Jimma University

School of Graduate Studies

Jimma Institute of Technology

School of Civil and Environmental Engineering

Civil Engineering Department

Highway Engineering Stream

Causes of Flexible Pavement Deterioration and Its Remedial Measures: A Case Study Bako to Nekemte Road section.

A Thesis Submitted to School of Graduate Studies of Jimma University, in Partial Fulfillment of the Requirements for Degree of Masters of Science in Civil Engineering (Highway Engineering Stream).

By

Dessalegn Geleta

October, 2016

Jimma, Ethiopia

Jimma University

School of Graduate Studies

Jimma Institute of Technology

School of Civil and Environmental Engineering

Civil Engineering Department

Highway Engineering Stream

Causes of Flexible Pavement Deterioration and Its Remedial Measures: A Case Study Bako to Nekemte Road section.

A Final thesis Submitted To School of Graduate Studies of Jimma University, in Partial Fulfillment of the Requirements for Degree of Masters of Science in Civil Engineering (Highway Engineering Stream).

By

Dessalegn Geleta

Main advisor: Prof. Emer T. Quezon

Co-advisor: Engr. Anteneh Geremew

October, 2016

Jimma, Ethiopia

DECLARATION

I, the undersigned, declare that this thesis entitled "**Causes of Flexible Pavement Deterioration and Its Remedial Measures: A Case Study Bako to Nekemte Road section**.." 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 dually acknowledged

Candidate:

Mr. DESSALEGN GELETA

Signature______________________

As Master research Advisors, we hereby certify that we have read and evaluate this MSc research prepared under our guidance, by Mr. Dessalegn Geleta entitled: **Causes of Flexible Pavement Deterioration and Its Remedial Measures: A Case Study Bako to Nekemte Road section**. We recommend that it can be submitted as fulfilling the MSc Thesis requirements

Prof. Emer T. Quezon Main Advisor	_____________ Signature	_____________ . Date
Engr. Anteneh Geremew Co-Advisor	_____________ Signature	_____________ Date

DEDICATION

This Thesis Is Dedicated To My Advisor and Family.

"ONLY WHEN YOU CLIMB THE HIGHEST MOUNTAIN, WILL YOU BE AWARE OF THE VASTNESS THAT LIES AROUND YOU"…..OSCAR WILDE.

ACKNOWLEDGMENT

Firstly, I would like to thank almighty God, for this blessed opportunity.

I would like to express my sincere gratitude to my advisor Prof. Emer T. Quezon, the advisor of this thesis, for his valuable feedback and guidance throughout the thesis work and his unlimited support. My special thanks also go to my co-advisor Engr. Anteneh Geremew for his support and encouragement.

I would also like to use this opportunity to convey my gratitude to the Ethiopian Roads Authority, academic staff of the Department of Civil Engineering and Jimma Institute of Technology, Jimma University without their support and encouragement I couldn't have this opportunity to complete my study.

Also, I wish to express my sincere gratitude to all who have contributed directly or indirectly to the success of this thesis work without whose constant support and help, the study would never have been accomplished on time. Those, Ethiopian Roads Authority Nekemte District laboratory, Ethiopian road construction corporation (and Mr. Mengistu Tolasa (material engineer)) and professionals of the sector who have participated in this research sacrificing their time are on the front line to receive my appreciation.

Finally, my acknowledgement also goes to my families and to my friends for their unconditional love, constant encouragement, and sincere care as they always did all through my life.

ABSTRACT

Road constructing in Ethiopia is increasingly in demand to meet its medium and long term development programs. Roads are constructed radiating from the metropolis of Addis Ababa towards the Western direction. Some portions along the alignment proposed and existing roads traversed low resistance of the subgrade that affect the stability of upper layers of pavements. Structural deteriorations are observed on Flexible Pavements, would it be constructed by good quality or low quality of materials.

Flexible pavement roads in Ethiopia often deteriorate in different ways, because of the harsh climatic conditions, lack of proper design and quality control, sudden increasing of traffic due to the construction of different industries, high loads and inadequate assessment for identifying causes of deteriorate before carrying out Maintenance and rehabilitation.

Bako to Nekemte Project which was completed in 2013 observed there were deterioration that needs to be addressed and a corresponding remedial measures must be drawn. Hence, a desk study, letter correspondence review, field visual inspection and in-depth field investigation have been undertaken to identify the causes of flexible Pavement deterioration. A possible remedial measures had been organized for every observed failure or deterioration to obtain normal pavement condition of the study area.

An investigation was made by using laboratory and field test to determine the adequacy of the underlain material to serve as a subgrade, sub base course and base course for road construction based on project specifications and Ethiopian Road Authority (ERA) Specification. The investigation covered field tests to determine the in-situ condition of the road materials while laboratory tests on representative samples were extracted from site to determine the engineering properties of soil materials and to compare test results with the existing conditions of pavement layers.

From the field tests and laboratory tests carried out, it is observed that the causes of flexible pavement deterioration of this road section are mainly due to the inadequacy of existing thickness of the base layer for the current traffic loading, engineering properties of the pavement layers material, poor method of construction, poor design quality and lack of side drainage. To reduce more damage for this road section reconstruction of the base and providing proper drainage at the deteriorated sections must be required.

Raveling, pumping, potholes, edge cracking and rutting are the most dominant type of deteriorations along Bako-Nekemte road section. It is expected that the results of this research will provide useful information about all causes of flexible pavement deteriorations and its remedial measures for the Bako-Nekemte road section.

Key words: *pavement, deterioration, Remedial, laboratory*

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

APPENDICES

ACRONYMS

ASTM	American Society for Testing and Materials
AASHTO	American Association of State Highway and Transportation Officials
AC	Asphalt Concrete
a.s.l	above sea level
CBR	California Bearing Ratio
ERA	Ethiopian Road Authority
ESA	Equivalent Standard Axle
EIRR	Economic Internal Rate of Return
GS1	Crushed Stone Sub-base
GB1	Crushed base course
GS2	Granular Sub base
GC	Granular capping layer
Gs	specific gravity
HMA	Hot Mix Asphalt
MDD	Maximum Dry Density
OMC	Optimum Moisture Content
TRL	Transport Research Laboratory
S4	Subgrade Class four
S	Subgrade Class
W	moisture content
USCS	Unified Soil Classification System

CHAPTER ONE

INTRODUCTION

1.1 Background

A road pavement is a structure of superimposed layers of selected and processed materials that are placed on the basement soil or subgrade. Flexible pavements constructed for heavy duty vehicles are composed of asphaltic layers (wearing, binder and base courses) and sound sub base layer which laid over a well compacted and strong subgrade foundation. The main structural functions of a pavement are to support the wheel loads applied to the carriageway and ultimately distribute them to the underlying subgrade layer [20]. The pavement is designed and constructed based on the most economical combination of layers that guarantees adequate dispersion of the incident wheel stresses so that each layer in the pavement does not become overstressed during the design life of the highway [16].

Roads have helped in developing cultural and social among the people by transporting them from one corner to the other corner of the country. Thus, we see that progress made by any country and well-being of a nation depends much on road facilities. Moreover, the development, civilization and efficiency of a nation can be easily judged by the extent of its roads facilities.

A flexible pavement is one of the largest infrastructure components in civil construction works and complex system that involves multiple layers of different materials subjected to various combinations of irregular traffic loadings and varying environmental conditions. It provides the road-user a smooth, quiet and skid-resistant riding surface, maximizes tire contact by providing more traction, and saves wear and tear on vehicles. Flexible pavements are safe, economical and most long-lasting roads that can be built very quickly thereby reducing costs due to traffic delays and save the traveling time. In addition, asphalt pavements are purely recyclable that old flexible pavements can be reprocessed, remixed with a portion of fresh materials and used again and again [32].

If we look back to Ethiopian history and briefly try to visualize the genesis of road construction works, we note that some roads and bridges were constructed in early times [23]. Emperors during their royal trips, used to exert efforts to make the roads suitable for their trips by having forests cleared and difficult terrains leveled.

Roads, in Ethiopia, are significant and potential means of transporting human and material over both short and long distance in the country. Roads have also helped in developing cultural and social ties among the people by transporting them from one corner to the other corner of the country. Thus, we see that progress made by any country and well-being of a nation depends much on road facilities. Moreover, the development, civilization and efficiency of a nation can be easily judged by the extent of its roads facilities. Most of the construction activities were used for protecting the country from the internal invasion and for showing the strength of the ruling dynasty. Additionally, road construction work, just like any other social undertaking, has vast and wonderful history of its own, since it evolved with the social development of mankind[23].

They were Emperors that ought to be mentioned in line with those that played a significant role in this constructive field of endeavor in early Ethiopia, are Emperors such as Emperor Fasil, Tewodros, Yohannes and Menelik. The main reason that prompted these Emperors to give emphasis to the need for development of road construction was because of their appreciation of the value that could be gained from the expansion of territory and from ensuring the unity of the nation. For the last twenty years in Ethiopia, with the increases of populations and demands of public customers, highways have been actively and continually constructed or rehabilitated or maintained. As result, the country‘s road network has increased from 26,550 kms in 1997 to 99,522 kms in 2014 Gc. Since an enhanced transportation infrastructure system would sustain the causes of the deterioration and remedies development [23].

In fact, many flexible pavements in Ethiopia require surface repairs or periodic maintenance every three to five years. The longer service life of rigid pavements has an economic impact because it requires fewer repairs, fewer materials, and fewer construction zones requiring traffic control etc. In most cases, the rigid structure of concrete pavement requires less than half the aggregate base used in an asphalt structure. This minimizes the amount of non-renewable resources being consumed for highway construction. Understanding the value of a rigid pavement structure and its performance over 40 years or more, including ride ability, rehabilitation costs, user delay costs, and life cycle costing tools to evaluate system costs have been developed and refined.

Different researchers are conducted that both traffic volume and loads on roads are going on increasing from year to year with alarming rate all over the world. Such heavy traffic growth demands better performance roads for efficient transport of commercial and industrial products without delay. The repetitive traffic loading that the road experiences during its service life combined with environmental factors causes deformation, fatigue cracking, instability and other forms of deterioration which ultimately degrade the serviceability and durability of pavement structures [25].

Flexible pavements have helped for the economic and industrial development of first world nations. More than 85% of road networks in the world are asphalt pavements. However, since the construction of asphalt pavements started in the early 20th century, it has also been experiencing a number of problems. One such major problem is a perception and arguably a reality that asphalt pavements were not being designed and constructed to sufficiently long lasting and cost effective. At the same time, there is a growing backlog of needs due to road deterioration. This deterioration includes such problems as early cracking, rutting and moisture damage. The pavement industry has invested millions of dollars in search for remedies to pavement distress [31].

Bako – Nekemte road project was signed between the Ethiopian Roads Authority and China Hyway Group Limited on 29 July 2009. The construction works of the project commenced on 10th October 2009 on receipt of the notice to commence by the Contractor from the Engineer. However, the commencement date has been adjusted to 21st January 2010 which has resulted in a revised completion time of 21st July 2012 (1015 calendar days of the revised Contract period plus 365 calendar days of the Defects Liability Period). .

The scope of works is upgrading of the existing Double Bituminous Surfaced Road to a DS3 Asphalt Concrete Road. The designed road is completed, which have a total length of 64.814 kilometers with a two lane 7-m wide AC surfaced carriageway and 1.5-m SBST shoulder on either side of the carriageway in rural areas and 13 to 14-m wide dual carriageway plus 2 to 2.5-m wide footpath in town sections.

The Bako-Nekemte road project was one of the main road projects included in the Ethiopian Government's 10 Years Road Sector Development Programmed (1997- 2007). Which was formulated to improve and expand the country's road network, and which has been accorded high priority by the Government of Ethiopia in order to stimulate growth of the economy and for the long term development of the country [27].

Figure 1.1 different deteriorations types which are exist along Bako-Nekemte currently

1.2 Statement of the problem

Pavement is an engineering structure placed on natural soils and designed to withstand the traffic loading and the action of the climate with minimal deterioration and in the most economical way [17]. Asphalt pavement roads are designed and constructed to serve the upcoming traffic that reveal during the service life of the road. Different factors taken in to account in the design and construction of flexible pavements include the characteristics of the traffic, climatic conditions, material as well as structural properties and other elements which have significant impact on the overall performance of the road [25].

Recently constructed roads in Ethiopia often deteriorate in different ways directly after opening to traffic because of many reasons. The deterioration, that may affect pavement roads, is fatigue cracking, potholes, corrugations, etc. These deteriorations affect the safety and riding quality on the pavement as they may lead to premature failure and traffic hazards. Identifying the causes of deterioration before carrying out maintenance and rehabilitation is very important task for wise use of resources and save country's budget.

The pavement can no longer absorb and transmit the wheel loading through the road structure when the layers have failed for various reasons, as through aging, inadequate design, poor construction and maintenance practices, low bearing capacity of the materials or the gradual degradation of the strength of the road due to increased traffic flow and load or due severe climatic conditions, decreasing subgrade strength in conjunction with inadequate surface/subsurface drainage facilities and so on [16]. Every vehicle, which passes over a road, causes a momentary, very small, but significant deformation of the road pavement structure. The passage of many vehicles has a cumulative effect, which gradually leads to permanent deformation and road surface deterioration [17].

The cost savings associated with knowing causes of flexible pavement deterioration may take some forms including reduced construction costs, reduction of maintenance costs throughout the life of the pavement or life extension of the normal condition of pavement. Thus the highway engineer tries to construct pavement without a problem that may last for its intended design life within the optimum economic cost. It is difficult to do this truly, but finding the causes of recently constructed pavement deterioration is the best to overcome.

The Road project of the Bako-Nekemte road was expected that make major contribution to sustainable development of the country, in particular, the project area which is considered for the growing of agricultural land. The existing asphalt road has shown signs of distress and it could not provide efficient service throughout its projected level of traffic. It is expected that the road will improve the level of transportation services to the rural areas and to support transporting of agricultural products, distribution of inputs, improved social services and reduce vehicle operating costs.

Unfortunately, Bako-Nekemte Road project is now experiencing early deterioration of the pavement and it is observable several stretches has already been deteriorated. The pavement defects may create both direct and indirect impacts to the travelling public. The direct impacts are the cost of maintenance on the deteriorated portions. Likewise, indirect impacts would greatly affect the project benefits which include reduction of vehicle operating costs, travel time to road users, access to markets, social and economic facilities and promotion of market integration between the rural and urban areas of Ethiopia in support of poverty reduction program of the government [30].

Many studies about causes of pavement deterioration focus on old pavement deterioration but the causes of deterioration of recently constructed pavement are equally important these days to avoid those maintenance cost and reconstruction.

Therefore the situational assessment should cover different causes and remedial measures of the recently constructed flexible asphalt pavement deterioration because these activities are not well developed in Ethiopia.

1.3 Research Questions

1. What are the types of deterioration of the flexible pavement of the road section?

2. What are the level severities of the deterioration of the road section?

3. What are the engineering properties of the flexible pavement layer of road section?

4. What is possible option or remedies for this deterioration of flexible pavement of the road section?

1.4 Objectives of the research

1.4.1 General objective

The General Objective of this research was to investigate the major causes and remedies of the flexible asphalt pavement deterioration of Bako-Nekemte road section.

1.4.2 Specific objective

To identify type of deterioration of the road section.

To determine level of severity of the deterioration of the road section.

To identify the engineering properties and to compare laboratory test results of the pavement layers with the Standard specifications.

To recommend possible remedial measures in order to correct pavement deteriorations for the study area.

1.5 Significance of the study

One of the main objectives of this study is evaluation of the causes of asphalt pavement deterioration, finding the possible solution with respect to the criteria, and then recommends remedial actions at Bako to Nekemte road section. And also it would have been a significance of pointing the problem of the deterioration of the recently constructed asphalt pavement. Generally it is best to know the major causes of the asphalt pavement deterioration without its design period. It is for this reason that the researcher had arrived with the following significance of the study:

1) The identified types of distress, possible causes of deterioration and remedial measures on asphalt concrete pavement would be of great helped in comparing with the Standard Specifications and ERA Manuals.

2) In the future, it could be used as reference material for related post graduate studies.

3) It could be served as an aid to users on how to maintain the asphalt pavement properly

4) Proper understanding the types of distress and possible causes of damage on asphalt pavement may lead to correct application of remedial measures.

5) Feasibility of appropriate remedial measure and proactive approach like "right activity at the right place, and at the right time". This could be helpful for the concerned agency to maintain properly the pavement in order to reduce the maintenance cost and vehicle operating costs.

1.6 Scope of the Study

The study of this research would cover the major determination of the causes and remedies of the flexible pavement deterioration layers **excluding asphalt** layer based on its type of major causes. It is possible to full fill the gap between deterioration of recently constructed roads and with its design period by minimizing the factors that affect the performance of asphalt pavement and considering the present condition. The scope of the study along Bako-Nekemte road rehabilitation comprised the following main tasks in identification of deterioration severity, deterioration types and causes of flexible pavement deterioration. Tests are conducted and analyzed the results and compared with project specification and ERA Standard Specifications:

(a) Identification of level of severity (low, medium , and high deterioration) and types of deterioration.

(b) Preparation of representative samples for laboratory tests

(c) Identification of exiting causes of deterioration on flexible pavement by conducting field and laboratory tests.

(d) Comparison on the results of calculations with ERA Standard Specifications.

(e) Conclusions would be developed based on the comparisons.

(f) Recommended appropriate remedial measures.

1.7 Structure of the thesis

This research study comprised of five chapters and their contents are outlined below:

In the first chapter an overview of the background of the research, statement of the problem, research questions, scope and the final objective of the thesis work were discussed. The second chapter deals with the literature review about site characterization of pavement materials and organization of subgrade, capping, subbase course and base course investigated. A discussion was made about pavement materials especially subgrade materials related to subgrade strength and finally about the pavement deterioration types. The third chapter deals with the materials and methods. The fourth chapter deals with assessments of test results that are gathered from field and laboratory tests, whether it satisfies the requirements set in the design specification of the project. Furthermore, a comparison was made between the quality requirements established for pavement material in the project specification to that of the requirements set for the same in the ERA’s specification.

Generally this chapter deals with the results, proposed remedial measures and discussions. Finally chapter Five, a conclusions and recommended remedial measures are derived based on the results of chapter four.

CHAPTER TWO

LITERATURE REVIEW

2.1 Introduction

Pavement deterioration is permanent deformation of any part of the pavement structure. Deterioration of highway pavement is a very serious problem that causes unnecessary delay in traffic flow, distorts pavement aesthetics, damages of vehicle and most significantly, causes road traffic accident that had resulted into loss of lives and properties, [2].

Pavement surface deformation affects the safety and riding quality on the pavement as it may lead to premature failures. A variety of factors contribute to pavement deterioration were investigated by many researchers [3]. On [3], some of the factors that cause highway failure have been identified. They include poor design, construction and maintenance, use of low quality construction materials, poor workmanship and poor supervision of construction work and the applying of heavy traffic that were not meant for the road. Furthermore, he also suggested that the following will lead to highway failure; poor highway facilities, no knowledge base, in adequate sanction for highway failure, no local standard of practice, poor laboratory and in-situ tests on soil and weak local professional bodies in highway design, construction and management. The most significant road defects observed in the field are potholes; cracks edge defects, Depressions and corrugation [4]. At the same time he emphasized that traffic overloading, Pavement age, road geometry, weather, drainage, construction quality as well as construction Materials, maintenance policy play the major role as road deteriorate agents. However, Understanding the causes for pavement deterioration failures is essential step towards minimizing risks to have good road performance. An intensive literature of the major factors that may lead to pavement deterioration will be reviewed in the following sections.

2.2 Road surface deterioration

Considering remedial measures for defects on reconstruction or overlay, it is imperative that the engineer takes into account, various parameters that are necessary for proper evaluation of the existing pavement condition. In particular, it is important to ascertain whether certain types of pavement distress are progressive, leading to eventual failure of the road, or whether they are non-progressive. Excessive movement of flexible pavements, which eventually result in uneven riding qualities, may mostly be caused by poor qualities of the sub grade, sub base, base course or wearing course and due to improper drainage system. A qualitative measure of the effect of the movement can be determined only after a detailed investigation had been undertaken. The investigations might take the form of trenching or bore logging in which visual inspection was made on the cross section of the pavement structure. Measurements of the thickness and analysis of the structural thickness of the various paving layers inside and outside the traffic lanes is certainly vital. Testing of various pavement components had been considered a great helped in the evaluation of the probable cause of distress. Each distress must be evaluated to determine whether the distress will be progressive or whether it represents an inactive condition.

Failed surfaces could be classified into different categories depending on the patterns of failures. The following section provides basic information on the most common types of pavement failures and their probable cause [11].

2.2.1 Flexible pavement deterioration

Flexible Pavement deterioration is the process by which distresses develop in pavement under the combined effects of traffic loading and environmental conditions. Deterioration of pavement greatly affects serviceability, safety, comfort and riding quality of the road. After construction, roads deteriorate with age as a result of use and therefore, they need to be maintained to ensure that the requirements for safety, efficiency and durability are satisfied. Normally, new paved roads deteriorate very slowly in the first five to ten years of their life, and then go on to deteriorate much more rapidly unless timely maintenance is undertaken, [6].

In Ethiopia, recently constructed roads were reported to deteriorate rapidly after opened to traffic. These deteriorations were contributed to many reasons such as excessive loads, climatic changes, poor drainage, and low quality pavement materials. The most common road distresses are cracks, potholes, rutting, raveling, depressions, and damaged edges. These distresses affect the safety and riding quality on the pavement as they may lead to premature failure and traffic hazards. Before going into maintenance strategies, engineers must look into the causes of road deterioration. Therefore, this paper aims to identify the causes and remedies of pavement deterioration shortly after construction or rehabilitation of roads from Bako to Nekemte.

2.2.2 Types of pavement deterioration

There are four major categories of flexible asphalt pavement surface distress

1. Cracking
2. Surface deformation
3. Disintegration (potholes, etc.
4. Surface defect (bleeding, etc.)

1. Cracking

The most common types cracking are:

a. Fatigue cracking b. Longitudinal cracking c. Transverse cracking d. Block cracking e. Slippage cracking f. Reflective cracking g. Edge cracking

a. Fatigue cracking (alligator cracking)

Fatigue cracking is commonly called alligator cracking. This is a series of interconnected cracks creating small, irregular shaped pieces of pavement. It is caused by failure of the surface layer or base due to repeated Traffic loading (fatigue). Eventually the cracks lead to disintegration of the surface, as shown in Figure. The final result is potholes. Alligator cracking is usually associated with base or drainage problems. Small areas may be fixed with a patch or area repair. Larger areas require reclamation or reconstruction. Drainage must be carefully examined in all cases.

Figure 2.1 alligator crack

b. Longitudinal cracking

Longitudinal cracking are long cracks that run parallel to the center line of the roadway. These may be caused by frost heaving or joint failures or they may be load induced. Understanding the cause is critical to selecting the proper repair. Multiple parallel cracks may eventually form from the initial crack. This phenomenon, known as deterioration, is usually a sign that crack repairs are not the proper solution

Figure 2.2 Longitudinal Cracking

c. Transverse cracks

Transverse cracking form at approximately right angles to the centerline of the roadway. They are regularly spaced and have some of the same causes as longitudinal cracks. Transverse cracks will initially be widely spaced (over 20 feet apart). They usually begin as hairline or very narrow cracks and widen with age. If not properly sealed and maintained, secondary or multiple cracks develop, parallel to the initial crack. The reasons for transverse cracking, and the repairs, are similar to those for longitudinal cracking. In addition, thermal issues can lead to low-temperature cracking if the asphalt cement is hard [28].

