



**ASSESSMENT AND REMEDIAL MEASURES OF ROAD FAILURE  
A CASE STUDY ALONG ADAMA-ASSELA TRUNK ROAD**

**MSc Thesis**

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ADDIS ABABA SCIENCE AND TECHNOLOGY UNIVERSITY**

**OCTOBER 2020**



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A Thesis Submitted as a Partial Fulfillment for the Degree of Master of  
Science in Geotechnical Engineering

to

**DEPARTMENT OF CIVIL ENGINEERING**

**ADDIS ABABA SCIENCE AND TECHNOLOGY UNIVERSITY**

**OCTOBER 2020**

## Certification


This is to certify that the M.Sc. thesis prepared by **Mr. ASHENAFI KEBEDE** entitled **"ASSESSMENT AND REMEDIAL MEASURE OF ROAD FAILURE CASE STUDY ALONG ADAMA-ASSELA TRUNK ROAD"** and submitted as partial fulfillment for the degree of Master of Science complies with the regulations of the University and meets the accepted standards with respect to originality, content and quality.

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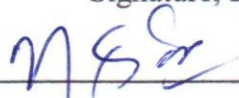
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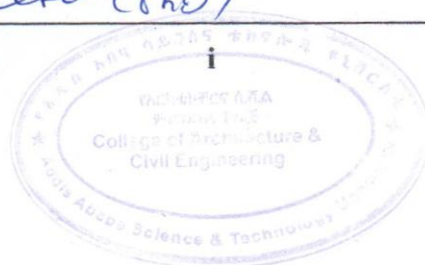
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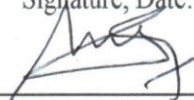
## Declaration

I hereby declare that this dissertation entitled “ASSESSMENT AND REMEDIAL MEASURE OF ROAD FAILURE CASE STUDY ALONG ADAMA-ASSELA TRUNK ROAD” was prepared by me, with the guidance of my advisor. The work contained herein is my own except where explicitly stated otherwise in the text, and that this work has not been submitted, in whole or in part, for any other degree or professional qualification.

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## **Abstract**

The construction of road asphalt pavement is increasing rapidly in the world, which in turn in our country Ethiopia. Not only constructing the road, but also each city in the world has been constantly repairing road infrastructure due to different types of deteriorations. Asphalt concrete pavement roads are always subjected to different failures without giving the intend services for the traffic flow. Ethiopia has been experiencing a continuous growth and change. Due to this there is high increase in traffic volume and demand. As a result most of asphalt pavement subjected to different defects.

This research work investigates types of failure, reasons that cause failures and suggest suitable remedial options for Trunk Road running from Adama to Assela. To achieve these objective 13 sampling areas were selected and pits were excavated to a maximum depth of three meters. Disturbed and undisturbed samples were collected. Field and laboratory investigations have been conducted. Laboratory tests including gradation, Atterbeg limit testes, compaction, CBR tests; direct shear were conducted.

Laboratory test results revealed that 62% of the subgrade soils of the alignment road indicate Silt soil. The remaining 38% indicate soil group which are low in plasticity but fine and  $LL > 32\%$ , maximum dry density range from 1.51 to 1.63 g/cm<sup>3</sup>, optimum moisture content ranges from 11.4 to 17.4, soaked California Bearing Ratio of 4.6 and %swell of 2.97 and direct shear implying that Angel of internal friction ranges 11 to 12 and apparent cohesion of 3 to 4 this indicates that the samples are uniform silty-clay soil and susceptible to water erosion. During heavy rainfall flood result and such flood erodes the unconsolidated collapsible soil and saturate the sub- grade materials. Hence, the research reviled that the failure was formed following heavy rain fall and in that the existing cross drainage structure was unable to accommodate the flood. Due to this the flood moved parallel to the road eroding the soil and thereby creating gully.

**Keywords:** Asphalt Pavement, Trunk Road, Pavement Failure, CBR, Flood, Gully,

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## **List of Abbreviation**

AASHTO - American association of State Highway Transportation Officials

AC - Aggregate Crushing Value

CBR- - Californian Bearing Ratio

ERA -Ethiopian road authority

LL -LiquidLimit

LHS - Left hand side

MDD - Maximum Dry Density

MER - Main Ethiopia Rift

NP - Non-Plastic

OMC - Optimum moisture content

PL - Plastic limit

RHS - Right hand side

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# CHAPTER ONE

## INTRODUCTION

### 1. Background

In the context of Ethiopia's geography, pattern of settlement and economic activity, transport plays a vital role in facilitating economic development. In particular, it is road transport that provides the means for the movement of people, utilization of land and natural resources, improved agricultural production and marketing, access to social services, and opportunities for sustainable growth. Recognizing the importance of the road transport in supporting social and economic growth and its role as a catalyst to meet poverty reduction targets, the Government of Ethiopia has placed increased emphasis on improvement of the quality and extent of road infrastructure in the country. However some of the newly constructed roads failed before serving the design life time because of problems associated with design, workmanship, soil problem, ground condition, and inter pavement & surface water drainage. [15]

Premature road pavement failure occurs when it can no longer performs its intended function of carrying vehicles and people from one location to another in safety and comfort before the anticipated design life. Failure is therefore defined as an unacceptable difference between the expected design life and the observed performance [8].

A classic example of premature road failure is the Addama-Assela road which failed after being opened to vehicular traffic. On Adama-Asela road, which is about 79.5km, wide and deep gully has been formed and endangered part of the existing road between Dera and Etaye (km 30+000 to km 42+800). The gully was formed following heavy rain fall and the existing cross drainage structure was unable to accommodate the flood, as a result the flood moved parallel to the road eroding the soil and thereby creating gully. [4]

This research work investigates types of failure, reasons that cause failures and suggest suitable remedial options for Trunk Road running from Adama to Assela.

## **1.2. Statement of the problem**

It is reported that some of newly constructed roads in Ethiopia have shown premature failures due to design, poor workmanship, inadequacy of the drainage system, geotechnical related problems and negligence of road maintenance. Hence, newly constructed roads are not long lasting and do not give intended purpose they are constructed for. Huge amount of resource was also allocated for the construction and maintenance thus entail an increased amount of maintenance and road users cost that directly affects the economy of the nation.

Various researches have been conducted previously to access the cause of road failure and suggesting remedial measures. However, most of the previous researches were focused on the road which performed several years after construction and most of them were done in countries other than Ethiopia and hence doesn't have potential to indicate the cause of failures in Ethiopia.

This research aims to fill in gap in literature by assessment the cause of failures for road running from adama-assela and suggesting remedial measures.

## **1.3. Objective**

### **1.3.1. General objective**

The general objective of this research is to assess and suggests remedial measures for road failure for the case study of adama-assela trunk road.

### **1.3.2. Specific objectives**

Specific objectives are

- ❖ To Study the physical and Engineering Property of the soil
- ❖ To Study and identify Different type of road failure
- ❖ To recommend remedial measures based on the findings.

## **1.4. Scope of the study**

This scope of the study is to identify the failure and suggest Remedies of the road. The study encompasses the road route running from Adama –Assela road will be considered. Both Laboratory and filed test have been conducted for the research.

### **1.5. Significance of the research**

Most of the previous researches were focused on the road which performed several years after construction and most of them were done in countries other than Ethiopia and hence doesn't have potential to indicate the cause of failures in Ethiopia. This research has a plan fill in gap in literature by assessment the cause of failures for road running from adama-assela and suggesting remedial measures. The research also used as a reference for other researchers who want to conduct further study in the area.

### **1.6. Limitation of the research**

This study is limited to the identification of failure and Remedies of the road running from Adama –Assela. Even though both Laboratory and filed test have been conducted for the research was not validated with previous research.

## **CHAPTER TWO**

### **LITERATURE REVIEW**

A highway pavement is a structure consisting of superimposed layers of processed materials above the natural soil sub-grade, whose primary function is to distribute the applied vehicle loads to the sub-grade. The pavement structure should be able to provide a surface of acceptable riding quality, adequate skid resistance, favorable light reflecting characteristics, and low noise pollution. The ultimate aim is to ensure that the transmitted stresses due to wheel load are sufficiently reduced, so that they will not exceed bearing capacity of the sub-grade. Two types of pavements are generally recognized as serving this purpose, namely flexible pavements and rigid pavement. Improper design of pavements leads to early failure of pavements affecting the riding quality. Pavement deterioration is due to traffic and environmental factors. [17].

Pavement failure is defined in terms of decreasing serviceability caused by the development of surface distresses such as cracks, potholes and ruts, [8]. They reported that before going into the maintenance strategies, engineers must look into the causes of failures of bituminous pavements. They found that failures of pavements are caused due to many reasons or combination of reasons. It has been seen that only three parameters i.e. unevenness index, pavement cracking and rutting are considered while other distresses have been omitted while going for maintenance operations.

Moisture affects the sub grade, sub base, or granular base, while temperature affects the asphalt mixture. For example; Materials of basic igneous rock origin are sometimes weathered and may release additional plastic fines during construction or in service. Problem is likely to worsen if water gains entry in to the pavement and this can lead to rapid and premature failure. The release of these minerals may lead to a consequential loss in the bearing capacity. Climatic factors include rainfall and annual variations in temperature are an important consideration in pavement deterioration. Rainfall has a significant influence on the stability and strength of the pavement layers because it affects the moisture content of the sub grade soil. [4].

