Q_{10} (Design)	slope	3.21/sqrt of slope	Q*n	Proposed Structure Dimensions		
								Type	Calculated Diameter	Design Diameter
	Km	Km²	Year	m³/s					m	m
1	0+330	0.340	10	3.64	0.015	26.21	0.06	PC	1.20	0.90
2	1+320	0.485	10	4.45	0.022	21.64	0.08	PC	1.20	1.00
3	2+850	0.264	10	2.85	0.024	20.59	0.05	PC	1.00	1.00
4	3+400	0.306	10	3.30	0.027	19.68	0.06	PC	1.04	1.00
5	4+650	0.125	10	1.43	0.011	30.61	0.02	PC	0.90	0.90
6	5+020	0.114	10	1.49	0.011	30.61	0.03	PC	0.91	0.90
7	5+440	0.280	10	4.44	0.013	27.83	0.08	PC	1.32	1.00
8	6+890	0.350	10	5.11	0.021	22.15	0.09	PC	1.28	1.20
9	9+780	0.140	10	1.61	0.010	32.10	0.03	PC	0.95	0.90

The above table shows that dimensions of all station drainage structures determined using Manning's equation and calculated discharge. These dimensions were very different with comparing the design culvert diameter. In order to mitigate the overtopping problem two-row pipe that has one-meter internal diameter on the left side and the right side was important additionally to mitigate the overtopping problem of the peak flood during the rainy season.

A flood event larger than the specified review flood might be used for analysis to ensure the safety of the drainage structure and downstream development. Always when its design the road it must be follow the drainage design manual.

The analysis of the peak rate of runoff, volume of runoff, and time distribution of flow is fundamental to the design of drainage structures. Therefore when construct the road drainage it consider hydraulic analysis. In addition, maintenance and cleaning of the block drainage system are important.

CHAPTER 5

CONCLUSION AND RECOMMENDATION

Conclusions are for warded from the investigations of the results of the research and recommendations are provided based on the findings of the results of the research.

5.1 Conclusions

The following conclusions are listed on the performance of gravel road based on material quality, gravel surface performance and the causes and effects of poor drainage system on the road.

According to the laboratory test results of the natural subgrade soil samples obtained during the present study, the engineering performance of soils of this class is concerned, such soils were expansive soils, which have high volume changing properties with variation in moisture content, free swell index is greater than 50%. Furthermore, the CBR value and percent swell of the soil samples indicates that the soils has a low load bearing capacity and high swelling potential when compared to ERA's specifications.

The results of the soil shows the subgrade soil is not achieve the requirements as a sub-grade and are determined to be unsuitable for sub-grade in road construction. Therefore, the existing road failed in the case of poor quality subgrade soil. Based on the specifications all gravel material samples were generally lack cohesion and highly susceptible to the formation of loose material (raveling) and corrugations.

The CBR value and percent swell of the soil samples indicates that the material has a low load bearing capacity and low swelling potential when compared to ERA's specifications except station 0+540km. Laboratory results of the material shows that, it does not satisfy the requirements as a gravel material and are determined to be unsuitable for road construction.

Generally, the existing road was failed based on material quality because subgrade soil and gravel materials were not fulfill the requirements of the standard specifications as shown in the laboratory result. Based on road user assessment the selected route segment the present serviceability rate of the existing road is grouped under fair.

The gravel loss of existing road does not consider as shown in the design profile, because the design thickness is different from the calculated thickness.

The design peak discharge calculated and the review peak discharge calculated of the return period is not equal. From this the road around this is damaged by over flooding of the water on the road because the calculated peak discharge for the design were less than the calculated peak discharge for the review at all station.

Generally, it can be concluded that road surface drainage of the study area found to be inadequate due insufficient road profile, insufficient drainage structures provision and flooding problems in the area.

5.2 Recommendations

Based on the findings of this research, the following recommendations are forwarded:

- ✓ It is recommended that more research be conducted in attempt to get more useful pavement data for evaluations the performance of gravel roads by collecting more data in order to improve results for these gravel roads, which exists widely and has high economic value.
- ✓ This study was done for specific area and specific data used, it is recommended as more investigation shall be performed on different parts of the country and used different data has to increase the quality of the research document.
- ✓ The study was conducted only the present serviceability rate, gravel loss, the other researchers should conduct the international roughness index and Present Serviceability Index are both indices that can be used as indicator of road roughness and serviceability.
- ✓ The present study was conducted by taking limited parameter such as Atterberg limit, grain size analysis, free swell index, moisture density relation, and CBR on evaluation of material quality. It is recommended to test additional tests should also be performed to have more realistic test results.
- ✓ The designer and contractor should follow the minimum requirement set by ERA regarding to material quality the drainage structure, size, length and alignments of road drainage structure to prevent the factors affecting the performance of gravel road.
- ✓ This study has certain limitations like those that the data for analysis was taken from only one project route so that it is uncertain to generalize for all type of gravel-unsealed roads, because every road project has its own unique condition and environmental situation.

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APPENDIX:

Appendix A: Laboratory Test Result of subgrade soil for all soil samples

1) Natural Moisture Content

Sample location	Stations				
	0+540	1+780	3+975	5+540	10+435
Can number	X-1	X-2	X-3	X-4	X-5
Mass of moisture can (Mc)	17.61	17.61	12.8	17.61	16.5
Mass of moisture can + Mass of moist soil (McMs)	90	85	90.4	81.96	81.24
Mass of Moisture can + Mass of oven dried soil (McDs)	78.6	71.5	75.6	69.5	68.36
Mass of water (Mw)	11.4	14.1	14.8	12.4	12.9
Mass of dry soil (Ms)	61	53.9	62.8	51.9	51.9
Water Content(W) %	18.69	26.16	23.57	23.95	24.84

2) Sieve Analysis of subgrade soil

Sieve Analysis of station 0+540					
Mass in g		1000			
Sieve No	Sieve size in (mm)	Mass of retained (g)	Percentage retained (%)	Percentage cumulative retained (%)	percentage of finer particle
3/8''	9.5	0	0.00	0.00	100.0
4	4.75	3.0	0.30	0.30	99.7
10	2.000	25.08	2.51	2.81	97.2
20	0.850	17.91	1.79	4.60	95.4
40	0.425	13.87	1.39	5.99	94.0
60	0.250	7.1	0.71	6.70	93.3
140	0.106	5.308	0.53	7.23	92.8
200	0.075	6.78	0.68	7.91	92.1
	PAN	920.922	92.09	100.00	0.0
Total		1000	100.0		

Sieve Analysis of station 1+780					
Mass in g		1000			
Sieve No	Sieve size in (mm)	Mass of retained (g)	Percentage retained (%)	Percentage cumulative retained (%)	percentage of finer particle
3/8"	9.5	0	0.00	0.00	100.0
4	4.75	4.0	0.40	0.40	99.6
10	2.000	26.3	2.63	3.03	97.0
20	0.850	18.5	1.85	4.88	95.1
40	0.425	12.91	1.29	6.17	93.8
60	0.250	8.25	0.83	7.00	93.0
140	0.106	6.43	0.64	7.64	92.4
200	0.075	5.496	0.55	8.19	91.8
	PAN	918.114	91.81	100.00	0.0
Total		1000	100.0		

Sieve Analysis of station 3+975					
Mass in g		1000			
Sieve No	Sieve size in (mm)	Mass of retained (g)	Percentage retained (%)	Percentage cumulative retained (%)	percentage of finer particle
3/8"	9.5	0	0.00	0.00	100.0
4	4.75	6.02	0.60	0.60	99.4
10	2.000	27.4	2.74	3.34	96.7
20	0.850	19.8	1.98	5.32	94.7
40	0.425	12.73	1.27	6.60	93.4
60	0.250	6.51	0.65	7.25	92.8
140	0.106	4.72	0.47	7.72	92.3
200	0.075	7.84	0.78	8.50	91.5
	PAN	914.98	91.50	100.00	0.0
Total		1000	100.0		

Sieve Analysis of station 5+540					
Mass in g		1000			
Sieve No	Sieve size in (mm)	Mass of retained (g)	Percentage retained (%)	Percentage cumulative retained (%)	percentage of finer particle
3/8''	9.5	0	0.00	0.00	100.0
4	4.75	5.7	0.57	0.57	99.4
10	2.000	30.12	3.01	3.58	96.4
20	0.850	14.7	1.47	5.05	94.9
40	0.425	12.6	1.26	6.31	93.7
60	0.250	8.9	0.89	7.20	92.8
140	0.106	7.4	0.74	7.94	92.1
200	0.075	9.2	0.92	8.86	91.1
	PAN	911.38	91.14	100.00	0.0
Total		1000	100.0		

Sieve Analysis of station 10+435					
Mass in g		1000			
Sieve No	Sieve size in (mm)	Mass of retained (g)	Percentage retained (%)	Percentage cumulative retained (%)	percentage of finer particle
3/8''	9.5	0	0.00	0.00	100.0
4	4.75	4.6	0.46	0.46	99.5
10	2.000	19.5	1.95	2.41	97.6
20	0.850	23.8	2.38	4.79	95.2
40	0.425	16.4	1.64	6.43	93.6
60	0.250	8.1	0.81	7.24	92.8
140	0.106	8.2	0.82	8.06	91.9
200	0.075	9.4	0.94	9.00	91.0
	PAN	910	91.00	100.00	0.0
Total		1000	100.0		

3) Hydrometer Analysis of subgrade soil

Station 0+540

Description of soil Fat Clay								Sample No 1				
Location 0+540								Dry weight soil (Ws)=50gram				
Spesfic Gravity (Gs) =2.65			Hydrometer Type ASTM152-H						Temperature of Test, T =20-23 degree			
		Meniscus Correction +1			Zero Correction -6							
Tested by Ashenafi A.									Date --			
HYDROMETER ANALYSIS FOR CALY AND SILT MATERIALS												
Time	Elapsed time(minute)	Temperature (0c)	Actual Hydrometer reading(RA)	Hydrometer correction for Miniscus	Effective depth (table)	K value from table	D(mm)	Ct value from table	Factors a from table	corrected Hydrometer reading (RC)	% Finer (P)	Corrected (PA)
3:00	1	22	51	52	7.9	0.01332	0.037	0.4	1	45.4	90.8	83.63
3:02	2	22	48	49	8.4	0.01332	0.027	0.4	1	42.4	84.8	78.10
3:06	4	22	47	48	8.6	0.01332	0.02	0.4	1	41.4	82.8	76.26
3:14	8	22	46	47	8.8	0.01332	0.014	0.4	1	40.4	80.8	74.42
3:29	15	22	45	46	8.9	0.01332	0.01	0.4	1	39.4	78.8	72.57
3:59	30	22	44	45	9.1	0.01332	0.0073	0.4	1	38.4	76.8	70.73
4:59	60	23	43	44	9.2	0.01317	0.0052	0.7	1	37.7	75.4	69.44
6:59	120	23	42	43	9.4	0.01317	0.0037	0.7	1	36.7	73.4	67.60
8:59	240	23	40	41	9.7	0.01317	0.0027	0.7	1	34.7	69.4	63.92
16:59	480	23	38	39	10.1	0.01317	0.002	0.7	1	32.7	65.4	60.23
	960	20	34	35	10.7	0.01365	0.0014	0	1	28	56	51.58
	1440	21	32	33	11.1	0.01348	0.0011	0.2	1	26.2	52.4	48.26

Station 1+780

Description of soil Fat Clay								Sample No 1				
Location 1+780								Dry weight soil (Ws)=50gram				
Spesfic Gravity (Gs) =2.65			Hydrometer Type ASTM152-H						Temperature of Test, T =20-23 degree			
		Meniscus Correction +1			Zero Correction -6							
Tested by Ashenafi A.									Date --			
HYDROMETER ANALYSIS FOR CALY AND SILT MATERIALS												
Time	Elapsed time(minute)	Temperature(0c)	Actual Hydrometer reading(RA)	Hydrometer correction for Miniscus	Effective depth (table)	K value from table	D(mm)	Ct value from table	Factors a from table	Hydrometer reading (RC)	% Finer (P)	Corrected (PA)
3:00	1	23	51	52	8.1	0.01297	0.037	0.7	0.99	45.7	90.49	83.07
3:02	2	23	51	52	8.1	0.01297	0.026	0.7	0.99	45.7	90.49	83.07
3:06	4	23	50	51	8.3	0.01297	0.019	0.7	0.99	44.7	88.51	81.25
3:14	8	23	49	50	8.4	0.01297	0.0133	0.7	0.99	43.7	86.53	79.43
3:29	15	23	48	49	8.6	0.01297	0.0098	0.7	0.99	42.7	84.55	77.61
3:59	30	23	47	48	8.8	0.01297	0.007	0.7	0.99	41.7	82.57	75.80
4:59	60	23	46	47	8.9	0.01297	0.005	0.7	0.99	40.7	80.59	73.98
6:59	120	24	45	46	9.1	0.01282	0.0035	1	0.99	40	79.20	72.71
8:59	240	24	44	45	9.2	0.01282	0.0025	1	0.99	39	77.22	70.89
16:59	480	25	42	43	9.6	0.01267	0.0018	1.3	0.99	37.3	73.85	67.80
	960	25	41	42	9.7	0.01267	0.0013	1.3	0.99	36.3	71.87	65.98
	1440	23	39	40	10.1	0.01297	0.0011	0.7	0.99	33.7	66.73	61.25

Station 3+975

Description of soil Fat Clay		Sample No 1	
Location 3+975		Dry weight soil (Ws)=50gram	
Spesfic Gravity (Gs) =2.65	Hydrometer Type ASTM152-H	Temperature of Test, T =20-23 degree	
Meniscus Correction +1	Zero Correction -6		
Tested by Ashenafi A.		Date ---	

HYDROMETER ANALYSIS FOR CALY AND SILT MATERIALS

Time	Elapsed time(minute)	Temperature (0c)	Actual Hydrometer reading(RA)	Hydrometer correction for Miniscus	Effective depth (table)	K value from table	D(mm)	Ct value from table	Factors a from table	Hydrometer reading (RC)	% Finer (P)	Corrected (PA)
3:00	1	22	50	51	8.1	0.01312	0.037	0.4	0.99	44.4	87.91	80.70
3:02	2	22	49	50	8.3	0.01312	0.027	0.4	0.99	43.4	85.93	78.89
3:06	4	22	48	49	8.4	0.01312	0.019	0.4	0.99	42.4	83.95	77.07
3:14	8	22	47	48	8.6	0.01312	0.014	0.4	0.99	41.4	81.97	75.25
3:29	15	22	46	47	8.8	0.01312	0.01	0.4	0.99	40.4	79.99	73.43
3:59	30	22	45	46	8.9	0.01312	0.007	0.4	0.99	39.4	78.01	71.62
4:59	60	22	44	45	9.1	0.01312	0.005	0.4	0.99	38.4	76.03	69.80
6:59	120	23	42	43	9.4	0.01297	0.0036	0.7	0.99	36.7	72.67	66.71
8:59	240	23	40	41	9.7	0.01297	0.0026	0.7	0.99	34.7	68.71	63.07
16:59	480	22	38	39	10.1	0.01312	0.0019	0.4	0.99	32.4	64.15	58.89
	960	22	36	37	10.4	0.01312	0.0014	0.4	0.99	30.4	60.19	55.26
	1440	22	32	33	11.1	0.01312	0.0011	0.4	0.99	26.4	52.27	47.99

Station 5+540

Description of soil Fat Clay		Sample No 4	
Location 5+540		Dry weight soil (Ws)=50gram	
Spesfic Gravity (Gs) =2.65	Hydrometer Type ASTM152-H	Temperature of Test, T =20-23 degree	
Meniscus Correction +1	Zero Correction -6		
Tested by Ashenafi A.		Date --	

HYDROMETER ANALYSIS FOR CALY AND SILT MATERIALS

Time	Elapsed time(minute)	Temperature (0c)	Actual Hydrometer reading(RA)	Hydrometer correction for Miniscus	Effective depth (table)	K value from table	D(mm)	Ct value from table	Factors a from table	Hydrometer reading (RC)	% Finer (P)	Corrected (PA)
3:00	1	24	49	50	8.3	0.01264	0.036	1	0.98	44	86.24	79.17
3:02	2	24	48	49	8.4	0.01264	0.026	1	0.98	43	84.28	77.37
3:06	4	24	47	48	8.6	0.01264	0.019	1	0.98	42	82.32	75.57
3:14	8	24	46	47	8.8	0.01264	0.013	1	0.98	41	80.36	73.77
3:29	15	24	45	46	8.9	0.01264	0.0097	1	0.98	40	78.40	71.97
3:59	30	24	43	44	9.2	0.01264	0.007	1	0.98	38	74.48	68.37
4:59	60	24	41	42	9.6	0.01264	0.0051	1	0.98	36	70.56	64.77
6:59	120	24	38	39	10.1	0.01264	0.0037	1	0.98	33	64.68	59.38
8:59	240	25	36	37	10.4	0.01249	0.0026	1.3	0.98	31.3	61.35	56.32
16:59	480	25	34	35	10.7	0.01249	0.0019	1.3	0.98	29.3	57.43	52.72
	960	24	32	33	11.1	0.01264	0.0014	1	0.98	27	52.92	48.58
	1440	23	30	31	11.4	0.01279	0.0011	0.7	0.98	24.7	48.41	44.44

Station 10+435

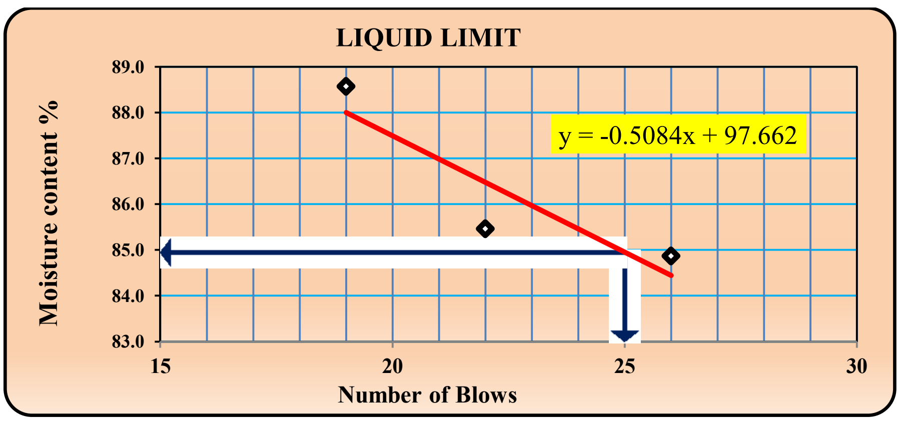
Description of soil Fat Clay							Sample No 5					
Location10+435							Dry weight soil (Ws)=50gram					
Spesfic Gravity (Gs) =2.65			Hydrometer Type ASTM152-H					Temperature of Test, T =20-23 degree				
		Meniscus Correction +1			Zero Correction -6							
Tested by Ashenafi A.								Date ---				
HYDROMETER ANALYSIS FOR CALY AND SILT MATERIALS												
Time	Elapsed time(minute)	Temperature (0c)	Actual Hydrometer reading(RA)	Hydrometer correction for Miniscus	Effective depth (table)	K value from table	D(mm)	Ct value from table	Factors a from table	Hydrometer Reading (RC)	% Finer (P)	Corrected (PA)
3:00	1	25	50	51	8.1	0.01267	0.036	1.3	0.99	45.3	89.69	82.34
3:02	2	25	50	51	8.1	0.01267	0.026	1.3	0.99	45.3	89.69	82.34
3:06	4	25	49	50	8.3	0.01267	0.018	1.3	0.99	44.3	87.71	80.52
3:14	8	25	49	50	8.3	0.01267	0.013	1.3	0.99	44.3	87.71	80.52
3:29	15	25	48	49	8.4	0.01267	0.0095	1.3	0.99	43.3	85.73	78.70
3:59	30	25	47	48	8.6	0.01267	0.0068	1.3	0.99	42.3	83.75	76.89
4:59	60	25	45	46	8.9	0.01267	0.0049	1.3	0.99	40.3	79.79	73.25
6:59	120	25	44	45	9.1	0.01267	0.0035	1.3	0.99	39.3	77.81	71.43
8:59	240	25	42	43	9.4	0.01267	0.0025	1.3	0.99	37.3	73.85	67.80
16:59	480	25	40	41	9.7	0.01267	0.0018	1.3	0.99	35.3	69.89	64.16
	960	25	38	39	10.1	0.01267	0.0013	1.3	0.99	33.3	65.93	60.53
	1440	25	36	37	10.4	0.01267	0.0011	1.3	0.99	31.3	61.97	56.89