Figure 2.3 Low severity transverse cracking

d. Block cracking

Block cracking is an interconnected series of cracks that divides the pavement into irregular pieces. This is sometimes the result of transverse and longitudinal cracks intersecting. They can also be due to lack of compaction during construction. Low severity block cracking may be repaired by a thin wearing course. As th e cracking gets more severe, overlays and recycling may be needed. If base problems are found, reclamation or reconstruction may be needed. Figure shows medium to high severity block cracking.

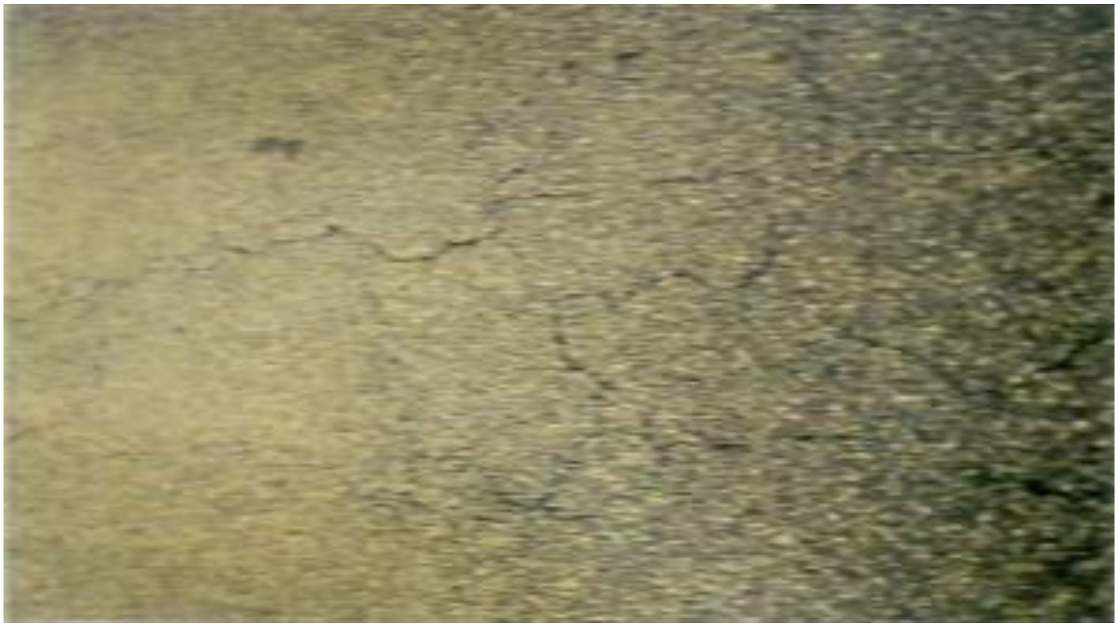


Figure 2.4 Block cracking

E. Slippage cracking

Slippage cracking are half-moon shaped cracks with both ends pointed towards the oncoming vehicles. They are created by the horizontal forces from traffic. They are usually a result of poor bonding between the asphalt surface layer and the layer below .The lack of a tack coat is a prime factor in many cases. Repair requires removal of the slipped area and repaving.

F. Reflective cracking

Reflective cracking occurs when a pavement is overlaid with hot mix asphalt concrete and cracks reflect up through the new surface. It is called reflective cracking because it reflects the crack pattern of the pavement structure below. As expected from the name, reflective cracks are actually covered over cracks reappearing in the surface. They can be repaired in similar techniques to the other cracking noted above. Before placing any overlays or wearing courses, cracks should be properly repaired.

G. Edge cracking

Edge cracking typically starts as crescent shapes at the edge of the pavement. They will expand from the edge until they begin to resemble alligator cracking. This type of cracking results from lack of support of the shoulder due to weak material or excess moisture. They may occur in a curbed section when subsurface water causes a weakness in the pavement. At low severity the cracks may be filled. As the severity increases, patches and replacement of distressed areas may be needed. In all cases, excess moisture should be eliminated, and the shoulders rebuilt with good materials. Figure shows high severity edge cracking

H. Surface deformation

Pavement deformation is the result of weakness in one or more layers of the pavement that has experienced Movement after construction. The deformation may be accompanied by cracking. Surface distortions can be a traffic hazard. The basic types of surface deformation are;

1. Rutting 2. Corrugation

3. Shoving 4.Depression 5. Swell

1. Rutting

Rutting is the displacement of pavement material that creates channels in the wheel path. Very severe rutting will actually hold water in the rut. Rutting is usually a failure in one or more layers in the pavement. The width of the rut is a sign of which layer has failed. A very narrow rut is usually a surface failure, while a wide one is indicative of a sub grade failure. Inadequate compaction can lead to rutting. Figure shows an example of rutting due to sub grade Failure. Minor surface rutting can be filled with micro paving or paver-placed surface treatments. Deeper ruts may be shimmed with a truing and leveling course, with an overlay placed over the shim. If the surface asphalt is unstable, recycling of the surface may be the best option. If the problem is in the sub grade layer, reclamation or reconstruction may be needed.

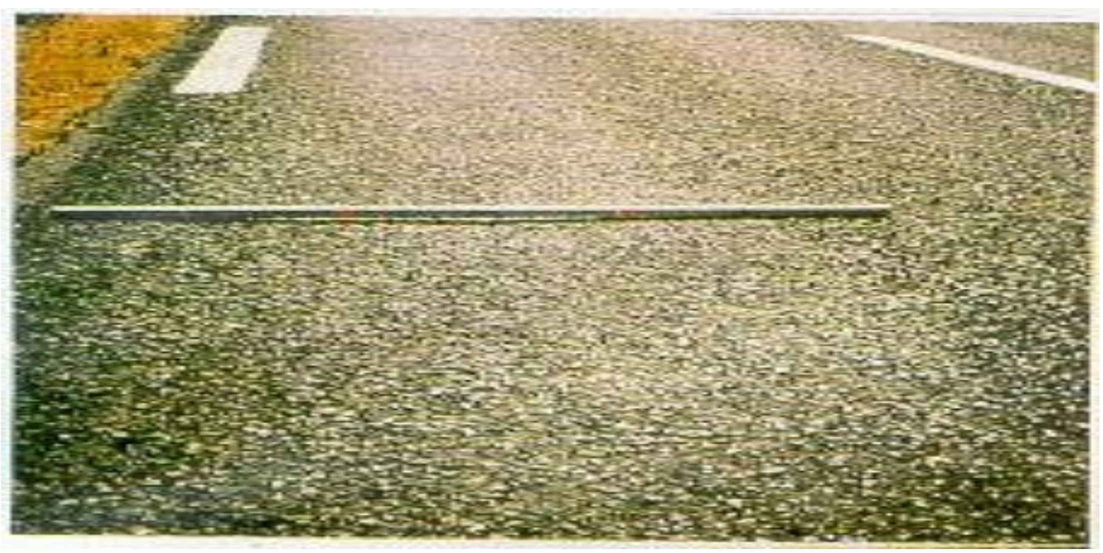


Figure 2.5 Rutting at wheel path

2. Corrugation

Corrugation is referred to as wash boarding because the pavement surface has become distorted like a Washboard. The instability of the asphalt concrete surface course may be caused by too much asphalt cement, too much fine aggregate, or rounded or smooth textured course aggregate. Corrugations usually occur at places where vehicles accelerate or decelerate. Minor corrugations can be repaired with an overlay or surface milling. Severe corrugations require a deeper milling before resurfacing.

3. Shoving

Shoving is also a form of plastic movement in the asphalt concrete surface layer that creates a localized bulging of the pavement. Locations and causes of shoving are similar to those for corrugations. Repair minor shoving by removing and replacing. For large areas, milling the surface may be required, followed by an overlay.

4. Depressions

Depressions are small, localized bowl-shaped areas that may include cracking. Depressions cause roughness, are a hazard to motorists, and allow water to collect. Depressions are typically caused by localized consolidation or movement of the supporting layers beneath the surface course due to instability. Repair by excavating and rebuilding the localized depressions Reconstruction is required for extensive depressions

5 swell

Swell is a localized upward bulge on the pavement surface. Swells are caused by an expansion of the supporting layers beneath the surface course or the sub grade. The expansion is typically caused by frost heaving or by moisture. Sub grades with highly plastic clays can swell in a manner similar to frost heaves (but usually in warmer months). Repair swells by excavating the inferior sub grade material and rebuilding the removed area. Reconstruction may be required for extensive swelling.

6. Disintegration

The progressive breaking up of the pavement into small, loose pieces is called disintegration. If the disintegration is not repaired in its early stages, complete reconstruction of the pavement may be needed. The two most common types of disintegration are: Potholes and patches.

1. Potholes

Potholes are bowl-shaped holes similar to depressions. They are a progressive failure. First, small fragments of the top layer are dislodged. Over time, the distress will progress downward into the lower layers of the pavement. Potholes are often located in areas of poor drainage, as seen in Figure. Potholes are formed when the pavement disintegrates under traffic loading, due to inadequate strength in one or more layers of the pavement, usually accompanied by the presence of water. Most potholes would not occur if the root cause was repaired before development of the pothole. Repair by excavating and rebuilding. Area repairs or reconstruction may be required for extensive potholes.

Figure 2.6 Potholes caused by poor drainage

2. Patch

Patch is defined as a portion of the pavement that has been removed and replaced. Patches are usually used to repair defects in a pavement or to cover a utility trench. Patch failure can lead to a more widespread failure of the surrounding pavement. Some people do not consider patches as a pavement defect. While this should be true for high quality patches as is done in a semi-permanent patch, the throw and roll patch is just a cover. The underlying cause is still under the pothole. To repair a patch, a semi-permanent patch should be placed.

Extensive potholes may lead to area repairs or reclamation. Reconstruction is only needed if base problems are the root source of the potholes.

4. Surface defects

Surface defects are related to problems in the surface layer. The most common types of surface distress are: 1 raveling 2.Bleeding 3.Polishing or Delimitation

1. Raveling

Raveling is the loss of material from the pavement surface. It is a result of insufficient adhesion between the asphalt cement and the aggregate. Initially, fine aggregate breaks loose and leave small, rough patches in the surface of the pavement. As the disintegration continues, larger aggregate breaks loose, leaving rougher surfaces. Raveling can be accelerated by traffic and freezing weather. Some raveling in chip seals is due to improper construction technique. This can also lead to bleeding. Repair the problem with a wearing course or an overlay.

Figure 2.7 High severities raveling of asphalt surface

2. Bleeding

Bleeding is defined as the presence of excess asphalt on the road surface which creates patches of asphalt cement. Excessive asphalt cement reduces the skid-resistance of a pavement, and it can become very slippery when wet, creating a safety hazard. This is caused by an excessively high asphalt cement content in the mix, using an asphalt cement with too low a viscosity (too flow able), too heavy a prime or tack coat, or an improperly applied seal coat. Bleeding occurs more often in hot weather when the asphalt cement is less viscous (more flow able) and the traffic forces the asphalt to the surface. Figure 13 shows an example of bleeding during hot weather

Figure 2.8 Bleeding

3. Polishing

Polishing is the wearing of aggregate on the pavement surface due to traffic. It can result in a dangerous low friction surface. A thin wearing course will repair the surface.

2.3 Causes of pavement deterioration

2.3.1 Pavement deterioration by heavy traffic

One of the defects caused by heavy traffic on the road is the deformation of the pavement surface due to overloading that is more than the design load. As stated by [5] that deterioration of pavements arises from deformation generally associated with cracking under heavy commercial vehicles. The increased traffic loading will then cause failures such as cracks and depressions on the pavement. Omer et al. [6] studied the pavement failures occurred in the ring road in Khartoum. They observed from the site visit to the road severe trenched on the west lane that might have been caused by the movements of heavy loaded truck-trailers, tippers, as well as loaded fuel tankers.

Road surfaces often wear under the action of traffic, particularly during the very early life of the road. However the action of traffic continues to wear the surface texture and thus gradually reduces the high speed skidding resistance, [7]. He reported that with the increase of traffic loads (volume and axle loads) the road network was experiencing a deterioration equivalent to a loss of billions dollars due to road deterioration and vehicle operating cost. Nowadays, the rate of traffic accident on roads due to the nature of the road is alarming. Okigbo [3] indicated that the defects that most often cause injuries to people and damage to vehicles include inadequate road shoulders, pavement surface that is uneven, improperly marked signs, malfunctioning stop lights, construction negligence, and municipal negligence. Traffic volume and size (especially for overloading) contributes to road safety and conditions. Recognizing of vehicles' uses and applications (industrial transportations) is the key for decreasing road deterioration.

2.3.2 Flexible pavement deterioration caused by climatic changes

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. 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. 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 [8]. 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 sub grade, weakening and hence lowering the CBR value of the sub grade on which the road pavement design was based upon [19].

Wee et al. [9] reported that climatic changes in temperature and rainfall can interact together. Rainfall can alter moisture balances and influence pavement deterioration while the temperature changes can affect the aging of bitumen resulting in an increase in embrittlement of the bitumen which causes the surface to crack, with a consequent loss of waterproofing of the

2.3.3 Flexible pavement deterioration caused by poor Drainage

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.

Patil Abhijit et al. [10] investigated the effect of poor drainage on road pavement condition and found that the increase in moisture content decreases the strength of the pavement. Therefore poor drainage causes the premature failure of the pavement.
Little and Jones [11] investigated moisture damage in asphalt pavements due to poor drainage. They found that the loss of strength and durability due to the effects of water is caused by loss of cohesion (strength) of the asphalt film, failure of the adhesion (bond) between the aggregate and asphalt, and degradation of the aggregate particles subjected to freezing. Moisture damage generally starts at the bottom of an asphalt layer or at the interface of two asphalt layers [12]. Eventually, localized potholes are formed or the pavement ravels or ruts. Surface raveling or a loss of surface aggregate can also occur, especially with chip seals. Occasionally, binder from within the pavement will migrate to the surface resulting in flushing or bleeding [13].

2.3.4 Pavement deterioration caused by Construction with Low Quality Materials.

The use of low quality materials for construction adversely affects the performance of the road. The use of marginal or substandard base materials for pavement construction will affect pavement performance [14]. He found that these materials may accelerate deterioration of the pavement and often result in rutting, cracking, shoving, raveling, aggregate abrasion, low skid resistance, low strength, shortened service life, or some combination of these problems. Osuolale etal. [15] investigated the possible causes of highway pavement failure along a road in south western Nigeria. He stated that the materials used as sub base have the geotechnical properties below the specification and this is likely to be responsible for the road failure. The base materials with high fines content are susceptible to loss of strength and load supporting capability upon wetting [16]. However, marginal base materials often lead to distress and can lead to premature failure in the form of severe shrinkage cracking followed by accelerated fatigue cracking and a general loss of stability [17]

2.3.5 Flexible pavement deterioration caused by expansive Sub grade Soil

Expansive soil as road sub grade is considered one of the most common causes of pavement distresses. Longitudinal cracking results from the volumetric change of the expansive sub grade, is one of the most common distresses form in low volume roads (see Fig. 1). This type of cracking is initiated from the drying highly plastic sub grade (PI>35) through the pavement structure during the summer [18], [19]. Other forms include fatigue (alligator) cracking, edge cracking, rutting in the wheel path, shoving, and pop outs.

Charlie et al. [20] investigated more than 30 sites and found that over one third of Sudan's area may have potentially expansive soils and recommended all potential construction sites in the clay plain be evaluated for expansive soils. Problem of expansive soils results from a wide range of factors such as swelling and shrinkage of clay soils result from moisture change, type of clay minerals, drainage– rise of ground water or poor surface drainage and compression of the soil strata resulting from applied load.

2.4 Subgrade Soils

2.4.1 General subgrade soil

The success or failure of a pavement is dependent on the subgrade material upon which the component of the pavement structure is to be built. Thus, the subgrade must be able to support the axle loads transmitted from the pavement layers without progressive excessive deformation. The performance of the subgrade material generally depends on its load bearing capacity, moisture content and volume change. The load bearing capacity depends on the degree of compaction, moisture content and soil type. Hence, the relationships among the strength, density and moisture content should be studied thoroughly [1].

2.4.2 Strength- density-moisture relationship

Desirable properties that the subgrade should possess must include strength, drainage, effortlessness of compaction, permanency of compaction and permanency of strength. Since subgrades vary considerably, it is necessary to make a thorough study of the soils in situ and, from this, to determine the design of the pavement.

The subgrade strength determination has been conducted for the design of the road pavement to ascertain the density-moisture content-strength relationships. It is a must to identify the density which will be representative of the compacted subgrade and the moisture content during and after construction phase.

2.4.3 Estimated design moisture content of the subgrade

Moisture conditions in the subgrade are controlled primarily by the local environment. Since design concepts for flexible pavements are based upon model-prototype principles, wherein samples of soil are tested in the laboratory simulated field condition, it is necessary to predict the optimum moisture content of the sub grade so that this value can be used in the testing program [14].

2.4.4 Representative density

The strength of the road subgrade for flexible pavements is commonly assessed in terms of the California Bearing Ratio (CBR). This is dependent on the type of soil, its density, and its moisture content. Direct assessment of the CBR of the subgrade soil under the completed road pavement is often difficult to undertaken. However, it can be inferred from an estimate of the density and 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 be controlled within limits by compaction at suitable moisture content at the time of construction. The moisture content of the subgrade soil is governed by the local climate and the depth of the water table below the road surface [6].

It is recommended that the top 25 cm of all subgrades should be compacted to a relative density of at least 100% of the maximum dry density achieved by ASTM Test Method D 698 (light or standard compaction). Alternatively, at least 93% of the maximum dry density achieved by ASTM Test Method D 1557 may be specified. With modern compaction equipment, a relative density of 95% of the density obtained in the heavier compaction test should be achieved without difficulty, but tighter control of the moisture content will be necessary. As a result, it is generally appropriate to base the determination of the design CBR on a density of 100% of the maximum dry density achieved by ASTM Test Method D 698 (light or standard compaction) or, alternately, on 93% of the maximum dry density achieved by ASTM Test Method D 1557 (heavy or modified compaction) [6]. The structural manual catalog given in the ERA Pavement Design Manual Volume I require that the subgrade strength for design be assigned to one of six strength classes reflecting the sensitivity of thickness design to subgrade strength. The classes are defined in Table 2-1 below [6].

Table 2.1 Subgrade strength classes

Class	Range (CBR %)
S1	2
S2	3-4
S3	5-7
S4	8-14
S5	15-29
S6	30+

2.5 Granular pavement materials

2.5.1 General gravel pavement material.

Granular pavement material is one of the important components of a flexible pavement structure. This material include crushed rock, semi-crushed, mechanically stabilized, and modified or naturally occurring ‘as dug’ or ‘pit run’ gravels. The suitability of rocks for road construction depends on their mineral, chemical and physical properties.

2.5.2 Properties of unbound pavement materials

Unbound granular materials are generally used in road pavements as base and sub-base courses, which are as important a component of roads as the surface composition and foundations. As a base course, they play a structurally important role, especially on medium and low volume roads. As a sub-base, they protect the soil, and act as a working platform and an insulating layer against frost action. Pavement failure due to inadequate support of upper layers, or to rutting, will usually necessitate complete pavement reconstruction, and not just the repair of the pavement surface where the problem is visible. According to the ERA Pavement Design Manual, the main categories of unbound pavement materials with a brief summary of their characteristics are shown in Table 2.2 [6].

Table 2.2 Properties of unbound materials

Code	Descripti	Summary of Specification
GB1	Fresh, crushed rock	Dense graded, un weathered crushed stone, non-plastic parent
GB2	Crushed weathered rock, gravel or boulders	Dense grading, PI<6, soil or parent fines
GB3	Natural coarsely graded granular material, including processed and	Dense grading, PI < 6 CBR after soaking > 80
GS	Natural gravel	CBR after soaking > 30

GC	Gravel or gravel-soil	Dense graded; CBR after soaking > 15

Notes: These specifications are sometimes modified according to site conditions, material type and principal use.

i) Base course materials

Materials such as crushed quarried rock, crushed and screened, mechanically stabilized, modified or naturally occurring "as dug" or "pit run" gravels can be used as a base course material. According to the ERA Pavement Design Manual the properties for base course materials is given below.

a. Crushed stone

Graded crushed stone (GB1). This material is produced by crushing fresh, quarried rock (GB1) and may be an all-in product, usually termed a 'crusher-run', or alternatively the material may be separated by screening and recombined to produce a desired particle size distribution, as per the specifications. Alternate gradation limits, depending on the local conditions for a particular project, are shown in Table 2.3. After crushing, the material should be angular in shape with a Flakiness Index (British Standard 812, Part 105) of less than 35%, and preferably of less than 30%. If the amount of fine aggregate produced during the crushing operation is insufficient, non-plastic angular sand may be used to make up the deficiency. In constructing a crushed stone base course, the aim should be to achieve maximum impermeability compatible with good compaction and high stability under traffic [6].

Table 2.3: Grading limits for graded crushed stone base course materials (GB1)

Test sieve (mm)	**Percentage by mass of total aggregate passing test sieve**		
	Nominal maximum particle size		
	A	B	C
	37.5mm	28mm	20mm
50	100	-	-
37.5	95-100	100	-
28	-	-	100
20	60-80	70-85	90-100

10	40-60	50-65	60-75
5	25-40	35-55	40-60
2.36	15-30	25-40	30-45
0.425	7-19	12-24	13-27
0.075(1)	5-12	5-12	5-12

Note 1 .For paver-laid materials lower fines content may be accepted. The fine fraction of a GB1 material should be non-plastic.

The in situ dry density of the placed material should be a minimum of 98% of the maximum dry density obtained in the ASTM Test Method D 1557 (Heavy Compaction).

The compacted thickness of each layer should not exceed 200mm. Crushed stone base courses constructed with proper care with the materials described above should have CBR values at least 100%. There is usually no need to carry out CBR tests during construction [1].

B. Naturally occurring granular materials, boulders, weathered rocks

Normal Requirements for natural gravels and weathered rocks (GB2, GB3). A wide range of materials including lateritic, calcareous and quartz tic gravels, river gravels, boulders and other transported gravels, or granular materials resulting from the weathering of rocks can be used successfully as base course materials. Table 2.4 contains three recommended particle size distributions for suitable materials corresponding to maximum nominal sizes of 37.5 mm, 20 mm and 10 mm. Only the two larger sizes should be considered for traffic in excess of 1.5 million equivalent standard axles. To ensure that the material has maximum mechanical stability, the particle size distribution should be approximately parallel with the grading envelope.

To meet the requirements consistently, screening and crushing of the larger sizes may be required. The fraction coarser than 10 mm should consist of more than 40% of particles with angular, irregular or crushed faces. The mixing of materials from different sources may be warranted in order to achieve the required grading and surface finish. This may involve adding fine or course materials or combinations of the two. The fines of these materials should preferably be non-plastic but should normally never exceed a PI of 6 [1].

When used as a base course, the material should be compacted to a density equal to or greater than 98% of the maximum dry density achieved in the ASTM Test Method D 1557 (Heavy Compaction). When compacted to this density in the laboratory, the material should have a minimum CBR of 80% after four days immersion in water (ASTM D 1883).

Recommended particle size distributions for mechanically stable natural gravels and weathered rocks for use as base course material (GB2, GB3) given in Table 2.4 [6].