Subgrade with CBR greater than 5% and plastic index less than 25% are described as good subgrade. Any failure in the subgrade will cause structural failure of the pavement. A minimum CBR of 30% is required at the highest anticipated moisture content when compacted to the specified field density, usually a minimum 95% of the maximum dry density achieved. Highly plastic sub base or base materials of highly weathered basaltic origin cause premature pavement failure [6]. Base course material should be angular in shape with Flakiness Index of less than 35%. In addition, to insure the material is sufficiently durable, it should have  $PI < 6\%$ , minimum TFV 110KN and  $CBR > 80\%$  [14]. Failure in road base leads to insufficient cover to subgrade and led to subgrade failure. Failure on road wearing course leads to infiltration of water to base, sub base and subgrade and hence causes failure to pavement structure.

The effect of rain on road pavements can be destructive and detrimental as most pavements are designed based on a certain period of rainfall data. In addition, rainfall is well established as a factor affecting the elevation of the water table, the intensity of erosion, and pumping and infiltration investigating causes of pavement deterioration. [5]. In many pavement failures, excess moisture is the main cause of failure or a contributing cause. Queensland Transport [17] reported the effect of moisture content changes on the strength and stiffness of pavement materials. They found that excess moisture reduces the strength and stiffness of pavement materials, being worse for the sub grade material, than for the sub base or base. Excess moisture and particularly high degrees of saturation result in significant pore pressures within the material. Depending on the degree of saturation, failure may occur as any of rapid shear or bearing failure, premature rutting, lifting of wearing course due to positive pore pressures, or embedment of cover aggregate due to weak base, [17].

Long periods of rainfall of low intensity can be more adverse than short periods of high intensity because the amount of moisture absorbed by the soil is greater under the former conditions [11]. He further emphasized that water is the critical factor that cause road



failures. Once water has entered a road pavement, the damage initially is caused by hydraulic pressure. Vehicles passing over the road pavement impart considerable sudden pressure on the water, this pressure forces the water further into the road fabric and breaks it up. This process can be very rapid once it begins. When vehicles pass over the weak spot, the pavement will start to crack and soon the crack generates several cracks. Water will then enter the surface voids, cracks and failure areas. This can weaken the structural capacity of the pavement causing existing cracks to widen. Eventually, the water will descend to the subgrade, weakening and hence lowering the CBR value of the subgrade on which the road pavement design was based upon.

It can be seen that for nearly all types of pavement failure, moisture is often the primary or a contributing cause of failure. Moisture entry through the surface may be caused by inadequate pavement surface drainage during construction; exposure of surface to rain during may be caused by pondage in pits or poorly constructed surface drainage, and lateral movement of water into pavement. Other factors affecting the moisture in a pavement include the general drainage condition, such as the effectiveness of drainage structures, shoulder cross-fall and condition, longitudinal grade, and whether the pavement is constructed on cut or fill. Pavement failure quantified by its extent (length of road affected) and its severity. [17]

Drainage becomes major problem and challenge for a number of highways in the country. A number of recently constructed roads are affected and damaged by flood. Sometimes the construction of improperly designed road creates considerable adverse impact on the environment and socio-economic activity. And also it is difficult to achieve the service from the infrastructure unless drainages are treated properly during design as well as implementation stage. [2].

Soil erodibility is an estimate of the ability of soils to resist erosion, based on the Physical characteristics of each soil. The physical factors which affect erodibility of soil are

aggregate stability, particle size distribution, base minerals, organic carbon content, clay mineralogy, infiltration capacity, pore size, pore stability, moisture holding capacity

of soil, topographic features and management of the land ((Hudson, 1996 and Shetha,

2002). Generally, soils with faster infiltration rates, higher levels of organic matter and improved soil structure have greater resistance to erosion. Loam-textured soils tend to be less erodible than silt and very fine sand. Tillage and cropping practices that lower soil organic matter levels, cause poor soil structure, and result in increases in soil erodibility. [5].

Gully erosion is the erosion process whereby water concentrates in narrow channels and over short periods removes the soil. Gully erosion produces channels larger than rills. As the volume of concentrated water increases and attains more velocity on slopes; it enlarges the rills into gullies. Gully can also originate from any depression such as cattle trails, footpaths, cart tracks and traditional furrows and indicates neglect of land over long period of time [9].

The gully channels carry water during and immediately after rains and distinguished from rills, gullies cannot be obliterated by normal tillage. The Soil Conservation Society of America defines a gully as "a channel or miniature valley cut by concentrated runoff but through which water commonly flows only during and immediately after heavy rains: it may be dendritic or branching or it may be linear, rather long, narrow and of uniform width". The rate of gully erosion depends primarily on the runoff producing characteristics of the watershed, soil characteristics, alignment, size and shape of the gully and the slope in the channel.

Gully erosion is one of the major environmental challenges that are widespread across the globe. It is considered as the worst stage of soil erosion due to the permanent damages caused to the landscapes and, gullies today stand as a significant contributor of sediments in water reservoirs, as indicated by studies in Australia, Ethiopia, China and USA (Poesen et al. 2003; Valentin, Poesen & Li 2005)

According to Woods and Adcox [21], pavement failure may be considered as structural, functional, or materials failure, or a combination of these factors. Structural failure is the loss of load carrying capability, where the pavement is no longer able to absorb and transmit the wheel loading through the structure of the road without causing further deterioration. Functional failure is a broader term, which may indicate the loss of any function of the pavement such as skid resistance, structural capacity,

and serviceability or passenger comfort. Materials failure occurs due to the disintegration or loss of material characteristics of any of the component materials.

The highway drainage system includes the pavement and the water handling system which includes pavement surface, shoulders, drains and culverts. These elements of the drainage system must be properly designed, built, and maintained. When a road fails, inadequate drainage often is a major factor. Poor design can direct water back onto the road or keep it from draining away. Too much water remaining on the surface combine with traffic action may cause potholes, cracks and pavement failure [10]. Inadequate drainage leads to major cause of pavement distress due to large amount of costly repairs or replacements long before reaching their design life. Drainage design for pavement is to keep the base, sub-base, subgrade, and other susceptible paving materials from becoming saturated or even being exposed to constant high moisture levels over time.

Caltrans [2] categorized the main types of pavement failures as either deformation failures or surface texture failures. Deformation failures include corrugations, depressions, and potholes, rutting and shoving. These failures may be due to either traffic (load associated) or environmental (non load associated) influences. It may also reflect serious underlying structural or material problems that may lead to cracking. Surface texture failures include bleeding, cracking, polishing, stripping and raveling. These failures indicate that while the road pavement may still be structurally sound, the surface no longer performs the function it is designed to do, which is normally to provide skid resistance, a smooth running surface and water tightness. Other miscellaneous types of pavement failures include edge defects, patching and roughness.

The Cracking consists of visible discontinuities in surface and can be an indication of the pavement's structural condition and serious, [15]. The main problem with cracks is that they allow moisture into pavement, giving accelerated deterioration of pavement. Cracks can occur in a wide variety of patterns. They may result from a large number of causes, but generally are the result of either ageing and embrittlement of surfacing, environmental conditions, structural or fatigue failure of the pavement, or any other causes, [15]. The formation of cracks in the pavement surface causes numerous problems such as discomfort to the users, reduction of safety, etc. In addition to the

above, intrusion of water causing reduction of the strength in lower layers as well as lowering of bearing capacity of subgrade soil by pumping of soil particles through the cracks is also a major problem associated with the pavements, [16]. This leads to the progressive degradation of the road pavement structure in the neighborhood of the cracks. The origin of cracks differs by their shapes, configuration, and amplitude of loading, movement of traffic and rate of deformation.

According to Ahmed [1], potholes are an indication of structural surface failure and they result from growth of a break in the surfacing, often as a result of severe alligator cracking. Once water enters pavement layers, the base and/or subgrade become wet and unstable, and the resultant degradation leads to rapid growth of pothole area and depth. Sikdar et al [16] reported that if the potholes are numerous or frequent, it may indicate underlying problem such as inadequate pavement or aged surfacing requiring rehabilitation or replacement. Water entering pavement is often the cause, and could be caused by a cracked surface, high shoulders or pavement depressions ponding water on pavement, porous or open surface, or clogged side ditches.

Road maintenance is one of the importance components of the entire road system. Even if the Highways are well designed and constructed, they may require maintenance. Road maintenance is Necessary and required to protect the road in its originally constructed condition, protect adjacent resources and user safety, and provide efficient, convenient travel along the route. Unfortunately, maintenance is often neglected or improperly performed resulting in rapid deterioration of the road and eventual failure from both climatic and vehicle use impacts. It follows that it is impossible to build and use a road the requires no maintenance. [17].The remedy for all types of cracks depends on whether pavement remains structurally sound or has become distorted or unsound. Where the pavement is structurally sound, cracks should be filled with low viscosity binder. Slurry seal or sand bituminous premix patching can be used to fill wide cracks

# CHAPTER THREE

## RESEARCH METHODOLOGY

### 3.1. Method of Investigation

It is adopted a both laboratory and field test method for this research. The research constitutes several stages. The following flow chart clearly indicates the methodology followed for the research.