4) Sieve analysis and hydrometer analysis combined

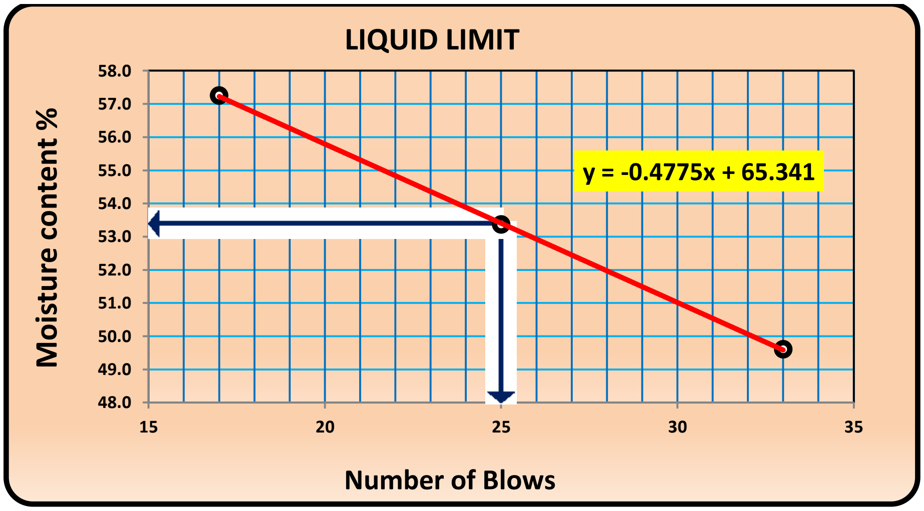
Stations	0+540		1+780		3+975		5+540		10+435	
Grain Size (mm)	Passing (%)	Combined passing (%)	Passing (%)	Combined passing (%)	Passing (%)	Combined passing (%)	Passing (%)	Combined passing (%)	Passing (%)	Combined passing (%)
9.5	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100
4.75	99.7	99.7	99.6	99.6	99.4	99.4	99.4	99.4	99.5	99.5
2	97.2	97.2	97.0	97.0	96.7	96.7	96.4	96.4	97.6	97.6
0.85	95.4	95.4	95.1	95.1	94.7	94.7	94.9	94.9	95.2	95.2
0.425	94.0	94.0	93.8	93.8	93.4	93.4	93.7	93.7	93.6	93.6
0.25	93.3	93.3	93.0	93.0	92.8	92.8	92.8	92.8	92.8	92.8
0.106	92.8	92.8	92.4	92.4	92.3	92.3	92.1	92.1	91.9	91.9
0.075	92.1	92.1	91.8	91.8	91.5	91.5	91.1	91.1	91.0	91.0
0.037	90.8	83.63	90.49	83.07	87.91	80.70	86.24	79.17	89.69	82.34
0.027	84.8	78.10	90.49	83.07	85.93	78.89	84.28	77.37	89.69	82.34
0.02	82.8	76.26	88.51	81.25	83.95	77.07	82.32	75.57	87.71	80.52
0.014	80.8	74.42	86.53	79.43	81.97	75.25	80.36	73.77	87.71	80.52
0.01	78.8	72.57	84.55	77.61	79.99	73.43	78.40	71.97	85.73	78.70
0.0073	76.8	70.73	100.0	100.0	100.0	100.0	74.48	68.37	83.75	76.89
0.0052	75.4	69.44	99.6	99.6	99.4	99.4	70.56	64.77	79.79	73.25
0.0037	73.4	67.60	97.0	97.0	96.7	96.7	64.68	59.38	77.81	71.43
0.0027	69.4	63.92	95.1	95.1	94.7	94.7	61.35	56.32	73.85	67.80
0.002	65.4	60.23	93.8	93.8	93.4	93.4	57.43	52.72	69.89	64.16
0.0014	56	51.58	93.0	93.0	92.8	92.8	52.92	48.58	65.93	60.53
0.0011	52.4	48.26	92.4	92.4	92.3	92.3	48.41	44.44	61.97	56.89

5) Atterberg Test Results for subgrade soil

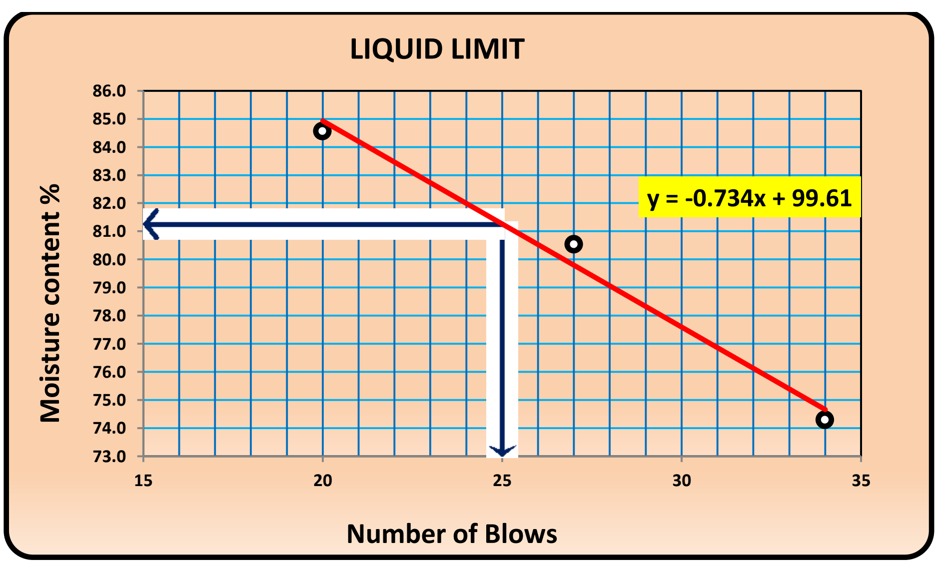
Determination	Subgrade Soil Station 0+540					
	Liquid Limit (AASHTO T 89-96)			Plastic Limit (AASHTO T 90-96)		
Number of blows	19	22	26			
Container No.	B1	D4	A3	A	B	C
Wt. of container + wet soil, (g)	19.7	18.5	26.2	16.21	16.38	16.70
Wt. of container + dry soil, (g)	13.03	12.5	17.95	13.53	13.68	14.02
Wt. of Container, (g)	5.5	5.48	8.23	6.2	6.39	6.58
Wt. of water, (g)	6.67	6.00	8.25	2.68	2.70	2.68
Wt. of dry soil, (g)	7.53	7.02	9.72	7.33	7.29	7.44
Moisture content, (%)	88.58	85.47	84.88	36.56	37.04	36.02
Average	**84.95**			**36.54**		
plastic index (PI)	**48.41**					



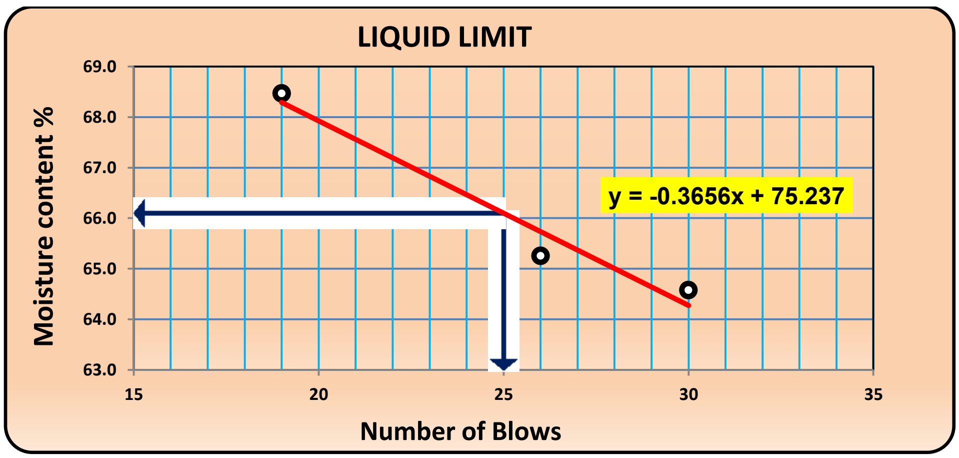
Determination	Subgrade Soil Station 1+780					
	Liquid Limit (AASHTO T 89-96)			Plastic Limit (AASHTO T 90-96)		
Number of blows	19	22	26			
Container No.	B1	D4	A3	A	B	C
Wt. of container + wet soil, (g)	19.7	18.5	26.2	16.21	16.38	16.70
Wt. of container + dry soil, (g)	13.03	12.5	17.95	13.53	13.68	14.02
Wt. of Container, (g)	5.5	5.48	8.23	6.2	6.39	6.58
Wt. of water, (g)	6.67	6.00	8.25	2.68	2.70	2.68
Wt. of dry soil, (g)	7.53	7.02	9.72	7.33	7.29	7.44
Moisture content, (%)	88.58	85.47	84.88	36.56	37.04	36.02
Average	**84.95**			**36.54**		
plastic index (PI)	**48.41**					



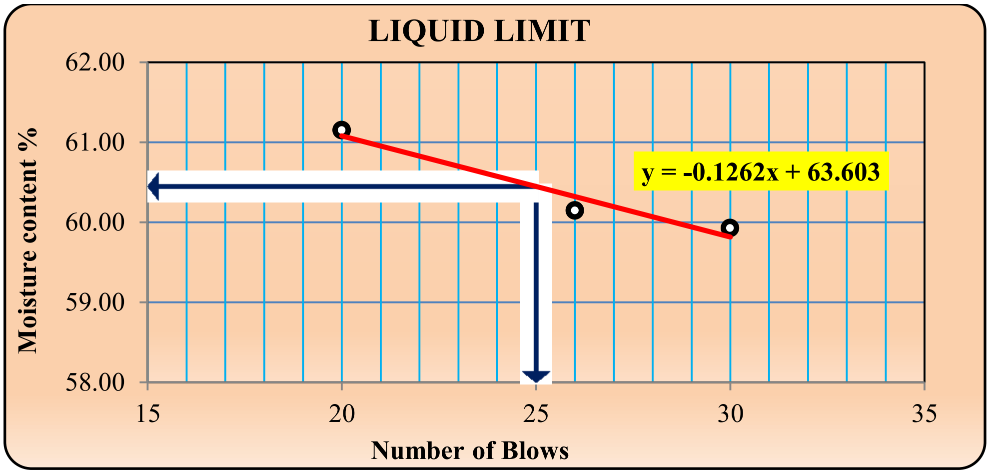
Determination	Subgrade Soil Station 3+975					
	Liquid Limit (AASHTO T 89-96)			Plastic Limit (AASHTO T 90-96)		
Number of blows	19	22	26			
Container No.	B1	D4	A3	A	B	C
Wt. of container + wet soil, (g)	19.7	18.5	26.2	16.21	16.38	16.70
Wt. of container + dry soil, (g)	13.03	12.5	17.95	13.53	13.68	14.02
Wt. of Container, (g)	5.5	5.48	8.23	6.2	6.39	6.58
Wt. of water, (g)	6.67	6.00	8.25	2.68	2.70	2.68
Wt. of dry soil, (g)	7.53	7.02	9.72	7.33	7.29	7.44
Moisture content, (%)	88.58	85.47	84.88	36.56	37.04	36.02
Average	**84.95**			**36.54**		
plastic index (PI)	**48.41**					



Determination	Subgrade Soil Station 5+540					
	Liquid Limit (AASHTO T 89-96)			Plastic Limit (AASHTO T 90-96)		
Number of blows	19	22	26			
Container No.	B1	D4	A3	A	B	C
Wt. of container + wet soil, (g)	19.7	18.5	26.2	16.21	16.38	16.70
Wt. of container + dry soil, (g)	13.03	12.5	17.95	13.53	13.68	14.02
Wt. of Container, (g)	5.5	5.48	8.23	6.2	6.39	6.58
Wt. of water, (g)	6.67	6.00	8.25	2.68	2.70	2.68
Wt. of dry soil, (g)	7.53	7.02	9.72	7.33	7.29	7.44
Moisture content, (%)	88.58	85.47	84.88	36.56	37.04	36.02
Average	**84.95**			**36.54**		
plastic index (PI)	**48.41**					

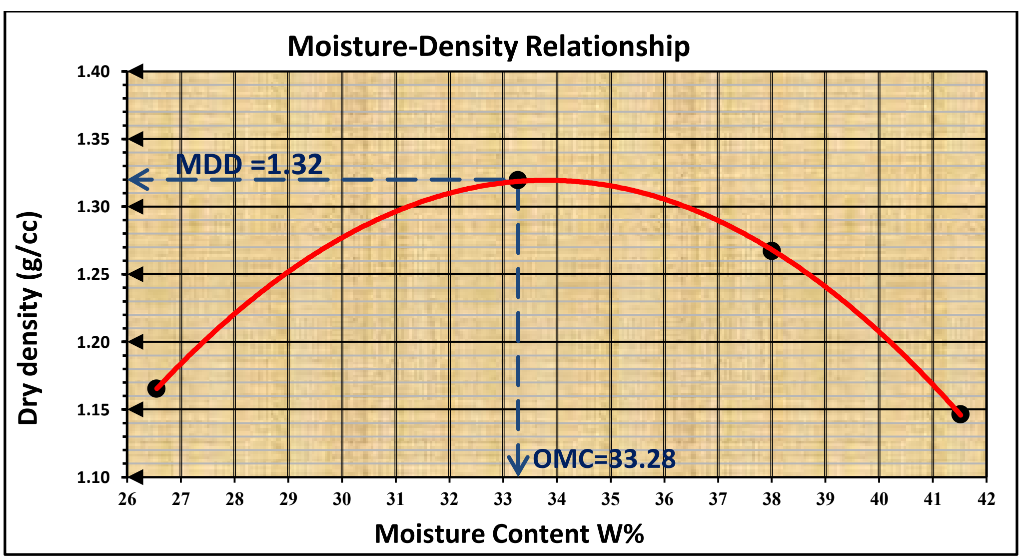


Determination	Subgrade Soil Station 10+435					
	Liquid Limit (AASHTO T 89-96)			Plastic Limit (AASHTO T 90-96)		
Number of blows	19	22	26			
Container No.	B1	D4	A3	A	B	C
Wt. of container + wet soil, (g)	19.7	18.5	26.2	16.21	16.38	16.70
Wt. of container + dry soil, (g)	13.03	12.5	17.95	13.53	13.68	14.02
Wt. of Container, (g)	5.5	5.48	8.23	6.2	6.39	6.58
Wt. of water, (g)	6.67	6.00	8.25	2.68	2.70	2.68
Wt. of dry soil, (g)	7.53	7.02	9.72	7.33	7.29	7.44
Moisture content, (%)	88.58	85.47	84.88	36.56	37.04	36.02
Average	**84.95**			**36.54**		
plastic index (PI)	**48.41**					

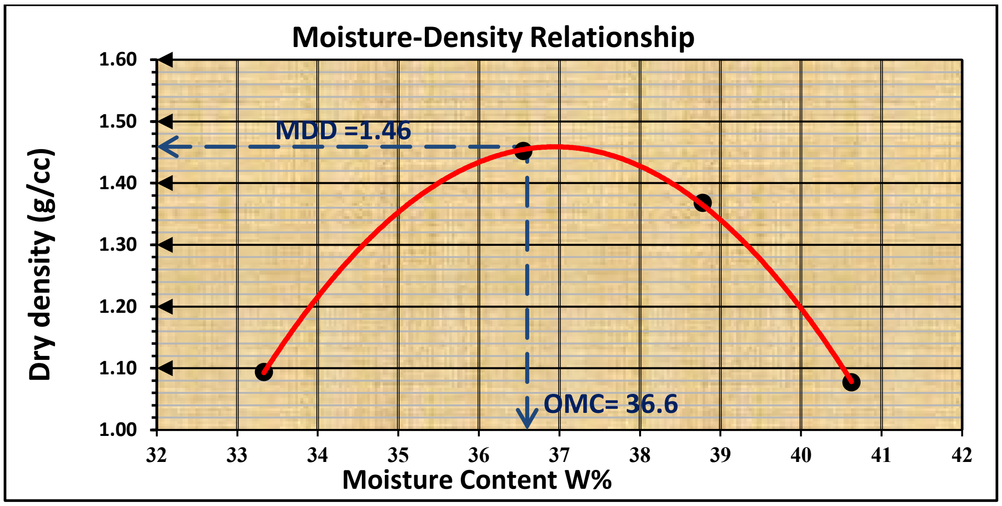


6) Compaction laboratory Result for subgrade soil

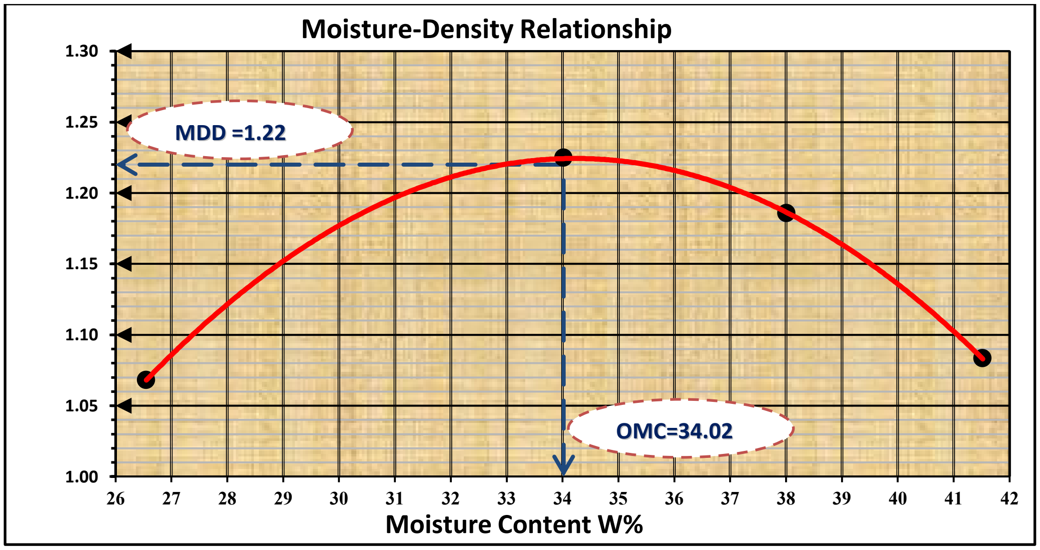
Sampled date:-				Type of Material :- subgrade soil			
Tested date:- .... Sample Ref.				Sampling Station:-		0+540	
Description:-			Sampled by:-Ashenafi A.			Tested by: - Ashenafi A.	
TEST DATA							
Trial no.		1	2	3	4		
Water added %		10	15	20	25		
Mass of wet soil + mould A (g)		4382	4650	4640.5	4521.3		
Mass of mould B (g)		2990	2990	2990	2990		
Mass of wet soil C=A-B (g)		1392	1660	1650.5	1531.3		
VOLUME OF MOULD (CC)		944	944	944	944		
Bulk density C / V = W		1.475	1.758	1.748	1.622		NMC
Moisture determination container No.		A	B	C	D		X-2
Mass of container + wet soil a (g)		76.075	85.11	90.73	90.33		90
mass of container + dry soil b (g)		61.415	65.365	67.315	65.62		78.6
Mass of container d (g)		6.2	6.03	5.71	6.105		17.61
Mass of dry soil b - d = e (g)		55.215	59.335	61.605	59.52		61.0
Mass of moisture a - b = f (g)		14.66	19.75	23.42	24.71		11.4
Moisture content f/e*100 = m (%)		**26.55**	**33.28**	**38.01**	**41.52**		**18.69**
Dry density W / (100+m) *100 (g/cc)		**1.17**	**1.32**	**1.27**	**1.15**		