Table 2.4: Recommended particle size distributions for GB2 and GB3

Test sieve (mm)	Percentage by mass of total aggregate passing test sieve		
	Nominal maximum particle size		
	A	B	C
	37.5mm	20mm	10mm
50	100	-	-
37.5	80-100	100	-
20	60-80	80-100	100
10	45-65	55-80	80-100
5	30-50	40-60	50-70
2.36	20-40	30-50	35-50
0.425	10-25	12-27	12-30
0.075	5-15	5-15	5-15

ii) Subbases

The sub-base is a pavement layer which enables traffic stresses to be reduced to acceptable levels in the subgrade. According to the ERA Pavement Design Manual the requirements to use as a sub-base material is discussed below

a. Bearing capacity

A minimum CBR of 30% is required at the highest anticipated moisture content when compacted to the specified field density, usually a minimum of 95% of the maximum dry density achieved in the ASTM Test Method D 1557 (Heavy Compaction). Under conditions of good drainage and when the water table is not near the ground surface the field moisture content under a sealed pavement will be equal to or less than the optimum moisture content in the ASTM Test Method D 698 (Light Compaction). In such conditions, the sub-base material should be tested in the laboratory in an unsaturated state. Except in arid areas, if the base course allows water to drain into the lower layers, as may occur with unsealed shoulders and under conditions of poor surface maintenance where the base course is pervious, saturation of the sub-base is likely. In these circumstances, the bearing capacity should be determined on samples soaked in water for a period of four days. The test should be conducted on samples prepared at the density and moisture content likely to be achieved in the field. In order to achieve the required bearing capacity, and for uniform support to be provided to the upper pavement, limits on soil plasticity and particle size-distribution may be required. Materials which meet the recommendations of Table 2.5 and 2.6 will usually be found to have adequate bearing capacity [6].

B. Use as a construction platform

In many circumstances the requirements of a sub-base are governed by its ability to support axle loads due to traffic without excessive deformation or raveling. A high quality sub-base is therefore required where loading or climatic conditions during construction are severe. Suitable material should possess properties similar to those of a good surfacing material for unpaved roads. The material should be well graded and have a plasticity index at the lower end of the appropriate range for an ideal unpaved road wearing course under the prevailing climatic conditions. These considerations form the basis of the criteria given in Tables 2.5 and 2.6. Material meeting the requirements for severe conditions will usually be of higher quality than the standard sub-base (GS). If materials to these requirements are unavailable, trafficking trials should be conducted to determine the performance of alternative materials under typical site conditions.

In Ethiopia, laterite is one of the widely available materials and can be used as a sub-base material. Laterite meeting the graduation requirements of Table 2.6 can be used for traffic levels up to 3x106 Equivalent Standard Axles (ESA) provided the following criteria is satisfied [6]:

Plasticity Index (%) <25

Plasticity Modulus (PM) <500

CBR (%) >30

Table 2.5 Recommended plasticity characteristics for granular sub-bases (GS)

Climate	Typical Annual Rainfall	Liquid Limit	Plasticity Index	Linear Shrinkage
Moist tropical and wet tropical	>500mm	<35	<6	<3
Seasonally wet trop	>500mm	<45	<12	<6
Arid and semi-arid	<500mm	<55	<20	<10

Table 2.6 Typical particle size distribution for sub-bases (GS)

Test Sieve (mm)	Percentage by mass of total aggregate passing test sieve (%)
50	100
37.5	80-100
20	60-100
5	30-100
1.18	17-75
0.3	9-50
0.075	5-25

c. Sub-Base as a filter or separating layer

This may be required to protect a drainage layer from blockage by a finer material or to prevent migration of fines and the mixing of two layers. The two functions are similar except that for use as a filter the material needs to be capable of allowing drainage to take place and therefore the amount of material passing the 0.075 mm sieve must be restricted. The criteria according to the ERA Pavement Design Manual [6] should be used to evaluate a sub base as a separating or filter layer.

iii) Selected subgrade materials and capping layers (GC)

These materials are often required to provide sufficient cover on weak subgrades. They are used in the lower pavement layers as a substitute for a thick sub-base to reduce costs, and a cost comparison should be conducted to assess their cost effectiveness. As an illustrative example, approximately 30 cm of "GC" material placed on an S1 or S2 subgrade will allow selecting a pavement structure as for an S3 subgrade. An additional 5cm of "GC" material may allow considering an S4 subgrade class.

The requirements are less strict than for sub-bases. A minimum CBR of 15% is specified at the highest anticipated moisture content measured on samples compacted in the laboratory at the specified field density. This density is usually specified as a minimum of 95% of the maximum dry density in the ASTM Test Method D 1557 (Heavy Compaction). In estimating the likely soil moisture conditions, the designer should take into account the functions of the overlying sub-base layer and its expected moisture conditions in the subgrade. If either of these layers is likely to be saturated during the life of the road, then the selected layer should also be assessed in this state. Recommended grading or plasticity criteria are not given for these materials. However, it is desirable to select reasonably homogeneous materials since overall pavement behavior is often enhanced by this. The selection of materials which show the least change in bearing capacity from dry to wet is also beneficial [6].

2.4 Requirements based on the Bako-Nekemte Rehabilitation Road Project Specification California

Bearing Ratio (CBR) requirements for the pavement material layers based on Bako-Nekemte Road Rehabilitation Project (project specification)

Table 2.7 CBR requirements for the pavement material layers [6].

Description	Pavement Material Type	Requirement
4-Day soaked CBR value (AASHTO T193) at 98% of MDD	Crushed stone base material (GB1)	Not less than 100%
4-Day soaked CBR value (AASHTO T193) at 97% of MDD of AASHTO T180 Method D	Crushed subbase	Not less than 30%
	Gravel sub base material	Not less than 30%
Laboratory CBR at 95% of MDD of (AASHTO T180)	Capping layer (select layer)	Not less than 15%
	Subgrade	Designed subgrade CBR

Thickness requirements for the pavement material layers based on Bako-Nekemte Road Rehabilitation Project working drawings [10]

Table 2.8: Pavement layer design thickness

Road section	Pavement layer thickness (cm)		
(Km to Km)	AC	GB1	GS2
130+423-130+580	5	20	25
102+040-102+191	5	20	25
76+240-76+568	5	20	25

Note: GB1 = crushed base course; GS2 = natural sub base and GC = capping layer. Table 2.8 shows the minimum design thickness of pavement components at the different sections of the road.

Table 2.9 Acceptance criteria for selected subgrade fill material and subgrade

Standard specification (acceptance)limit		
Test description	subgrade	Capping layer (Selected subgrade fill material)
Liquid limit	$\prec 60$	$\prec 40$
Plastic limit	$\prec 30$	$\prec 20$

Table 2.10 Los Angeles Abrasion Value (AASHTO T-96)

Description	Requirement
Crushed stone Base course(GB1)	Shall not exceed 30%
Crushed Subbase	Not more than 45%
Gravel Subbase Material	Not more than 51%

CHAPTER THREE

METHODOLOGY AND MATERIALS

3.1 Location of the study area

3.1.1 Location and accessibility

The road project under review is a portion of the Gedo – Nekemte Road Rehabilitation Project named Contract -2, Bako – Nekemte which has been implemented by the Ethiopian Roads Authority under a package of the Road Sector Development Stage III Project initiated through the Ethiopian Government Agricultural Development-Led Industrialization (APL I) long term program. The project road is part of the country's road network that connects the capital Addis Ababa with the western parts of the country. The road was originally built during the Italian occupation in the 1930's and was rehabilitated in the 1950's. The road was again upgraded by the Ethiopian Roads Authority own force to its present standard (Double Bituminous Surfaced Treatment) in the 1980's. Currently, the road has severely deteriorated and developed deep and wide potholes and cracks which affect the smooth riding quality of the road leading to discomfort to the road users

The project road, begins at a point 66.1km from the exit of Gedo town towards Nekemte, and generally traverses in a westerly direction, en-routes through the small towns of Ano, Sire, Cheri, Chingi, Gutte and ends at the Y- junction in the center of the town of Nekemte. This road, when it is completed, Joins the recently upgraded road of the Nekempte – Mekenajo Road via Gimbi town. The whole route of the project is located in the Oromia Regional State, Eastern Wellega Zone, with a total project length of 64.814 km.

3.2 Terrain classification

The project road traverses through different terrains with an altitude of 1850m at the beginning near the rural town of Ano, it then drops to 1700m and then rises to 2110m at its end in Nekemte town. The terrain along which the project road traverses can be classified as flat, rolling and mountainous where on the largest portion being classified as rolling as indicated below.

Table 3.1 Terrain Classification of the Project

Terrain Type	**Road Stretches in kilometre**	**Length in km.**	**%age of total**
Flat	68.1-68.8,114.8-115.62, 118.7-123.8	6.62	10.22%
Rolling	66.1 - 68.1, 69.4 - 76.22, 77.1 - 100.3,100.8 - 105.2, 107.4 - 114.8, 115.62 - 118.7, 123.8 - 130.913	54.01	83.33%
Mountainous	68.8-69.4, 76.22-77.1, 100.3-100.8, 105.2-107.4	4.18	6.45%

The maximum gradient of the road is in the range of 7 - 9 % with an absolute maximum of 10 - 11% at few mountainous stretches of the route and on approaches to bridges, which is considered as a deviation from the ERA's standard for DS3 roads. The minimum gradient is 0.022% in the flat terrain portions of the project. However, all those deviations from the standard are reviewed during the construction phase to 10% maximum gradient and 0.5% minimum gradient, except bridge approaches [0%] and the revision has completed for the whole project length.

3.3 Climate

The climatic condition of the area through which the project traverses, can be classified as Weina Dega (Temperate) and Kolla (Warm climatic) zones. The records on the Engineering report, as obtained from the meteorological stations at Bako, Sire and Nekempte towns show the following:

a. Mean monthly minimum temperature in the project area is in the range of 10 0C – 11 0C, occurring during the months of November to January.

b. Mean monthly maximum temperature in the project area is in the range of 27 0C – 31 0C, occurring during the months of February to April.

c. Mean annual rainfall in the area is 1222 mm for Bako, 1360 mm for Sire and 2011 mm for Nekemte towns.

As observed from the monthly records of the rainfall in the vicinity of the project area, June to September are the wet seasons, which are a bit longer rainy duration compared to other locations of the Country; however, with others period relatively drier and favorable for construction of any temporary and permanent works.

Table 3.2 Mean Monthly Minimum Temperatures (0C)

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Bako	11.3	12.9	14.8	15.3	15.6	15.1	14.8	14.9	14.3	12.4	11	10.6
Nekemte	11.5	12.6	13.1	13.4	12.9	11.7	11.9	12.1	11.6	12.2	11.7	11.1

Table 3.3 Mean Monthly Maximum Temperatures (0C)

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Bako	29.6	30.2	30.9	30.3	28.3	25.7	24	24	25.1	27.3	28.3	28.9
Nekemte	25.5	26.6	26.7	26.7	24.2	21.3	19.9	20.1	21.4	23.1	23.8	24.5

Table 3.4 Mean Monthly Rainfall (mm)

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Bako	9.5	17.7	58.2	65.1	146.2	216.6	251.4	229	152.5	50	13.9	11.9
Sire	9.5	19.3	64.6	77	144.3	223.1	273.9	242.4	194	74.3	23.2	14.5
Nekempte	**8.3**	17	49	75.5	212.5	359.1	391	388.2	285.9	144.4	66.4	15.2

Source: National Meteorological Service Agency

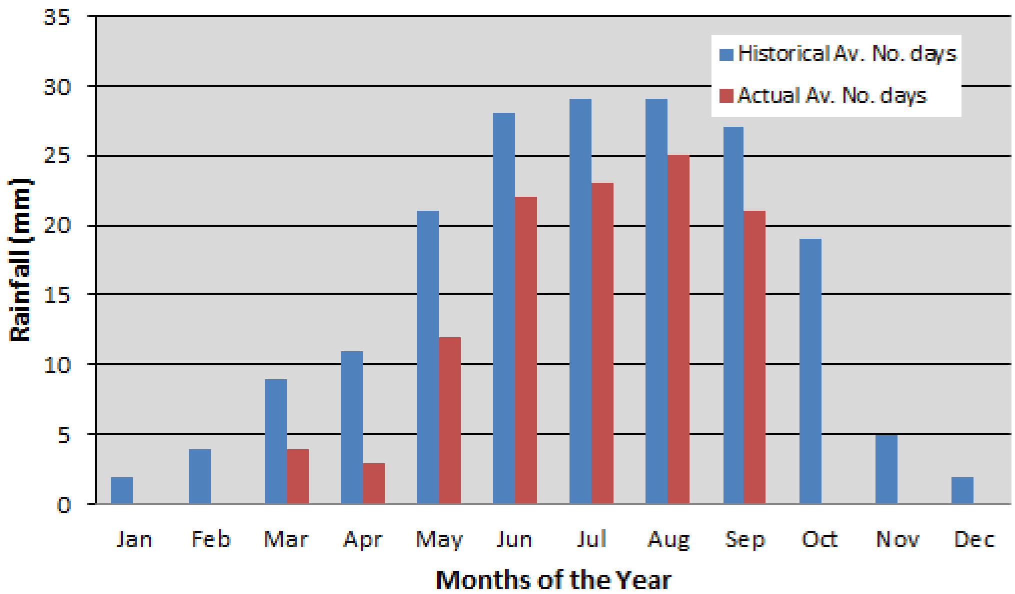


Figure 3.1 Annual rainfall of the road section.

The project road is located in the Oromia Regional State connecting the western part of Showa to the eastern part of Wollega of the Oromia Regional State provinces. In effect, it is part of the Federal road network that connects the capital Addis Ababa with the western part of the country. Moreover, the project road passes through towns and villages found along the road corridor as detailed in the table below

Table 3.5 List of Towns and Villages Traversed by the Project Road

Name	Location within the project chainage	Classification (Town/Village)	Distance from start of the project (66+100)
Ano	70+300	Town	4.2 km
Sire	83+200	Town	17.1 km
Cheri	90+200	Town	24.1 km
Jalelie	94+300	Village	28.2 km

Name	Location within the project chainage	Classification (Town/Village)	Distance from start of the project (66+100)
Chingi	103+500	Town	37.4 km
Arib Gebeya	112+000	Village	45.9 km
Gutie	118+200	Town	52.1 km
Nekempte	130+900	Town	64.8 km

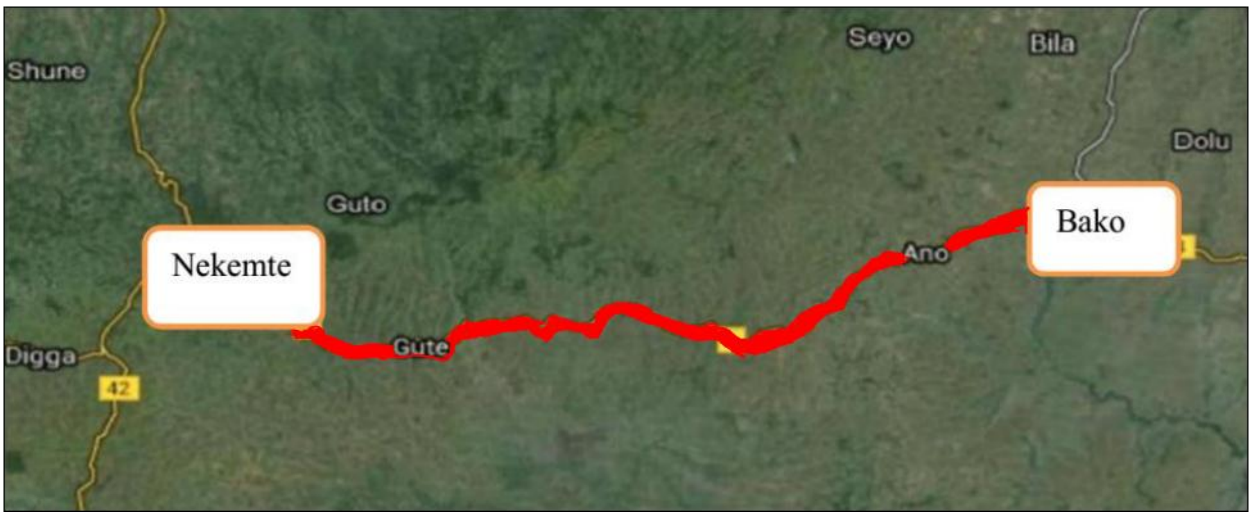


Figure 3.2 Project Location Map

3.5 Land cover

According to the Consultant's field assessment most of the land along the project road is intensively used for crop cultivation, livestock grazing and settlement. The major agricultural crops that have been grown along and in the vicinity of the project area include maize, sorghum, teff, and coffee. Coffee is a good cash crop that grows near Nekemte and also Nug (oil crop) is growing in the area as cash crop. The remaining land covers are grazing/grass land, bush, forest and urban or rural settlement area. The eucalyptus tree plantation is widely observed on either sides of the existing road in proximity and within the road limits as well.

3.6 Geology

According to the Geological Map of Ethiopia (second edition, 1996), the following four types of formations are encountered along the Road Gedo - Nekemte, where the project road is part of it.

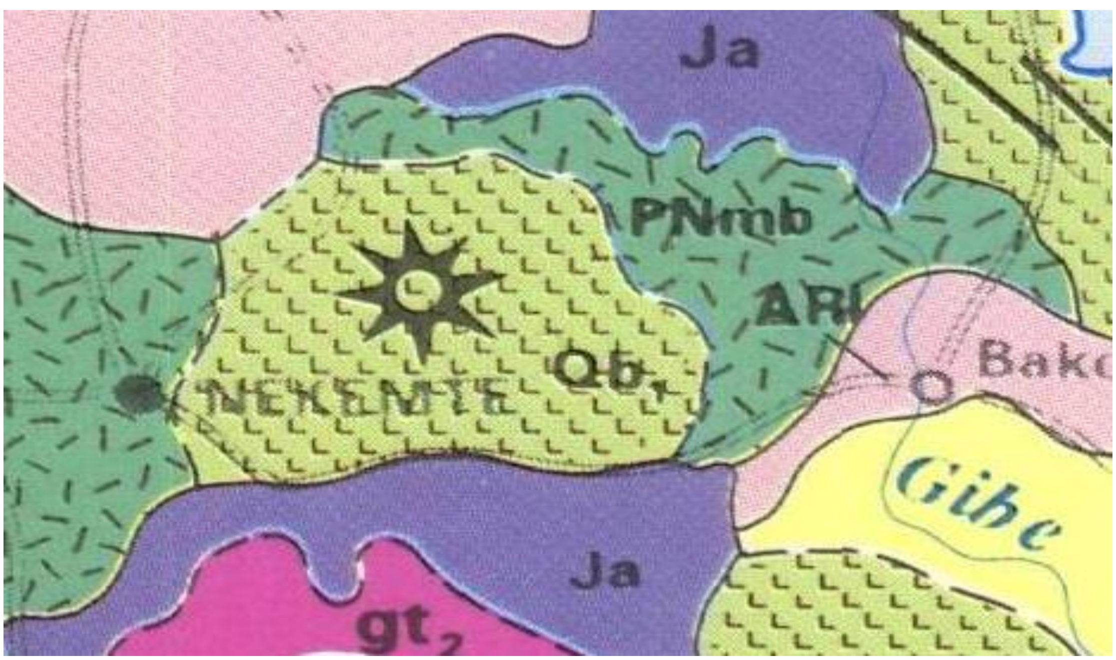
A. Quaternary Plateau Basalts (Qb1): The formation consists of alkaline basalts and trachytes and is found along the first 10 km of the project road approximately and from around km 85 to around km 130.

B. Alluvial and lacustrine deposits (Q) in the low lying areas consisting of sand, silt and clay.

C. The Quaternary sediments are mainly found from around km 10 to around km 40.

D. Alghe Group (ARI): The Precambrian Alghe Group consists mainly of boitite, hornblend and gneiss and extends from about km 40 to around km 70. Bako town is covered with the Alghe Group rocks.

E .Mekonen Basalts (PNmb): The group consists of Tertiary Flood Basalts overlying the crystalline basement and is mainly found from around km 70 to around km 85 and from around km 130 to around km 140.5. Nekempte is covered with the Mekonen Basalts.



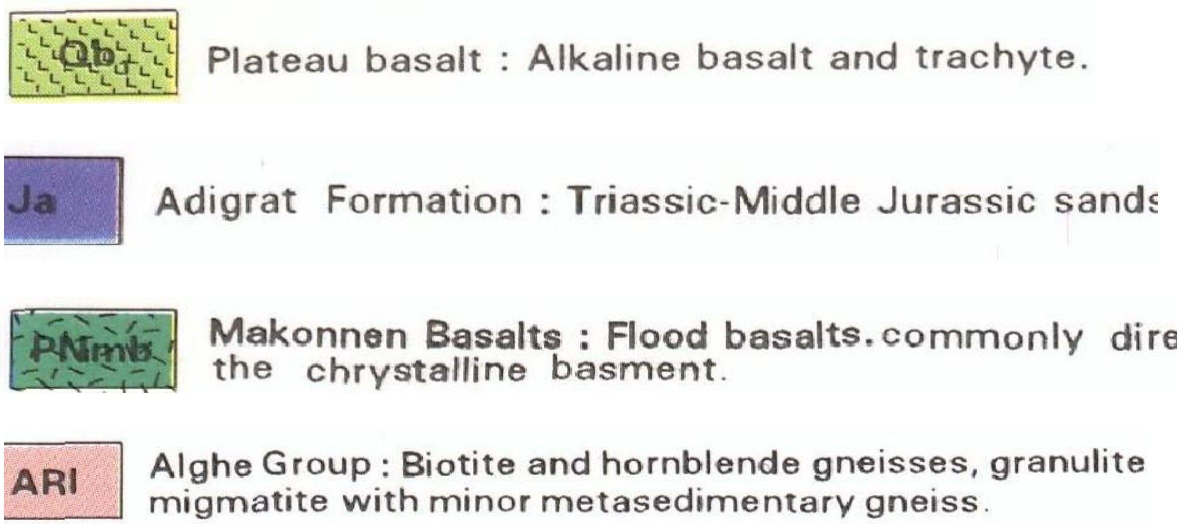


Figure 3.3 Geological Map of Ethiopia in the corridor of Bako - Nekemte Road

3.7 Project typical cross sections

The carriageway width of the road is 7.0-m paved with Asphalt Concrete (AC) and 1.5-m wide shoulder of single bituminous surface treatment on each side for sections in rural areas. In the largest towns like Nekemte, the carriageway width is 14-m paved with AC and 2.5-m walkway single bituminous surface treatment on each side. In other smaller towns, the carriageway width is reduced to 13-m and the walkway to 2-m with the same type of construction as the largest town.

3.8 Study procedure

The procedure utilized throughout the conduct of this study are as follows: Reviewed related literatures on relevant areas of causes of damages and remedial measures of asphalt concrete pavement including , Asphalt overlay which includes articles, reference books, research papers, class lecture notes, project specifications, standards specifications like ERA, AASHTO and ASTM. Necessary data collection, organization, comparison and analysis were obtained, and then subsequently compared the results with preexisting literature and standard specifications. A conclusion and recommendation on remedial measure had been formulated based on the possible causes of deterioration on the asphalt pavement.

3.9 Sampling technique

Purposive sampling technique was used by selecting particular parameters to make it sure that the parameters have certain characteristics as applied for this study. It is projected to be normally targets at particular geotechnical parameters

3.10 Study design

The research study was conducted by using both experimental and analytical methods. Qualitative and quantitative study was employed in this study area. Qualitative study gives impression of the findings where a quantitative study was used to describe the numerical aspects of the research findings.

3.11 Study variables

Independent variable: The independent variables for this research included:-moisture content, Method of compaction, Asphalt pavement parameters and subgrade parameters such as, base course, sub base and selected sub grade material quality , existing thickness of road, level of severity, type of deterioration and traffic volume.

Dependent variable: Flexible pavement deterioration

3.12 Data collection process

Quantitative and qualitative data were utilized based on the necessary input parameters for the analysis by comparing with project specification and ERA manuals. Data collection process included but not limited to: -Desk study (reviewing letter correspondences, reports, design documents such as template of the road, working drawings etc.), Field visual inspection and inventory, Sampling representative samples, finally Field measurements and Laboratory tests were conducted. Sorting data was done by grouping into different comparison groups by type and level of severity.