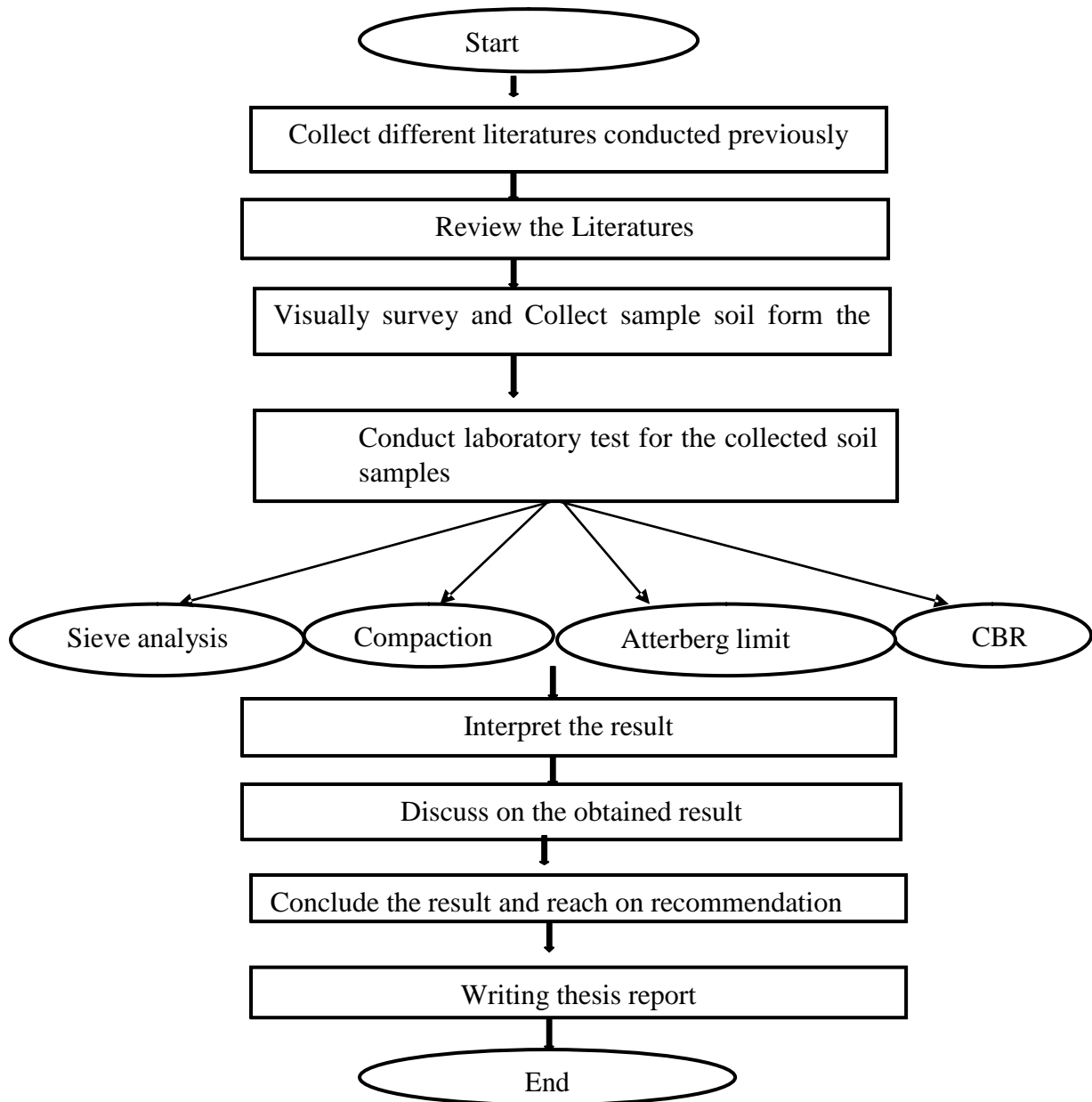


Figure 3.1: Flow chart showing the methodology to be followed

### 3.2. Description of Study Area

The study area lies in south eastern Oromia regional state along Adama-Asela road section particularly between Sodere junction (i.e., which is 113km from Addis Ababa) up to Iteya town (which is about 135km from Addis Ababa). The UTM Coordinate of Adama town is, Easting 529836.5 and Northing 943512.6 and UTM coordinate of Iteya town is Easting 524967.4 and Northing 899695.9 (Fig.3.2).

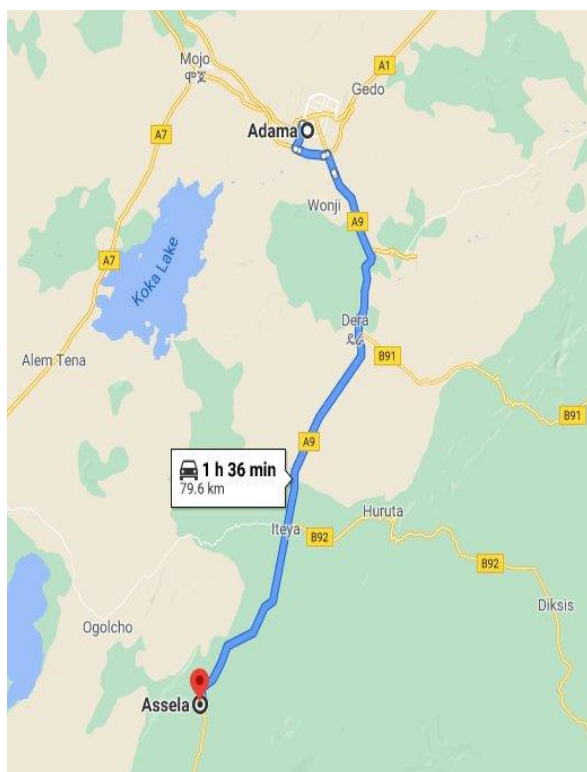


Figure 3.2: map of study area (Source:-Google map)

### 3.3. Geology of the route corridor

The study area is found in south central part of the country at the verge of the Eastern Plateau and the Main Ethiopian Rift system. The area is found making the shoulder of the rift from the east direction with an elevation of ranges from 1600m to 2000m amsl. The gully hazards are mainly observed starting from Sodera junction up to Iteya town. The main features of this area include flat morphology with frequent appearance of greyish silty sand soil. The geological formation of the study area is mainly covered by quaternary sediments of lacustrine origin. Lacustrine beds are interbedded with plio-pleistocene. Lacustrine beds are mostly redeposited volcanic sand and tuff with calcareous material and diatomite. According to Mohr.(1966) at the beginning of the Quaternary an ancestral lake which was almost certainly continuous from south to Awash basin to the north existed until it shrinks to the smaller ones by late Pleistocene tectonic movements. As observed during the site investigation, the lacustrine deposit found in study area is greyish to whitish silty sandy soil. Moreover, gypsiferous and fossiliferous limestone of lacustrine origin is observed in study area.



Figure 3.3: Geology of the route corridor

### 3.4. Topography

The topography of the project route is classified as flat, rolling, Mountainous and Escarpment based on ERA quality manual. This manual defines terrain classification based on both transverse and longitudinal slope of the road x-section with the following criteria.

The project corridor falls on Flat terrain. The terrain classification table is as shown

Table 3.1 Terrain of project corridor

Adama -Assela			
Start	End	Length (Km)	Terrain
0+000.000	5+504.18	5.5	Flat
Assela Existing Road			
119+123	131+849.12	12.73	Flat

### 3.5. Climatic condition of the project area

Climate is influenced by latitude, altitude, land and water surfaces, mountain barriers, local topography, and such atmospheric features as prevailing winds, air masses and pressure Centre's. Although Ethiopia is located in the tropics, temperatures vary greatly with altitude and large climate variation, from hot arid to cool temperate, exist in the country. Generally, project is located Weina Dega Zone and Altitude 1500-2500. Climate also has a strong influence on the pavement performance, and may be accounted for in the design to some extent.

Table 3.2: Design Rainfall of the Project Area

Return period	25	50	Rainfall region
24hr rain fall (mm)	78	86	A3



### 3.6. Hydrological study

Hydrological study is the basic step that should be done carefully in every flood management, road drainage facilities or protection projects design. The hydrological study was necessary in order to estimate the design discharge for Addama-Assela road section which require urgent rehabilitation measures based on the site investigation. Calculation of these peak discharge values helps in the determination appropriate remedial measure design that makes the road and surrounding area safe from prevailing flooding problem. The design floods are estimated using both HEC-HMS hydrologic model software and spread sheet program that developed to estimate the design flood using SCS method for road section under consideration.

### 3.7. Field Investigation

Field investigation of the existing pavement condition has been conducted in order to identify the type and extent of the pavement failure. Test pit excavation and sampling of soil for laboratory test performed to identify the major pavement failure .In addition; it provided valuable site information that has an influence on the best remedial option.



(a)



(b)



(c)



(d)

Figure 3.4(a), (b), (c) and (d) Soil survey and sampling

### 3.8. Laboratory Tests

Laboratory tests performed on the Subgrade Materials. Laboratory test conducted on representative samples taken from subgrade layers to determine physical characteristics of the materials. Samples obtained at the test pit of the road, and dug out using digger from subgrade. They were suitably packed into sacks and labeled in such a manner that each material can be identified distinctly. They were transported to the laboratory for the following tests: Sieve Analysis, Atterberg limit, Compaction, moisture content, free swell, California Bearing Ratio and Direct Shear in accordance with ASTM and AASHTO testing manual. The following chart shows the laboratory tests conducted for the collected soil samples.