TEST DATA						
Trial no.			1	2	3	4
Water added %			10	12	14	16
Mass of wet soil + mould	A (g)		5479.3	5974.23	5894.67	5532.48
Mass of mould	B (g)		4102.7	4102.7	4102.7	4102.7
Mass of wet soil	C=A-B (g)		1376.6	1871.53	1791.97	1429.78
VOLUME OF MOULD	(CC)		944	944	944	944
Bulk density	C / V = W		1.458	1.983	1.898	1.515
Moisture determination container No.			A	B	C	D
Mass of container + wet soil	a (g)		83.4	85.6	87.3	89.8
mass of container + dry soil	b (g)		64.1	64.3	64.5	65.62
Mass of container	d (g)		6.2	6.03	5.71	6.105
Mass of dry soil	b - d = e (g)		57.9	58.27	58.79	59.52
Mass of moisture	a - b = f (g)		19.30	21.30	22.80	24.18
Moisture content	f/e*100 = m (%)		**33.33**	**36.55**	**38.78**	**40.63**
Dry density	W / (100+m) *100 (g/cc)		**1.09**	**1.45**	**1.37**	**1.08**

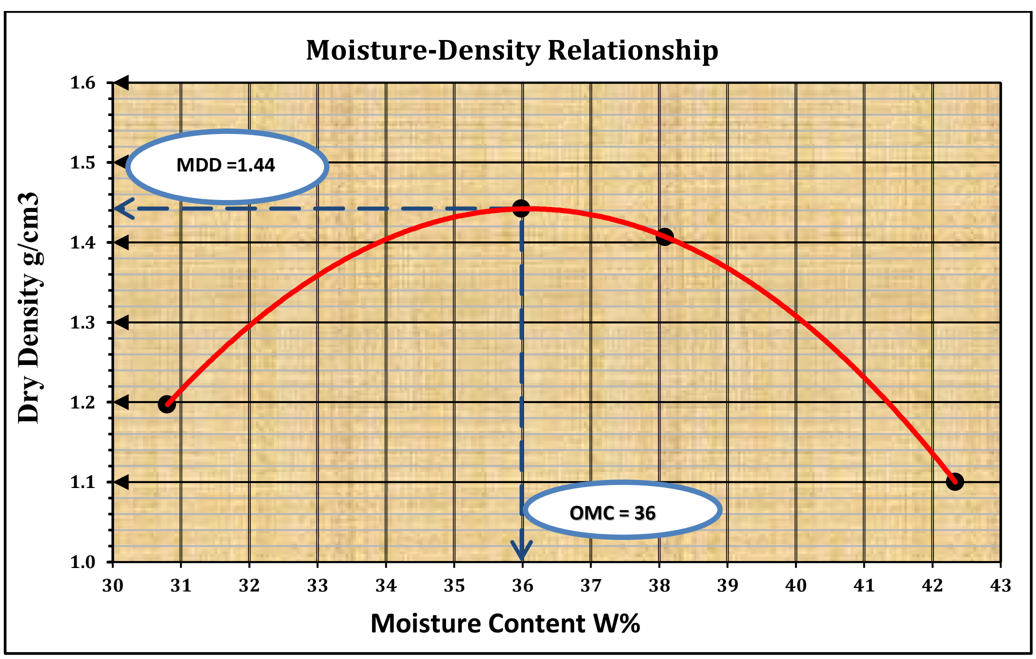


TEST DATA						
Trial no.			1	2	3	4
Water added %			8	12	16	20
Mass of wet soil + mould	A (g)		5321.89	5595.38	5590.75	5493.27
Mass of mould	B (g)		4046	4046	4046	4046
Mass of wet soil	C=A-B (g)		1275.89	1549.38	1544.75	1447.27
VOLUME OF MOULD	(CC)		944	944	944	944
Bulk density	C / V = W		1.352	1.641	1.636	1.533
Moisture determination container No.			A	B	C	D
Mass of container + wet soil	a (g)		76.075	85.55	90.73	90.33
mass of container + dry soil	b (g)		61.415	65.365	67.315	65.62
Mass of container	d (g)		6.2	6.03	5.71	6.105
Mass of dry soil	b - d = e (g)		55.215	59.335	61.605	59.52
Mass of moisture	a - b = f (g)		14.66	20.19	23.42	24.71
Moisture content	f/e*100 = m (%)		**26.55**	**34.02**	**38.01**	**41.52**
Dry density	W / (100+m) *100 (g/cc)		**1.07**	**1.22**	**1.19**	**1.08**

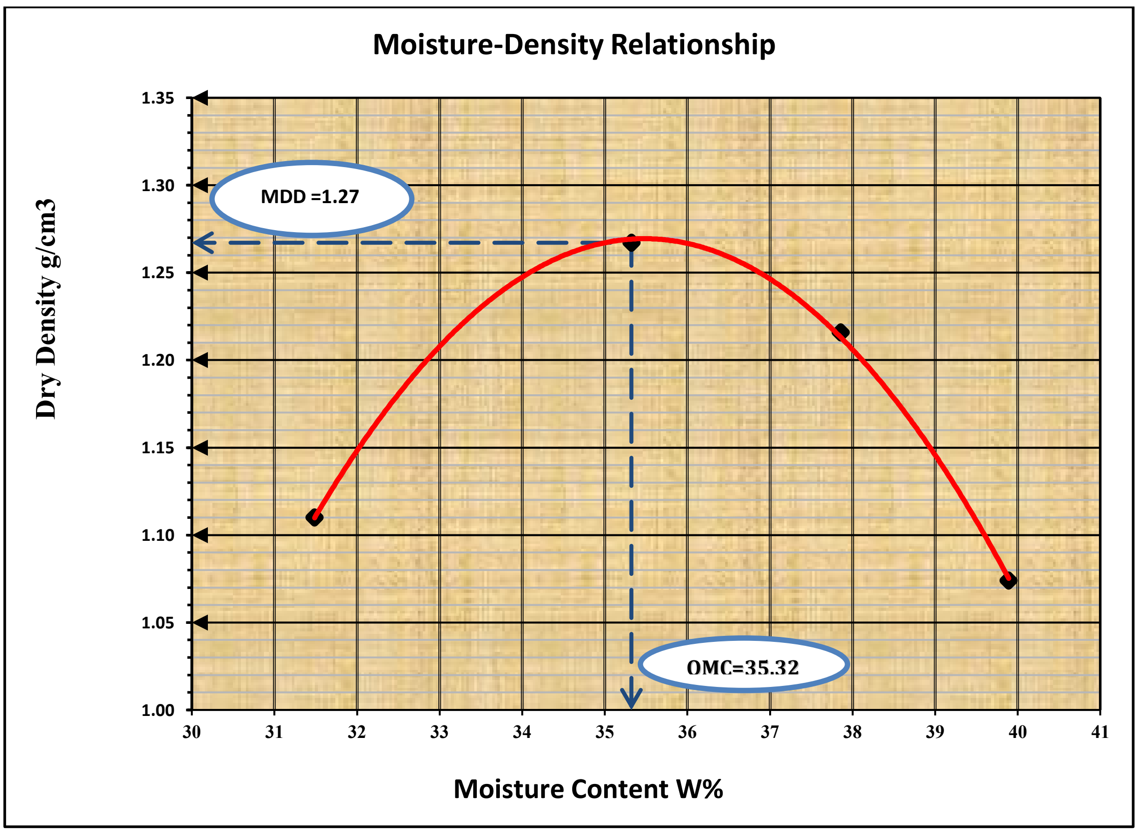


TEST DATA

Trial no.		1	2	3	4
Water added %		9	13	17	21
Mass of wet soil + mould	A (g)	5524	5897.67	5879.54	5524.38
Mass of mould	B (g)	4046	4046	4046	4046
Mass of wet soil	C=A-B (g)	1478	1851.67	1833.54	1478.38
VOLUME OF MOULD	(CC)	944	944	944	944
Bulk density	C / V = W	1.566	1.962	1.942	1.566
Moisture determination container No.		A	B	C	D
Mass of container + wet soil	a (g)	78.35	86.71	89.71	95.2
mass of container + dry soil	b (g)	61.36	65.36	66.54	68.7
Mass of container	d (g)	6.2	6.03	5.71	6.105
Mass of dry soil	b - d = e (g)	55.16	59.33	60.83	62.60
Mass of moisture	a - b = f (g)	16.99	21.35	23.17	26.50
Moisture content	f/e*100 = m (%)	**30.80**	**35.99**	**38.09**	**42.34**
Dry density	W / (100+m) *100 (g/cc)	**1.20**	**1.44**	**1.41**	**1.10**



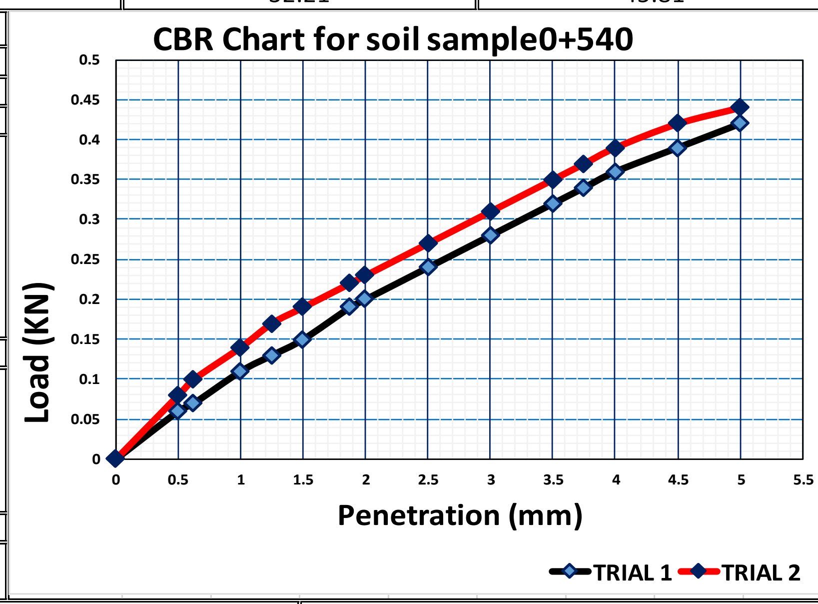
TEST DATA						
Trial no.			1	2	3	4
Water added %			8	12	16	20
Mass of wet soil + mould	A (g)		5423.98	5664.7	5628.37	5464.4
Mass of mould	B (g)		4046	4046	4046	4046
Mass of wet soil	C=A-B (g)		1377.98	1618.7	1582.37	1418.4
VOLUME OF MOULD	(CC)		944	944	944	944
Bulk density	C / V = W		1.460	1.715	1.676	1.503
Moisture determination container No.			A	B	C	D
Mass of container + wet soil	a (g)		83.25	86.10	87.72	89.36
mass of container + dry soil	b (g)		64.80	65.20	65.20	65.62
Mass of container	d (g)		6.20	6.03	5.71	6.11
Mass of dry soil	b - d = e (g)		58.60	59.17	59.49	59.52
Mass of moisture	a - b = f (g)		18.45	20.90	22.52	23.74
Moisture content	f/e*100 = m (%)		**31.48**	**35.32**	**37.86**	**39.89**
Dry density	W / (100+m) *100 (g/cc)		**1.11**	**1.27**	**1.22**	**1.07**



7) CBR Laboratory Result of subgrade soil

Natural Subgrade soil sample station 0+540					
Density of soil from CBR mold(g/cm3)					
Soaking Condition		**56 blows**			
		Before soaking		**After soaking**	
Mold number		**Trial 1**	**Trial 2**	**Trial 1A**	**Trial 2A**
Weight of wet soil + mold(g)	W2	10925.5	10977.5	11112	11254.5
Weight of mold (g)	W1	7055.5	7046	7055.5	7046
Weight of wet soil (g)	W2-W1	3870	3931.5	4056.5	4208.5
Volume of mold (cm3)	V	2250	2250	2250	2250
Wet density of soil (g/cm3)	W2-W1/V	1.72	1.75	1.80	1.87
Dry density of soil (g/cm3)	W2-W1/V(1+w)	1.30	1.32	1.24	1.28
Average dry density (g/cm3)		1.31		1.26	
Moisture Content Determination					
Soaking Condition		**56 blows**			
		Before soaking		**After soaking**	
Container		1	2	A1	A2
Weight of wet soil + Container (g)	W3	125	135.5	96.5	145
Weight of dry soil + Container (g)	W2	108.7	117.6	79.5	110.5
Weight of Container (g)	W1	59	61	40	39.5
Weight of Moisture(g)	W3-W2	16.3	17.9	17	34.5
Weight of dry soil (g)	W2-W1	49.7	56.6	39.5	71
Moisture Content (%)	W3-W2/(W2-W1)	32.80	31.63	43.04	48.59
Average moisture content (%)		32.21		45.81	

CBR Penetration determination				
Penetration after 96 hrs. soaking period				
Pen. (mm)	LOAD (KN) 1	LOAD (KN) 2	CBR (%) 1	CBR (%) 2
0	0	0		
0.5	0.06	0.08		
0.625	0.07	0.1		
1	0.11	0.14		
1.25	0.13	0.17		
1.5	0.15	0.19		
1.875	0.19	0.22		
2	0.2	0.23		
2.5	0.24	0.27	**1.82**	**2.05**
3	0.28	0.31		
3.5	0.32	0.35		
3.75	0.34	0.37		
4	0.36	0.39		
4.5	0.39	0.42		
5	0.42	0.44	**2.1**	**2.2**
10	0.43	0.46		
12.5	0.44	0.515		



Swell Determination				**Proctor Data's**	
No of blows	Gauge rdg. Mm		Swell in %	**OMC = 33.28%**	**MDD = 1.32%**
56	Initial	23.71			
	Final	28.5	**4.12**		
MDD	**1.32 g/cc**				
CBR (%)	**2.20%**				

Natural Subgrade soil sample station 1+780

Density of soil from CBR mold(g/cm3)

Soaking Condition		56 blows			
		Before soaking		After soaking	
Mold number		Trial 1	Trial 2	Trial 1A	Trial 2A
Weight of wet soil + mold(g)	W2	10895.5	10954.5	11215	11254.5
Weight of mold (g)	W1	7048.5	6957.5	7048.5	6957.5
Weight of wet soil (g)	W2-W1	3847	3997	4166.5	4297
Volume of mold (cm3)	V	2250	2250	2250	2250
Wet density of soil (g/cm3)	W2-W1/V	1.71	1.78	1.85	1.91
Dry density of soil (g/cm3)	W2-W1/V(1+w)	1.29	1.34	1.30	1.34
Average dry density (g/cm3)		1.32		1.32	

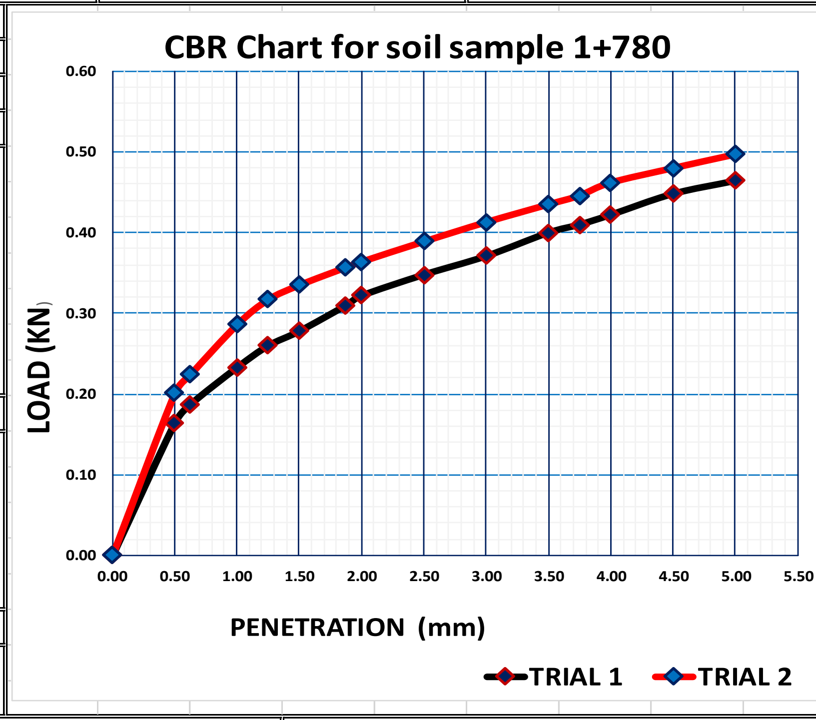
Moisture Content Determination

Soaking Condition		56 blows			
		Before soaking		After soaking	
Container		1	2	A1	A2
Weight of wet soil + Container (g)	W3	134.5	135.5	124	121
Weight of dry soil + Container (g)	W2	115	117.6	97	99
Weight of Container (g)	W1	57.8	59.5	40	39.5
Weight of Moisture(g)	W3-W2	19.5	17.9	27	22
Weight of dry soil (g)	W2-W1	57.2	58.1	57	59.5
Moisture Content (%)	W3-W2/(W2-W1)	34.09	30.81	47.37	36.97
Average moisture content (%)		32.45		42.17	

CBR Penetration determination

Penetration after 96 hrs. soaking period

Pen. (mm)	LOAD (KN) 1	LOAD (KN) 2	CBR (%) 1	CBR (%) 2
0.00	0.00	0.00		
0.50	0.16	0.20		
0.63	0.19	0.22		
1.00	0.23	0.29		
1.25	0.26	0.32		
1.50	0.28	0.34		
1.88	0.31	0.36		
2.00	0.32	0.36		
2.50	0.35	0.39	2.64	2.95
3.00	0.37	0.41		
3.50	0.40	0.44		
3.75	0.41	0.45		
4.00	0.42	0.46		
4.50	0.45	0.48		
5.00	0.46	0.50	2.32	2.49
10.00	0.51	0.57		
12.50	0.63	0.65		