The study population by grouped were classified as low deterioration, medium deterioration and highly deteriorated. Laboratory test result, were compared with ERA Standard and project Specification.

3.13 Traffic volume

Manual classified traffic volume counts by vehicle category was conducted at which is located Bako town to Nekemte town along the road section. The Supervision consultant carried out classified traffic volume counts for seven consecutive days for twelve hours (06:00AM to 6:00

PM each day) among those days, for two identified days have taken a night count, that means 24 hours, for the day July 5,2016 and July9,2016 for both directions.

The traffic counts were conducted at a single selected count section, to reflect the flow of the traffic over the project length to provide information related to the current traffic

3.13.1 Vehicle Categories

Eight vehicle categories were considered in the count. The types of vehicles are defined according to the breakdown adopted by ERA for traffic counts; the types of vehicles are classified and presented in simplified form for reporting purpose in the following table:

Table 3.6 Vehicle Classification

S/No.	Type of Vehicle	Description
1	Cars	Passenger cars and taxis
2	S/Wagon& Pickup	Pick-ups, Land Cruisers, Land rovers, etc.
3	Small Bus	Minibuses (up to 27-Passenger seats).
4	Large Bus	Large size buses above 27 passenger seats
5	Small Truck	Small sized trucks up to 3 tons load
6	Medium Truck	Medium sized trucks including tankers from 3 to 7 tons load
7	Heavy Truck	Trucks above 7 tons load
8	Trucks and Trailer	Trucks with trailer or semi- trailer and Tanker Trailers

3.13.2 Average Daily Traffic

The daily traffic in each direction estimated as the sum of the daily equivalent 24 hours counts as described below. The average daily traffic (ADT) considered as the sum of the daily traffic in both directions. Accordingly, the average daily traffic at Bako – Nekemte is described in the table below:

Table 3.7 Average Daily Traffic (ADT) estimated from July 03 to 09, 2016 at project road sections in both directions.

Count Direction	Cars	S/Wagon & Pickup	Small Bus	Large Bus	Small Trucks	Medium Truck	Heavy Truck	Trucks & Trailer	Total
Bako-Nekemte									
To Bako	3	69	43	23	3	132	106	55	***434***
To Nekemte	2	95	56	24	7	95	90	48	***417***
ADT (Total)	**5**	**164**	**99**	**47**	**10**	**227**	**196**	**103**	***851***
% of Veh/ Category	1	19	12	6	1	27	23	12	***100***

3.13.3 Composition of Vehicles in the Traffic stream over Project road Sections

The composition of the individual vehicles per category in the project road count sections for Bako-Nekemte presented here under:

3.13.4 Passenger vehicle category

The Passenger vehicle category includes Car, S/Wagon& pickup, small Bus and Large buses. The percentage composition of passenger vehicle category estimated about 1%, 19%, 12% and 6% respectively of total traffic on the project road section. Higher percentage of **Station wagon** observed dominantly.

3.13.5 Freight vehicle category

The Freight vehicle category includes small Truck, Medium Truck, heavy trucks and truck &Trailer. The percentage composition of the Freight vehicle category estimated about 1%, 27%, 23%, and 12% respectively. Higher percentage of **medium trucks** observed dominantly and used for transporting goods at different town along the project.

3.14 Field operation sequence

During the field observation, it was necessary to begin by conducting visual inspection and site inventory of the whole stretch of the Bako-Nekemte Road rehabilitation Project. The initial site visit was taken on the whole portion of the road and at the same time the deteriorated sections were identified for further detailed site observation. There were **81** deterioration locations were inspected and approximate measurements of **width** and **length** of the deterioration were recorded to obtain the extent of defects on deteriorated and the stations and their corresponding extent of defects . Several photos were taken from selected road sections and few representative ones were used in the results and discussion section. The rest of the photos are presented in Appendix part. These photos clearly show the part of the road section that were badly deterioration.

After finishing the initial visual inspection and categorizing the conditions of the road into three, the deterioration, were categorized accordingly as high deterioration, medium deterioration and low deterioration. The next step was then to select the representative locations for sampling based on their failure conditions. The researcher selected nine (9) test pits that represent the three conditions. For each condition three test pits were dug for laboratory testing as well as field tests. Since the detailed Field investigation was carried out at dry season and at the beginning of the rainy season (monsoon). Test pits were taken to assess the strength at the finished subgrade level and the suitability of said material for road construction. Samples were extracted at three sections at different Stations test pits excavated which were approximately 540KG.

3.15 Pavement condition survey

Condition surveys are essentially required to assess a pavement's physical distress and form the basis of a diagnosis regarding the maintenance or rehabilitation needs [11]. The main objective of the pavement condition survey for this study was to evaluate the state of the existing pavement and that of the subgrade by inspecting the physical conditions of the existing pavement. Before the commencement of the detailed pavement evaluation, the entire road length was visually assessed and an attempt was made to identify the types of distresses occurred on the road prism.

3.16 Field work

Preliminary visual survey was undertaken along Nekemte-Bako road rehabilitation. Field observations, Field tests and measurements were carried out and representative samples were taken to laboratory tests. Results from field tests and measurements were compared with the results from laboratory tests. Moreover, results from laboratory tests were compared with ERA

Standard Specifications.

3.17 Field observation and investigation

During field observation and investigation stage the width of the existing road surface was taken and the thickness of the road materials was measured in each test pit using a meter tape. Accordingly the corresponding width of the road and the thickness of the pavement layers were recorded.

3.18 In situ density survey

An in-situ density measurement had been carried out inside each test pit where samples have been taken for laboratory tests. The test was conducted in accordance with AASHTO T-191 (1993) [10]. The subgrade density was conducted within depths of about 10cm to 25 cm bellow the capping layer or selected subgrade [10].

Field tests were conducted on the selected sections of Bako-Nekemte road rehabilitation project. The field tests performed by checking thickness of the pavement layers, field moisture content and field density for each pavement layer. In each location, the surface of the material layer that had been tested was trimmed and smoothed to form platform for the measuring apparatus. Then a pit was excavated through the guide of the pavement material cross section of the road prism. The material from the pit, to a depth of the actual pavement material thickness, was carefully collected and put inside of polyethene bag, then, tightly sealed, and labeled for subsequent natural moisture content determination. The bulk density had been computed after completion of each test. The field dry densities of all samples were later computed based on the results of the natural moisture content at determined in the laboratory.

Figure 3.4 field density tests (photo 9, 7, 2016)

3.19 Laboratory tests

The representative samples collected during detailed field investigation were brought to Nekemte branch ERCC District, and the following tests were undertaken. Atterberg Limits, Grain size Analysis, Compaction Tests and California Bearing Ratio (CBR) Tests were made to understand the general behavior of the road materials with standards.

The researcher collected samples from each test pits of the three sections for every pavement layer to perform the required laboratory tests. Representative samples were collected and labeled accordingly. Immediately after extracting samples, these were transported to the laboratory. The tests were performed according to AASHTO specification [6], [7] and ASTM following the procedures discussed on the soil mechanics laboratory manual by Braja, M. D. [19]. The necessary tests were conducted for all the samples and the summary of the results is presented in a tabulated form. The laboratory data analysis is attached in appendix page.

3.20 1Atterberg's limit

Most of the methods for soil identification and classification are based on certain physical properties of the soils. The commonly used properties for the classification are the grain size distribution, liquid limit and plasticity index. These properties have also been used in empirical design methods for flexible pavements, and in deciding the suitability of subgrade soils. Tests were undertaken on subgrade and Capping layer or selected subgrade fill materials at selected test pits of the three sections. The testing procedure was done according to ASTM D 4318 [16].


.

Figure 3.5 atterberg's limit (photo 14, 7,2016)

3.20.2 Grain size analysis

The mechanical analysis consists of the determination of the amount and proportion of coarse material by the use of sieves and the analysis for the fine grained fraction by sedimentation method. For the materials passing 75 microns, hydrometer method was used. The combined grading of the material shall be a smooth continuous curve falling within the grading limits. When determined in accordance with the requirements of AASHTO T-27. The mass of material passing the 0.075 mm sieve shall be determined in accordance with the requirements of AASHTO T-11[1] Sedimentation method. For the materials passing 75 microns hydrometer method was used.

3.20.3 Compaction test

This laboratory test is performed to determine the relationship between the moisture content and the dry density of a soil for a specified comp active effort. The compaction effort is the amount of mechanical energy that is applied to the soil mass.

The Compaction tests are designed to simulate the density of soils compacted by field methods.

Modified Proctor Test was used for this study area. The soil tested was thoroughly mixed with measured quantity of water and, it was then filled in the mold in five layers of approximately equal thickness. Each layer was subjected to 56 numbers of blows using modified hammer weighing 44.5 N, which was allowed to drop freely from a height of 46 cm. After compaction of five layers, the soil was trimmed at the top of the mold. The mold with its content was removed from the base plate and weighed. Moisture content determination was undertaken on a sample of soil and the dry density was then calculated. This procedure was repeated with addition of water content and a compaction curve was drawn. The co-ordinates of the curve that represents peak gave the maximum dry density and the optimum moisture content [14]. The compaction curve is shown in Appendix.

$$\rho_d = \rho/(1+w)$$

Where: w= moisture content in percent divided by 100, and ρ = wet density in grams per centimeter cubic.

The moisture content of each compacted soil specimen was calculated using the average of the two water contents. To compute the wet density in grams per cubic centimeter of the compacted soil sample was divided the wet mass by the volume of the mold used, then the dry density computed using the wet density and the water content [24].

Figure 3.6 Procter test (photo 17, 7, 2016)

3.20.4 California Bearing Ration (CBR) test

California Bearing Ratio is a measure of shearing resistance of the material under controlled density and moisture conditions. The test consisted of causing a cylindrical plunger of 50 mm diameter to penetrate a pavement component material at 1.25 mm/minute. The loads for 2.54 mm and 5.08 mm were recorded. This load is expressed as a percentage of standard load value at a respective deformation level to obtain CBR value.

$$CBR = 100 \times (X/Y)$$

Where: 'x' = material resistance or the unit load on the piston (pressure) for 2.54mm or 5.08mm of penetration, y = standard unit load (pressure) for well graded crushed stone. For 2.54mm penetration = 6.9mpa and for 5.08mm penetration = 10.3mpa [24].

The standard load values were obtained from the average of a large number of tests on different crushed stones. Three point CBR Test were made for all samples. California bearing ratio test results (CBR test) for four days soaked samples at their maximum dry density were compared with the standard specifications. The density versus CBR was plotted and the required CBR "for the 56 blows" had computed from the graph for the maximum dry density.

Figure 3.7 soaking of sample for four days (photo 18, 7, 2016)

3.21 Subgrade soil classification

Soil classification is the arrangement of soils into different group in order that the soils in a particular group would have similar behavior. The method of classification used in this study was the AASHTO System. The AASHTO Classification system is useful for classifying soils for highways. The particle size analysis and the plasticity characteristics are required to classify a soil. The soils with the lowest number, A-1, is the most suitable as a highway material or subgrade. Range of liquid limit and plasticity index for soils in groups A -2, A -4, A-5, A -6 and A-7 [10] based on AASHTO Classification system.

$$PI = LL - 30$$

$$For A - 7 - 5, PI \prec LL - 30$$

$$For A - 7 - 6, PI \succ LL - 30$$

CHAPTER FOUR

RESULT AND DISCUSSIONS

4.1 Field test results

4.1.1 Pavement condition survey results

In order that on the extent of deterioration depending on the the **visual inspection** and **area** of deteriorated sections, proper identification was made to select three representative sections. These sections were categorized as section 1 (Km station 130+430 to Km station 130+580) where high deterioration (more severe) was observed, section 2 (Km station 102+040 to Km station 102+191)where medium deterioration (severe) was observed and section 3 (Km station 76+240 to km station 76+568) where observed relatively low deterioration.

Table 4.1 Test pit locations

Sections	**Station of the section.**	**Test pits**	Total area of deteriorated sections	**Severity**
1	Km Station 130+430-130+580	1. Km 130+523 2. Km 130+570 3. Km 130+600	**676.5m2**	High deterioration(More severe)
2	Km Station 102 +030-102 + 191	1. Km 102 +030 2. Km 102+175 3. Km 102+187	**421.6m2**	Medium deterioration(severe)
3	Km Station 76+240 -76 +568	1.Km 76+212 2. Km 76+457 3. Km 76+ 568	**331m2**	Low deterioration(Less severe)

Table 4.1 the locations where highly deteriorated, medium deteriorated and low deteriorated occurred for every section, three (3) test pits were extracted for each road section for laboratory

tests. The road width and pavement material thickness were measured using a meter tape. In situ density was measured in the laboratory for each layer of the three sections. The following pictures show the type and extent of deterioration along the Bako-Nekemte road rehabilitation project

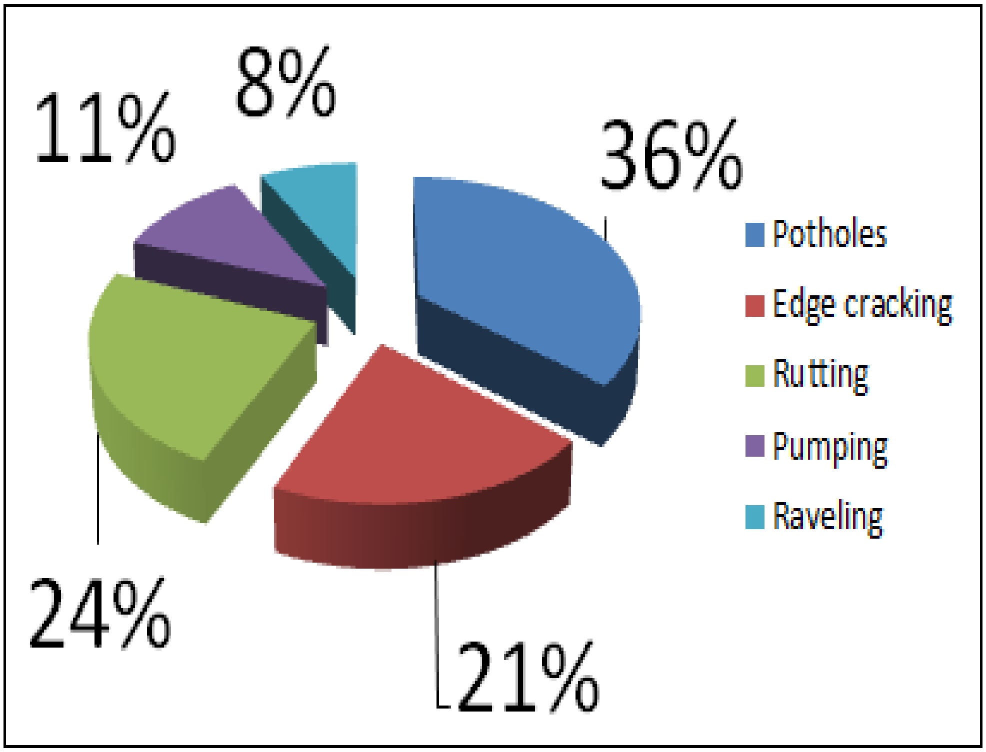


Figure 4.1 the percentage of common deterioration types and their severity.

The major types of deterioration observed along the study area of Bako-Nekemte rehabilitation Road Project. Relative to this, the researcher had organized the possible deterioration types based on existing condition of the pavement surface together with the extent of deterioration and had been ranked accordingly. The following Figures show the different photos taken from the field observation of the different types of deterioration along Bako-Nekemte road project

.

Rutting at wheel path at section 2 (photo10, 6,2016)

Potholes at section 1 (photo 3 ,6, 2016)

Raveling at section 3 (photo 15, 6, 2016)

Edge cracking at section 3 (photo 15, 6, 2016)

Potholes at section 2(photo10, 6, 2016)

Potholes at section 1 (photo 17, 6, 2016)

pumping type at section 1(photo 12, 6, 2016)

Figure 4.2 Types of deterioration of flexible pavement along the road section.

Figure 4.3 Taking of samples at section 1 (photo, 04, 7, 2016)

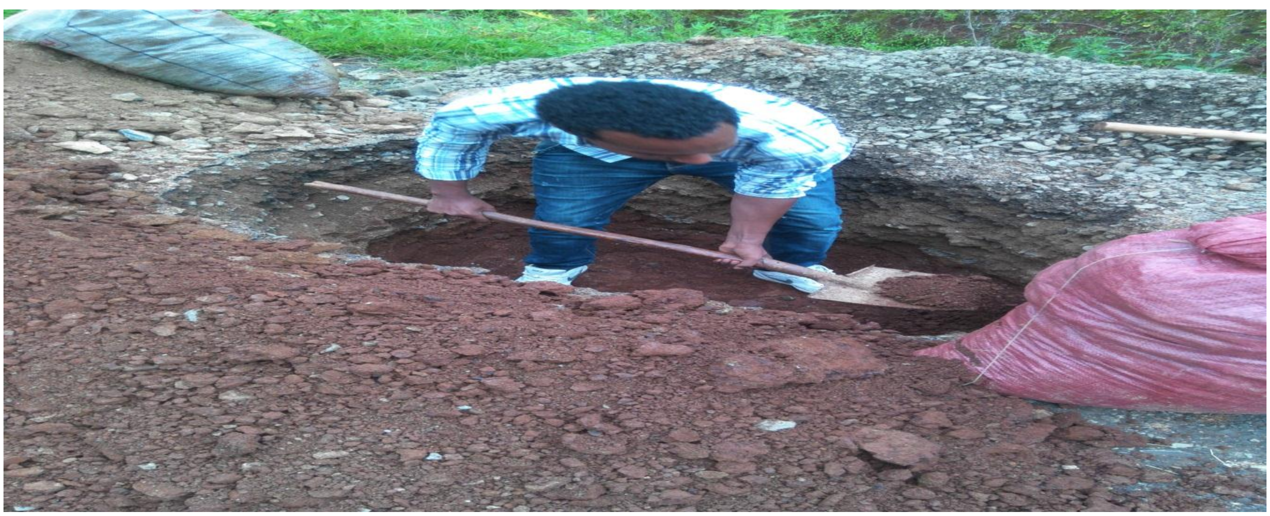


Figure 4.4 shows taking of samples at section 2(photo, 22, 6, 2016)

Figure 4.5 taking of samples at section 3 (photo, 2, 8, 2016)

Figure 4.6 Measuring the thickness of the pavement layers (photo 22, 6, 2016-17, 8, 2016)

4.1.2 Field observation and investigation results

Based on field observation and investigation the width of the road is 7 meter carriageway and 1.5 meters shoulders on both sides; whereas, in town sections the roadway was provided with 3.5 meters parking lane and 1.5 meters pedestrian walkway on both sides.

The thickness of the pavement layers is listed below.

Table 4. 2 Summary of actual pavement materials thickness

Station	Thickness of the pavement layers		
	Base course (GB1)	**Sub base course (GS)**	**Capping layer(GC)**
130+523	20.0	25	28
130+570	20.0	25	23
130+600	20.0	25	33
102 +040	19.50	22	30
102+175	20.00	25	26
102+187	18.77	21	29
76+212	19.00	25	31
76+457	20.00	25	27
76+ 568	19.60	25	36

Based on the results of pavement layers measurement in the project study area, at Km Station 130+430-130+580, Km 102 +040-102+ 191and Km 76+240 -76 +568, the actual asphalt thickness as compiled the minimum design thickness of 5 centimeter in the study area.

4.1.3 Field density test results

The field dry densities and the field moisture contents obtained on the subgrade, capping layer, sub base, and the base course materials are tabulated in Tables 4.3, 4.4, and 4.5.

Table 4.3 Test results on field density of subgrade material

Section	**Station (km)**	Water content %	**Bulk density (g/cu.cm)**	**Dry density (g/cu.cm)**
Section 1, Km Station 130+430-130+580	130+530	30.7	2.18	1.48
	130+580	25.36	2.19	1.94
	Average	28.03	2.19	1.71
Section 2, Km Station 102 +030-102+ 191	102 +040	25.4	2.02	1.61
	102+ 191	13.5	2.18	1.93
	Average	19.45	2.1	1.77
Section 3, Km Station 76+240 -76 +568	76+240	19.5	2.19	1.93
	76 +568	25.4	1.82	1.51
	Average	26.45	2.01	1.72

Table 4.4 Test results of field density of capping layer and natural sub base material

Section	Station	Capping layer			Natural subbase		
		Water content (%)	Bulk density (g/cu.cm)	Dry density (g/cc)	Water content (%)	Bulk density (g/cu.cm)	Dry density (g/cc)
1	130+523	24	1.49	1.23	15.2	2.18	1.94
	130+570	23.83	2.20	1.85	12	2.16	2.13
	130+600	17.1	2.05	1.75	13.5	2.18	1.95
	Average	21.64	1.91	1.61	13.56	2.17	2.00
2	102 +030	13.5	2.06	1.82	15.6	2.13	1.78
	102+175	17.7	2.09	1.87	11.23	2.20	2.14
	102+187	13.5	2.09	1.87	17.6	2.16	1.62
	Average	14.9	2.08	1.85	14.81	2.16	1.87
3	76+212	17.7	2.18	1.97	23.83	2.00	1.85
	76+457	17.7	2.14	1.83	14.7	2.18	1.78
	76+ 568	13.5	2.00	1.72	19	1.99	1.68
	Average	16.3	2.11	1.84	19.17	2.05	1.77

Table 4. 5 Results of field density test of base course material

Section	Station	Base course		
		Water content (%)	Bulk density (g/cu.cm)	Dry density
1	130+523	3.05	2.16	1.91
	130+570	2.24	2.16	2.26
	130+600	3.40	2.19	2.23
	Average	2.9	2.17	2.13
2	102 +030	5.00	2.14	1.76
	102+175	4.4	2.21	1.85
	102+187	2.28	2.16	2.21
	Average	3.89	2.17	1.94
3	76+212	2.28	2.23	2.21
	76+457	2.11	2.18	1.84
	76+ 568	3.12	1.99	1.78
	Average	2.50	2.13	1.96

4.2 Laboratory tests

4.2.1 Atterberg's limit test results

The Plasticity of base course and sub base materials were found to be non-plastic and for fill and subgrade materials are tabulated below. The laboratory data analysis is attached in Appendix.

Table 4.6 Atterberg's limit of pavement materials.

Atterberg's limit of subgrade				**Atterberg's limit of selected subgrade fill material (capping)**		
Location	**Liquid Limit**	**Plastic Limit**	**Plasticity Index**	**Liquid Limit**	**Plastic Limit**	**Plasticity Index**
130+523	56	30	26.00	45.2	30.5	14.6
130+570	53	30	23.00	40.5	28.8	11.7
130+600	61	39	22.00	43.0	31	12
102 +030	55.4	36.30	19.10	38	33	5
102+175	63.3	32.00	31.30	40.9	27.8	13.1
102+187	53.8	31.30	22.50	31	27	4
76+212	64.4	43.00	21.40	35	28	7
76+457	52.2	27.70	24.50	41	35	6
76+ 568	54.8	35.80	19.00	37.8	29.5	8.3

4.2.2 Grain size analysis result

The results of grain size analysis are plotted in Figures below while the analysis data is given in Appendix.