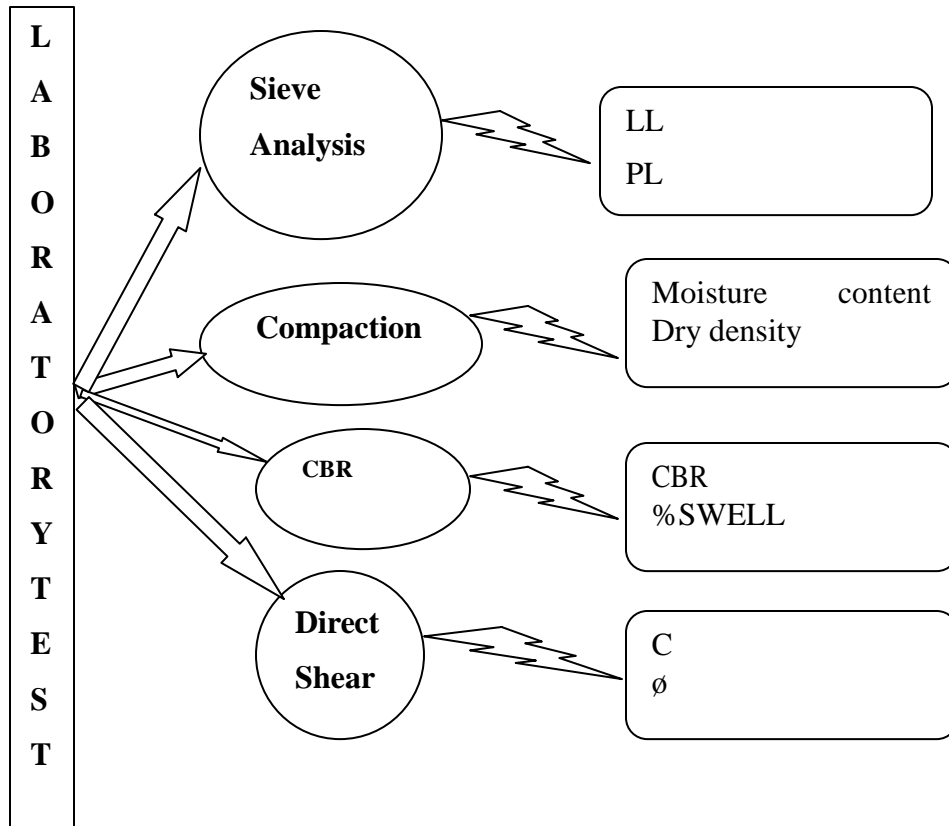


Figure 3.5: Flow chart showing the conducted Laboratory Tests

## CHAPTER FOUR

### RESULT AND DISCUSSION

#### 4.1. Visual Road Condition Survey

The existing condition of the road on site was surveyed visually and experimentally so as to collect best representative data. This has helped to relate the pavement distress noticed on the surface with possible causes and also to relates which distress are caused by the effect of geotechnical problems like subgrade materials. Different distresses are recorded during visual condition survey including: Rutting, potholes, cracks and settlement due to gully erosion etc.

The following pavement failures were noted and recorded,

- Cracks
- Ruts/Deformation
- Potholes
- Settlement due to gully erosion

Table 4.1 Chainage and severity level

Road Section	Chainage		Remark
	From	To	
Uniform Section 1	5+200	8+650	Minor distresses [Severity-level-1)
Uniform Section 2	9+700	15+150	Intermediate Distress (Severity- level 2)
Uniform Section 3	15+150	41+500	Severely Distressed areas ( Severity -level-3)



(a)



(b)



(c)

Figure 4.1(a),(b) and (c): Road condition of the failed section

#### 4.1.1. Cracking failure

Longitudinal Cracks are parallel slits to the pavement's centerline; these cracks are structural defects (weakness of paving layer) and Functional defects (roughness of the paving surface). The loads and moisture accelerate the deterioration of these cracks

#### **4.1.2. Pothole failure**

Minimum numbers of Potholes were observed on the roads section. However, the appearance of Potholes shows failure on base or subgrade due to poor drainage or due to structural deficient pavement. Moreover, the intrusion of water to the pavement through the cracking or ponding of water on the rutting of pavement may cause pothole creation.

#### **4.1.3. Rutting Failure**

The analysis indicates that there is type of rutting failure along the study area this failure type is mainly due to the low-quality subgrade materials it can be concluded that some part of the road section severely affected by rutting. This failure type is mainly due to the low quality subgrade materials as and mostly due to excessive traffic loading above the design values.

### **4.2. Laboratory Analysis of Subgrade Soils**

The subgrade soils of the alignment have been analyzed based on the findings of the field investigations. The analysis is made to assess the suitability and stability of the subgrade and then to identify the design parameters required for the design of the pavement layers. The thicknesses and the layers of pavement design determined based on the engineering properties of the subgrade soils and following procedures outlined. Lab test results analysis of the subgrade soils are presented below

#### **4.2.1. Soil Classification**

The purpose of soil classification system is to group soils with similar properties or attributes. As a means of obtaining general behavior, soils are systematically categorized on the basis of some common characteristics obtained from visual inspection/description and laboratory tests. Various soil classification systems are in use throughout the world in different areas of study. In highway engineering, soils are classified by conducting relatively simple tests on disturbed samples to serve as a means of identifying suitable materials and predicting the probable behavior when used as subgrade or sub base material.

#### 4.2.1.1. AASHTO Classification System

In this system of classification, soils are categorized into seven groups, A-1 through A-7, with several subgroups, as shown in Table 43. The classification of a given soil is based on its particle size distribution, LL, and PI. Soils are evaluated within each group by using an empirical formula to determine the group index (GI) of the soils, given as

$$GI = (F - 35) [0.2 + 0.005(LL - 40)] + 0.01(F - 15) (PI - 10) \quad (4.7)$$

Where, GI = group index

F = % of soil particles passing 0.075 mm (No. 200) sieve in whole number based on material passing 75 mm (3 in.) sieve,

LL = liquid limit expressed in whole number, and

PI = plasticity index expressed in whole number

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

Table 4.2: Grain size distribution

Sieve No	Mass Retained (grams) Mr	% Retained M /Mr*100	Σ% Retained	% Finer
2	21	3.18	3.18	97
1.18	150	22.73	25.91	74
0.6	160	24.24	50.15	50
0.3	210	31.8	81.95	18
0.15	88	13.3	95.25	5
0.075	31	4.7	99.97	0.3
660				

Under average conditions of good drainage and thorough compaction, the supporting value of a material as subgrade may be assumed as an inverse ratio to its group index; that is, a group index of 0 indicates a "good" subgrade material and a group index of 20 indicates a "very poor" subgrade material.

Subgrade soils of the realignment have generally low group index, which is the



characteristics of good roadbed soil, indicates that the subgrade soils of the realignment section are suitable to be used as roadbed/foundation for the overlying pavement.

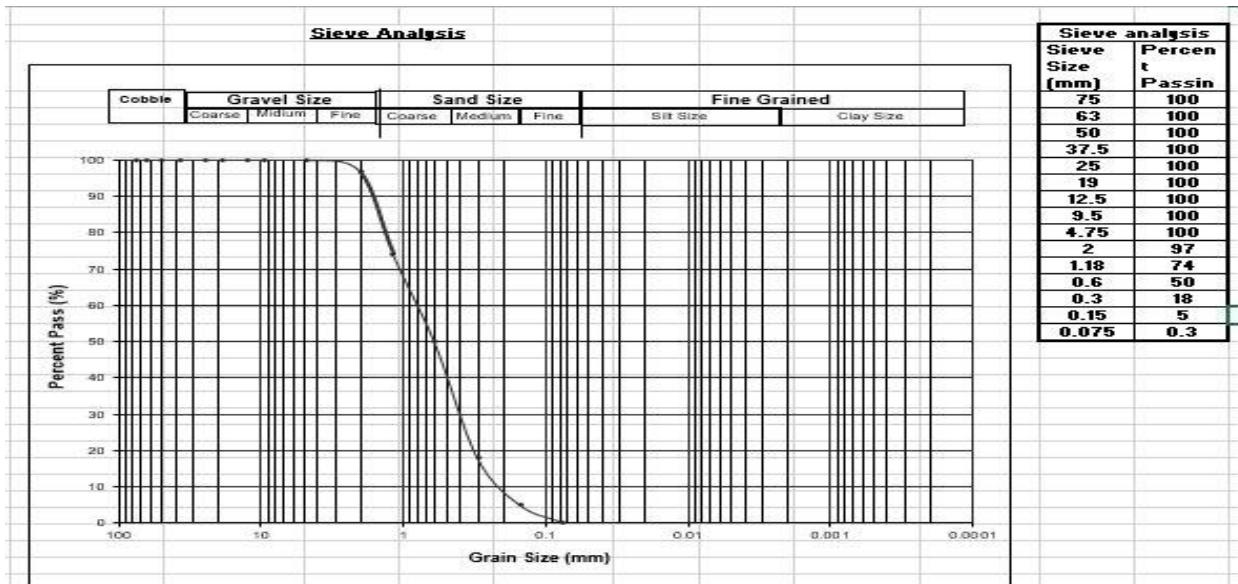


Figure 4.2: Gradation curve graph

The tests conducted for AASHTO soil classifications were the liquid limit, plastic limit and wet sieve analysis. These tests are indicators of the physical properties of the subgrade soils.

The subgrade soils sampled for laboratory testing were subjected to various tests and classified using ASSHTO soil classification method.