Swell Determination

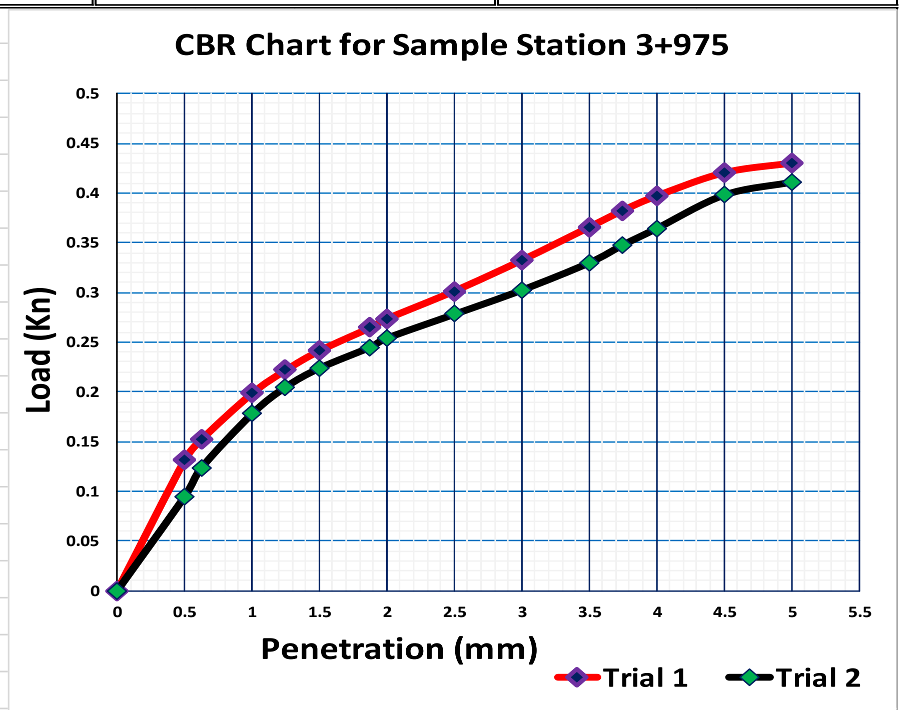
No of blows	Gauge rdg. Mm		Swell in %
56	Initial	25.63	3.25
	Final	29.41	
MDD	1.46g/cc		
CBR (%)	2.95%		

Proctor Data's

OMC =36.6% MDD = 1.46%

Natural Subgrade soil sample station 3+975					
Density of soil from CBR mold(g/cm3)					
Soaking Condition		**56 blows**			
		Before soaking		**After soaking**	
Mold number		**Trial 1**	**Trial 2**	**Trial 1A**	**Trial 2A**
Weight of wet soil + mold(g)	W2	10785	10835.5	11154.5	11215.7
Weight of mold (g)	W1	6959.5	6959	6959.5	6959
Weight of wet soil (g)	W2-W1	3825.5	3876.5	4195	4256.7
Volume of mold (cm3)	V	2250	2250	2250	2250
Wet density of soil (g/cm3)	W2-W1/V	1.70	1.72	1.86	1.89
Dry density of soil (g/cm3)	W2-W1/V(1+w)	1.31	1.32	1.28	1.30
Average dry density (g/cm3)		1.31		1.29	
Moisture Content Determination					
Soaking Condition		**56 blows**			
		Before soaking		**After soaking**	
Container		1	2	A1	A2
Weight of wet soil + Container (g)	W3	117.5	120.5	161	153
Weight of dry soil + Container (g)	W2	99.5	102.5	124	116.5
Weight of Container (g)	W1	40.5	42.5	39	40.5
Weight of Moisture(g)	W3-W2	18	18	37	36.5
Weight of dry soil (g)	W2-W1	59	60	85	76
Moisture Content (%)	W3-W2/(W2-W1)	30.51	30.00	43.53	48.03
Average moisture content (%)		30.25		45.78	

CBR Penetration determination				
Penetration after 96 hrs. soaking period				
Pen. (mm	OAD (KN)	OAD (KN)	CBR (%) 1	CBR (%) 2
0	0	0		
0.5	0.131	0.0948		
0.625	0.152	0.1237		
1	0.1984	0.178		
1.25	0.2218	0.205		
1.5	0.2413	0.224		
1.875	0.2645	0.245		
2	0.2735	0.2542		
2.5	0.3014	0.2782	**2.28**	**2.11**
3	0.3324	0.3025		
3.5	0.3657	0.33		
3.75	0.3825	0.348		
4	0.397	0.3645		
4.5	0.4203	0.3981		
5	0.43	0.411	**2.15**	**2.06**
10	0.551	0.5432		
12.5	0.6724	0.5968		



Swell Determination			
No of blows	Gauge rdg. Mm		Swell in %
56	Initial	30.64	
	Final	35.25	**3.96**
MDD	**1.22g/cc**		
CBR (%)	**2.28%**		

Proctor Data's	
OMC = 34.02%	**MDD = 1.22%**

Natural Subgrade soil sample station 5+540

Density of soil from CBR mold(g/cm3)

Soaking Condition		56 blows			
		Before soaking		After soaking	
Mold number		Trial 1	Trial 2	Trial 1A	Trial 2A
Weight of wet soil + mold(g)	W2	10957.5	10987.5	11135.5	11248
Weight of mold (g)	W1	7049	7026.5	7049	7026.5
Weight of wet soil (g)	W2-W1	3908.5	3961	4086.5	4221.5
Volume of mold (cm3)	V	2250	2250	2250	2250
Wet density of soil (g/cm3)	W2-W1/V	1.74	1.76	1.82	1.88
Dry density of soil (g/cm3)	W2-W1/V(1+w)	1.31	1.33	1.24	1.28
Average dry density (g/cm3)		1.32		1.26	

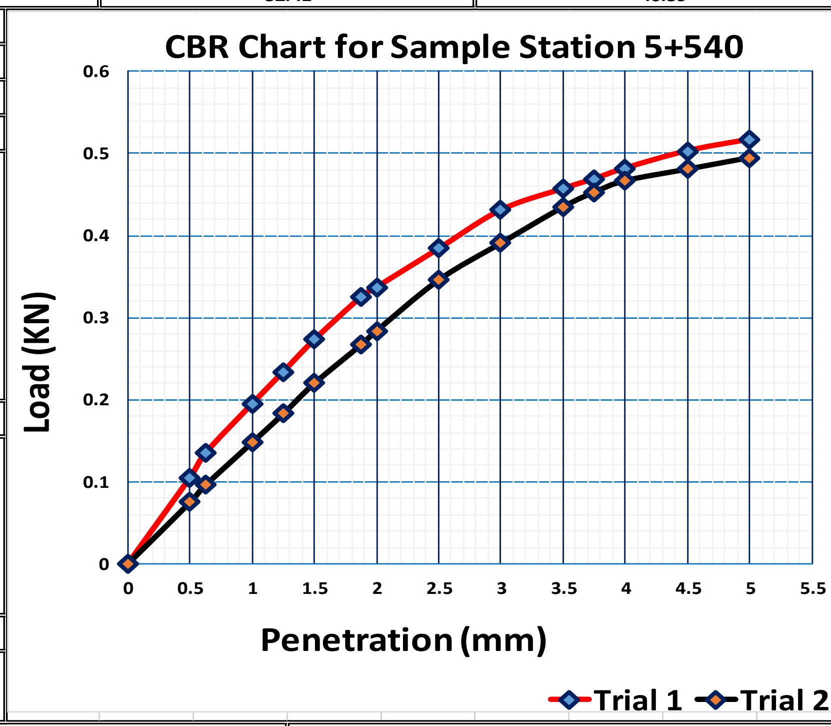
Moisture Content Determination

Soaking Condition		56 blows			
		Before soaking		After soaking	
Container		1	2	A1	A2
Weight of wet soil + Container (g)	W3	137	135.5	124.5	133
Weight of dry soil + Container (g)	W2	113	112.5	98.5	103.5
Weight of Container (g)	W1	39	41.5	40.5	42.5
Weight of Moisture(g)	W3-W2	24	23	26	29.5
Weight of dry soil (g)	W2-W1	74	71	58	61
Moisture Content (%)	W3-W2/(W2-W1)	32.43	32.39	44.83	48.36
Average moisture content (%)		32.41		46.59	

CBR Penetration determination

Penetration after 96 hrs. soaking period

Pen. (mm)	LOAD (KN) 1	LOAD (KN) 2	CBR (%) 1	CBR (%) 2
0	0	0		
0.5	0.105	0.075		
0.625	0.135	0.096		
1	0.195	0.148		
1.25	0.234	0.183		
1.5	0.273	0.22		
1.875	0.325	0.267		
2	0.337	0.284		
2.5	0.384	0.346	2.91	2.62
3	0.431	0.391		
3.5	0.457	0.435		
3.75	0.469	0.453		
4	0.482	0.467		
4.5	0.503	0.481		
5	0.517	0.495	2.585	2.48
10	0.642	0.514		
12.5	0.825	0.725		



Swell Determination

No of blows	Gauge rdg. Mm		Swell in %
56	Initial	27.46	3.35
	Final	31.36	
MDD	1.44 g/cc		
CBR (%)	2.91%		

Proctor Data's

OMC = 36%	MDD = 1.44%

Natural Subgrade soil sample station 10+435

Density of soil from CBR mold(g/cm3)

Soaking Condition		56 blows			
		Before soaking		After soaking	
Mold number		Trial 1	Trial 2	Trial 1A	Trial 2A
Weight of wet soil + mold(g)	W2	10976.5	10898	11246.5	11357.5
Weight of mold (g)	W1	7026.5	7094.5	7026.5	7094.5
Weight of wet soil (g)	W2-W1	3950	3803.5	4220	4263
Volume of mold (cm3)	V	2250	2250	2250	2250
Wet density of soil (g/cm3)	W2-W1/V	1.76	1.69	1.88	1.89
Dry density of soil (g/cm3)	W2-W1/V(1+w)	1.37	1.32	1.27	1.29
Average dry density (g/cm3)		**1.34**		**1.28**	

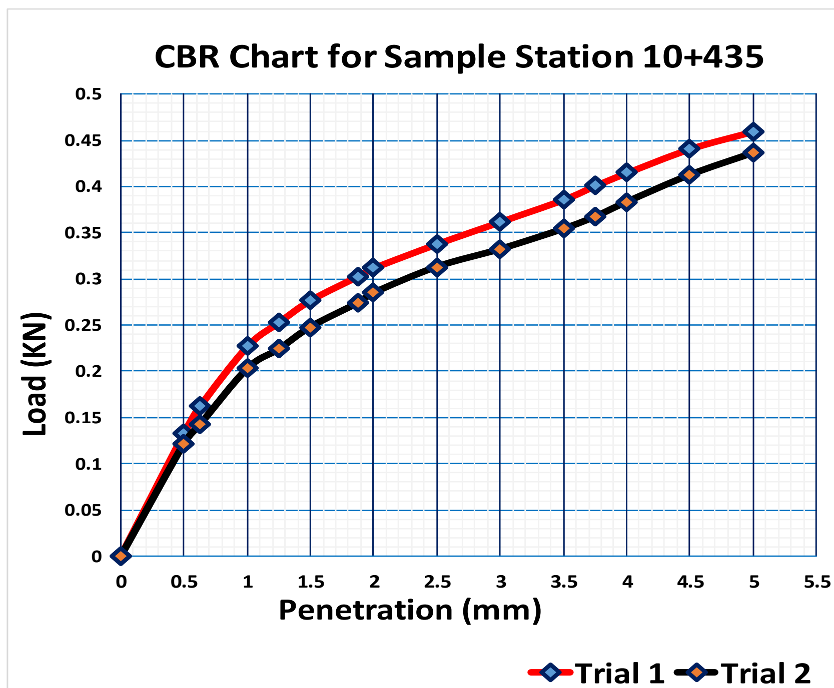
Moisture Content Determination

Soaking Condition		56 blows			
		Before soaking		After soaking	
Container		1	2	A1	A2
Weight of wet soil + Container (g)	W3	117.5	120.5	133.5	145.2
Weight of dry soil + Container (g)	W2	98.7	104.5	104	112
Weight of Container (g)	W1	39.5	39.5	40.5	43
Weight of Moisture(g)	W3-W2	18.8	16	29.5	33.2
Weight of dry soil (g)	W2-W1	59.2	65	63.5	69
Moisture Content (%)	W3-W2/(W2-W1)	31.76	24.62	46.46	48.12
Average moisture content (%)		**28.19**		**47.29**	

CBR Penetration determination

Penetration after 96 hrs. soaking period

Pen. (mm)	LOAD (KN) 1	LOAD (KN) 2	CBR (%) 1	CBR (%) 2
0	0	0		
0.5	0.133	0.122		
0.625	0.162	0.1422		
1	0.227	0.203		
1.25	0.253	0.224		
1.5	0.277	0.248		
1.875	0.303	0.274		
2	0.312	0.2849		
2.5	0.338	0.313	**2.56**	**2.37**
3	0.362	0.3321		
3.5	0.386	0.3541		
3.75	0.401	0.3672		
4	0.415	0.3831		
4.5	0.441	0.4124		
5	0.46	0.4362	**2.30**	**2.18**
10	0.512	0.4821		
12.5	0.6017	0.6009		



Swell Determination

No of blows	Gauge rdg. Mm		Swell in %
56	Initial	25.4	
	Final	30.24	**4.16**
MDD	**1.27g/cc**		
CBR (%)	**2.56%**		

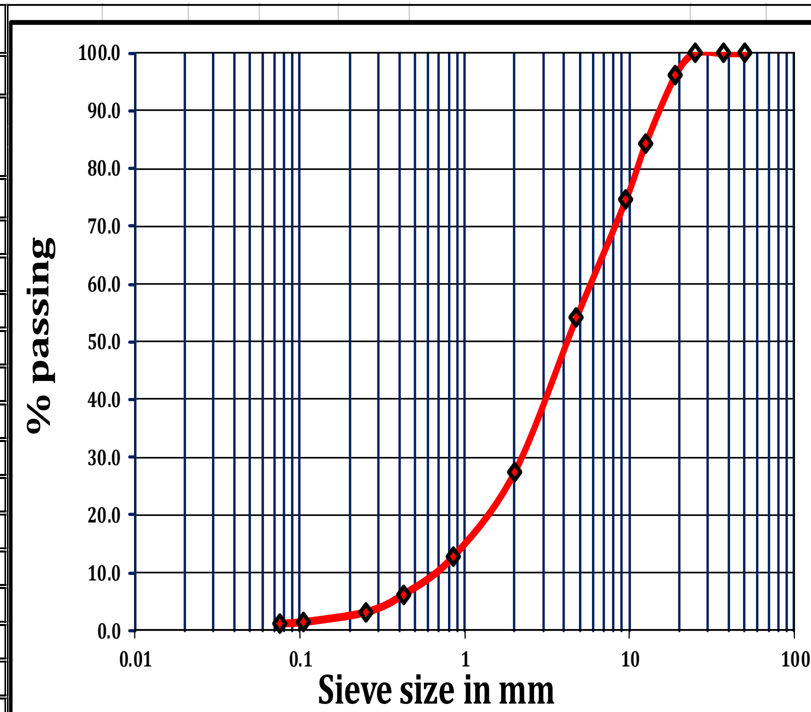
Proctor Data's

OMC =35.32%	**MDD = 1.27%**

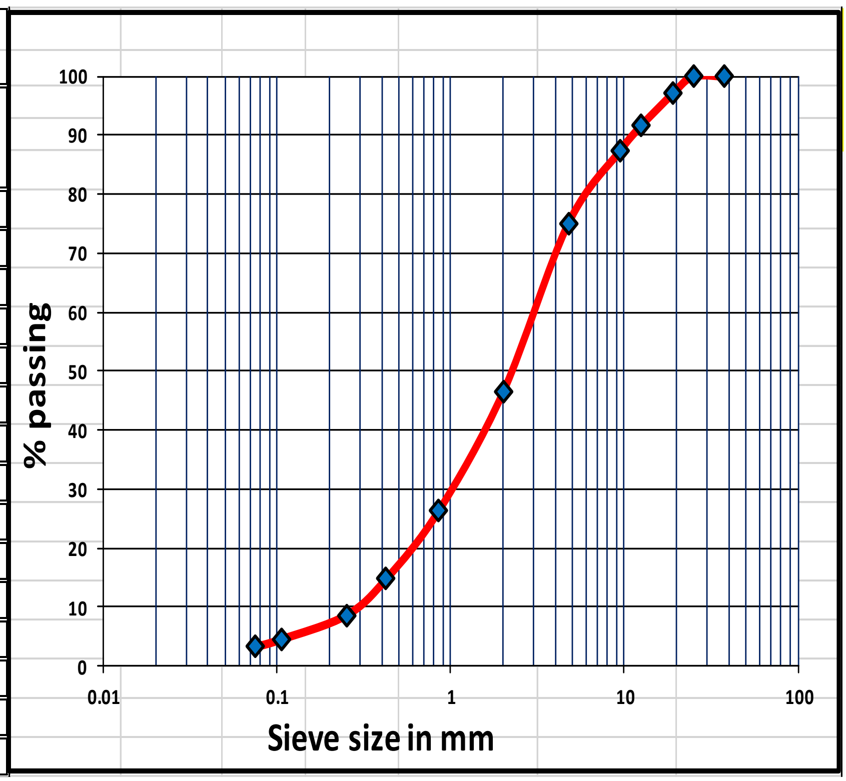
Appendix B: Laboratory Test Results of Gravel materials

1. Particle Size Analysis

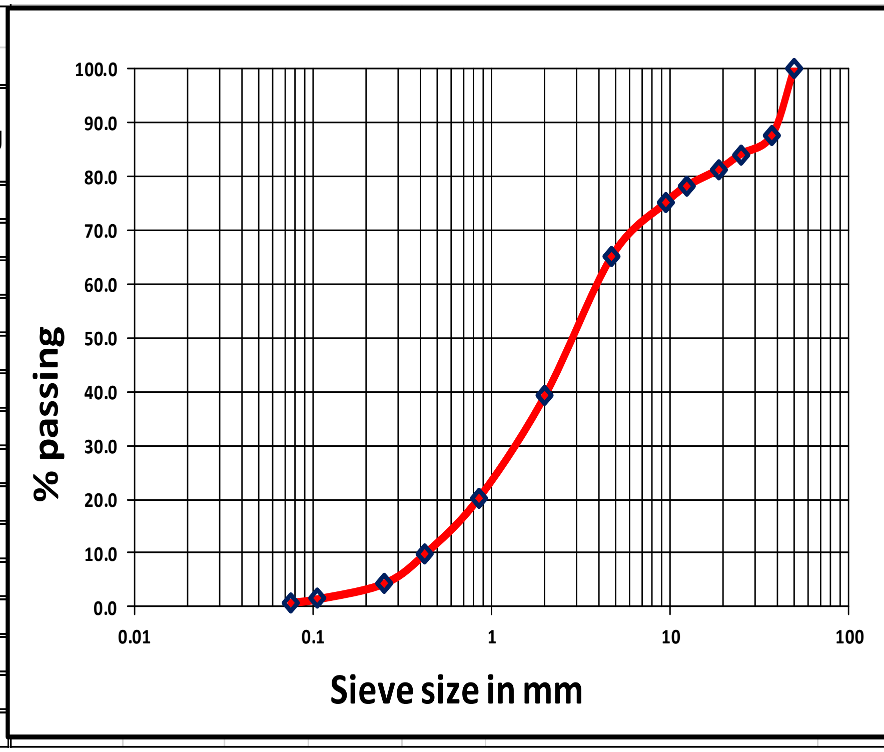
Sieve Analysis station 0+540				
Mass in g	3000.0			
Sieve size in mm	wt.ret	% ret	CU	%pass
50	0	0	0	100.0
37.5	0	0.00	0.00	100.0
25	0	0.00	0.00	100.0
19	114.8	3.83	3.83	96.2
12.5	358.9	11.96	15.79	84.2
9.5	288.6	9.62	25.41	74.6
4.75	615.2	20.51	45.92	54.1
2.000	802.7	26.76	72.67	27.3
0.850	437.2	14.57	87.25	12.8
0.425	197.3	6.58	93.82	6.2
0.250	92.2	3.07	96.90	3.1
0.106	50.8	1.69	98.59	1.4
0.075	8.4	0.28	98.87	1.1
PAN	33.9	1.13	100.00	0.0
Total	**3000**	**100.0**		