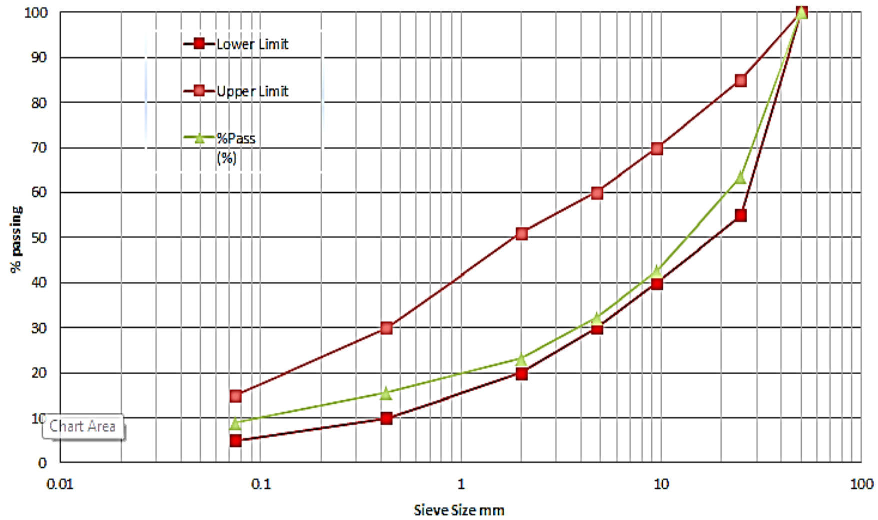


Figure 4.7 Base course material particle size distribution at section 1

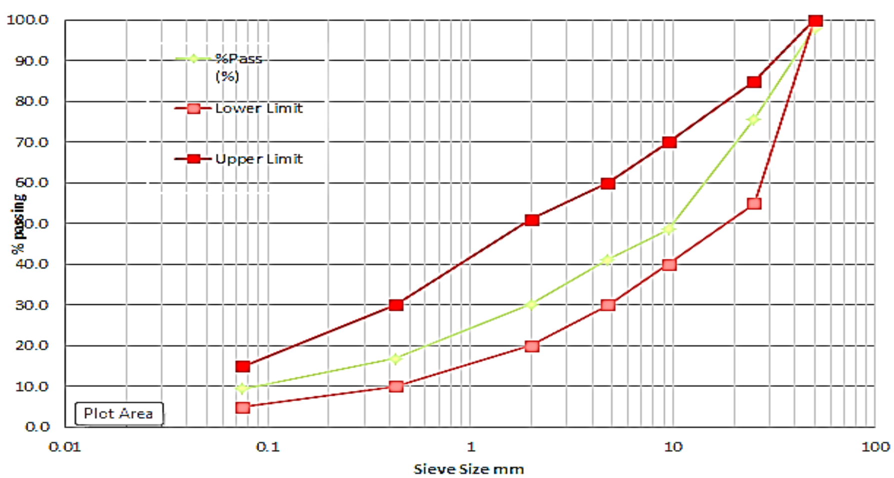


Figure 4.8 Base course material particle size distributions at section 2

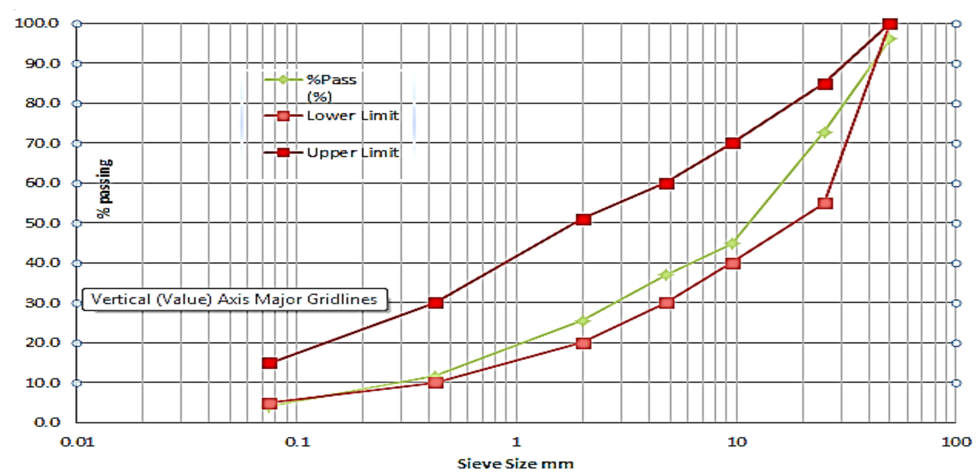


Figure 4.9 Base course material particle size distribution at section 3

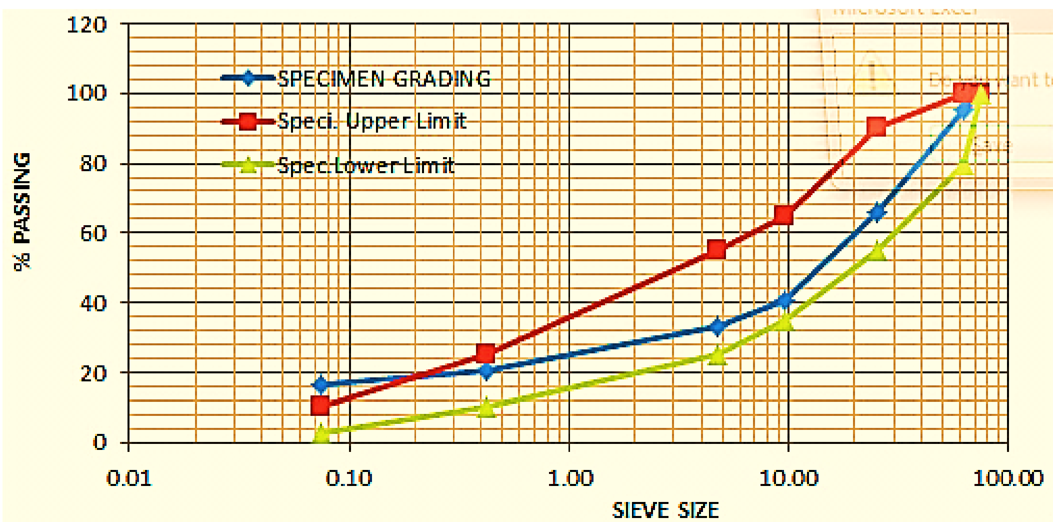


Figure 4.10 Natural Sub base material particle size of section 1

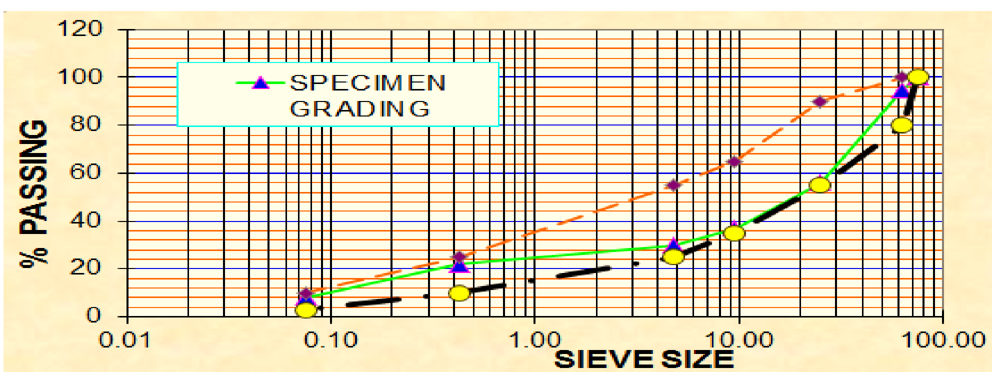


Figure 4.11 Natural Sub base material particle size of section 2

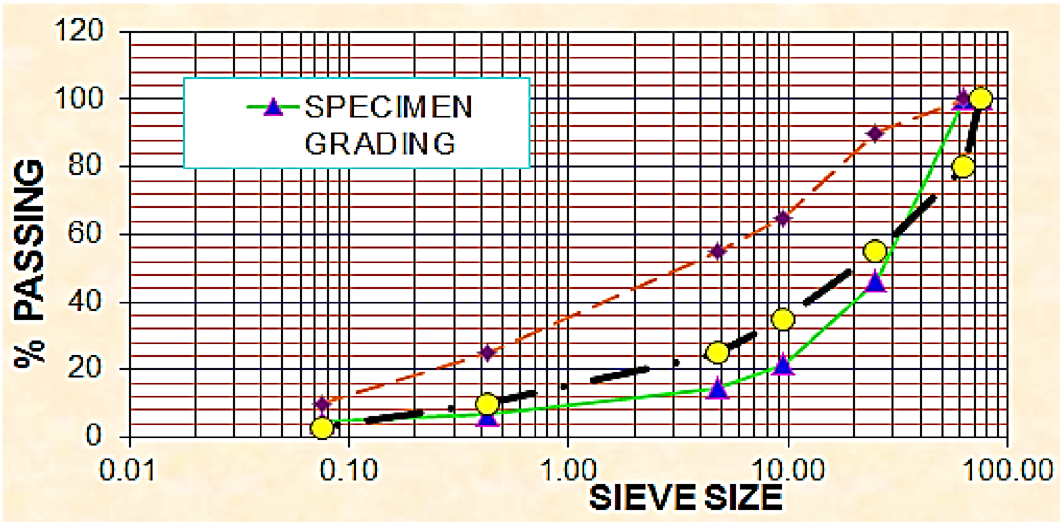


Figure 4.12 Natural Sub base material particle size of section 3

4.2.5 Moisture-density relation (compaction) test

The dry density values on the y-axis and the moisture contents on the x-axis were plotted. And a smooth curve connecting the plotted points was drawn

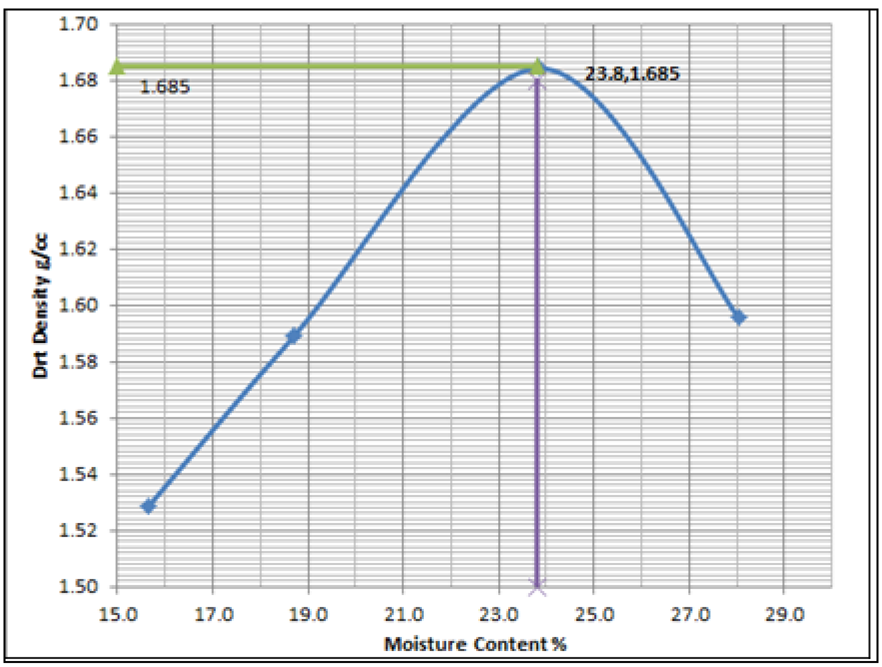


Figure 4.13 Moisture content against dry density

Table 4.7 Laboratory compaction test of subgrade material

	Section 1			Section 2			Section 3		
	130+523	130+570	Average	102 +030	102 +175	Average	76+212	76+457	Average
OMC	30.75	17.6	24.18	25.4	16.72	21.06	16.7	25.8	21.25
MDD	1.48	1.62	1.55	1.61	1.83	1.72	1.74	1.47	1.605

Table 4. 8 Laboratory compaction tests of natural sub base and capping layer material.

Section	**Station of test.KM**	**Capping layer**		**Natural sub base**	
		OMC (%)	**MDD (g/cc)**	**OMC (%)**	**MDD (g/cc)**
1	130+523	23.83	1.685	18.8	1.6
	130+570	24.00	1.690	14.00	2.15
	130+600	18.7	1.73	13.44	1.86
	Average	22.17	1.70	15.41	1.87
2	102 +030	11.4	1.84	11.23	2.14
	102+175	11.74	1.83	12.00	1.79
	102+187	16.83	1.84	13.5	1.95
	Average	13.32	1.84	12.24	1.96
3	76+212	17.41	1.72	11.69	2.18
	76+457	24.18	1.48	10.87	2.14
	76+ 568	24.95	1.469	15.7	1.83
	Average	22.18	1.56	12.75	2.05

Table 4. 9 Laboratory compaction test of base course material

Section	Station of test. KM	Base course	
		OMC (%)	MDD (g/cc)
1	130+523	2.20	2.26
	130+570	3.40	2.23
	130+600	5.00	2.15
	Average	3.53	2.21
2	102 +030	2.30	2.24
	102+175	7.40	2.10
	102+187	2.30	2.21
	Average	4.00	2.18
3	76+212	2.40	2.19
	76+457	3.22	2.15
	76+ 568	3.52	2.08
	Average	3.04	2.14

4.2.6 California Bearing Ratio (CBR) test results

The test results from the low deterioration, medium deterioration and high deterioration of road sections are listed in Tables 4.10. Based on test results, the values indicate that the materials used for all the three conditions have very good CBR values when compacted at their maximum dry density and optimum moisture content except sub base materials. The summary of the test result is tabulated below and the laboratory test analysis and plots are given in Appendix.

Table 4.10 Average values of CBR test results for the three sections:

Section	Base course	Natural Sub base	Capping layer	Sub grade
Section-1	105.33	36.67	21.67	5.22
Section-2	115.00	29.17	21.23	6.47
Section-3	124.0	43.67	18.20	5.57

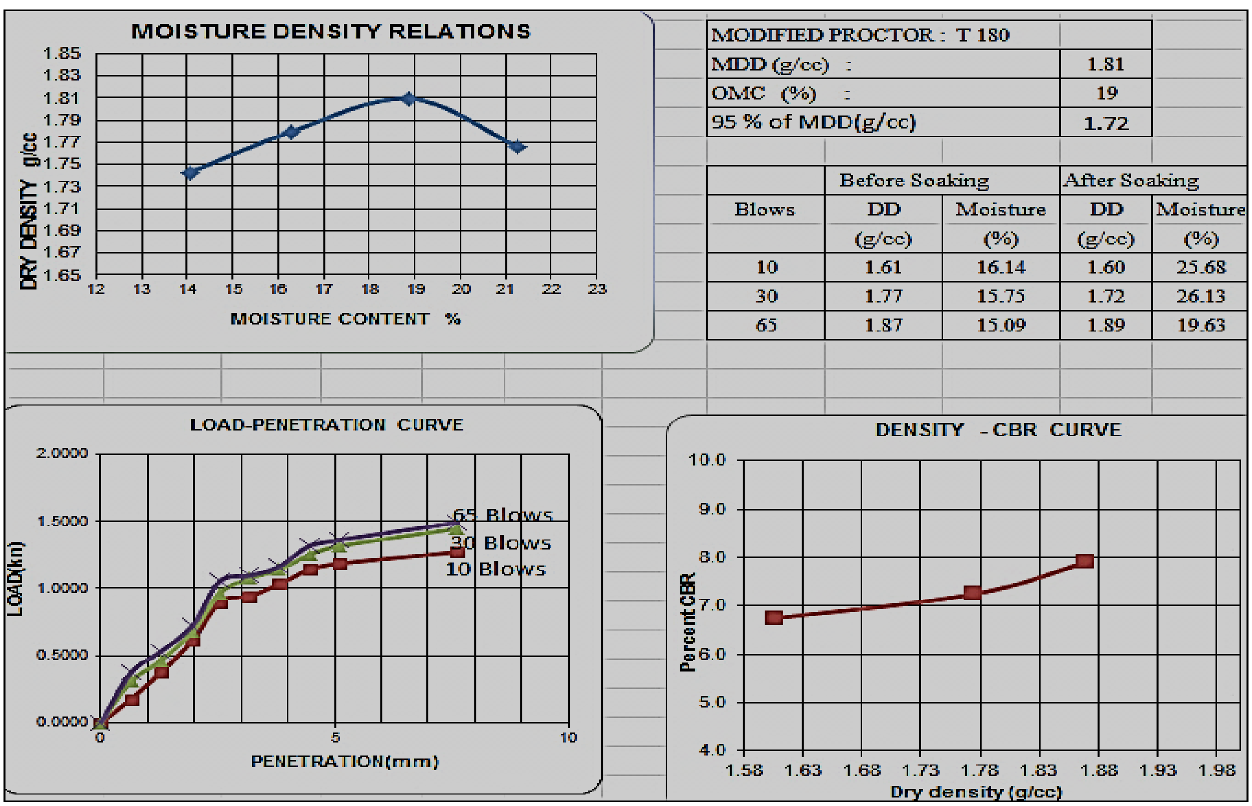


Figure 4.14 Stress vs. Penetration relationship for subgrade material

The above figure expresses the results of the stress-penetration relationship for the subgrade materials at station KM 102+030 CBR soaked and dry density values. This section was presented to visualize the process results, while the other sections were given in Appendix.

4.3 Discussion

The following comparisons and discussions are based on the interpretation of all data gathered from the study sections that had been obtained at Bako-Nekemte Rehabilitation of Road Project were presented in orderly manner.

4.3.1 Road pavement surface deterioration

The different types of deterioration observed in the study area are presented on Figure 4.2

The observation results show that Edge cracking, pumping, raveling, pothole and rutting types of deterioration were recognized along the selected sections. The causes of the observed deterioration could be moisture fluctuation, poor drainage facilities, and poor method of compaction as described by Keith RK. (1992). Similar observation have been reported by ERA (2000),Awoke (2005), Hagos (2006) and Berhanu (2009) in which moisture fluctuation is the main cause of deterioration in tropical climate. These may indicate that the analyzed road sections are in bad conditions. It can be seen from the photos taken that the status of the road section was in bad condition (Figures 4.2).

The pothole was due to the removal or disintegration of materials in weakened spots on the pavement surface. In order to evaluate the carrying capacity of the pavement materials due to the traffic loadings throughout its service design life, the following analyses were made.

From the point of view of traffic analysis and according to the ERA pavement design manual, for the traffic Class is “T5” with ESAs ranging from 3-6 million [10]. Based on project design data of the study area, it comprised of about 5.5 Million Cumulative Standard Axle for the design period. This means that the project design data was within the class “T5” [6].

The thickness of each layer is a function of the Equivalent standard Axles (ESAs) and the CBR of the subgrade layer. For the CBR test, the subgrade strength class was classified as S3 with CBR range of 5-7 [10]. According to TRL Road Note 31 Chart 1 [23], the thickness of the base course aggregates and sub base course for traffic class T5 with corresponding ESAs ranging from 3.0-6.0 million must be 20cm thick and 25 cm thick respectively according to project specification. From Table 4-2, it can be seen that the average thickness of the base course at section 1 is 20 cm, at section 2 is 19.42 cm, and the average thickness of natural sub base course at 2 and 3 is less than 25 cm. This indicates that the base course at section 2 and section 3 are less than the minimum thickness of 20 cm and the sub base course of the section 2 section 3 are also less than the minimum thickness, so except base course at section 1 and sub base at section 1 would not be able to carry the traffic loads throughout its service life based on the requirements of standard specification.

On the other hand, the base course and sub base course at section 1 showed that it would be able to carry the traffic loads throughout its service life because it is equal to the minimum required thickness as per approved plan of the project

4.3.2 Traffic volume

Road surfaces often wear under the action of traffic, particularly during the very early life of the road. However the action of traffic continues to wear the surface texture and thus gradually reduces the high speed skidding resistance, [12]. **Traffic volume** and **size** contributes to road safety and conditions. Recognizing of vehicles' volume and sizes (industrial transportations) is the key for decreasing road deterioration. The composition of the individual vehicles per category in the project road count sections for Bako-Nekemte presented here under and observed the traffic volume and types of the vehicles are determined Table 3.6 and 3.7. There are two types of vehicles category on this road section currently.

A. Passenger vehicle category

The Passenger vehicle category includes Car, S/Wagon& pickup, small Bus and Large buses. The percentage composition of passenger vehicle category estimated about 1%, 19%, 12% and 6% respectively of total traffic on the project road section. Higher percentage of Station wagon observed dominantly.

B. Freight vehicle category

The Freight vehicle category includes small Truck, Medium Truck, heavy trucks and truck

&Trailer. The percentage composition of the Freight vehicle category estimated about 1%, 27%, 23%, and 12% respectively. Higher percentage of medium trucks observed dominantly and used for transporting goods at different town along the project. So recognizing of vehicles' volume and sizes or category is the key for decreasing road deterioration. As, observed the composition of the individual vehicles per category and volume on the Bako-Nekemte road section currently not changed as per design project specifications.

4.3.3 Compaction test

The subgrade material had been evaluated and it can be observed from Table 4.11 that there was increased of about 1.85% at section 1 and decrease 0.4% at section 2 and an increase of 1.8% at section 3 on moisture content at the field than the optimum moisture content obtained in the laboratory. The increased of the moisture content at section 1 was mainly due to percolation of ground water through the cracks and potholes and at section 3 due to poor drainage during the rainy season on the road surface. Aside from this, it was observed that the main cause of the deterioration of these road sections which was known to have seepage of the water within the embankment. This will resulted decrease in dry density. The compaction at sections 1 and section 3 were relatively not good because of this decreased dry density. Hence these finding indicates that the subgrade material at section 1 and section 3 consisted of less at the required dry density as per the standard specification. Based on the ERA Pavement Design Manual [6], it is recommended that the top 25 cm of all subgrades should be compacted to a relative density of at least 95% of the maximum dry density achieved by heavy compaction. Likewise section 2 is greater than this required specification.

Based on the sample conducted, it was found out that the selected material used as a capping layer or selected subgrade fill material at sections 1, 2 and 3 were fine grained soils. At section 1, the average MDD for the selected capping material was 1.70 gm. /cc, but the average of MDD of field result of section 1 of this material is 1.90 gm./cc. In comparison percentage of compaction the MDD is 93.21% to 95.79% which was less than the required of 95% based on specification. The average OMC for the selected material was 22.17% and the range of moisture content that was found during field test had been recorded between 17.1% and 24.00% or averagely 21.64 which is greater than the OMC.

The selected material used for medium deterioration road at section 2 had an average MDD of 1.96 gm./cc and an average OMC of 13.42 %. The field test results for the dry density and moisture content were in the range of 1.82gm/cc to 1.87 gm. /cc, (1.85 gm. /cc) and 13.50% to 17.7%, 14.9% respectively. From these results, it was seen that the percentage of compaction ranges from 96.56% to 98.78 %, which was greater than the required and the moisture content was nearly equal to the OMC.

At section 3, the field moisture content was 18.3% and MDD was 1.84 gm. /cc averagely. The average OMC and MDD for the selected material of the section 3 were 19.17% and 1.77 respectively. From these results, it was found out that the percentage of compaction ranges from 97.35% to 100.00 % which was greater than the required of 95% based on specification.

The natural sub-base material for the highly deteriorated road section at section 1 had an average OMC of 15.41% and an average MDD of 1.87gm/cc. The field dry density observed 2.00gm/cc while the moisture content was 15.56%. It was found that the average in moisture content is equal in both field and laboratory by using given tolerance -2% or +2% of OMC [5]. Because of this result the percentage of compaction had found in the range 95.70 % to 96.80% which was greater than the required of 95% based on specification.

The average optimum moisture content observed during the laboratory test results for natural sub base course of medium deterioration road section 2 was 12.24%, while the average of the field density test results was 12.81%. The field result of moisture content is much higher than the OMC. Likewise, the average MDD result, was found to have 1.96 gm./cc, during laboratory test, on the other hand, the average values found in the field test is 1.87gm/cc and the percent compaction was observed from 95.56 % to 97.76 %, relatively which is greater than the specified requirements.