Table 4.3: Soil sample classification as per AASHTO

Station (km)	Depth (cm)	% pass this sieve (mm)			LL	PL	PI	AASHTO
		2	0.425	0.075	%	%	%	Soil class
0+000	1.00	93.9	87.3	67.3	37	29	8	A-4(6)
0+500	2.5	99.4	92.3	77	39	30	9	A-4(8)
1+000	3.0	99.5	94.3	82.8	44	33	11	A-7-5(9)
1+500	1.50	96.9	91.0	78.1	32	25	7	A-4(8)
2+000	1.5	99	94.3	84.5	42	32	10	A-5(8)
2+500	2.0	99.6	96.5	89.1	45	35	10	A-5(9)
3+000	3.0	95.5	90.7	74.9	38	28	10	A-4(8)
3+500	3.0	99.4	94.4	83.7	42	32	10	A-5(8)
4+000	1.5	99.4	95.0	85.4	44	30	14	A-7-5(10)
4+500	1.5	99.5	95.9	88	44	32	12	A-7-5(10)
5+000	3.0	99.4	97.1	89.3	40	31	9	A-4(10)
5+500	1.5	99.7	97.4	92.2	47	35	12	A-7-5(12)
6+000	1.5	99.5	97.2	92.6	48	36	12	A-7-5(12)

From the results of classification of the soils along the Road project, 62% of the subgrade soils of the alignment road indicate Silt soil (A-4 and A-5 groups). The remaining 38% indicate A-7-5 soil group which are low in plasticity but fine and  $LL > 41\%$ . In general, the project road can be grouped into homogenous sections of low plastic clayey SILT soil which are good in terms of load bearing but vulnerable to erosion.

#### **4.2.2. Atterberg Limits**

Soils containing clay exhibit a property called plasticity. Plasticity is the ability of a material to be molded (irreversibly deformed) without fracturing. This behavior is unique to clays and arises due to the electrochemical behavior of clay minerals.

The stiffness or consistency of fine grained soils depends on their moisture content, and varies with variations in the amount of moisture present. Depending on its moisture content, a soil can exist in one of the following states: viscous liquid, plastic solid, semi-solid and solid. Atterberg in 1911 proposed a series of tests, mostly empirical, for the determination of the consistency properties/states of fine grained soils. Atterberg limits define the moisture contents at which the soil changes from one state to another. These include the liquid limit (LL), the plastic limit (PL), shrinkage limit (SL). They are determined by tests carried out on the fine soil fraction passing the 425 $\mu$ m (No. 40) sieve.

Consistency limits and the plasticity index are used in the identification and classification of soils. Generally, soils having high values of liquid limit and plasticity index are poor as subgrades/engineering materials. Both the liquid limit and plastic limit depend on the type and amount of clay in the soils. In soils having same values of liquid limit, but with different values of plasticity index; it is generally found that rate of volume change and dry strength increases and permeability decreases with increase in plasticity index. On the other hand, in soils having same values of plasticity index but different values of liquid limit, it is seen that compressibility and permeability increase, and dry strength decreases with increase in liquid limit. Soils that cannot be rolled to a thread at any water content are termed as Non-Plastic (NP).

Table: 4.4: Atterberg Limits test

		Liquid Limit Test				Plastic Limit Test	
		Trial 1	Trial 2	Trial 3	Trial 4	Trial 1	Trial 2
No. of Blows	N	38	26	24	18	-----	-----
Mass of Can	$M_c$	30.6	30.6	30.6	30.6	30.6	30.6
Mass of moist Sample + Can	$M_1$	60.5	64	60	62	57	52
Mass of Dried Sample + Can	$M_2$	53	55	51	52.4	49	48
Mass of Moist	$M_m = M_1 - M_c$	7.5	9	9	9.6	6.5	4.5
Mass of Dried Sample	$M_d = M_2 - M_c$	22.40	24.40	20.40	21.80	18.40	17.40
Moisture	$w = \frac{(M_m - M_d)}{M_d} \times 100$	33.48	36.89	44.12	44.04	35.33	25.86
LL		39.63				PL =	<u>30.59</u>

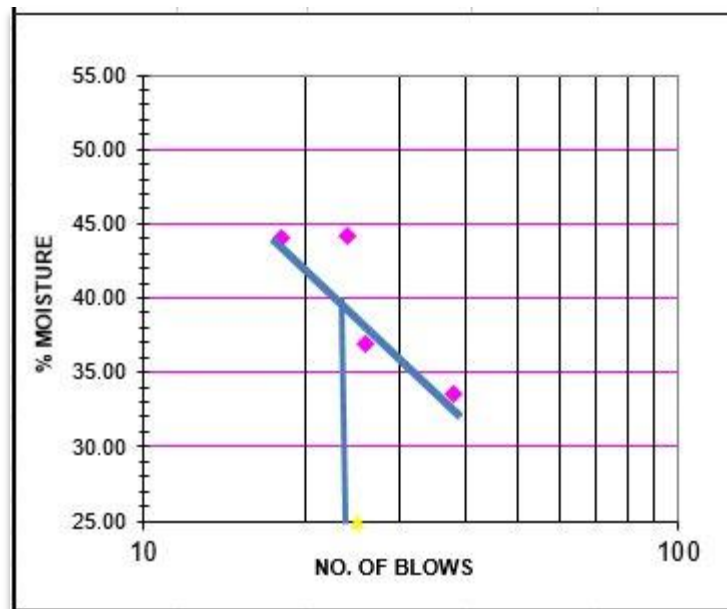


Figure 4.3: Graph of no of blows to percentage moisture

The plasticity index, the difference of the liquid limit and the plastic limit, is an important index of soil. It is one of the important parameters used in the classification of soils and is also an indicator of whether the soil is expansive or not. Plasticity index is also used as an input in the extended investigation of unsuitable soils to determine the expansiveness of the subgrade soil. The plasticity of subgrade soils of the realigned road was determined. According to ERA- 2013 Manual, the upper limit of plasticity for suitable subgrade soil to be used as foundation for pavement and as fill material is 30%. As it is noted from the table, all the subgrade soils have shown plasticity index of less than 30% which is an indicator of suitable subgrade soil

Table 4.5 Atterberg limit test result of each stations

		Station and Atterberg limit test result		
Station (km)	Depth (cm)	LL	PL	PI
		%	%	%
0+000	1.00	37	29	8
0+500	2.5	39	30	9
1+000	3.0	44	33	11
1+500	1.50	32	25	7
2+000	1.5	42	32	10
2+500	2.0	45	35	10
3+000	3.0	38	28	10
3+500	3.0	42	32	10
4+000	1.5	44	30	14
4+500	1.5	44	32	12
5+000	3.0	40	31	9
5+500	1.5	47	35	12
6+000	1.5	48	36	12

Disturbed soil samples were collected from road bed and walls at various depths ranging from 1.7 meters to 12 meters. The plasticity test results shows, all the soil samples from the various sample have clustered within the ranged from NP to 2.5%, which is categorized as non-plastic or low plastic, hence they are cohesionless. Therefore, the non-cohesive nature of the soils in the area account for the formation of gully erosion since water flows through the soil with ease and moves the soil particles down slope with increase in velocity of motion of the water.

Generally, the soil along the study area is non-plastic or low plastic range and they are cohesionless. Therefore, the non-cohesiveness in the area accounts for the formation of gully erosion.

#### **4.2.3. Compaction Test**

Compaction is the process by which air is excluded from a soil mass to bring the particles closer together and thus increase its density (dry density). The state of compaction of a soil is appropriately expressed in terms of the dry density which is a measure of the state of packing of soil particles.

In order to determine the degree of compaction and percentage of moisture content of the subgrade materials, ASTM D 698 test method was adopted. The Dry density was obtained in the laboratory by using the standard compaction method. Charts show the optimum moisture content and maximum dry density of the project soil.

The original test involved compacting the soil in three approximately equal layers in a standard mould, using a 2.5kg hammer falling through a height of 305mm (standard compaction test). However, with the advent of heavier compaction equipment, greater densities were now achievable in the field.

The bulk density of the soil for each trial is obtained by dividing the weight of the soil by the total volume

( $\gamma_b = W/V$ ). = The dry density of the soil is determined by:

$$\gamma_d = \frac{\gamma_m}{1 + w} \quad \text{.....equation 4.2}$$

Where  $\gamma_b$  = bulk unit weight,  $w$  = moisture content

Table 4.6: Compaction sample for lab analysis

		Mass of mold + base (g)			7100
		Volume of mold $V_m =$			2124
Test Point	1	2	3	4	
Mass of Mold + Compacted Soil + Base = $M_1$ (g)	16675	10980	11095	11080	
Mass of Compacted Soil $M_c(g) = M_1 - M_m$	3575.0	3880.0	3995.0	3986	
Wet Density of Soil $d_w(Kg/m^3) = M_c/V_m$	1683.1	1826.7	1880.9	1873.8	
Mass of Pan $M_2(g)$	31	31	31	31	
Mass of Wet Soil + Pan = $M_3(g)$	172.5	175	176	1736	
Mass of Dry Soil + Pan = $M_4(gm)$	158	157.8	156.8	152.5	
Moisture Content $w(\%) = (M_3 - M_4)/(M_4 - M_2) \times 100$	11.4	13.6	15.3	17.4	
Dry Density $\rho_d(gm/cc) = \rho_d/(w+100) \times 100$	1.51	1.6085	1.6318	1.5966	

Table 4.7: Compaction test result of each stations

Sites	Station	Depth (m)	(MDD)	(OMC)
Sites 1	0+000	1.5m	1.51gm/cc	11.4%
Sites 2	0+500	2.0m	1.60gm/cc	13.6%
Sites 3	1+000	3.0m	1.52gm/cc	16.1%.
Sites 4	1+500	3.0m	1.485gm/cc	16.2%.