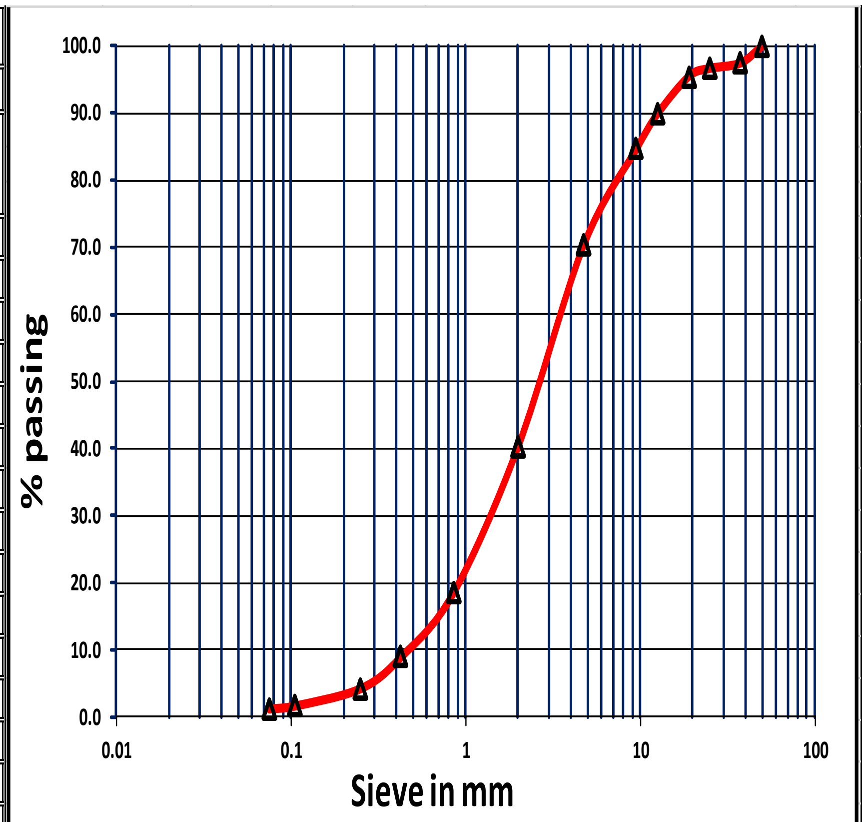
Sieve Analysis station 1+780				
Mass in g	3000.0			
Sieve size in mm	wt.ret	% ret	CU	%passing
50	0			
37.5	0	0.00	0.00	100.0
25	0	0.00	0.00	100.0
19	89.9	3.00	3.00	97.0
12.5	162.4	5.41	8.41	91.6
9.5	124.8	4.16	12.57	87.4
4.75	373.4	12.45	25.02	75.0
2.000	857.5	28.58	53.60	46.4
0.850	603.3	20.11	73.71	26.3
0.425	344	11.47	85.18	14.8
0.250	185.9	6.20	91.37	8.6
0.106	121.8	4.06	95.43	4.6
0.075	39.7	1.32	96.76	3.2
PAN	97.3	3.24	100.00	0.0
Total	3000	**100.0**		



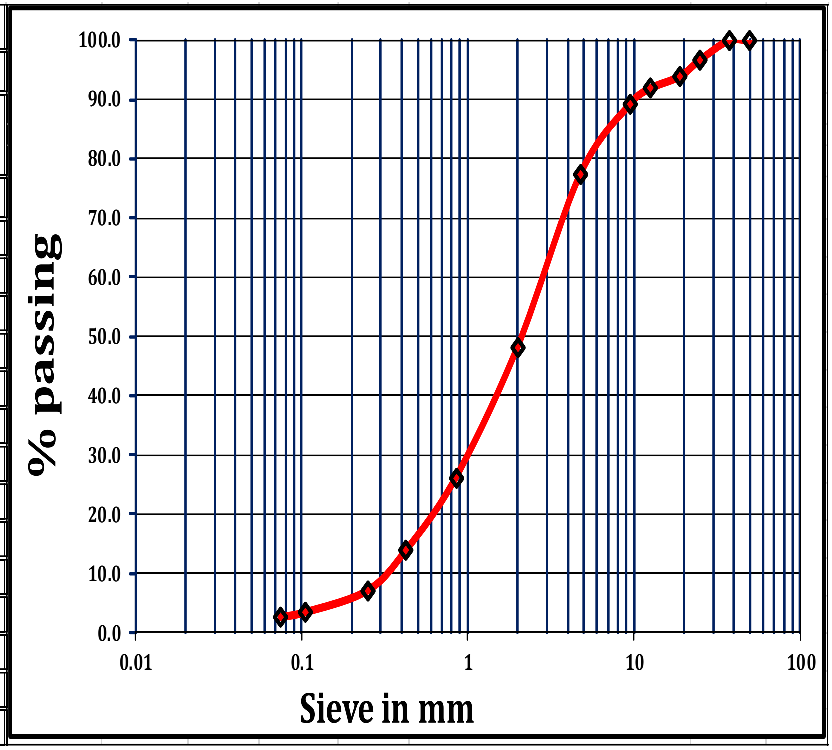
Sieve Analysis station 3+975			
Mass in g	3000.0		
Sieve size in mm	wt.ret	% ret	%passing
50	0	0	100.0
37.5	373	12.43	87.6
25	104.7	3.49	84.1
19	82.4	2.75	81.3
12.5	93.0	3.10	78.2
9.5	94.4	3.15	75.1
4.75	296.9	9.90	65.2
2.000	777.6	25.92	39.3
0.850	571.6	19.05	20.2
0.425	309.6	10.32	9.9
0.250	166	5.53	4.4
0.106	86.5	2.88	1.5
0.075	20.3	0.68	0.8
PAN	24	0.80	0.0
Total	**3000**	**100.0**	



Sieve Analysis station 5+540				
Mass in g	3000			
Sieve size in mm	wt.ret	% ret	CU	%pass
50	0	0	0	100.0
37.5	75.4	2.51	2.51	97.5
25	23.7	0.79	3.30	96.7
19	36.4	1.21	4.52	95.5
12.5	170.0	5.67	10.18	89.8
9.5	155.4	5.18	15.36	84.6
4.75	429.3	14.31	29.67	70.3
2.000	900.8	30.03	59.70	40.3
0.850	656	21.87	81.57	18.4
0.425	286.3	9.54	91.11	8.9
0.250	139.7	4.66	95.77	4.2
0.106	76.9	2.56	98.33	1.7
0.075	13.9	0.46	98.79	1.2
PAN	36.2	1.21	100.00	0.0
Total	3000	**100.0**		

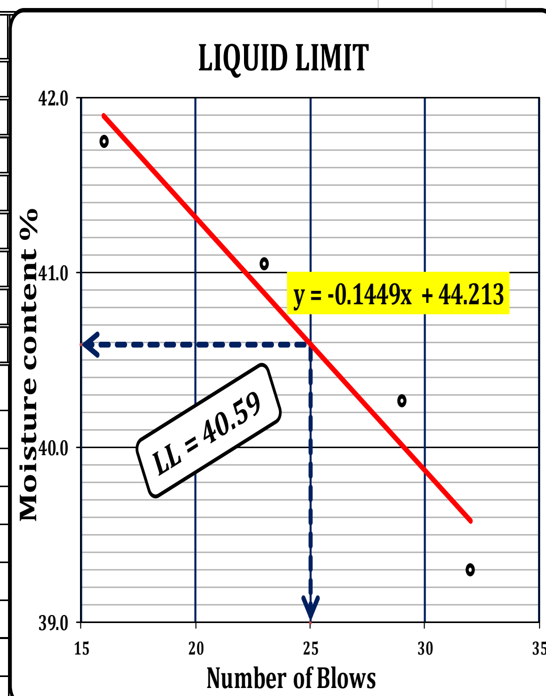


Sieve Analysis station 10+435				
Mass in g	3000.0			
Sieve size in mm	wt.ret	% ret	CU	%pass
50	0	0	0	100.0
37.5	0	0.00	0.00	100.0
25	100.1	3.34	3.34	96.7
19	82.0	2.73	6.07	93.9
12.5	61.8	2.06	8.13	91.9
9.5	78.9	2.63	10.76	89.2
4.75	358.7	11.96	22.72	77.3
2.000	872.5	29.08	51.80	48.2
0.850	661.8	22.06	73.86	26.1
0.425	369.5	12.32	86.18	13.8
0.250	203.2	6.77	92.95	7.1
0.106	109.6	3.65	96.60	3.4
0.075	25	0.83	97.44	2.6
PAN	76.9	2.56	100.00	0.0
Total	**3000**	**100.0**		



2. Atterberg Limit Test Result

DETERMINIATION OF LIQUID LIMIT & PLASTIC LIMIT ASTM D 423/424 AASHTO T89/T90					
Date sampled:-D/M/Y		Sampled by:-Ashenafi A.		Material Type: Gravel	
Test date :-			Location	0+540	
Description :-			Tested by:-Ashenafi A.		`
TEST DATA					
LIQUID LIMIT		1	2	3	4
No. of blows		32	29	23	16
Container No.		A	B	C	D
Mass of wet soil + container (a)	g	90.00	81.00	80.00	85
Mass of dry soil + container (b)	g	76.60	68.80	68.30	72.6
Mass of container (c)	g	42.5	38.50	39.8	42.9
Mass of moisture (a-b)	g	13.40	12.20	11.70	12.40
Mass of dry soil (b-c)	g	34.10	30.30	28.50	29.70
Moisture content (w=a-b/b-c x 100)	%	39.30	40.26	41.05	41.75
PLASTIC LIMIT					
Container No.		A	B	C	
Mass of wet soil + container (a)	g	50.5	52	51	
Mass of dry soil + container (b)	g	47.80	49.30	48.6	
Mass of container (c)	g	39.1	39.8	39.8	
Mass of moisture (a-b)	g	2.70	2.70	2.40	
Mass of dry soil (b-c)	g	8.70	9.50	8.80	
Moisture content (w=a-b/b-c x 100)	g	31.03	28.42	27.27	
Average moisture content (wa)	%	28.91			



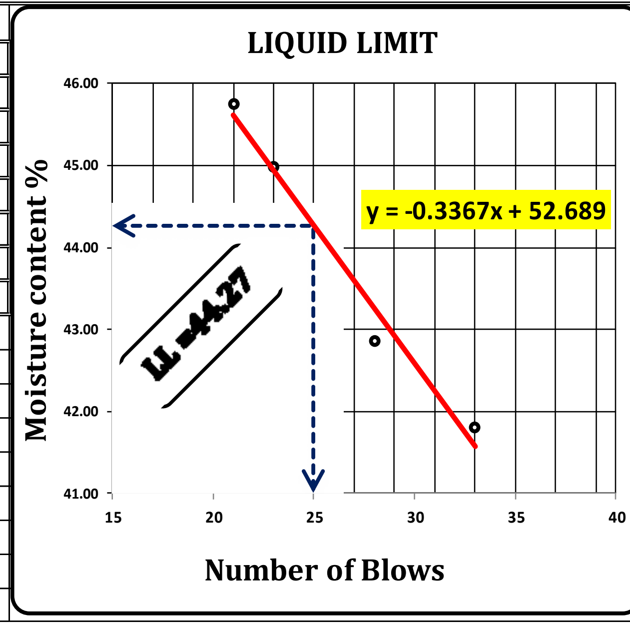
DETERMINIATION OF LIQUID LIMIT & PLASTIC LIMIT ASTM D 423/424 AASHTO T89/T90

Date sampled:-D/M/Y	Sampled by:-Ashenafi A.	Material type: Gravel
Test date :-	Sample Ref.:-	Location km1+780
Description :-	Tested by:-Ashenafi A.	
Remarks :-		

TEST DATA

LIQUID LIMIT				
No. of blows	33	28	23	21
Container No.	A	B	C	D
Mass of wet soil + container (a) g	89.00	78.30	78.50	58
Mass of dry soil + container (b) g	76.50	67.50	66.40	48.3
Mass of container (c) g	46.6	42.30	39.5	27.1
Mass of moisture (a-b) g	12.50	10.80	12.10	9.70
Mass of dry soil (b-c) g	29.90	25.20	26.90	21.20
Moisture content (w=a-b/b-c x 1%	41.81	42.86	44.98	45.75

PLASTIC LIMIT			
Container No.	A	B	C
Mass of wet soil + container (a) g	55.2	57.5	59
Mass of dry soil + container (b) g	52.10	53.80	55.2
Mass of container (c) g	42.1	42.9	40.4
Mass of moisture (a-b) g	3.10	3.70	3.80
Mass of dry soil (b-c) g	10.00	10.90	14.80
Moisture content (w=a-b/b-c x 1g	31.00	33.94	25.68
Average moisture content (wa) %	30.21		

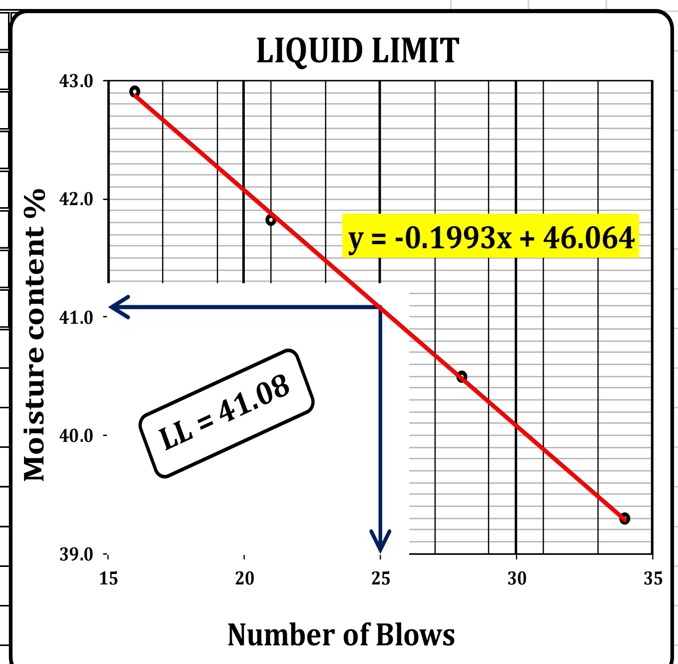


DETERMINIATION OF LIQUID LIMIT & PLASTIC LIMIT ASTM D 423/424 AASHTO T89/T90

Date sampled:	Sampled by:-Ashenafi A.	Material type :Gravel
Test date :-	Sample Ref.:-	Location 3+975
Description :-		Tested by:-Ashenafi A.
Remarks :-		

No. of blows	34	28	21	16
Container No.	A	B	C	D
Mass of wet soil + container g	82.00	87.80	81.70	77
Mass of dry soil + container (g	70.80	74.60	70.70	65.8
Mass of container (c) g	42.3	42.00	44.4	39.7
Mass of moisture (a-b) g	11.20	13.20	11.00	11.20
Mass of dry soil (b-c) g	28.50	32.60	26.30	26.10
Moisture content (w=a-b/b-c %	39.30	40.49	41.83	42.91

PLASTIC LIMIT			
Container No.	A	B	C
Mass of wet soil + container g	55.5	52.5	53.5
Mass of dry soil + container (g	52.50	50.40	50.6
Mass of container (c) g	43.3	40.5	40.8
Mass of moisture (a-b) g	3.00	2.10	2.90
Mass of dry soil (b-c) g	9.20	9.90	9.80
Moisture content (w=a-b/b-c g	32.61	21.21	29.59
Average moisture content (w %	27.80		

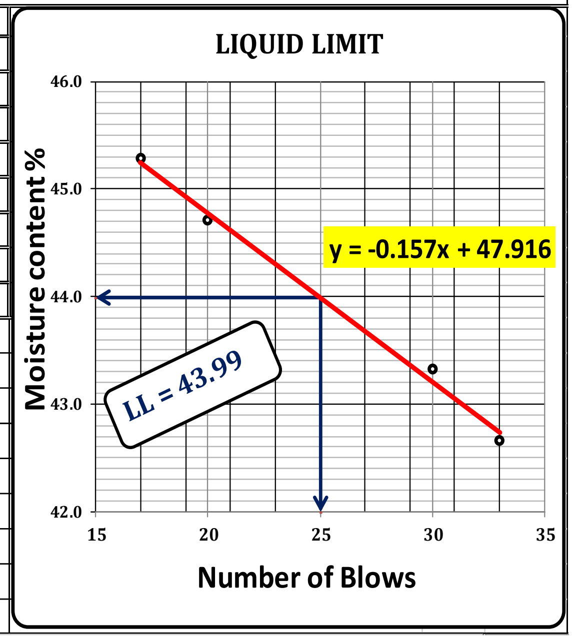


DETERMINIATION OF LIQUID LIMIT & PLASTIC LIMIT ASTM D 423/424 AASHTO T89/T90

Date sampled:-D/M/Y	Sampled by:-Ashenafi A.		
Test date :-		Location	5+540
Description :-		Tested by:-Ashenafi A.	
Remarks :-			

TEST DATA

LIQUID LIMIT		1	2	3	4
No. of blows		33	30	20	17
Container No.		A	B	C	D
Mass of wet soil + container (a)	g	89.60	84.70	88.30	80.4
Mass of dry soil + container (b)	g	74.50	71.40	73.10	68.4
Mass of container (c)	g	39.1	40.70	39.1	41.9
Mass of moisture (a-b)	g	15.10	13.30	15.20	12.00
Mass of dry soil (b-c)	g	35.40	30.70	34.00	26.50
Moisture content (w=a-b/b-c x 100)	%	42.66	43.32	44.71	45.28
PLASTIC LIMIT					
Container No.		**A**	**B**	**C**	
Mass of wet soil + container (a)	g	48	33	49.6	
Mass of dry soil + container (b)	g	46.60	31.50	48	
Mass of container (c)	g	40.5	27.1	42.6	
Mass of moisture (a-b)	g	1.40	1.50	1.60	
Mass of dry soil (b-c)	g	6.10	4.40	5.40	
Moisture content (w=a-b/b-c x 100)	g	22.95	34.09	29.63	
Average moisture content (wa)	%	28.89			