The field moisture test results for section 3, the subbase material represented value averagely 17.17% while the average OMC found from laboratory test results was 12.75%. The specification tolerance for moisture content is **+/- 2% of OMC [5].** This study showed that the value is out of the tolerance based on standard specifications. The average MDD for the natural subbase material is 2.05 gm./cc. The results had been found in the field 1.77 gm. /cc averagely. Because of this result the percentage of compaction ranges from 91.44% to 94.63%. These values are below the standard specification which requires at least 95% of the maximum compaction.

The field moisture content of the base course material as shown in Table 4.5 is 2.50% less than the optimum moisture content obtained in the laboratory 3.04% averagely by using given tolerance at sections 3. The average decrease in moisture is 0.540% at section 3, but the field moisture content of the base course materials at sections 1 and 2 are respectively 2.90% and 3.89% averagely, while the moisture content obtained in the laboratory test of section 1 and 2 are respectively 3.53% and 4.00% which is below the optimum moisture content obtained in the laboratory. The field dry density at all sections are ranges from 98.56% to 101.78%, which is greater than the requirements set in the standard specification which is comparatively greater than 98%.

Table 4.11 Comparison of field density and laboratory compaction test of subgrade material

Station (Km)	Field test		Laboratory test	
	Water content (%)	Dry density (g/cm^3)	OMC (%)	MDD (g/cm^3)
130+523	30.7	2.18	30.75	1.48
130+580	25.36	2.19	17.6	1.62
Average	28.03	1.71	24.18	1.55
102+030	25.4	1.61	25.40	1.61
102+191	13.5	1.93	16.72	1.83
Average	19.45	1.77	21.06	1.72
76+240	13.5	1.93	16.70	1.74
76+568	21.4	1.51	25.80	1.47
Average	17.45	1.72	21.25	1.61

Table 4.12 Comparison of field density and laboratory compaction test of capping materia1

Section	Station	Field test		Laboratory test	
		Water content (%)	Dry density (g/cc)	OMC (%)	MDD (g/cm3)
1	130+523	24	1.23	23.83	1.685
	130+570	23.83	1.85	24.00	1.690
	130+600	17.1	1.75	18.7	1.73
	Average	21.64	1.61	22.17	1.70
2	102 +030	13.5	1.82	11.4	1.84
	102+175	17.7	1.87	11.74	1.83
	102+187	13.5	1.87	16.83	1.84
	Average	14.9	1.85	13.32	1.84
3	76+212	17.7	1.97	17.41	1.72
	76+457	17.7	1.83	24.18	1.48
	76+ 568	13.5	1.72	24.95	1.469
	Average	16.3	1.84	22.18	1.56

Table 4.13 Comparison of field density and laboratory compaction test of natural sub base material

Section	Station	Field test		Laboratory test	
		Water content (%)	Dry density (g/cc)	OMC (%)	MDD (g/cm3)
1	130+523	15.2	1.94	18.8	1.60
	130+570	12	2.13	14.00	2.15
	130+600	13.5	1.95	13.44	1.86
	Average	13.56	2.00	15.41	1.87
2	102 +030	15.6	1.78	11.23	2.14
	102+175	11.23	2.14	12.00	1.79
	102+187	17.6	1.62	13.5	1.95
	Average	14.81	1.87	12.24	1.96
3	76+212	23.83	1.85	11.69	2.18
	76+457	14.7	1.78	10.87	2.14
	76+ 568	19	1.68	15.7	1.83
	Average	19.17	1.77	12.75	2.05

Table 4.14 Comparison of field density and laboratory compaction test of base course

Section	Station	Field test		Laboratory test	
		Water content (%)	Dry density (g/cc)	OMC (%)	MDD (g/cm3)
1	130+523	3.05	1.91	2.20	2.26
	130+570	2.24	2.26	3.40	2.23
	130+600	3.05	2.23	5.00	2.15
	Average	2.9	2.13	3.53	2.21
2	102 +030	5.00	1.76	2.30	2.24
	102+175	4.4	1.85	7.40	2.10
	102+187	2.28	2.21	2.30	2.21
	Average	3.89	1.94	4.00	2.18
3	76+212	2.28	2.21	2.40	2.19
	76+457	2.11	1.84	3.22	2.15
	76+ 568	3.12	1.78	3.52	2.08
	Average	2.50	1.96	3.04	2.14

4.3.4 Atterberg's limit

From the laboratory results, the average liquid limit, Plasticity limit and Plasticity index of the subgrade and that of the selected subgrade fill material is summarized below

Table 4.15 Atterberg's Limits of pavement Materials

Section	**Subgrade**			**Capping layer**		
	LL (%)	**PL (%)**	**PI (%)**	**LL (%)**	**PL (%)**	**PI (%)**
1	56.67	33.00	23.67	42.9	30.10	12.77
2	57.5	33.20	24.30	36.63	29.27	7.37
3	57.13	35.50	21.63	37.93	30.83	7.10

4.3.5 Grain size analysis

Comparing the laboratory test results for gradation with that of the specification for base course, the results showed that for all the three sections were within the limit. Besides the base course materials used for all the three sections of the road were similar based on the results of sieve analysis.

The comparison of the particle size distribution curve of the laboratory test results of all samples with the recommended particle size distribution for mechanically stable graded crushed stone used as base course material (GB1) is given in Table 2.3, while for granular sub base materials (GS) is shown in Table 2.6 based on ERA Pavement Design Manual [1] is given from Figure. 4.7 Up to Figure. 4.9

From Figure 4.10 to Figure.4.12, it can be observed that the gradation of the base course at all the three sections, the gradation is within the recommended range. However, this recommended range will come to invalid if there is a possibility of moisture fluctuation because of poor drainage condition. Since this moisture fluctuation can be affected or washed out the fine ingredients of the base course material. The stability of coarse grained materials would depend on the grain-to-grain contact and are difficult to compact. Hence, such materials could not attain the required percent of field compaction because of low density. This was one of the reasons for the low field density obtained from the field test shown in Table 4.1. Moreover, such materials observed that it was displaced under load due to less supporting resistance. This

could also be the reason for the rutting observed along the road as discussed above.
Furthermore, coarse-graded mixes can be excessively permeable to water due to their inherently larger void sizes. These larger void sizes increase the chances of their interconnection, which easily allow the water entry and infiltration and in turn, leads to higher permeability. This higher permeability resulted the underlain layers to moist that decrease its density.

From figure 4.12 to figure. 4.14, it can be observed that the gradation of the natural sub base at all the three sections. Gradation analysis was made for this layer at all sections and have different gradation curve with the section. However, it could be noted that section 1 and 2 lie within the recommended range provided at the project and ERA specifications. But the gradation curve of section 3 was not in the range of provided at the project and ERA specifications

Based on, the gradation curve, it can be summarized that, the materials used at section 1 and 2 are relatively stable enough and have a good resistant to erosion. This can be compacted to a very dense condition which produce and develop good bearing capacity and shearing resistance. However, at section 3 the material used was course which is not good resistant to erosion stable enough.

Sieve analysis was also conducted for selected subgrade fill material and for the subgrade material. There was no recommended grading or plasticity criteria for these materials. However, according to the ERA Pavement Design manual, it is desirable to select reasonably homogeneous materials, since the overall pavement behavior is often enhanced the quality while proper selection of materials were show the least change in bearing capacity from dry to wet which is important.

4.3.6 Soil classification

From AASHTO classification system for subgrade and capping layer based on the Table 4-15 The plasticity index value of subgrade material at all sections were found to have comprised A-7-6 which is greater than LL-30.
The plasticity index value of the selected subgrade materials or capping materials at 1 and 2 sections were classified as A-7-5, which is less than LL-30. Whereas, the plasticity index value

of the selected subgrade materials or capping materials at section 3 was classified as A-7-6, which is greater than LL-30,

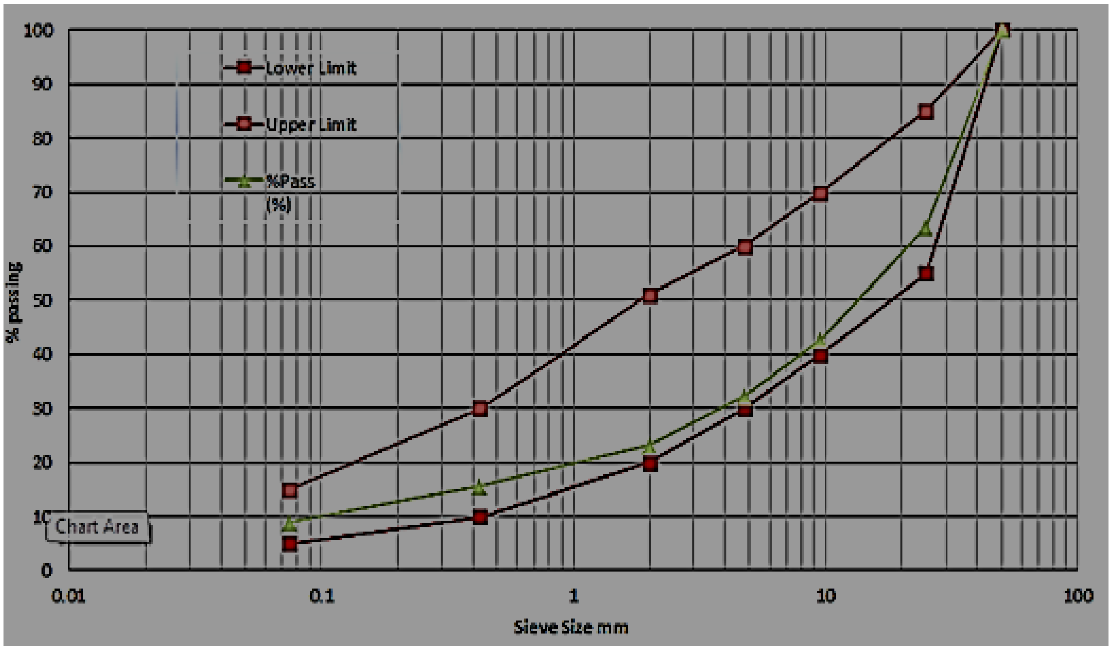


Figure 4.15 Comparison of base course material at section 1 with GB1

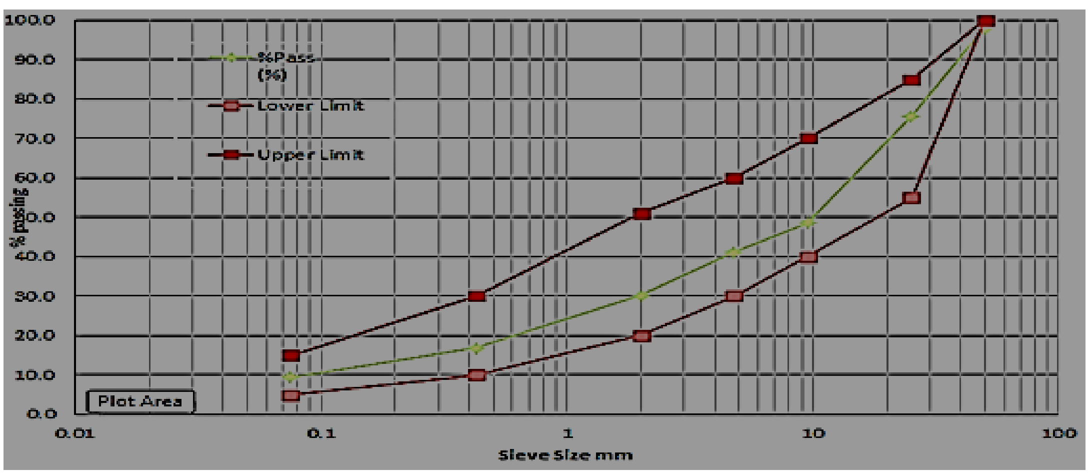


Figure 4.16 Comparison of base course material at section 2 with GB1

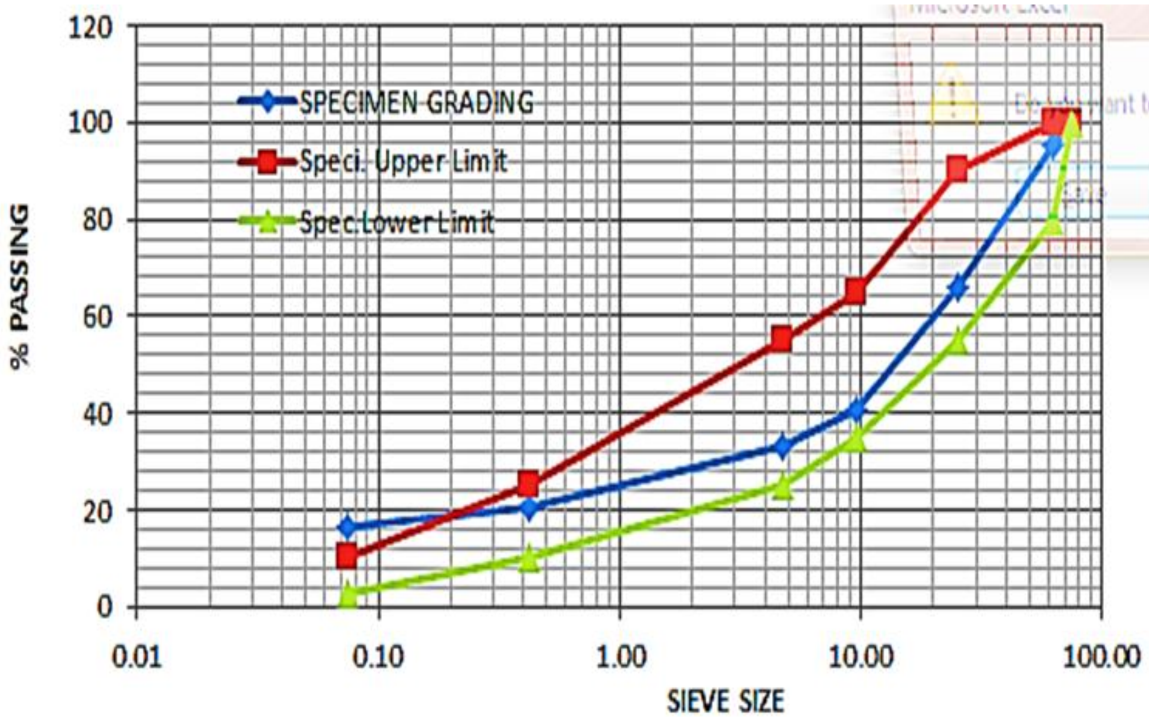


Figure 4.17Comparison of base course material at section 3 with GB1

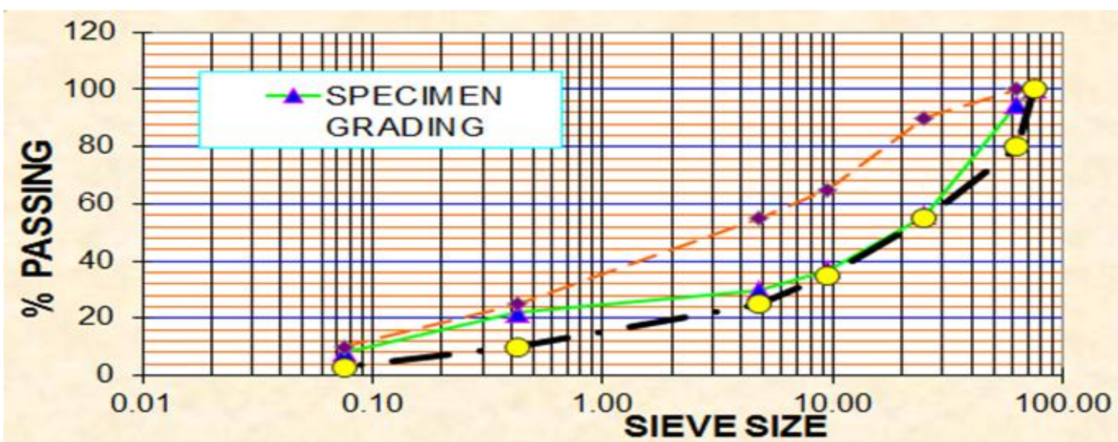


Figure 4.18 Comparison of natural sub base course material at section 1 with GS2

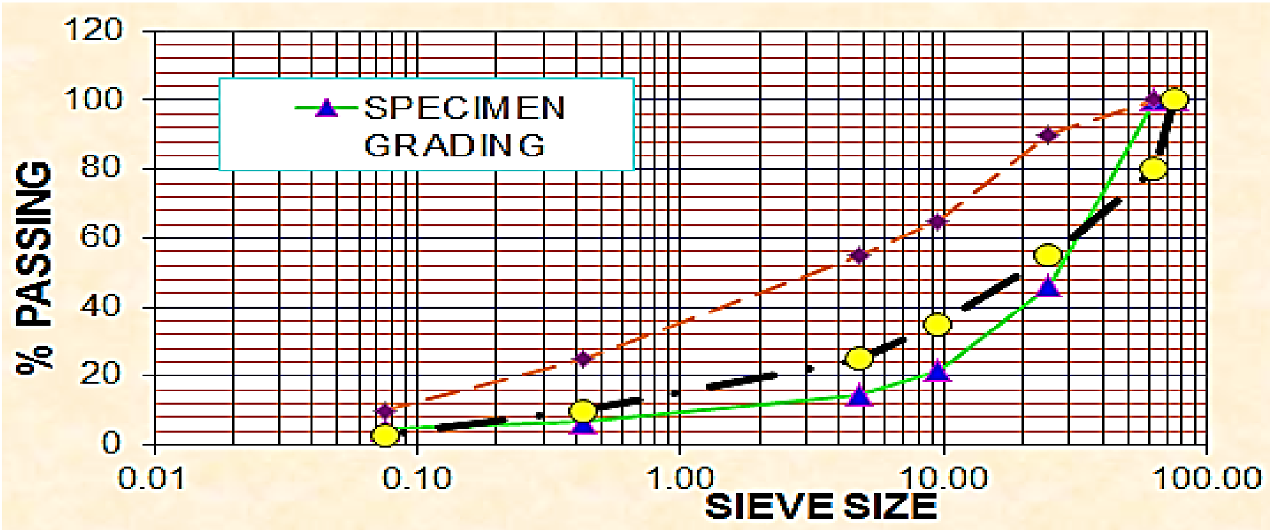


Figure 4.19 Comparison of natural sub base course material at section 2 with GS2

4.3.7 California Bearing Ratio (CBR) Test

Based on the project specification in table 2.8 and the recommendation given in Table 2.2 taken from ERA Pavement Design Manual volume I [5], for fresh, crushed rock (GB1), the CBR after soaking should be greater than 100 percent. As can be seen from the laboratory test results of this study area given in Table 4.10, the CBR of the Crushed base course material used ranges from 105.33 to 124 averagely. Thus, from the point of CBR view the material is suitable to use as base course when compacted at its optimum moisture content and compacted to its maximum dry density. The CBR value at the field dry density is read from the laboratory dry density versus CBR curve. Example, the CBR value at the field dry density at section 1 is 130% > 100%, at section 2 is 115% >100 and at section 3 is 100=100%.

Considering natural subbase, the recommendation given in Table 2.2 taken from ERA Pavement Design Manual volume I [10] and the recommendation in table 2.7 taken from the project specification, for natural gravel (GS2), the CBR after soaking should be greater than 30. And the result obtained from the laboratory was greater than 30 ranging from 36.67 to 47.67averagely at section 2 and 3. The CBR values obtained from the dry density versus CBR curve at section 1 is 29.17, at section 2 is 48.00 and at section 3 are 37.00.

According to the specification given by ERA, Gravel or gravel-soil (GC) with CBR after soaking greater than 15 can be used as capping layer or selected subgrade fill material. The material used as selected subgrade fill material at section 1 is with an average CBR of 21.67 and at section 2 and section 3 is with an average CBR of 21.23 and 18.20 respectively. This indicates that, the selected material at the three sections used as capping layer is greater than the requirement given in the specification.

The average CBR obtained for Subgrade material when compacted at its optimum moisture content and maximum dry density obtained 5.22%, at section1, 6.47% at section 2 and 5.57% at section3. From table 2.1, the subgrade strength class for CBR range of 5-7 is S3, 8-14 is S4 and for 15-29 is S5. As most of the laboratory results lay on the range 5-7, it can be classified as S3.

4.4 Observations and proposed remedial measures

Based on the findings and observation of the study, at section 1 all the deterioration of the road could be related to Presence of water on or in the subgrade or sub base like percolation of water under the ground to the subgrade, infiltration of rain to the surface of the pavement during rainy season and the seepage of water at the interface of the of the pavements. As the water is ejected, the carries material was fine, thus resulting in progressive material deterioration and loss of support.

Generally, the main cause of pavement deterioration of this section, as observed in the field, was due Presence of water on or in the subgrade or sub base along with heavy loads passing over the pavement surface and cracking the pavement. The Contractor must provide proper material for these voids beneath the pavement should be filled with high softening under sealing asphalt to prevent the intrusion of water in to the subgrade and sub base. The remedial measure of this highly deteriorated section (6) road should be reconstruction of layers up to subgrade level must be undertaken in order to repair. Additionally, in areas where cracks had already occurred, sealing of cracks with bitumen or any other suitable material to prevent further cracking and to minimize infiltration of water during rainy seasons.

Depending on the observation and finding of the road section at section 2 the deterioration of the road could be related to the insufficient thickness of the base course of the road section. At this section the speed breaker was provided, because of this the speed of the vehicles decrease around that, so that the pavement was consolidated or deflected under traffic in one or more of the underlying courses or displacement in the surface layer itself. The main cause of pavement deterioration of this section, as observed in the field, was due insufficient thickness of the base course. The remedial measures of this medium deterioration road section is reconstruction of base course and layers above it, by considering this speed breaker which increase the load by decreasing speed of the vehicles as per the requirement set in the ERA Standard Manual.

Addionally for the low deterioration of the road at section 3 the observed remedial measure is providing of proper side ditch which minimize the ejection of water in to the subgrade or sub base. The main cause of this road section deterioration, as observed in the field, was because of lack proper side ditch of the road. It was observed that the failure was within the edge of the road already deteriorated. But the results of the field test conducted during the field investigation showed that the dry density values of the subgrade were greater than the values stipulated in the specification and the CBR values found which is related to the field dry density for subgrade materials is less than the values stipulated in the ERA specification. So the Contractor should provide proper drainage facilities farther away from the toe of the shoulder and under the foundation of subgrade to prevent the ingression of water into the pavement layers.

Table 4.16 Summary of types of deterioration which are common along the road section,

Types of deterioration	Description	Observed possible causes	Recommended remedial measures
Pumping	This refers to pavement movement under passing loads resulting in ejection of a mixture of water, sand, clay, or silt.	Presence of water on or in the subgrade or sub base along with heavy loads passing over the pavement surface and deflecting the pavement	The voids beneath the pavement should be filled with high softening under sealing asphalt to prevent the intrusion of water in to the subgrade and sub base
Rutting	Refers to longitudinal surface depression in the wheel- paths of the asphalt pavement.	Consolidation or lateral movements under traffic in one or more of the pavement layer or displacement in the surface layer itself.	Level the pavement by filling the channels with hot plant-mixed asphalt material. Follow with a thin asphalt plant-mixed overlay
Raveling	The wearing away of the pavement surface caused by the dislodging of aggregate particles.	Lack of compaction, construction of a thin lift during cold weather, dirty or disintegrating aggregate, too little asphalt in the mix, or overheating of the asphalt mix.	Surface treatment, such as seal coating, surface dressing, thin overlaying of Bituminous Surface Course(Hot-Laid), SBST, DBST

Potholes	Bowl-shaped holes of various sizes in the pavement surface.	Weakness in the pavement resulting from too little asphalt, failure of base due to poor drainage	Temporary repair through filling with pre-mixed asphalt patching material and Permanent repair through filling with new base and surface material.
Edge cracking	This edge cracking typically starts as crescent shapes at the edge of the pavement, which will expand from the edge until they begin to resemble alligator cracking.	This type of cracking results from lack of support of the shoulder due to weak material or excess moisture due to lack of the proper drainage.	The possible remedial measures of this edge cracking type deterioration are elimination of the excess moisture by building shoulders and providing proper drainage with good materials.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

Depending on the observation and investigation of pavement condition survey, the field tests and laboratory tests the following conclusions are drawn.