Sites 5	2+000	3.0m	1.46gm/cc	15.1%.
Sites 6	2+500	1.0m	1.63gm/cc.	16.3%.
Sites 7	3+000	3.5m	1.43gm/cc	14.4%
Sites 8	3+500	4.5m	1.54gm/cc.	14.2%.
Sites 9	4+000	4.5m	1.40gm/cc	13.2%.
Sites 10	4+500	9.0m	1.51gm/cc	10.0%.
Sites 11	5+000	6.0m	1.47gm/cc	16.0%.
Average			1.478gm/cc	16.04%
Min			1.40 gm/cc	10.0%
Max			1.63 gm/cc	16.3%

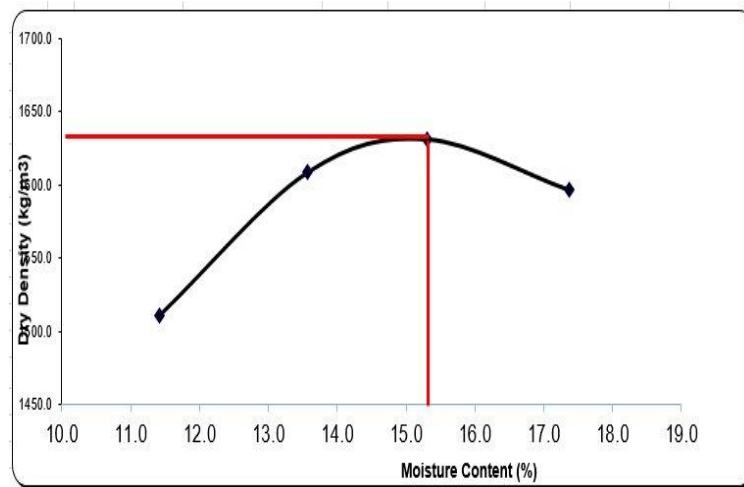


Figure 4.4: Moisture versus dry density curve

The optimum moisture content of the sampled subgrade soil as obtained from the laboratory is 15.3%. Whereas the maximum dry density value of 1632kg/m<sup>3</sup>.

Disturbed soil samples from sites of the study area were analyzed. The compaction tests result shows that the optimum moisture content ranges from 11.4% to 17.4%, while the maximum dry density ranges from 1.51g/cm<sup>3</sup> to 1.63g/cm<sup>3</sup> the moisture

content of the subgrade soil is at maximum dry density of 1.63 is 15.3 .The maximum dry density values (MDD) are generally low because the soil is loose and thus susceptible to erosion. The low values of the dry density indicate that the natural deposits are loose and account for the high void ratio. This high void ratio of the soil will generate high flow velocities, high seepage pressure and high internal erosion potential. Therefore, the low maximum dry density of the soils in the area account for the formation of gully erosion since this high void ratio of the soil will create high internal erosion potential.

#### **4.2.4 CBR-Swell Test**

CBR-Swell test was conducted on CBR specimen and is equivalent to swelling potential test. It is used to identify whether the soil is suspected of expansiveness or not. The expansiveness of a soil is classified according to its swelling potential

In this test, a plunger is made to penetrate the soil, which is compacted to the prevalent dry density and moisture content anticipated in the field (or to MDD and OMC as specified) in a standard mould (CBR mould) at a specified rate of penetration. The resulting load-penetration curve is compared with that obtained for a standard crushed rock material, which is considered an excellent base course material. Depending upon the prevailing climatic conditions of the site, the compacted specimens are immersed in water for four days before the penetration test. The soaking process is to simulate the worst moisture condition of the soil that may occur in the field. During this period, the sample is loaded with a surcharge load that simulates the estimated weight of pavement layers over the material tested. Any swell due to soaking is also measured.

The load is applied by cylindrical metal plunger of 50 mm diameter, the standard penetration rate used is 1.27mm/minute and readings of the applied load are taken at appropriate intervals of penetration (0.5mm, 1.27mm (0.5")) up to a total penetration of usually not more than 7.5 mm-12.7mm.

The CBR is then determined by reading off from the curve the load that causes a penetration of 2.54 mm and dividing this value by the standard load (13.34kN)



required to produce the same penetration in the standard crushed stone as

$$CBR = \frac{\text{Unit load for 2.54 mm penetration in test specimen}}{\text{Unit load for 2.54 mm penetration in standard crushed rock}} \times 100 \quad \dots \text{equation 4.3}$$

Similarly, the CBR at 5.08 mm penetration is obtained by dividing the load causing a penetration of 5.08 mm with the standard load of 20kN required to produce the same penetration in standard crushed stone. The CBR corresponding to 2.54mm penetration is normally greater than that at 5.08mm penetration and is accepted as the CBR of the soil (provided that it is greater than that obtained at 5.08mm penetration). AASHTO T193 test procedure stipulates that, if the CBR at 5.08 mm penetration is greater than that at 2.54mm penetration the entire test should be repeated on a fresh sample. If the 5.08 mm penetration CBR in the repeat test is still greater, then it is accepted as the CBR of the soil.

Table: 4.8: CBR lab analysis

Compacted Dry Density						
			MDD	1.632g/cm <sup>3</sup>		
Moisture Content			OMC	15.3		
			Specimen 1	Specimen 2	Specimen 3	
Mass of Container (g)		M <sub>c</sub>	31.0	31.0	31.0	
Mass of Sample + Container (g)		M <sub>1</sub>	400.0	420.0	421.0	
Mass of Dried Sample + Container (g)		M <sub>2</sub>	365.0	385.0	389.0	
Moisture Content (%)		w = [(M <sub>1</sub> -M <sub>2</sub> ) / (M <sub>2</sub> -M <sub>c</sub> ) x 100]	10.5	9.9	8.9	
			Specimen 1	Specimen 2	Specimen 3	
No. of Blows			10	30	65	
Mass of Mold + Base (g)		M <sub>3</sub>	7250	7650	7800	
Mass of Compacted Specimen + Mold + Base (g)		M <sub>4</sub>	10200	11065	11680	

Mass of Compacted Specimen (g)		$M_5 = M_4 - M_3$	2950	3415	3880
Volume of Mold - Volume of Spacer Disk (cm <sup>3</sup> )		V	2124	2124	2124
Wet Density (g/cm <sup>3</sup> )		$\square_w = M_5 / V$	1.389	1.608	1.827
Dry Density (g/cm <sup>3</sup> )		$\square_d = [\square_w / (100 + w)] \times 100$	1.257	1.463	1.677

Penetration Test (After 4 days Soaking)							RING CALIBRATION		0.04166	KL
		Specimen 1			Specimen 2			Specimen 3		
		Penetration Gauge Ave.	Load Gauge	Applied Load (KN)	Penetration Gauge Ave.	Load Gauge	Applied Load (KN)	Penetration Gauge Ave.	Load Gauge	Applied Load (KN)
0.0	mm	0.00	0.0	0.00	0.00	0.0	0.00	0.00	0.0	0.0
0.64	mm	0.64	1.0	0.04	0.64	3.0	0.12	0.64	5.0	0.2
1.27	mm	1.27	2.0	0.08	1.27	6.0	0.25	1.27	9.0	0.3
1.96	mm	1.96	4.0	0.17	1.96	8.0	0.33	1.96	13.0	0.5
2.54	mm	2.54	6.0	0.25	2.54	14.0	0.58	2.54	19.0	0.7
3.18	mm	3.18	7.0	0.29	3.18	15.0	0.62	3.18	19.0	0.7
3.81	mm	3.81	8.0	0.33	3.81	16.0	0.67	3.81	21.0	0.8
4.45	mm	4.45	9.0	0.37	4.45	17.0	0.71	4.45	23.0	0.9
5.08	mm	5.08	10.0	0.42	5.08	19.0	0.79	5.08	28.0	1.1
7.62	mm	7.62	11.0	0.46	7.62	20.0	0.83	7.62	28.0	1.1
10.16	mm	10.16	12.0	0.50	10.16	22.0	0.92	10.16	30.0	1.2
<b>Applied Load vs. Penetration</b>					<b>Soaked CBR vs. Dry Density</b>					
		Standard	Specimen 1		Specimen 2		Specimen 3			
		Load Ls (KN)	Applied Load La	CBR	Applied Load La	CBR	Applied Load La	CBR		
At corrected 2.5mm penetration		13.2	0.25	2	0.58	4	0.79	6		
At corrected 5.08mm penetration		20	0.42	2	0.79	4	1.17	6		
CBR %		2.08	3.96	5.83	<b>CBR (%)</b>					
Dry density (g/cm <sup>3</sup> )		<b>1.257</b>	<b>1.463</b>	<b>1.677</b>	<b>4.60</b>					
<b>95% OF MDD</b>			<b>1.55</b>							
			Specimen 1 (10)		Specimen 2 (30)		Specimen 3 (65)			