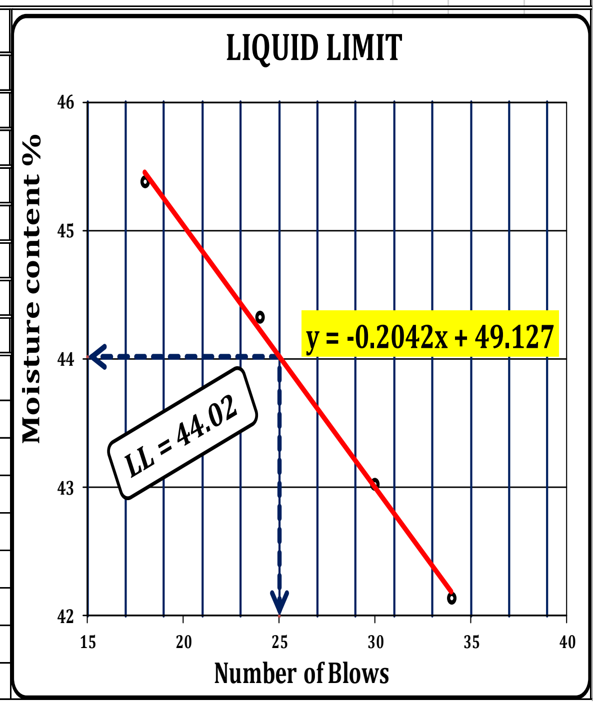


DETERMINIATION OF LIQUID LIMIT & PLASTIC LIMIT ASTM D 423/424 AASHTO T89/T90

Date sampled:-D/M/Y	Sampled by:-Ashenafi A.	Material Type: Gravel	
Test date :-		Location	10+435
Description :-		Tested by:-Ashenafi A.	`

TEST DATA

LIQUID LIMIT		1	2	3	4
No. of blows		34	30	24	18
Container No.		A	A-6	D	G
Mass of wet soil + container (a)	g	96.50	78.40	80.20	79.3
Mass of dry soil + container (b)	g	80.70	67.00	67.70	67.5
Mass of container (c)	g	43.2	40.50	39.5	41.5
Mass of moisture (a-b)	g	15.80	11.40	12.50	11.80
Mass of dry soil (b-c)	g	37.50	26.50	28.20	26.00
Moisture content (w=a-b/b-c x 100)	%	42.13	43.02	44.33	45.38
PLASTIC LIMIT					
Container No.		A	B	C	
Mass of wet soil + container (a)	g	54	58.5	60.8	
Mass of dry soil + container (b)	g	50.30	55.50	57.5	
Mass of container (c)	g	42.4	41.9	38.6	
Mass of moisture (a-b)	g	3.70	3.00	3.30	
Mass of dry soil (b-c)	g	7.90	13.60	18.90	
Moisture content (w=a-b/b-c x 100)	g	46.84	22.06	17.46	
Average moisture content (wa)	%	28.78			



3. Compaction laboratory Result

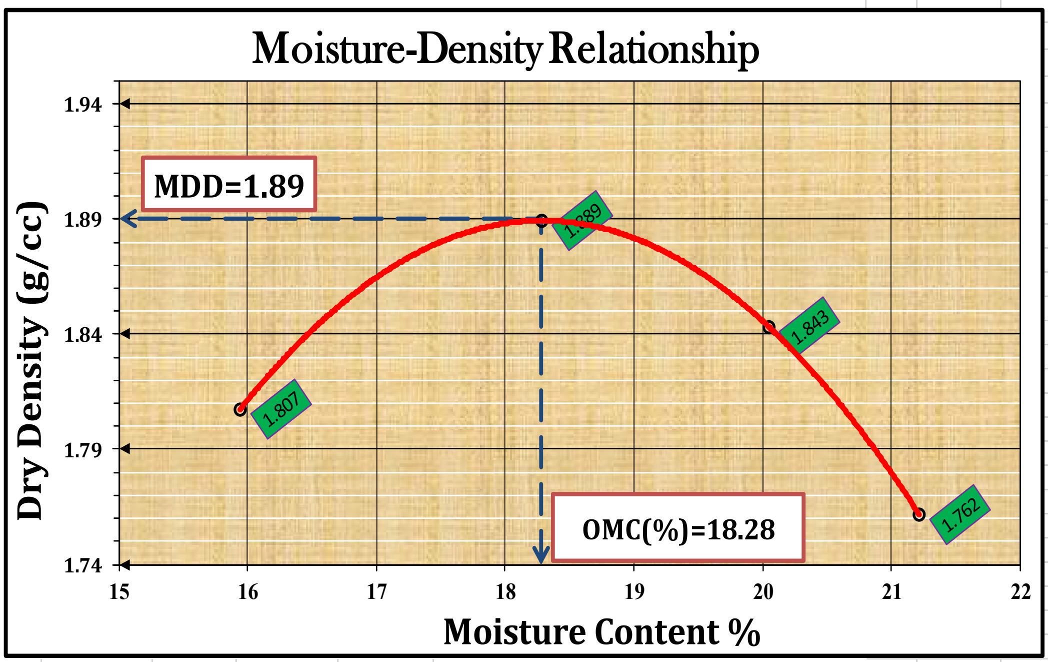
DETERMINIATION OF STANDARD PROCTOR COMPACTION ASTM D-698 AASHTO T-99

Sampled date:-		**Type of Material :- Gravel**	
Tested date:-	**Sample Ref.**	**Sampling Station:-**	**1+780**
Description:-		**Sampled by:-Ashenafi A.**	

TEST DATA

Trial no.		1	2	3	4	
Mass of wet soil + mould	A (g)	6081	6212.4	6191.5	6119	
Mass of mould	B (g)	4102.7	4102.7	4102.7	4103	
Mass of wet soil	C=A-B (g)	1978.3	2109.7	2088.8	2016	
VOLUME OF MOULD	(CC)	944	944	944	944	**NMC**
Bulk density	C / V = W	2.096	2.235	2.213	2.135	X-6
Moisture determination container No.		X-5	D	X-2	B	113
Mass of container + wet soil	a (g)	130.5	127.00	142.00	178	107.50
mass of container + dry soil	b (g)	118.00	114.00	125.50	155.00	41.50
Mass of container	d (g)	39.60	42.90	43.20	46.60	66.0
Mass of dry soil	b - d = e (g)	78.4	71.1	82.3	108.40	5.5
Mass of moisture	a - b = f (g)	12.50	13.00	16.50	23.00	**8.33**
Moisture content	f/e*100 = m (%)	**15.94**	**18.28**	**20.05**	**21.22**	
Dry density	W / (100+m) *100 (g/cc)	**1.807**	**1.889**	**1.843**	**1.762**	

MC	DD
18.28	1.740
18.28	1.890
15	1.890



MAXIMUM DRY DENSITY (g / cc)	1.890
OPTIMUM MOISTURE CONTENT (%)	18.3

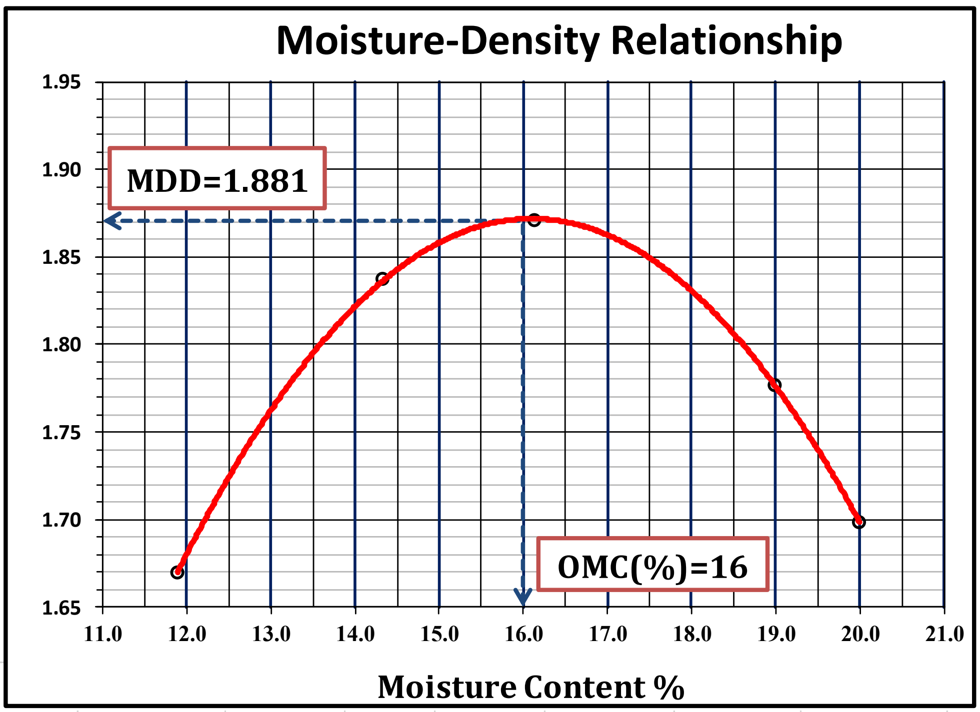
Amount of wate	459.1	ml
mass of sample	5000	g
NMC	8.33	%
OMC	18.28	%

DETERMINIATION OF STANDARD PROCTOR COMPACTION ASTM D-698 AASHTO T-99

Sampled date:-		Type of Material :- Gravel	
Tested date:-	Sample Ref.	Sampling Station:-	3+975
Description:-	Sampled by:-Ashenafi A.	Tested by: -Ashenafi A.	

TEST DATA

Trial no.	1.0	2.0	3.0	4.0	5.0	
Water added %	8.0	12.0	16.0	20.0	24.0	
Mass of wet soil + mould A (g)	10092.0	10586.5	10740.0	10615.0	10453.0	
Mass of mould B (g)	6126.5	6126.5	6126.5	6126.5	6126.5	
Mass of wet soil C=A-B (g)	3965.5	4460.0	4613.5	4488.5	4326.5	
VOLUME OF MOULD (CC)	2123.0	2123.0	2123.0	2123.0	2123.0	
Bulk density C / V = W	1.9	2.1	2.2	2.1	2.0	**NMC**
Moisture determination container No.	X-5	D	X-2	B		X-6
Mass of container + wet soil a (g)	178.0	187.5	169.0	206.5	195.0	159.5
mass of container + dry soil b (g)	163.5	169.0	151.0	180.0	167.0	152.0
Mass of container d (g)	41.5	40.0	39.5	40.5	27.0	43.0
Mass of dry soil b - d = e (g)	122.0	129.0	111.5	139.5	140.0	109.0
Mass of moisture a - b = f (g)	14.5	18.5	18.0	26.5	28.0	7.5
Moisture content f/e*100 = m (%)	**11.9**	**14.3**	**16.1**	**19.0**	**20.0**	**6.9**
Dry density W / (100+m) *100 (g/cc)	**1.7**	**1.8**	**1.9**	**1.8**	**1.7**	



PRO COMPACTION

METHOD C

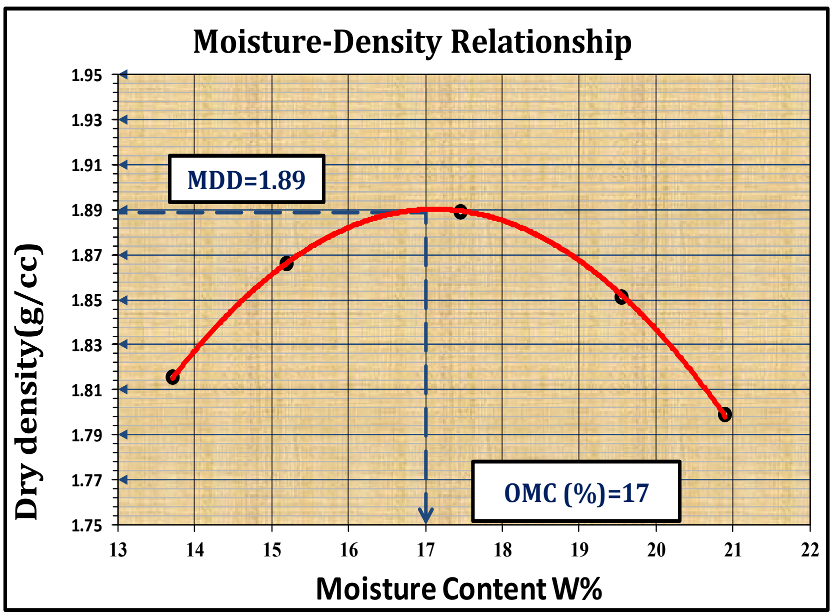
MAXIMUM DRY DENSITY (g / cc)	1.871	Amount of water required for CBR	426.6	ml
OPTIMUM MOISTURE CONTENT (%)	16.0	mass of sample	5000	g
		NMC	6.88	%
		OMC	16.0	%

DETERMINIATION OF STANDARD PROCTOR COMPACTION ASTM D-698 AASHTO T-99

Sampled date:-		Type of Material :- GRAVEL
Tested date:-	Sampling Station:- 5+540	
Description:-	Sampled by:-Ashenafi A.	Tested by: - Ashenafi A.

TEST DATA

Trial no.		1.0	2.0	3.0	4.0	5.0			
Water added %		8.0	10.0	12.0	14.0	16.0			
Mass of wet soil + mould	A (g)	6052.0	6132.5	6197.3	6192.5	6156.2			
Mass of mould	B (g)	4103.0	4103.0	4103.0	4103.0	4103.0			
Mass of wet soil	C=A-B (g)	1949.0	2029.5	2094.3	2089.5	2053.2			
VOLUME OF MOU	(CC)	944.0	944.0	944.0	944.0	944.0			
Bulk density	C / V = W	2.1	2.1	2.2	2.2	2.2	**NMC**		
Moisture determination container No.		A	B	C	D	E	X-3		
Mass of container + wet soil	a (g)	143.2	158.7	165.9	177.0	186.1	186.8		
mass of container + dry soil	b (g)	131.0	143.0	147.5	155.0	162.0	176.90		
Mass of container	d (g)	42.0	39.7	42.1	42.5	46.7	42.40		
Mass of dry soil	b - d = e (g)	89.0	103.3	105.4	112.5	115.3	134.5		
Mass of moisture	a - b = f (g)	12.2	15.7	18.4	22.0	24.1	9.9		
Moisture content	f/e*100 = m (%)	**13.7**	**15.2**	**17.5**	**19.6**	**20.9**	**7.36**	MC	DD
Dry density	W / (100+m) *100 (g/cc)	**1.8**	**1.9**	**1.9**	**1.9**	**1.8**		17.00	1.75
								17.00	1.89
								13	1.89



P OR COMPACTION
METHOD B

MAXIMUM DRY DENSITY (g / cc)	1.89
OPTIMUM MOISTURE CONTENT (%)	17.0

Amount of water requir		448.96	ml
mass of samp	5000	gram	
NMC	7.36	%	
OMC	17	%	

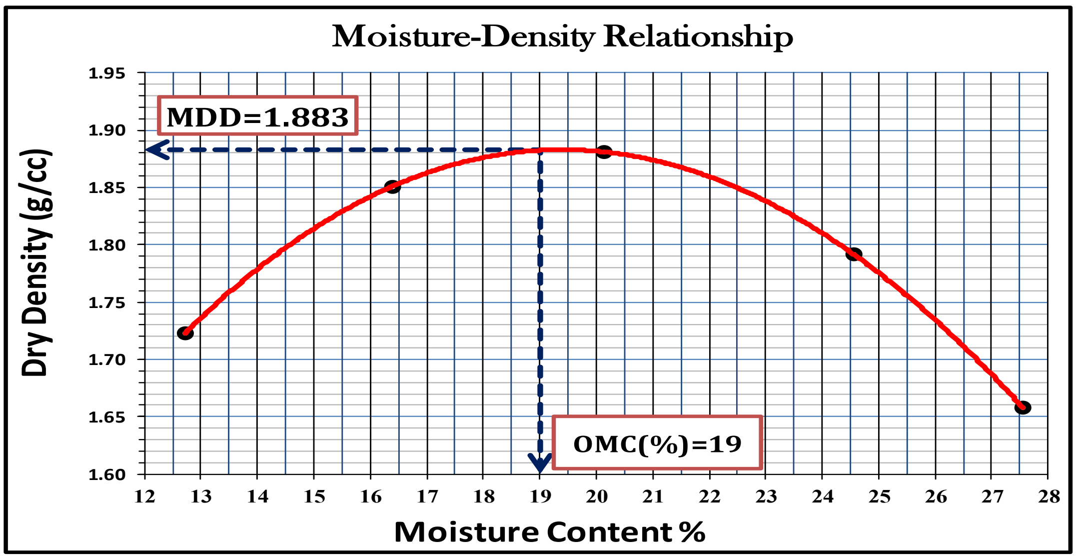
DETERMINIATION OF STANDARD PROCTOR COMPACTION ASTM D-698 AASHTO T-99

Sampled date:-		**Type of Material :-Gravel**	
Tested date:-	**Sample Ref.**	**Sampling Station:**	**10+435**
Description:-	**Sampled by:-Ashenafi A.**	**Tested by: -Ashenafi A.**	

TEST DATA

Trial no.	1	2	3	4	5	
Water added %	8	12	16	20	24	
Mass of wet soil + mould A (g)	5923.5	6124.3	6224	6197	6087	
Mass of mould B (g)	4090.6	4090.6	4091	4091	4091	
Mass of wet soil C=A-B (g)	1832.9	2033.7	2133	2107	1996	
VOLUME OF MOULD (CC)	944	944	944	944	944	
Bulk density C / V = W	1.942	2.154	2.260	2.232	2.115	**NMC**
Moisture determination container No.	A	B	C	B	E	N-4
Mass of container + wet soil a (g)	183	153.50	214.40	222.5	253	98.2
mass of container + dry soil b (g)	169.00	140.50	189.00	191.00	212.00	92.50
Mass of container d (g)	58.90	61.20	62.90	62.70	63.20	18.70
Mass of dry soil b - d = e (g)	110.1	79.3	126.1	128.30	148.80	73.8
Mass of moisture a - b = f (g)	14.00	13.00	25.40	31.50	41.00	5.7
Moisture content f/e*100 = m (%)	**12.72**	**16.39**	**20.14**	**24.55**	**27.55**	**7.72**
Dry density W / (100+m) *100 (g/cc)	**1.723**	**1.851**	**1.881**	**1.792**	**1.658**	

PROC COMPACTION METHOD B



MC	DD
19	1.600
19	1.883
12	1.883

MAXIMUM DRY DENSITY (g / cc)	**1.883**
OPTIMUM MOISTURE CONTENT (%)	**19.0**

Amount of water required for CBR	**523.4**	
mass of sample	**5000**	**g**
NMC	**7.72**	**%**
OMC	**19.0**	**%**

4. CBR Laboratory Result

Gravel material station 0+540

Density of soil from CBR mold(g/cm3)

Soaking Condition		56 blows			
		Before soaking		After soaking	
Mold number		1	2	1A	2A
Weight of wet soil + mold(g)	W2	11914	11983.5	11981	12049.5
Weight of mold (g)	W1	7043	7092.5	7043	7092.5
Weight of wet soil (g)	W2-W1	4871	4891	4938	4957
Volume of mold (cm3)	V	2250	2250	2250	2250
Wet density of soil (g/cm3)	W2-W1/V	2.16	2.17	2.19	2.20
Dry density of soil (g/cm3)	W2-W1/V(1+w)	1.90	1.91	1.86	1.87
Average dry density (g/cm3)		**1.90**		**1.87**	