1. The laboratory and field test results that had been obtained on the pavement materials used for the construction of the road section under the research discussed and compared with the requirements of the standard specification, which shows poor results at section 1 and relatively good results at section 2 and 3.

2. The investigation have revealed that different type and degree of deteriorations observed on the pavement surface.

- ✓ High deteriorations observed at section 1, (pumping and potholes) type of deterioration are observed at this section.
- ✓ Medium deterioration is observed at section 2, (rutting and potholes) type of deterioration are observed at this section.
- ✓ Relatively low deterioration is noticed at section 3, (Edge cracking and Raveling) types of deterioration are observed at this section.

3. The major causes of deterioration of these flexible pavement along the Bako-Nekemte road section are presence of water on or in the subgrade or sub base, lack of proper drainage at some points , lack of routine and timely maintenance on the sections of potholes and cracks, insufficient thickness, poor shoulder design, seepage of groundwater and infiltration of surface water into the pavement layer, poor field compaction, poor method of construction (i.e. construction of pavement layers during rainy season).

4. The thickness of the base course and sub-base course for traffic class T5 with ESAL of 3.0-6.0 million should be 20cm and 25cm respectively.

However, the average thickness of the existing road layer of the base course is 19.42 cm and that of the sub-base course is 22.67cm. Based on these values for base and sub-base course, the researcher concludes that the pavement would not be able to carry the existing traffic loading through-out its service life

5. CBR value related to the field dry density for base course is greater than the standard specification for low, medium and high deterioration road sections relative to the dry density obtained.

6. Averagely in areas of low and high deterioration road sections, the moisture contents obtained on the field for natural sub base and capping materials are much higher than the optimum moisture contents obtained in the laboratory test without considering given tolerances.

7. However, averagely in area of medium deterioration section the moisture content obtained on the field was relatively equal or less than the OMC obtained in laboratory by considering given tolerance.

8. The average of the observed CBR values of the natural sub bases is good (greater than 30) for section 2 and 3 according to standard specification. But at section 1 the CBR value of natural sub base is less than 30 (minimum of the specification). Therefore, the deterioration frequently observed on the road surface is significantly influenced by natural sub base strength. Related to the field dry density the value of the CBR, for capping layer materials are good or greater than the minimum requirement of standard specification at all sections.

5.2 Recommendation

The following recommendations are given based on the field and laboratory test results discussed below.

The base of the deterioration for all road section must be reconstruction and maintenance. Moreover, it is important to protect or improve the shoulder areas to enhance both serviceability and structural capacity levels.

The sections with various sizes of cracks and potholes must be patched with good quality asphalt. Seal coats shall be applied to prevent infiltration of water through cracked surfaces should at different sections of the pavement. The deteriorated part of the pavement at section 3, with poor material due to lack of proper drainage must be removed and replaced with required depths which have weak subgrade.

Adequate longitudinal, turn- outs and other drainage facilities should be provided in order to overcome the drainage problem at this section which is especially important due to the water functionless of the materials used for bases course and sub-base course. Additionally, Concrete side ditches should be constructed to sufficient depth below the sub base to ensure that free water level in the ditch will always be below the pavement layers (i.e. the side ditch on the left side shall be constructed by concrete so as to prevent infiltration of water to the underlying strata.

The subgrade should be properly compacted to meet the minimum requirements of the standard specifications and adequate surface as well as sub-surface drains, turn- outs and protections should be provided. Moreover, Seal coats shall be applied to prevent infiltration and seepage of water through cracked surfaces should at section 1.

When studying with the problems of subgrade drainage, the attention must be given to ground water pumping, seepage and surface infiltration that needs in depth geotechnical investigation. To control these points, construction of consultant must be identified locations of bad ground water table condition when encountered during excavation and determine the proper location of subgrade drains or out nets. Likewise water-bearing strata which will possibly feed water into the pavement structure should be intercepted at some distance away from the roadway section.

The designer and contractor should follow the minimum requirement set by ERA regarding the properties and structural thickness of layers of a road; so as to prevent the pavement from deterioration.

After the pavement is deteriorated, further investigating the causes and giving remedial measures is not enough in order to have normal pavement structure. In addition, starting at the time of the opening for traffic, protecting or keeping the safety of the pavement by creating group member is the best for the long lasting.

REFERENCES

[1] W.Paterson, “Road Deterioration and Maintenance Effects: Models for planning and Management,” Baltimore: The John Hopkins University press, 1987.

[2] O. M. Ogundipe, “Road pavement failure caused by poor soil properties along Aramoko-Ilesa highway, Nigeria,” Journal of engineering and applied sciences, 2008, 3(3), pp. 239-241.

[3] N. Okigbo, “Road maintenance in Nigeria, the way forward,” International journal of research in engineering science, Pan African journal series, Accra, Ghana, 2012.

[4] A. S. Harischandra, Identification of road defects, causes of road deterioration and relationship among them for bitumen penetration macadam roads in Sri Lanka. Master Thesis at the University of Moratuwa, Sri Lanka, 2004.

[5] P. Croney and D. Croney, The Design and Performance of Road Pavements. New York City: McGraw-Hill Professional, 1998.

[6] ERA. Ethiopian Roads Authority Standard Manuals. Pavement Design Manual: Flexible Pavements and Gravel Roads, Volume I. 2002.

[7] Keith RK. Northwest pavement Management systems users Group and Washington State Department of Transportation. March 1992.

[8] AASHTO. Standard Specification for Transportation Materials and Methods of Sampling and Testing. 1993. (Twenty first edition 2001 (Part 1)).

[9] Ethiopian Roads Authority Standard Manuals. Site Investigation Manual. 2002.

[10]Ethiopian Roads Authority Standard Manuals.Gedo-Nekemte Road Upgrading Project.2009.

[11] Ethiopian Roads Authority Standard Manuals. Pavement Rehabilitation and Asphalt Overlay Manual. 2002.

[12] Chen FH. Foundations on Expansive Soils. New York Elsevier Science Publisher. 1998.

[8] [13] Awoke S. Further Investigation of Road Failures Constructed on Expansive Soils of Central Ethiopia Addis Ababa-Nekemte Road as a Case Study. A Master's Thesis, Addis Ababa University. 2005.

[14] Hagos G. Assessment of Road Failures Constructed on Red Clay Soils of Western Ethiopia and their Remedial Measures. A Master's Thesis, Addis Ababa University.2006.

[15] AASHTO. Standard Specification for Transportation Materials and Methods of Sampling and Testing. 1993. (Twenty first edition 2001 (Part 2)).

[16] Flexible Pavement Design Manual, Department of Transportation, State of Florida. 2002

[17] Hudson, W.R., Monismith, C.L., Dougan, C.E., and Visser, W. (2003), Use Performance Management System Data for Monitoring Performance: Example with Superpave, Transportation Research Record 1853, TRB, Washington D.C.

[18] Transport Research Laboratory. A Guide to the Structural Design of Bitumen- Surfaced Roads in Tropical and Sub-tropical Countries. Overseas Road Note No. 31. TRL, Crowthorne, for ODA, London, UK, 1993.

[19] Braja, M. D., Soil mechanics Laboratory manual. (Fourth edition) [15] Ethiopian Roads Authority Standard Specification.2002.

[20] Ethiopian Roads Authority Standard Specification.2002.

[21] AASHTO. Standard Specification for Transportation Materials and Methods of Sampling and Testing. 1993. (Twenty first edition 2001 (Part 2)).

[22] Ethiopian Roads Authority. Project Design Review, Gedo-Nekemte Road Upgrading Project Draft Submission. 2007.

[23] Flexible Pavement Design Manual, Department of Public Works and highways, Philippines. [24]. Tanzania Pavement and Material Design Manual. 1999

[25]. Berhanu A. Analysis and Modeling of Rutting for Long Life Asphalt Concrete pavement. A PhD. Thesis, Technische Universität Darmstadt, German. 2009.

[26] A Policy on Geometric Design of Highways and Streets, AASHTO, ISBN: 1-56051-156-7, Fourth Edition, Washington DC. 2001

[27] Ethiopian Roads Authority. Road Sector Development: 1997-2007. Final Draft. Addis Ababa Ethiopia.1996.

[28] Yang. H. Huang. Pavement Analysis and Design," University of Kentucky.1993.

[29] Ethiopian Roads Authority. Pavement and Shoulder Cracking Report of Addis Ababa – Nekemte Road Project.2000.

[30] Federal Democratic Republic of Ethiopia. Bako-Nekemte Road Upgrading Project. Appraisal Report. Addis Ababa.2006.

[31] Portugal EPAM3-3rd European Pavement and Asset Management Conference Study of

the Causes and Remedies of Premature Surface Cracking of Asphalt Pavements. Coimbra.7-9 July.2008.

[32] Government of India Ministry of Road Transport and Highways Transport Research Wing. July 2010.

[33] HUDSON, W.R., R. HAAS, AND R.D. Pavement Management System Development, NCHRP Report 215. PEDIGO.1979.

[34] BCEOM. Pavement Management System. Draft Final Report.1998.

[35] ASPHALT INSTITUTE Manual Series No 16 (MS-16): Asphalt in Pavement Maintenance. and PIARC International Road Maintenance Handbook, Volume III. Maintenance of Paved Roads.

[36] TRANSPORT AND ROAD RESEARCH LABORATORY. Overseas Road Note 1 (2nd Edition, Reprinted 1995). Maintenance Management for District Engineers.

[37] ASPHALT INSTITUTE. Manual Series No. 17 (MS-17). Asphalt Overlays for Highway and Street Rehabilitation. The Asphalt Institute, Lexington, Kentucky. June .1983.

[38] ASPHALT INSTITUTE (February 1990 Edition). Research Report No. 83-1 (RR-83-1) Deflection Method for Designing Asphalt Concrete Overlays for Asphalt Pavements. [34] CSRA. Draft TRH 12:- Bituminous Pavement Rehabilitation Design. Pretoria. South Africa. 1983.

[39] Yoder and Witczak. Principles of Pavement Design, Second Edition (Part V: Pavement Evaluation and Rehabilitation). John Wiley and Sons.1975.

[40] TRANSPORT RESEARCH LABORATORY. Road Building in the Tropics. State-of- the-Art Review No.9.1993.

[41] ASPHALT INSTITUTE. Information Series No. 178 (IS-178). Alternatives in Pavement Maintenance, Rehabilitation, and Reconstruction. May. 1981.

[42] ASPHALT INSTITUTE. Manual Series No. 20 (MS-20) Revised Second Edition. Asphalt Hot-Mix Recycling.

[43] TRANSPORT AND ROAD RESEARCH LABORATORY. Overseas Road Note 2 (2nd Edition, Reprinted 1987). Maintenance Techniques for District Engineers.

[44] CSRA. TRH 6. Nomenclature and Methods for Describing the Condition of Asphalt Pavements.1985.

[45] CSRA. Draft TRH 19. Standard Nomenclature and Methods for Describing the Condition of Jointed Concrete Pavements. Pretoria, South Africa.1989.

[46] Mangesha el at., Geology Map of Ethiopia. Second Edition.1996.

[47] Tesfaye C. Hydrogeological map of Ethiopia, Ethiopian institute of geological survey, Addis abeba.1988.

[48] African Development Fund. Environmental and Social Impact Assessment - Gedo-Nekemte road upgrading project.2006

[49] O. G. Omer, A. M. Elsharief and A. E. Mohamed, "Failure Investigation for Recently Constructed Road in Khartoum State," Journal of Geological Resource and Engineering 2,David Publishing Press, 2014, pp 20-31.

[50] T. M. Oguara, A management model for road infrastructure Maintenance. Book of proceedings, 19th engineering assembly, Council for the regulation of engineering in Nigeria, 2010.

[51] M. Sargious, Pavement and Surfacing for Highways and Airports.
London: Applied Science Publishers Ltd, 1975.

[52] S. Y. Wee, R. B. Chan, and H. W. Teo, "Potential modeling of pavement deterioration rate due to cracking," UNIMAS E-Journal of Civil Engineering, Vol 1, issue 1, August 2009.

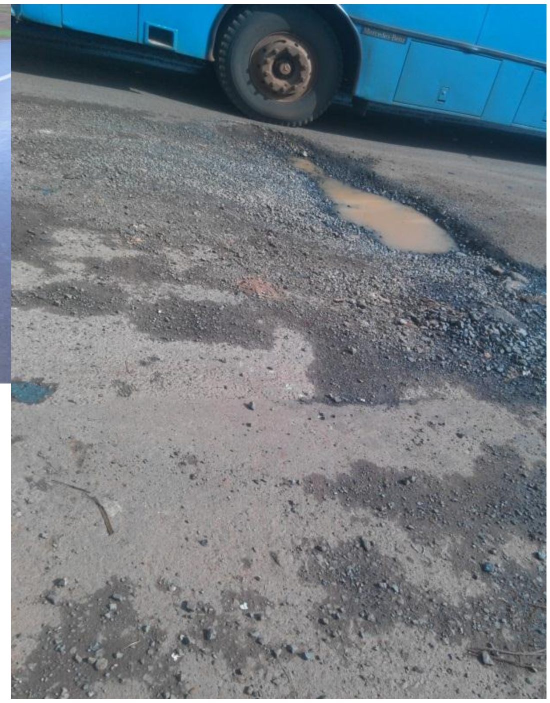
[53] D.Y. Patil Abhijit, Y. B. Patil Prathisthan's, and A. Patil Polytechnic,
"Effects of Bad Drainage on Roads," Civil and Environmental Research, Vol. 1, No.1, 2011.

[54] N. D. Little, and R. D. Jones, "Chemical and Mechanical Processes of Moisture Damage in Hot-Mix Asphalt Pavements," Moisture Sensitivity of Asphalt Pavements- A National Seminar, February 4–6, 2003 San Diego, California. Transportation Research Board of the National Academics, 2003.

[55] N. P. Khosla, G. B. Birdsall, and S. Kawaguchi, "An In-Depth Evaluation of Moisture Sensitivity of Asphalt Mixtures," NCDOT Research Project 1998-08 FHWA/NC/2002-102, 1999.

[56] K. D. Stuart, "Moisture Damage in Asphalt Mixtures- A State-of-Art Report, FHWARD-90-019, Federal Highway Administration, U.S Department of Transportation, Washington, D.C., 20001., 1990.

Appendix A: types of deteriorations



Raveling type of deterioration

Potholes type of deterioration

Edge crack type of deterioration

Pot hole type

Rutting at wheel path type

Pumping types of deterioration

Edge cracking type of deterioration

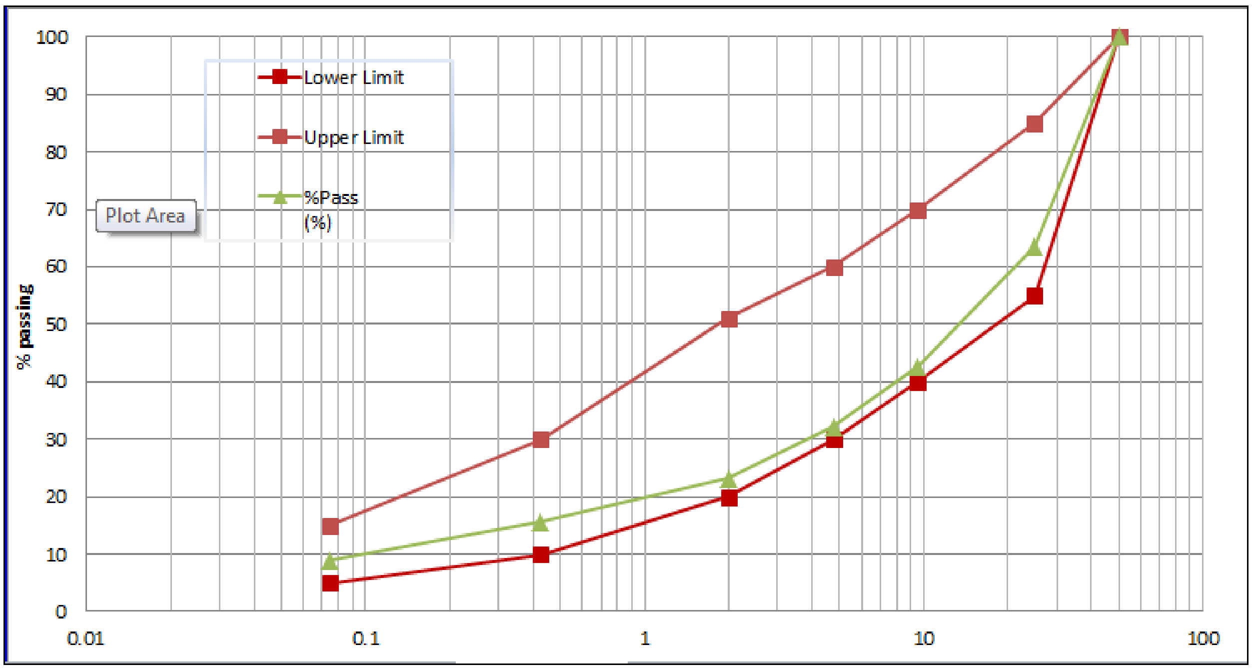
Pumping types of deterioration

pothole at edge damage type

Pot hole and patch

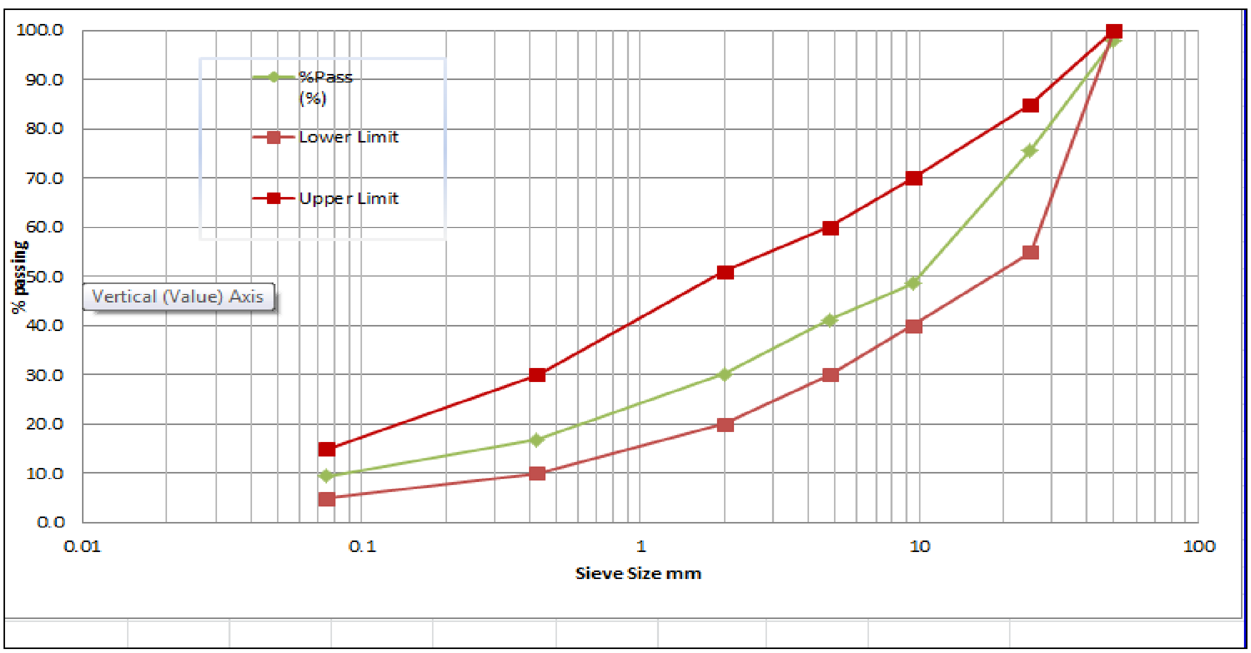
Appendix B: Grain size analysis data of base course sample at section 1

SIEVE ANALYSIS OF BASE COURSE MATERIAL AASHTO T - 27						
REPRESENTING SECTION	section 1		DATE SAMPLED		19/7/2016	
MATERIAL DESCRIBTION	crushed Aggregate		DATE TESTED		23/07/2016	
PURPOSE	Basecourse					
Sieve Size (mm)	Weight Retained (g)	Percent Retained (%)	Percent Cumm. Retained (%)	%Pass (%)	Specification Limit	
					Lower Limit	Upper Limit
50	0.0	0.0	0.0	100.0	100	100
25	4726.0	36.4	36.4	63.6	55	85
9.5	2730.0	21.0	57.4	42.6	40	70
4.75	1340.0	10.3	67.8	32.2	30	60
2.00	1170.0	9.0	76.8	23.2	20	51
0.425	990.0	7.6	84.4	15.6	10	30
0.075	864.0	6.7	91.1	8.9	5	15
Pan	1210.0	9.3	100			
Total	12980.0	100				
		ERA 2002 Grading B				



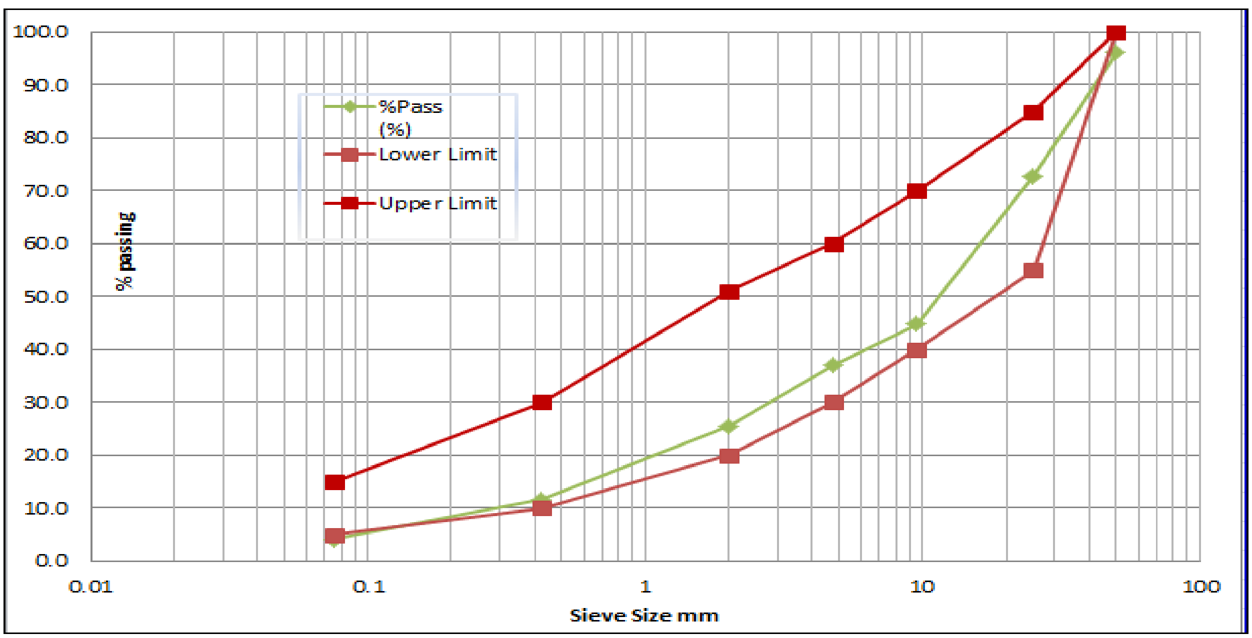
Appendix B: Grain size analysis data of base course sample section 2