Mass of Annular Weights (kg)			5	5	5
Initial Penetration Gauge Reading (mm)		R <sub>1</sub>	55.00	80.00	95.00
Final Penetration Gauge Reading (mm)		R <sub>2</sub>	520.00	480.00	360.00
Penetration		P = R <sub>2</sub> - R <sub>1</sub>	465.000	400.000	265.000
% Swell	(P/127)		3.66	3.15	2.09
Average				2.97	
No. Of Blows		CBR values %			Swell (%)
		at 2.54mm	at 5.08mm	at 95% MDD	
10		4.20	4.20		0.73
30		3.80	3.75	4.0	0.70
65		3.60	3.70		0.64
10		4.20	4.20		1.13
30		4.00	3.65	3.2	1.06
65		4.20	3.70		0.83
10		3.40	3.30		1.29
30		3.90	4.00	4.2	1.02
65		3.86	4.10		0.80
10		3.80	3.60		1.07
30		3.75	3.89	3.9	0.87
65		4.10	4.30		0.78

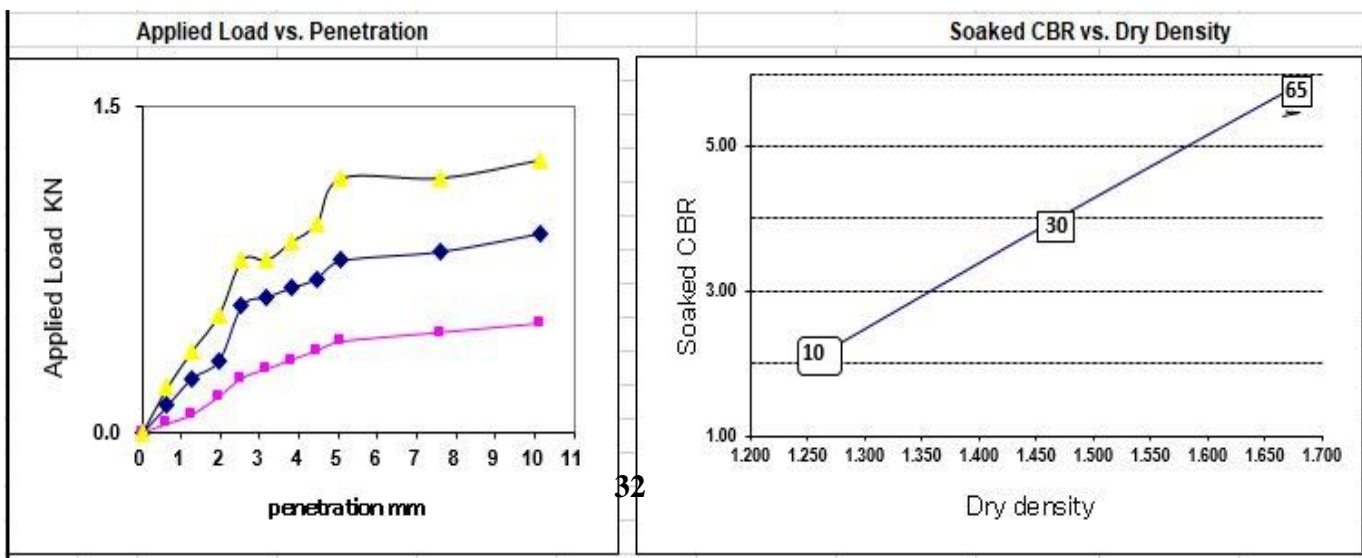


Figure 4.5: Load versus penetration curve

Table 4.9: Degree of Expansiveness based on CBR-SWELL

CBR-SWELL (%)	Classification of Potential
0 –2.5	Low
2.5 – 5	Medium
5 – 25	High
25+	Very high

As it is observed from the chart, the CBR-Swell of the subgrade soils are less than 3 % which is an indicator of low or non-expansiveness and hence suitable to be used as road bed.

The strength of the soil materials along the realigned road also determined. The 3 point CBR values at 95% of the Modified AASHTO Density have been interpolated from the CBR at densities obtained by compacting each layer in the mould at 10, 30 & 65 blows. As it is observed from the CBR test result, the CBR-Swell of the subgrade soils are less than 3 % which is an indicator of low or non-expansiveness. The native subgrade material is highly erodible as it consists of silty soil. It is the erodability nature of the soil that resulted in the failure of the road.

#### **4.2.5. Direct Shear Test**

One of the most important and the most controversial engineering properties of soil is its shear strength or ability to resist sliding along internal surfaces within a mass. The stability of a cut, the slope of an earth dam, the foundations of structures, the natural slopes of hillsides and other structures built on soil depend upon the shearing resistance offered by the soil along the probable surfaces of slippage.

In the direct shear test, a sample of soil is placed into the shear box. The size of the box used is 6 x 6 cm and the sample is 2.5 cm thick. The soil is used for the test is undisturbed samples. After the specimen is placed in the box, all the other necessary

adjustments are made, a known normal load is applied. Then a shearing force is applied. The normal load is held constant throughout the test but the shearing force is applied at a constant rate of strain (which will be explained later on). The shearing displacement is recorded by a dial gauge. Dividing the normal load and the maximum applied shearing force by the cross-sectional area of the specimen at the shear plane gives respectively the unit normal pressure and the shearing strengths at failure of the sample. These results may be plotted on a shearing diagram where  $\sigma$  is the abscissa and  $\tau$  the ordinate. The result of a single test establishes one point on the graph representing the Coulomb formula for shearing strength. However, it is the usual practice to draw the best straight line through the test points to establish the Coulomb Law. The slope of the line gives the angle of shearing resistance and the intercept on the ordinate gives the apparent cohesion.

The strength of a soil depends of its resistance to shearing stresses. It is made up of basically the components;

1. Frictional – due to friction between individual particles.
2. Cohesive - due to adhesion between the soil particles the two components are combined in Coulomb's shear strength equation,

$$\tau_f = c + \sigma_f \tan \phi \dots\dots\dots \text{equation 4.3}$$

Where  $\tau_f$  = shearing resistance of soil at failure  $c$  = apparent cohesion of soil  $\sigma_f$  = total normal stress on failure plane  $\phi$  = angle of shearing resistance of soil (angle of internal friction)

For cohesionless soil  $c = 0$ , then Coulomb's equation becomes

$$\tau_f = \sigma_f \tan \phi \dots\dots\dots \text{equation 4.4}$$

In Coulomb's equation  $c$  and  $\phi$  are empirical parameters, the values of which for any soil depend upon several factors; the most important of these are:

- ❖ The past history of the soil.
- ❖ The initial state of the soil, i.e., whether it is saturated or unsaturated.
- ❖ The permeability characteristics of the soil.

The conditions of drainage allowed taking place during the test.

Table 4.10 Direct Shear Test Result of soil sample no 1

Sample		1	
Depth (m)		1.70	
Moisture content (%)		10.8	
Bulk density (kg/m <sup>3</sup> )		1621	
Dry density (kg/m <sup>3</sup> )		1463	
Normal stress	Shear stress	C=Cohesion (KN/m <sup>2</sup> )	Angle of internal friction (Degrees)
100	18	3	11
200	33		
300	58		

Horizontal dial (mm)	Shear Dial	Shear Dial	Shear dial
	100Kpa	200 Kpa	300 Kpa
0	0	0	0
0.2	2	4	7
0.4	5	7	12
0.6	6	8	15
0.8	8	9	19
1	10	10	25
1.2	11	12	29
1.4	12	15	33
1.6	15	18	36
1.8	19	22	39
2	21	25	40
2.2	21	26	46
2.4	22	29	49
2.6	23	31	55
2.8	24	34	56
3	25	36	60
3.2	25	39	63
3.4	25	41	65
3.6	23	42	68
3.8	20	45	71
4	21	45	75
4.2		46	79
4.4		46	80
4.6		42	80
4.8		40	76

5		38	74
5.2			72

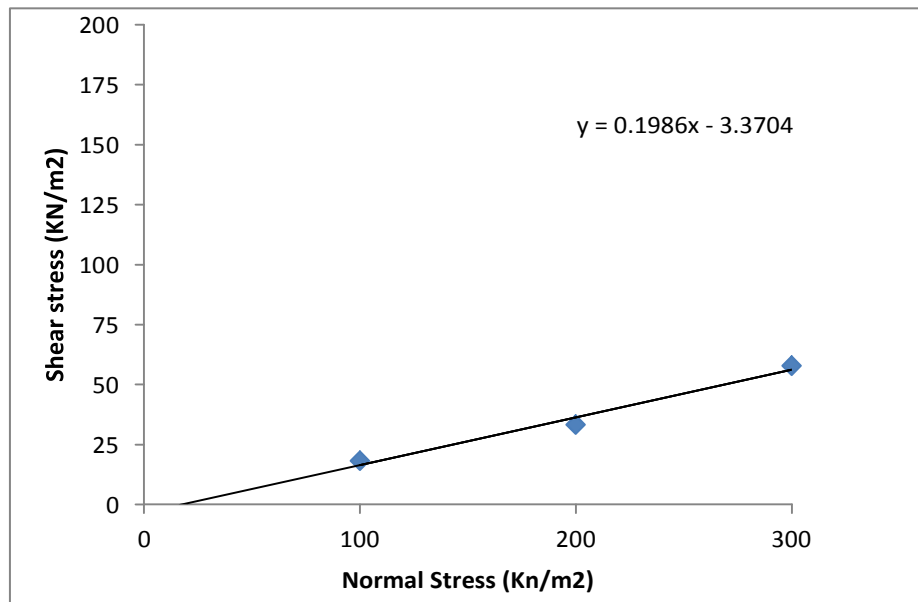


Figure 4.6: Direct Shear graph for soil sample no 1

Table 4.11 Direct Shear Test Result sample no 2

Sample		2	
Depth (m)		4.00	
Moisture content (%)		10.2	
Bulk density (kg/m <sup>3</sup> )		1614	
Dry density (kg/m <sup>3</sup> )		1465	
Normal stress	Shear stress	C=Cohesion (KN/m <sup>2</sup> )	Angle of internal friction ( Degrees)
100	22	4	12
200	33		
300	65		