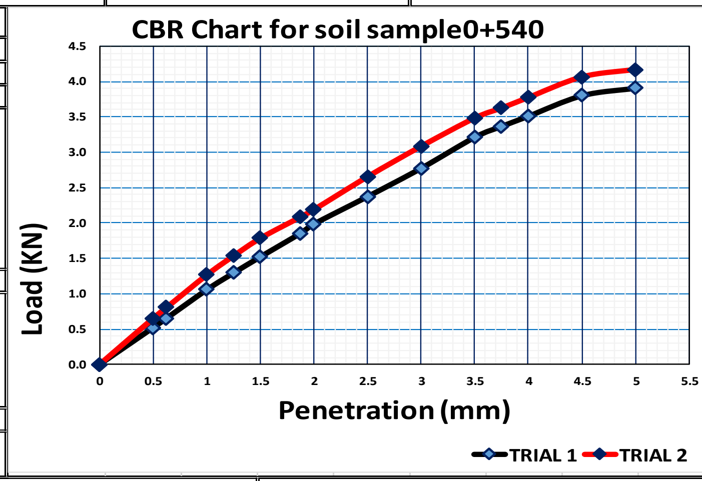
Moisture Content Determination

Soaking Condition		56 blows			
		Before soaking		After soaking	
Container		A	B	A1	B1
Weight of wet soil + Container (g)	W3	155	156.5	151.5	153.5
Weight of dry soil + Container (g)	W2	141.5	142.5	134.5	136.5
Weight of Container (g)	W1	43	44.5	39.5	40.5
Weight of Moisture(g)	W3-W2	13.5	14	17	17
Weight of dry soil (g)	W2-W1	98.5	98	95	96
Moisture Content (%)	W3-W2/(W2-W1)	13.71	14.29	17.89	17.71
Average moisture content (%)		14.00		17.80	

CBR Penetration determination

Penetration after 96 hrs. soaking period

Pen. (mm)	LOAD (KN) 1	LOAD (KN) 2	CBR (%) 1	CBR (%) 2
0	0.0	0		
0.5	0.521	0.652		
0.625	0.654	0.805		
1	1.057	1.265		
1.25	1.294	1.534		
1.5	1.522	1.784		
1.875	1.857	2.084		
2	1.987	2.189		
2.5	2.375	2.654	**17.99**	**20.11**
3	2.776	3.084		
3.5	3.215	3.484		
3.75	3.368	3.624		
4	3.512	3.774		
4.5	3.802	4.064		
5	3.908	4.17	**19.54**	**20.85**
10	4.033	4.295		
12.5	4.212	4.474		



Swell Determination

No of blows	Gauge rdg. mm		Swell in %
56	Initial	13.5	
	Final	14.5	**0.86**
MDD	**1.98g/cc**		
Calculated CBR (%)		**20.85%**	

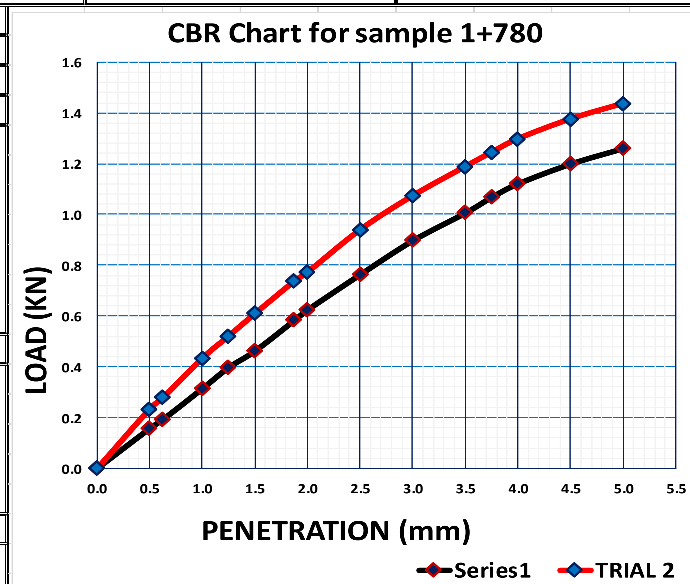
Proctor Data's

OMC = 15.49%	MDD = 1.98%

Gravel material station 1+780					
Density of soil from CBR mold(g/cm3)					
Soaking Condition		56 blows			
		Before soaking		After soaking	
Mold number		1	2	1A	2A
Weight of wet soil + mold(g)	W2	11896.5	11760	11953.5	11819.5
Weight of mold (g)	W1	7048.5	6957.5	7048.5	6957.5
Weight of wet soil (g)	W2-W1	4848	4802.5	4905	4862
Volume of mold (cm3)	V	2250	2250	2250	2250
Wet density of soil (g/cm3)	W2-W1/V	2.15	2.13	2.18	2.16
Dry density of soil (g/cm3)	W2-W1/V(1+w)	1.85	1.83	1.79	1.77
Average dry density (g/cm3)		1.84		1.78	

Moisture Content Determination					
Soaking Condition		56 blows			
		Before soaking		After soaking	
Container		1	2	A1	A2
Weight of wet soil + Container (g)	W3	170.5	164.5	173.5	167.5
Weight of dry soil + Container (g)	W2	153.2	146.7	152.9	141.5
Weight of Container (g)	W1	45.5	42.5	41.5	39.5
Weight of Moisture(g)	W3-W2	17.3	17.8	20.6	26
Weight of dry soil (g)	W2-W1	107.7	104.2	111.4	102
Moisture Content (%)	W3-W2/(W2-W1)	16.06	17.08	18.49	25.49
Average moisture content (%)		16.57		21.99	

CBR Penetration determination				
Penetration after 96 hrs. soaking period				
Pen. (mm)	LOAD (KN) 1	LOAD (KN) 2	CBR (%) 1	CBR (%) 2
0.00	0.0000	0.0000		
0.50	0.1580	0.2348		
0.63	0.1930	0.2789		
1.00	0.3150	0.4345		
1.25	0.3980	0.5215		
1.50	0.4630	0.6099		
1.88	0.5840	0.7380		
2.00	0.6250	0.7730		
2.50	0.7642	0.9402	5.79	7.12
3.00	0.8980	1.0740		
3.50	1.0070	1.1890		
3.75	1.0690	1.2450		
4.00	1.1220	1.2980		
4.50	1.2000	1.3760		
5.00	1.2610	1.4370	6.31	7.19
10.00	2.4540	2.6300		
12.50	3.2578	3.0210		



Swell Determination			
No of blows	Gauge rdg. Mm		Swell in %
56	Initial	15.5	
	Final	17.5	1.72
MDD	1.89 g/cc		
Calculated CBR (%)	7.19%		

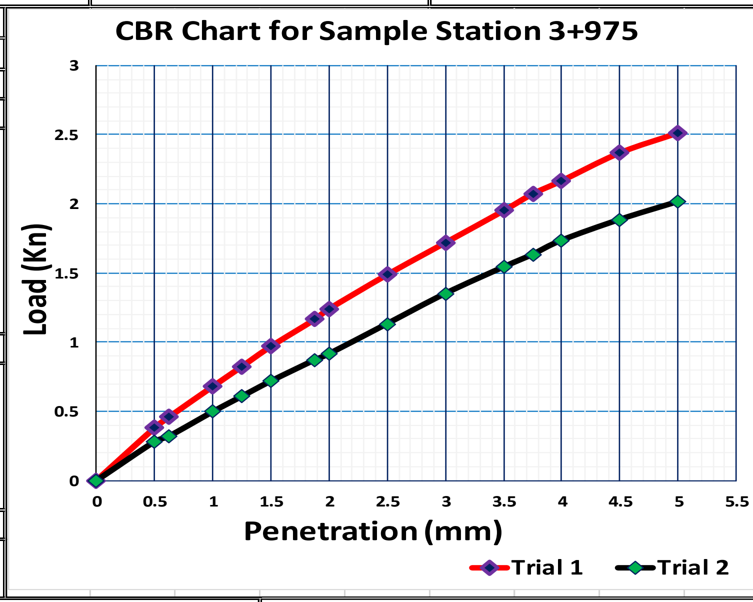
Proctor Data's	
OMC =18.3%	MDD = 1.89%

Gravel material sample station 3+975					
Density of soil from CBR mold(g/cm3)					
Soaking Condition		56 blows			
		Before soaking		After soaking	
Mold number		1	2	1A	2A
Weight of wet soil + mold(g)	W2	11966.5	11964.5	12013.5	11009.5
Weight of mold (g)	W1	6959.5	7038	6959.5	7038
Weight of wet soil (g)	W2-W1	5007	4926.5	5054	3971.5
Volume of mold (cm3)	V	2250	2250	2250	2250
Wet density of soil (g/cm3)	W2-W1/V	2.23	2.19	2.25	1.77
Dry density of soil (g/cm3)	W2-W1/V(1+w)	1.93	1.89	1.91	1.50
Average dry density (g/cm3)		1.91		1.70	

Moisture Content Determination

Soaking Condition		56 blows			
		Before soaking		After soaking	
Container		1	2	A1	A2
Weight of wet soil + Container (g)	W3	134	131	170	163.5
Weight of dry soil + Container (g)	W2	121.5	119	150.5	142.5
Weight of Container (g)	W1	40.5	42.5	39	25
Weight of Moisture(g)	W3-W2	12.5	12	19.5	21
Weight of dry soil (g)	W2-W1	81	76.5	111.5	117.5
Moisture Content (%)	W3-W2/(W2-W1)	15.43	15.69	17.49	17.87
Average moisture content (%)		15.56		17.68	

CBR Penetration determination				
Penetration after 96 hrs. soaking period				
Pen. (mm)	LOAD (KN) 1	LOAD (KN) 2	CBR (%) 1	CBR (%) 2
0	0	0		
0.5	0.38	0.281		
0.625	0.46	0.321		
1	0.68	0.497		
1.25	0.824	0.608		
1.5	0.97	0.718		
1.875	1.164	0.87		
2	1.241	0.915		
2.5	1.489	1.132	11.28	8.58
3	1.72	1.35		
3.5	1.951	1.541		
3.75	2.073	1.632		
4	2.168	1.734		
4.5	2.37	1.884		
5	2.514	2.013	12.57	10.07
10	5.6241	5.2141		
12.5	6.4863	6.0763		



Swell Determination			
No of blows	Gauge rdg. Mm		Swell in %
56	Initial	14	
	Final	15.2	1.03
MDD	1.87g/cc		
CBR (%)	12.57%		

Proctor Data's	
OMC = 16%	MDD = 1.87%

Gravel material sample station 5+540

Density of soil from CBR mold(g/cm3)

Soaking Condition		56 blows			
		Before soaking		After soaking	
Mold number		1	2	1A	2A
Weight of wet soil + mold(g)	W2	12011.5	11907	12051.5	11931.5
Weight of mold (g)	W1	7049	6958.5	7049	6958.5
Weight of wet soil (g)	W2-W1	4962.5	4948.5	5002.5	4973
Volume of mold (cm3)	V	2250	2250	2250	2250
Wet density of soil (g/cm3)	W2-W1/V	2.21	2.20	2.22	2.21
Dry density of soil (g/cm3)	W2-W1/V(1+w)	1.92	1.92	1.84	1.83
Average dry density (g/cm3)		1.92		1.83	

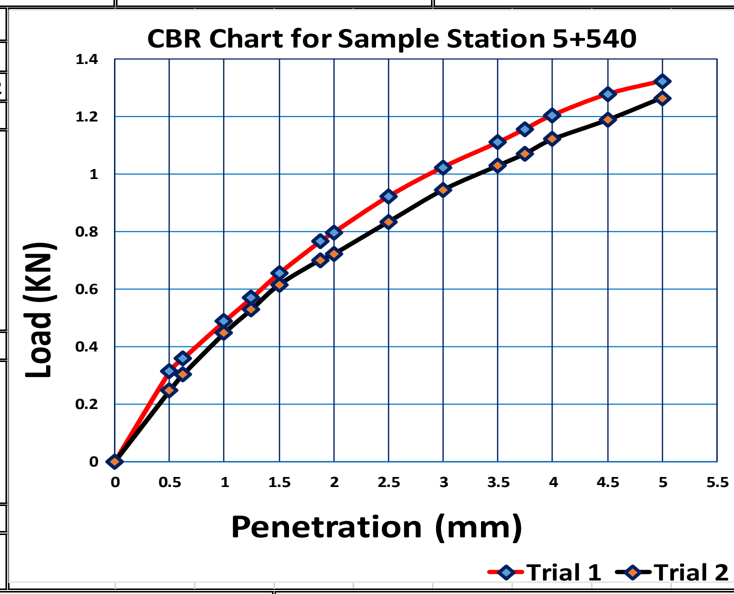
Moisture Content Determination

Soaking Condition		56 blows			
		Before soaking		After soaking	
Container		1	2	A1	A2
Weight of wet soil + Container (g)	W3	154.5	162	166.5	155.5
Weight of dry soil + Container (g)	W2	140	146.4	145.1	135.5
Weight of Container (g)	W1	40.5	42.5	43.5	40.5
Weight of Moisture(g)	W3-W2	14.5	15.6	21.4	20
Weight of dry soil (g)	W2-W1	99.5	103.9	101.6	95
Moisture Content (%)	W3-W2/(W2-W1)	14.57	15.01	21.06	21.05
Average moisture content (%)		14.79		21.06	

CBR Penetration determination

Penetration after 96 hrs. soaking period

Pen. (mm)	LOAD (KN) 1	LOAD (KN) 2	CBR (%) 1	CBR (%) 2
0	0	0		
0.5	0.312	0.246		
0.625	0.358	0.303		
1	0.486	0.446		
1.25	0.568	0.529		
1.5	0.653	0.614		
1.875	0.764	0.7		
2	0.796	0.721		
2.5	0.921	0.833	6.98	6.31
3	1.023	0.944		
3.5	1.1089	1.028		
3.75	1.1558	1.07		
4	1.2039	1.12		
4.5	1.2764	1.187		
5	1.3228	1.264	6.614	6.32
10	2.0779	2.046		
12.5	2.8791	2.755		



Swell Determination

No of blows	Gauge rdg. Mm		Swell in %
56	Initial	14.25	
	Final	16.5	1.93
MDD	1.89 g/cc		
CBR (%)	6.98%		

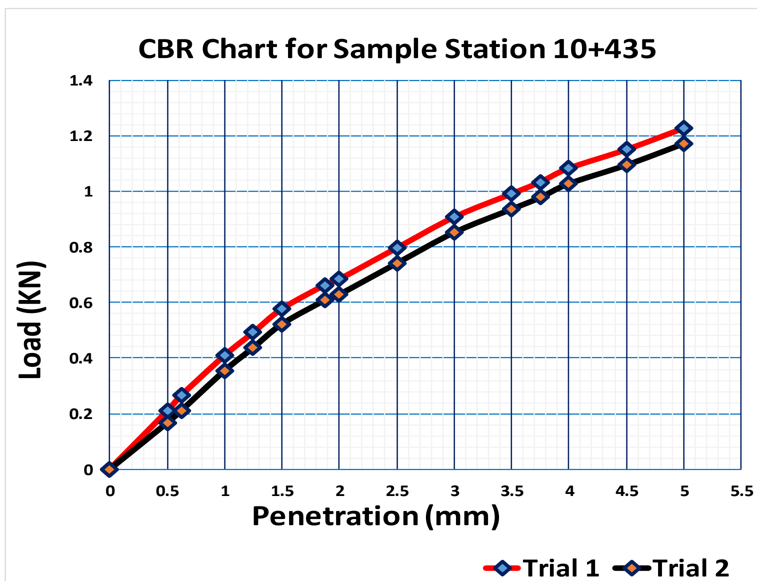
Proctor Data's

OMC = 17%	MDD = 1.89

Gravel Material sample station 10+435					
Density of soil from CBR mold(g/cm3)					
Soaking Condition		56 blows			
		Before soaking		After soaking	
Mold number		1	2	1A	2A
Weight of wet soil + mold(g)	W2	11799	11863	11835	11905
Weight of mold (g)	W1	7026.5	7094.5	7026.5	7094.5
Weight of wet soil (g)	W2-W1	4772.5	4768.5	4808.5	4810.5
Volume of mold (cm3)	V	2250	2250	2250	2250
Wet density of soil (g/cm3)	W2-W1/V	2.12	2.12	2.14	2.14
Dry density of soil (g/cm3)	W2-W1/V(1+w)	1.79	1.79	1.77	1.77
Average dry density (g/cm3)		1.79		1.77	

Moisture Content Determination					
Soaking Condition		56 blows			
		Before soaking		After soaking	
Container		1	2	A1	A2
Weight of wet soil + Container (g)	W3	158.5	156.5	166.5	168
Weight of dry soil + Container (g)	W2	140.5	138.1	145.3	145.9
Weight of Container (g)	W1	39.5	40.5	39.5	43
Weight of Moisture(g)	W3-W2	18	18.4	21.2	22.1
Weight of dry soil (g)	W2-W1	101	97.6	105.8	102.9
Moisture Content (%)	W3-W2/(W2-W1)	17.82	18.85	20.04	21.48
Average moisture content (%)		18.34		20.76	

CBR Penetration determination				
Penetration after 96 hrs. soaking period				
Pen. (mm)	LOAD (KN) 1	LOAD (KN) 2	CBR (%) 1	CBR (%) 2
0	0	0		
0.5	0.209	0.167		
0.625	0.266	0.212		
1	0.409	0.355		
1.25	0.492	0.438		
1.5	0.577	0.523		
1.875	0.663	0.609		
2	0.684	0.63		
2.5	0.796	0.742	6.03	5.62
3	0.907	0.853		
3.5	0.991	0.937		
3.75	1.033	0.979		
4	1.083	1.029		
4.5	1.15	1.096		
5	1.227	1.173	6.14	5.87
10	2.009	1.955		
12.5	2.718	2.664		



Swell Determination			
No of blows	Gauge rdg. Mm		Swell in %
56	Initial	14.5	
	Final	16.85	2.02
MDD	1.88g/cc		
CBR (%)	6.14%		

Proctor Data's	
OMC = 19%	MDD = 1.88g/cc

Appendix C: Traffic Count Result

Table C.1 Summary of both direction traffic count for Mechare to Arssema road

No	Type	Monday	Tuesday	Wednesday	Thursday	Friday	Saturday	Sunday
1	Cart		6	1	1			1
2	Bic	2	4	1	3	2	1	4
3	Mc	2	2	3	1	2	1	2
4	Bajaj	48	64	44	41	52	39	59
5	Taxi	1	2		1	1	1	2
6	S/B	13	24	2	12	15	13	21
7	M/B		6					7
8	L/B							
9	Pickup		2		2	1		2
10	S/T	2	13	6	2	2	2	14
11	M/T		2		1	2	2	2
12	L/T							1
13	T/T							
Total		**68**	**125**	**57**	**64**	**73**	**56**	**89**
ADT		**76 Vehicle/day**						

Appendix D: Discharge Determination Using Rational Formula for Catchment Area Less Than 0.50km^2

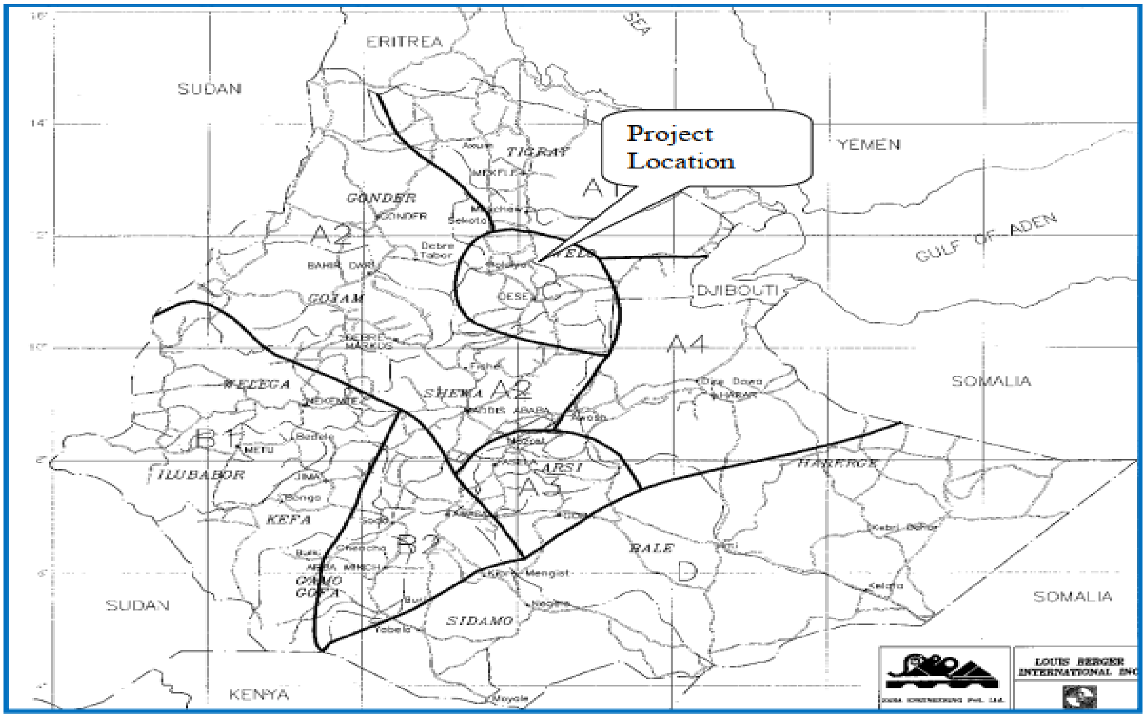


Figure D.1 Rainfall Regions of Ethiopia (Source: ERA drainage design manual 2013)

Table D.1 Storm Design Return Period (Source: ERA LVR)

Structure type	Geometric design standard							
	DC4		DC3		DC2		DC1	
	Design	Check	Design	Check	Design	Check	Design	Check
Side drains	10	25	5	10	5	10	2	5
Fords and drifts	10	25	5	10	5	10	2	5
Culvert diameter <2m	15	25	10	25	10	25	5	10
Large culvert diameter >2m	25	50	15	25	10	25	5	10
Gabion abutment bridge	25	50	20	25	15	25	-	-
Short span bridge (<10m)	25	50	25	50	15	25	10	25
Masonry arch bridge	50	100	25	50	25	50	-	-
Medium span bridge (15m–50m)	50	100	50	10	25	50	-	-
Long span bridge >50m	100	200	100	200	50	100	-	-

Table D.2 Runoff Coefficients, C Values (Adopted from ERA, 2013 manual)

Factor		Description	Runoff Coefficient
Cs	Average slope of catchment	< 3.5% Flat	0.05
		3.5% - 10% Soft to moderate	0.1
		10% - 25% Rolling	0.15
		25% - 45% Hilly	0.2
		> 45% Mountainous	0.25
Cp	Permeability of soil	Well drained soil e.g. sand and gravel	0.05
		Fair drained soil e.g. sand and gravel with fines	0.1
		Poorly drained soil e.g. silt	0.15
		Impervious soil e.g. clay, organic silts and clay	0.25
		Water-logged black cotton soil	0.5
		Rock	0.4
Cv	Vegetation	Dense forest/thick bush	0.05
		Sparse forest/dense grass	0.1
		Grassland/scrub	0.15
		Cultivation	0.2
		Space grassland	0.25
		Barren	0.3
C = Cs + Cp + Cv			

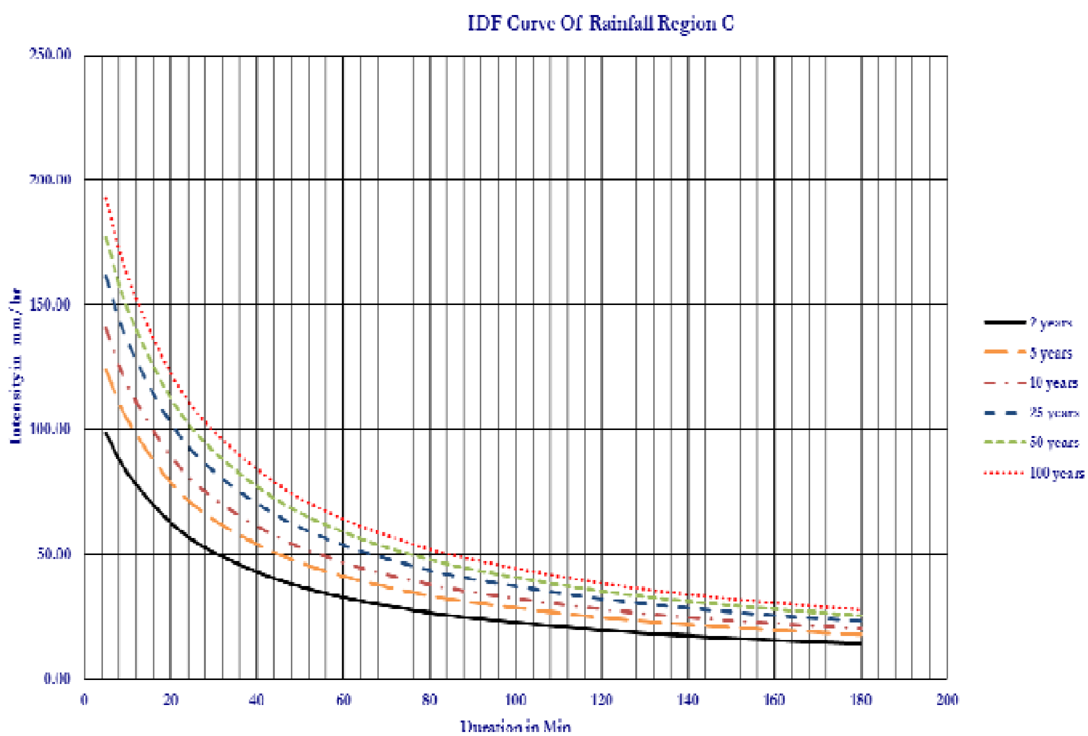


Figure D.2 IDF Curve of Rainfall Region C (Source: ERA drainage design manual 2013)

Table D.3 Frequency Factors for Rational Formulas (ERA DDM 2013)

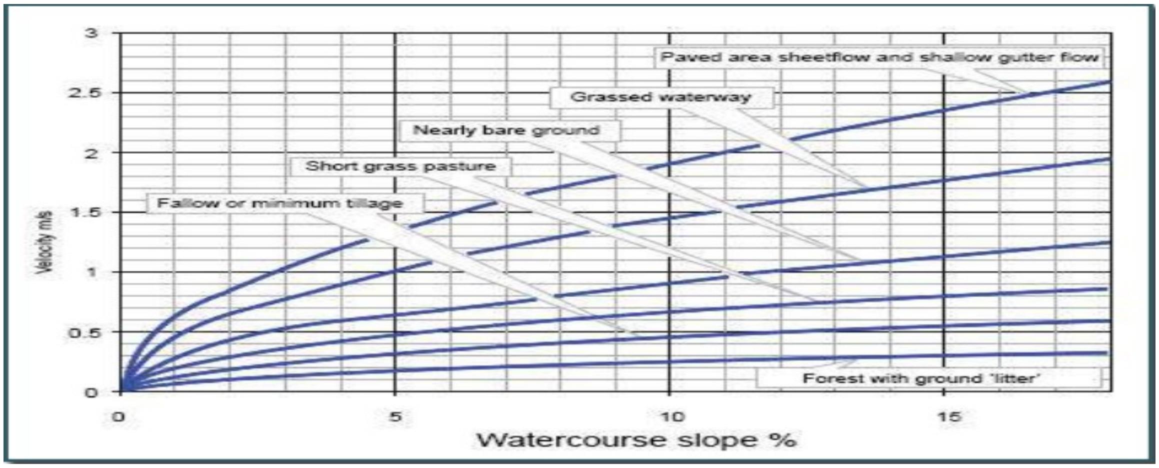
Recurrence Interval (years)	C_f
5	1.0
10	1.0
25	1.1
50	1.2
100	1.25

Table D.5 summary of Runoff determination using rational method for all stations

RUNOFF DETERMINATION USING RATIONAL FORMULA FOR CATCHMENT AREA LESS THAN 0.50km²																						
MECHARE TO ARSSEMA ROUTE SEGMENT																						
No	Station	Area No	Catch. Area	Stream L.	Cat. Terrain	Soil Type	Land Use	slope %	Tc	INTENSITY (mm/hr)			FREQ. FACTOR			RUNOFF COE.				DESIGN DISCHARGE		
	km		km2	km					min	I_5	I_{10}	I_{25}	C_{f5}	C_{f10}	C_{f25}	C_s	C_p	C_V	C	Q_5	Q_{10}	Q_{25}
1	0+330	A1	0.340	1.280	Flat to Moderate	Clay	Intensivily Cultivated	0.22	33	65.0	70	79	1.0	1.0	1.1	0.10	0.25	0.20	0.55	3.38	3.64	4.49
2	1+320	A2	0.485	1.460	Flat to Moderate	Clay	Intensivily Cultivated	0.43	44	51	60	75	1.0	1.0	1.1	0.10	0.25	0.20	0.55	3.78	4.45	6.12
3	2+850	A3	0.264	0.47	Flat to Moderate	Rock	Moderately Cultivated land	0.2	32	66	71	79	1.0	1.0	1.1	0.10	0.25	0.20	0.55	2.64	2.85	3.51
4	3+400	A4	0.306	0.83	Flat to Moderate	Rock	Moderately Cultivated land	0.21	32	66	71	79	1.0	1.0	1.1	0.10	0.25	0.20	0.55	3.06	3.30	4.07
5	4+650	A5	0.125	0.35	Flat to Moderate	Clay	Moderately Cultivated land	0.17	28	70	75	81	1.0	1.0	1.1	0.10	0.25	0.20	0.55	1.34	1.43	1.70
6	5+020	A6	0.114	0.24	Rolling	Clay	Moderately Cultivated land	0.11	24	75	79	85	1.0	1.0	1.1	0.15	0.25	0.20	0.60	1.43	1.49	1.78
7	5+440	A7	0.28	0.464	Rolling	Rock	Moderately Cultivated land	0.2	26	73	76	83	1.0	1.0	1.1	0.15	0.40	0.20	0.75	4.26	4.44	5.33
8	6+890	A8	0.35	0.84	Rolling	Clay	Moderately Cultivated land	0.23	33	65	70	79	1.0	1.0	1.1	0.15	0.40	0.20	0.75	4.74	5.11	6.30
9	9+780	A9	0.140	0.252	Flat to Moderate	Clay	Moderately Cultivated land	0.18	28	70	75	81	1.0	1.0	1.1	0.10	0.25	0.20	0.55	1.50	1.61	1.91

Table D.6 Roughness Coefficients (Manning's n)

Surface Description	n[1]
Smooth surfaces (concrete, asphalt, gravel, or bare soil	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover ≤ 20%	0.06
Residue cover > 20%	0.17
Grasses:	
Short grass	0.15
Dense Grasses	0.24
Range (natural)	0.13
Woods:[2]	
Light underbrush	0.40
Dense underbrush	0.80



FigureD.3 Velocity of Flow chart (Source: ERA 2016 design standard for LVR)

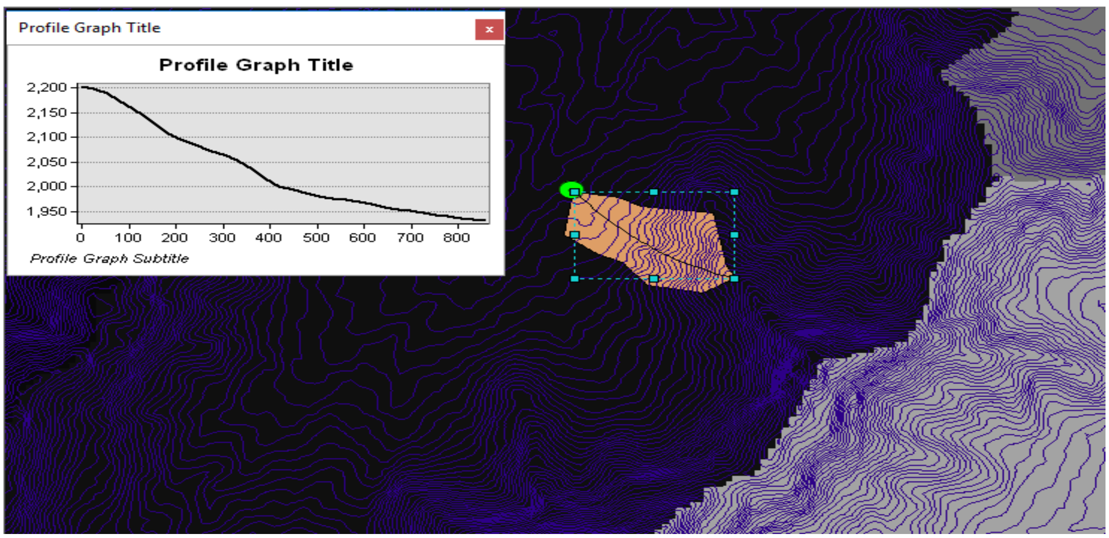


Figure D.4 Slope computation for station 2+850 using Arc GIS

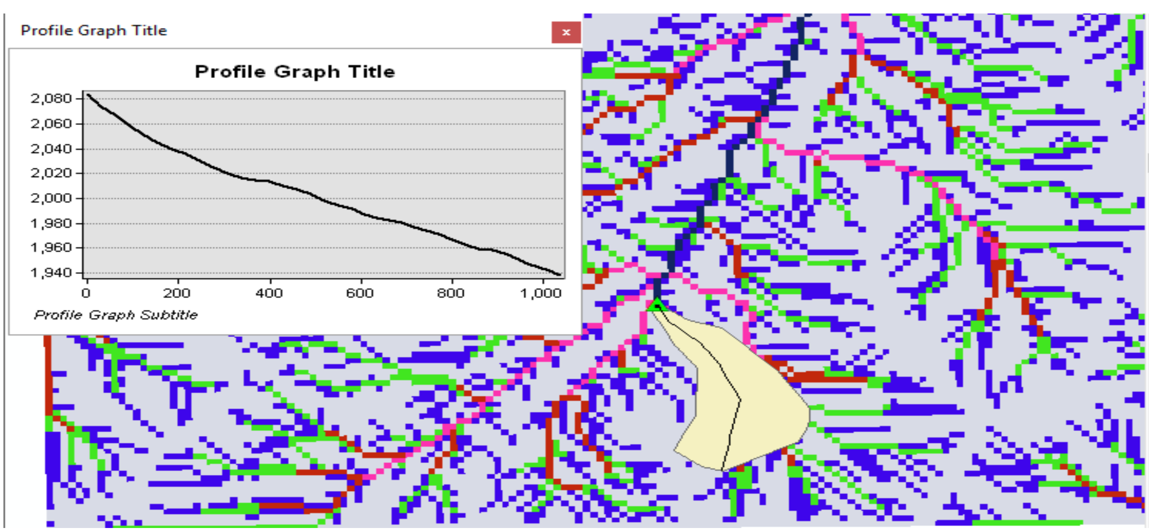


Figure D.5 Slope computation for station 3+400 using Arc GIS

Table D-7 Manning's "n" values for culvert

Type of Culvert	Roughness or Corrugation	Manning's n
Concrete Pipe	Smooth	0.010 - 0.011
Concrete Box	Smooth	0.012 - 0.015
Spiral Rib Metal Pipe	Smooth	0.012 - 0.013
Corrugated Metal Pipes, Pipe-Arch and Box (Annular or Helical Corrugations, Manning's n varies with barrel size)	2⅔ by ½ inch Annular	0.022 - 0.027
	2⅔ by ½ inch Helical	0.011 - 0.023
	6 by 1 inch	0.022 - 0.025
	5 by 1 inch	0.025 - 0.026
	3 by 1 inch	0.027 - 0.028
	6 by 2 inch Structural Plate	0.033 - 0.035
	9 by 2 ½ inch Structural Plate	0.033 - 0.037
Corrugated Polyethylene	Smooth	0.009 - 0.015
Corrugated Polyethylene	Corrugated	0.018 - 0.025
Polyvinyl Chloride (PVC)	Smooth	0.009 - 0.011

Appendix E: Data Collection Instrument

A. Questionnaires to be filled by Road Professionals and Contractors

Company Name: Amhara rural road authority (ARRA)

Position: Project manager

1. What are gravel road design considerations?

Table E.1 - Data collection instrument for research

	1	2	3	4	5	6	7	8	I don't know
Road performance									
Traffic									
Roadbed soil									
Materials of construction									
Environmental									
Reliability									
Life cycle costs									
Shoulder design									

2. What are the different types of gravel road distresses that you encounter?

✓ Use this mark

Table E.2 Types of gravel road deterioration

Pothole		Erosion		Stoniness	
Loose of gravel		Corrugations		Dust	
Rutting		Loose material		Cracking	

Others, Specify--

3. What causes of pavement structure damage?

Table E.3 Causes of gravel road deterioration

Type of Causes	1	2	3	4	5	6	7	8	I don't know
Overload vehicles									
Design factors									
Type of materials used									
Poor Construction Quality									
Environmental Factors									
Poor drainage system									
Poor Compaction									
Others, Specify									

4. Are the type of material used and gravel road deterioration related?

Yes---------------- No--------------------- I don't know------------------------

If your answer is yes, how? ....

5. Do you think that poor subgrade soil causes gravel road structure to Damage?

Yes, --------------- No --------------------- I do not Know…………….

If yes how? ....

6. How severe the effect of overloaded vehicles on the road pavement service life.

Very sever----------- less severe--------- moderately severe-------------

7. How significant do heavy trucks cause damage?

Most significant----------- more significant----------- not significant-------------------

B. Interview Questions to Local Administrators and Road Users

i. Which parts of the road corridor are more prone to pavement failure?

ii. How do the poor drainage installation and type of material used affect the gravel road deterioration?

iii. What are the major factors affecting the performance of gravel road? In addition, which factors are more affect the existing road?

iv. What is the extent and intensity of gravel road damages in this Zone?

v. How is gravel road deterioration affecting the traffic accident and crowding in the town?

vi. What are the remedial measurements of gravel road defects? And, which measurements are suitable for controlling the problem?

vii. If Road maintenance actions are to be applied on the Arssema to Mechare road segment?

viii. Do local contractors have the technical capacity to undertake gravel road damage in the town?

Appendix F: Photos taken during the study

A. During sample collection

B. During sieve analysis

C. During Atterberg limit test

D. During compaction

E. During CBR test and CBR swell determination