Horizontal dial (mm)	Shear Dial	Shear Dial	Shear dial	
	100Kpa	200 Kpa	300 Kpa	
0	0	0	0	
0.2	2	3	6	
0.4	3	5	7	
0.6	5	6	10	
0.8	8	8	15	
1	9	10	19	
1.2	10.2	11	25	
1.4	12.3	11.3	30	
1.6	12.5	15	35	
1.8	15	18	39	
2	18	21	45	
2.2	21	22	49	
2.4	25	23	53	
2.6	26	25	56	
2.8	26.3	27.3	63	
3	26.4	28.2	66	
3.2	28	29	69	
3.4	28.4	30	75	
3.6	29.3	35	76	
3.8	29.3	36	79	
4	30	39	82	



4.2	30	40	85	
4.4	29	42	89	
4.6	28	42	90	
4.8	27	45	90	
5	25	45	88	
5.2		43	86	

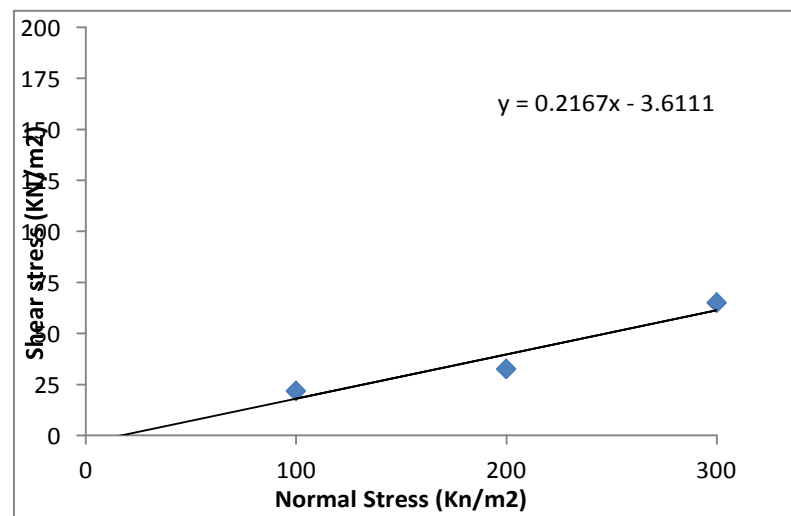


Figure 4.7: Direct Shear graph for soil sample no 2

Table 4.12 Direct Shear Test Result sample no 3

Sample		3	
Depth (m)		8.00	
Moisture content (%)		11.3	
Bulk density (kg/m3)		1598	
Dry density (kg/m3)		1436	
Normal stress	Shear stress	C=Cohesion (KN/m2)	Angle of internal friction
100	26	4	11
200	35		
300	64		
Horizontal dial (mm)	Shear Dial	Shear Dial	Shear dial
	100Kpa	200 Kpa	300 Kpa

0	0	0	0
0.2	3	7	9
0.4	4	9	13
0.6	6	10	16
0.8	9	12	18
1	11	15	25
1.2	14	19	29
1.4	17	25	33
1.6	19	28	38
1.8	25	30	42
2	26	31	49
2.2	26	34	56
2.4	26	36	59
2.6	25	36	66
2.8	23	38	68
3	20	39	70
3.2	21	40	72
3.4		43	76
3.6		46	78
3.8		48	81
4		49	85
4.2		49	87
4.4		48	88
4.6		46	88
4.8		43	86
5		40	82
5.2			<b>78</b>

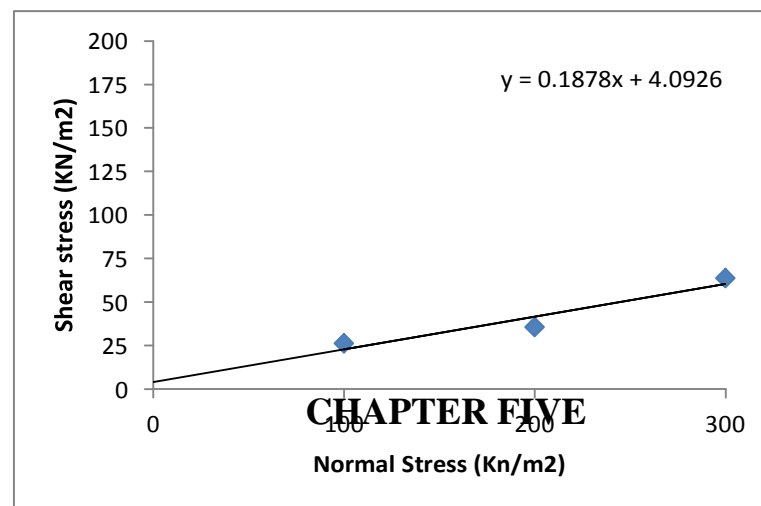


Figure 4.8: Direct Shear graph for soil sample no 3

The tested samples recorded reasonably low values of cohesion ( $c$ ) with moderately high values of angle of shearing resistance ( $\phi$ ). Low  $c$  could result from the silty nature of the soil, while the high  $\phi$  is attributable to cohesionless nature of the soil.

## CHAPTER FIVE

### CONCLUSION AND RECOMMENDATION

#### 5.1. Conclusion

Pavement deterioration is the process by which distress develop in the pavement under the combined effects of traffic loading and environmental conditions. Some Causes of Road failure is due to poor construction quality, Soil problem, poor highway facilities, Poor maintenance policy, Poor supervision and others. Based on the findings the following conclusions were drawn on the selected road route:

- ❖ The failure in the study area was initiated by improper termination of the storm drain of the area and concentrated runoff from the road system to the poor erosion resistance soil.
- ❖ The abrupt formation and expansion of the gully is due to the absence of appropriate side drainage facilities along the road side alignment.
- ❖ The results obtained in this investigation so far, (Atterberg limit, sieve analysis, compaction test and CBR) show that the soils in the study area are cohesionless, loose, erosive and non-plastic; hence the area is highly susceptible to water erosion
- ❖ Laboratory test results revealed that 62% of the subgrade soils of the alignment road indicate Silt soil (A-4 and A-5 groups). The remaining 38% indicate A-7-5 soil group which are low in plasticity but fine and  $LL > 32\%$ , maximum dry density range from 1.51 to 1.63 g/cm<sup>3</sup>, optimum moisture content ranges from 11.4 to 17.4, soaked California Bearing Ratio of 4.6 and %swell of 2.97 and direct shear implying that Angle of internal friction ranges 11 to 12 and apparent cohesion of 3 to 4 this indicates that the samples are uniform silty-clay soil and susceptible to water erosion.
- ❖ The gully erosion along the study area has depth varying from 1.7 m to 12m, width varying from 1m to 12m and length has varying from 150m to 1600m.
- ❖ According to the gully classification, most of the erosion gully sites are classified as medium to large.

- ❖ Most of the identified gullies are very close to the existing asphalt road and thus, it is dangerous for the existing road.
- ❖ Biophysical mitigation and structural measure is recommended to control gully erosion hazard in the area early detection and repair of road defects are important to maintain the permanence of road.

## **5.2. Recommendation**

The road failure should be controlled and stabilized with properly designed mitigation measure so that it reduces expansion rate. Hence, careful attention in selection and designing should be given. Arising from the conclusion the following recommendations can be made;

- ❖ It is recommended that, the erosion gully along the study area should be mitigated by doing a model which is appropriate for slope stability by using structural measure like gabion and retaining structure.
- ❖ Filling the gully with high plastic reddish clay ash which is abundantly available near to the study area i.e., around Iteya town. Thus it is economical than any other stabilization measure.
- ❖ Proper termination of road side drainage structures (concrete or masonry ditch) should be provided, this may convey an adequate storm drain from carriageways and other areas. Ditches shall be kept free of silt, debris, large amounts of vegetation, or any other material that restricts the flow of water. A complete cleaning shall be carried out at least once every year.
- ❖ Continuous periodic inspection of roads.
- ❖ Biophysical mitigation and structural measure is recommended to control gully erosion hazard in the area early detection and repair of road defects are important to maintain the permanence of road.
- ❖ Filling gullies by high plastic clay soil, Road side ditch (concrete or Masonry side ditch), Vegetative control measures
- ❖ The Suggested maintenance for the cracks and failure in the road are Crack seal (fill the crack) to prevent moisture entering into subgrade through the cracks,

Improve drainage system, Reconstruction of the edge and support the edge with paving stones and patching.

Thus further study is required on Hydrological investigation. It's the basic step that should be done carefully in every flood management and road drainage facilities design. The hydrological study was necessary in order to estimate the design discharge for Addama-Assela road section which require urgent rehabilitation measures based on the site investigation. Calculation of these peak discharge values helps in the determination appropriate remedial measure design that makes the road and surrounding area safe from prevailing flooding problem. The design floods are estimated using both HEC-HMS hydrologic model software and spread sheet program that developed to estimate the design flood using SCS method for road section Hydrological investigation is necessary and new research on the design floods are Gap for further study

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