SIEVE ANALYSIS OF BASE COURSE MATERIAL AASHTO T - 27					
REPRESENTING SECTION		section 2	DATE SAMPLED		11/07/16
MATERIAL DESCRIBTION		Creshed Aggregate	DATE TESTED		15/07/16
PURPOSE		Basecourse			
Sieve Size (mm)	Weight Retained (g)	Percent Retained (%)	%Pass (%)	Specification Limit	
				Lower Limit	Upper Limit
50	107.0	2.0	98.0	100	100
25	1201.6	22.4	75.6	55	85
9.5	1448.3	27.0	48.6	40	70
4.75	396.9	7.4	41.2	30	60
2.00	590.0	11.0	30.2	20	51
0.425	718.5	13.4	16.8	10	30
0.075	391.6	7.3	9.5	5	15
Pan	510.1	9.5			
Total	5364.0	100			
		ERA2002Grading B			



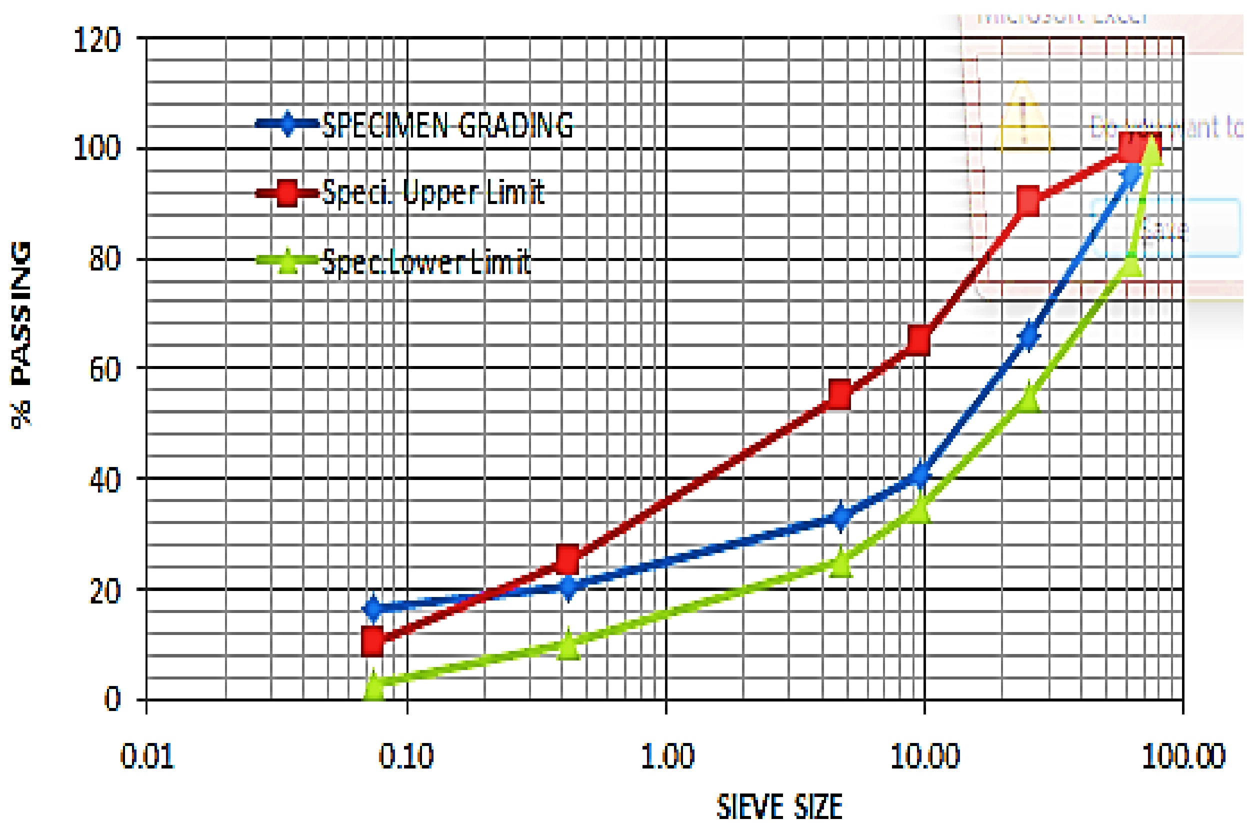
Appendix B: Grain size analysis data of base course sample section 3

SIEVE ANALYSIS OF BASE COURSE MATERIAL AASHTO T - 27					
REPRESENTING SECTION		section 3	DATE SAMPLED		26/07/16
MATERIAL DESCRIBTION		Creshed Aggregate	DATE TESTED		29/07/16
PURPOSE		Basecourse			
Sieve Size (mm)	Weight Retained (g)	Percent Retained (%)	%Pass (%)	Specification Limit	
				Lower Limit	Upper Limit
50	200.6	3.9	96.1	100	100
25	1200.0	23.3	72.8	55	85
9.5	1447.5	28.1	44.7	40	70
4.75	396.9	7.7	37.0	30	60
2.00	590.0	11.4	25.6	20	51
0.425	718.5	13.9	11.6	10	30
0.075	390.0	7.6	4.1	5	15
Pan	210.1	4.1			
Total	5153.6	100			
		ERA2002Grading B			



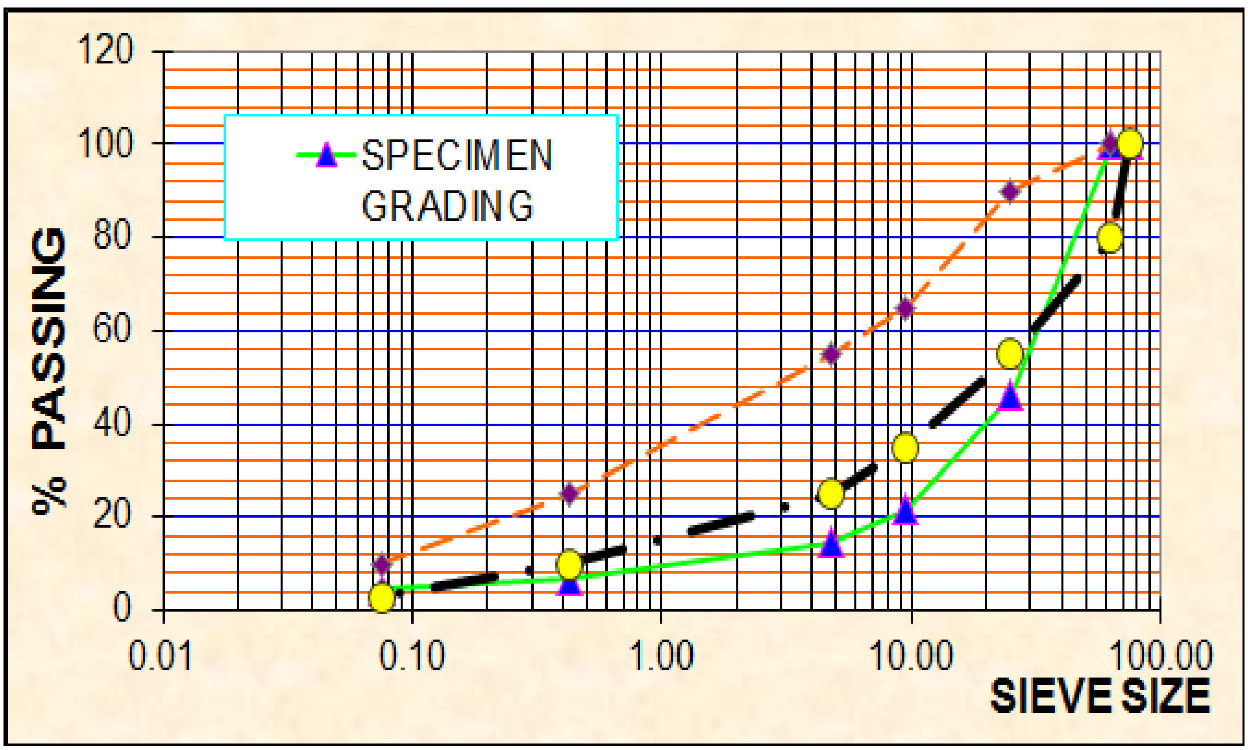
Appendix C: Grain size analysis data of natura subbase sample at section 1

Sampling Date : 29/07/2016			TEST DATE	2/8/2016		
SECTION 1						
PURPOSE	NATURAL	SUBBASE				
Sieve size(mm)	wt. of Ratained	%Ratained	%passing	Specification	Lower limit	Upper Limit
75	0	0.00	100.0	100	100	100
63	0	0.00	100.0	80--100	80	100
25	5477	31.31	68.7	55--90	55	90
9.5	4755	27.18	41.5	35--65	35	65
4.75	1697	9.70	31.8	25--55	25	55
0.425	2464	14.08	17.7	10--25	10	25
0.075	678	3.88	13.9	3--10	3	10
pan						



Appendix C: Grain size analysis data of natura subbase sample at section 2

Sampling Date : 27/07/2016			Sample of : Sub-base Material			
SECTION 2						
PURPOSE	NATURAL	SUBBSAE				
Sieve size(mm)	wt. of Ratained	%Ratained	%passing	Specification	Lower limit	Upper Limit
75	0	0.00	100.0	100	100	100
63	0	0.00	100.0	80--100	80	100
25	7897	53.89	46.1	55--90	55	90
9.5	3585	24.46	21.6	35--65	35	65
4.75	1045	7.13	14.5	25--55	25	55
0.425	1161	7.92	6.6	10--25	10	25
0.075	320	2.18	4.4	3--10	3	10
pan						



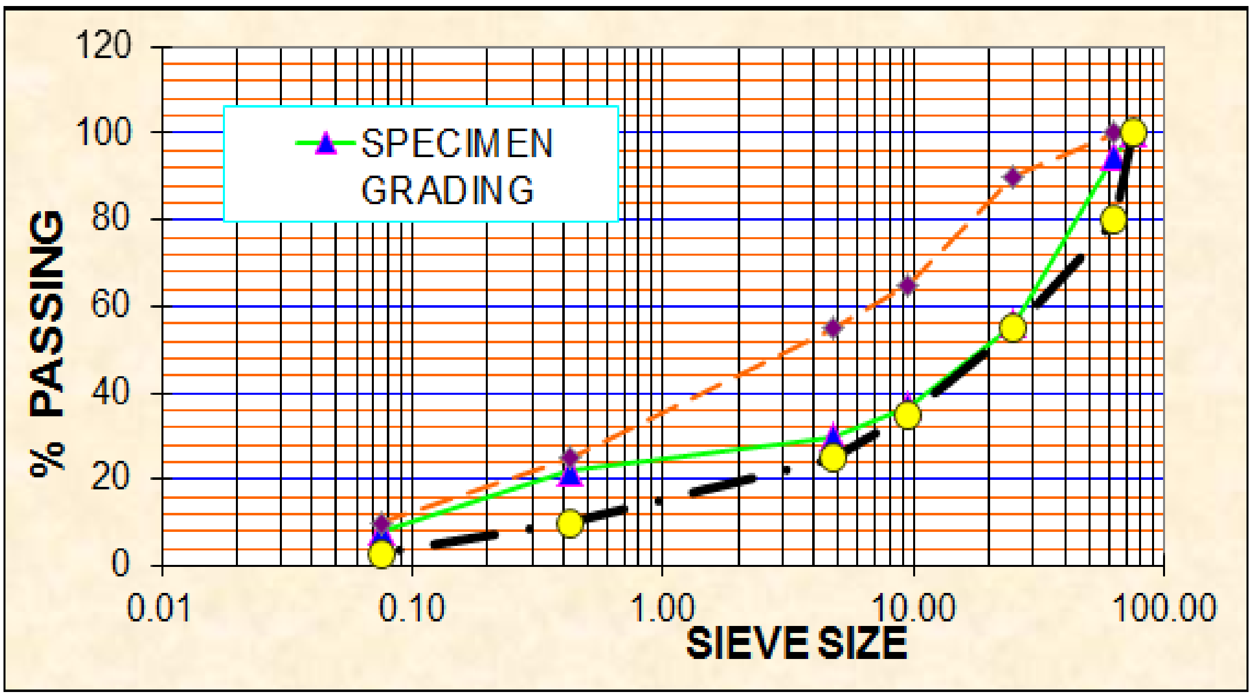
Appendix C: Grain size analysis data of natura subbase sample at section 3

Sieve Analysis (Test Method AASHTO T- 27) NATURAL SUBBASE

Sampling Date : 09/08/2016 **Sample of : Sub-base Material**

SECTION 3

Sieve size(mm)	wt. of Ratained	%Ratained	%passing	Specification	Lower limit	Upper Limit
75	0	0.00	100.0	100	100	100
63	0	0.00	95.0	80--100	80	100
25	7897	53.89	56.0	55--90	55	90
9.5	3585	24.46	37.0	35--65	35	65
4.75	1045	7.13	29.9	25--55	25	55
0.425	1161	7.92	21.9	10--25	10	25
0.075	320	2.18	8.0	3--10	3	10
pan						



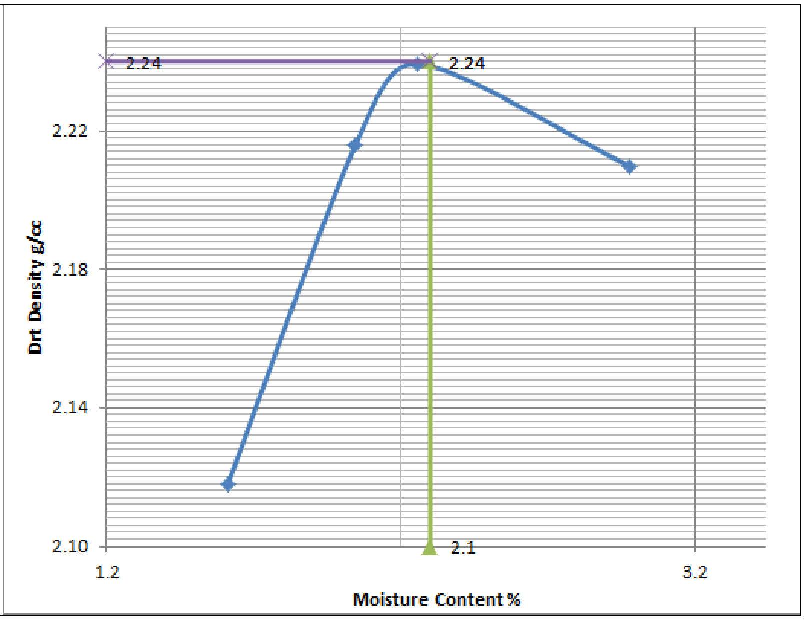
Appendix D: Moisture content determination laboratory test sample at section 1

MOISTURE DENSITY RELATIONSHIP OF SOIL
TEST METHOD : AASHTO T-180 METHOD D

REPRESENTING SECTION	section 1	DATE SAMPLED	22/07/2016
MATERIAL DESCRIBTION	crushed Aggregate	DATE TESTED	25/07/2016
PURPOSE	Base course		

DENSITY	TRIAL NUMBER	1	2	3	4	5		
	WEIGHT OF SAMPLE (g)		6000	6000	6000			
	WATER ADDED litre		0.0	50.0	100.0	150.0		
	WEIGHT OF SOIL + MOLD (g)		11,120	11,350	11,410	11380		
	WEIGHT OF MOLD (g)		6590	6590	6590	6590		
	WEIGHT OF SOIL (g)		4530	4760	4820	4790		
	VOLUME OF MOLD (cc)		2105	2105	2105	2105		
	Wet DENSITY OF SOIL (g/cc)		2.15	2.26	2.29	2.28		NMC
MOISTURE	CONTAINER NUMBER		A4	CH	A2			A1
	WET SOIL + CONTAINER (g)		243.46	290.17	342.79	338.78		275.2
	DRY SOIL + CONTAINER (g)		240	284.91	335.78	329.74		273.1
	WEIGHT OF WATER (g)		3.46	5.26	7.01	9.04		2.1
	WEIGHT OF CONTAINER (g)		25.3	27.8	25.3	26.3		23.7
	WEIGHT OF DRY SOIL (g)		214.7	257.1	310.5	303.4		249.4
	MOISTURE CONTENT (%)		1.6	2.0	2.26	2.98		0.8
	DRY DENSITY OF SOIL (g/cc)		2.12	2.22	2.24	2.21		

MDD (gm/cc) :	2.24
OMC (%) :	2.2

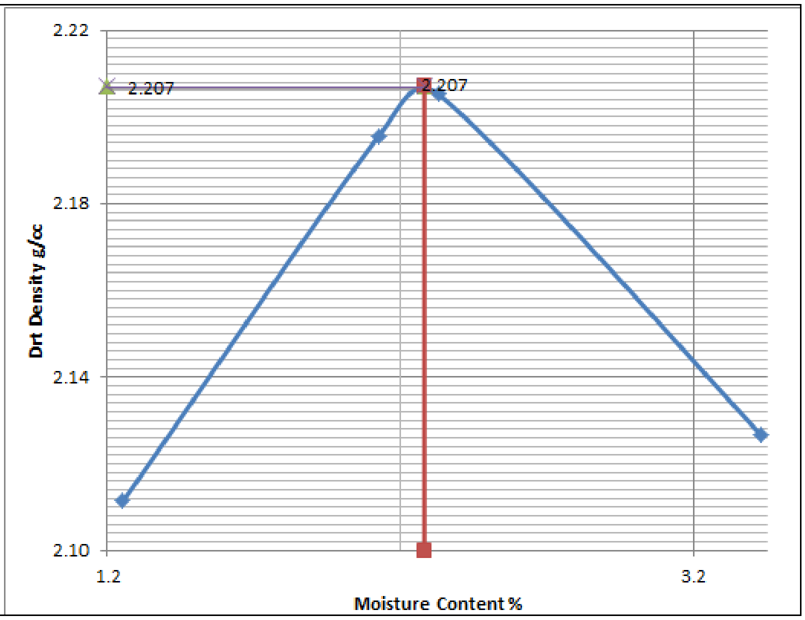


Appendix D: Moisture content determination laboratory test sample at section 2

MOISTURE DENSITY RELATIONSHIP OF SOIL TEST METHOD : AASHTO T-180 METHOD D			
REPRESENTING SECTION	section 2	DATE SAMPLED	18/07/2016
MATERIAL DESCRIBTION	Creshed Aggregate	DATE TESTED	21/07/2016
PURPOSE	Basecourse		

DENSITY	TRIAL NUMBER		1	2	3	4	5		
	WEIGHT OF SAMPLE	(g)	6000	6000	6000	6000			
	WATER ADDED	litre	0.0	50.0	100.0	150.0			
	WEIGHT OF SOIL + MOLD (g)		11,090	11,310	11,340	11,220			
	WEIGHT OF MOLD (g)		6590	6590	6590	6590			
	WEIGHT OF SOIL (g)		4500	4720	4750	4630			
	VOLUME OF MOLD (cc)		2105	2105	2105	2105			
	Wet DENSITY OF SOIL (g/cc)		2.14	2.24	2.26	2.20			NMC
MOISTURE	CONTAINER NUMBER		C	X	1	B			
	WET SOIL + CONTAINER	(g)	305.03	280.17	332.79	328.78			
	DRY SOIL + CONTAINER	(g)	301.57	274.91	325.83	318.74			
	WEIGHT OF WATER	(g)	3.46	5.26	6.96	10.04			
	WEIGHT OF CONTAINER	(g)	25.3	27.4	26.9	25.9			
	WEIGHT OF DRY SOIL	(g)	276.3	247.5	298.9	292.8			
	MOISTURE CONTENT (%)		1.3	2.1	2.3	3.43			
	DRY DENSITY OF SOIL (g/cc)		2.11	2.20	2.21	2.13			

MDD (gm/cc) :	2.21
OMC (%) :	2.3



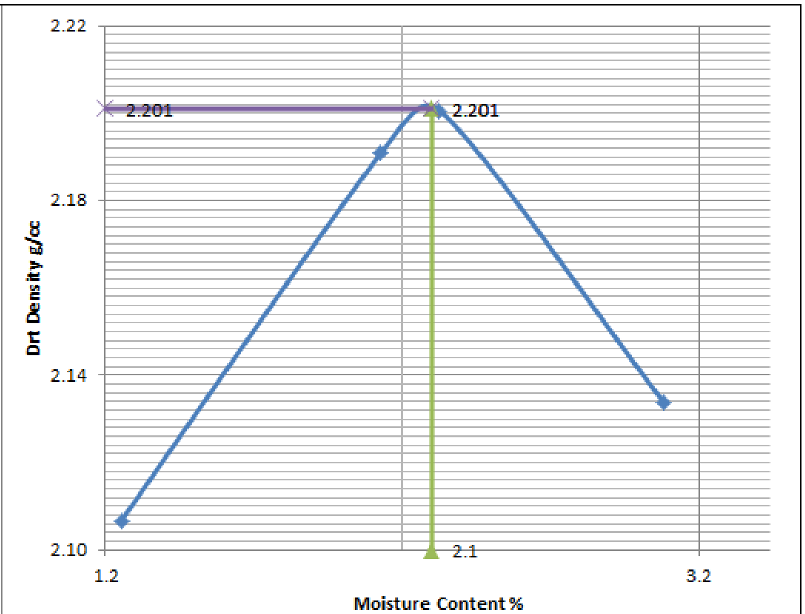
Appendix D: Moisture content determination laboratory test sample at section 3

MOISTURE DENSITY RELATIONSHIP OF SOIL
TEST METHOD : AASHTO T-180 METHOD D

REPRESENTING SECTION	section 3	DATE SAMPLED	23/07/16
MATERIAL DESCRIBTION	crushed Aggregate	DATE TESTED	26/07/16
PURPOSE	Basecourse		

		1	2	3	4	5		
DENSITY	TRIAL NUMBER	1	2	3	4	5		
	WEIGHT OF SAMPLE (g)		6000	6000	6000			
	WATER ADDED litre		0.0	50.0	100.0	150.0		
	WEIGHT OF SOIL + MOLD (g)		11,080	11,300	11,330	11220		
	WEIGHT OF MOLD (g)		6590	6590	6590	6590		
	WEIGHT OF SOIL (g)		4490	4710	4740	4630		
	VOLUME OF MOLD (cc)		2105	2105	2105	2105		
	Wet DENSITY OF SOIL (g/cc)		2.13	2.24	2.25	2.20		NMC
MOISTURE	CONTAINER NUMBER		A4	CH	A2			A1
	WET SOIL + CONTAINER (g)		305.03	280.17	332.79	328.78		265.2
	DRY SOIL + CONTAINER (g)		301.57	274.91	325.8	319.74		263.1
	WEIGHT OF WATER (g)		3.46	5.26	6.99	9.04		2.1
	WEIGHT OF CONTAINER (g)		25.3	27.8	25.3	26.3		23.7
	WEIGHT OF DRY SOIL (g)		276.3	247.1	300.5	293.4		239.4
	MOISTURE CONTENT (%)		1.3	2.1	2.33	3.08		0.9
	DRY DENSITY OF SOIL (g/cc)		2.11	2.19	2.20	2.13		

MDD (gm/cc) :	2.20
OMC (%) :	2.3



Appendix E: CBR analysis base course sample result at section1

CALIFORNIA BEARING RATIO TEST

TEST METHOD : AASHTO T-193

REPRESENTED SECTION	SECTION 1	DATE SAMPLED	24/06/2016
MATERIAL DESCRIPTION	crushed Aggergate	DATE AND TIME SOAKED	28/06/2016
PUROPSE	**Basecourse**	DATE AND TIME TESTED	2/7/2016

DENSITY DETERMINATION

SOAKING CONDITION			10 Blows		30 Blows		65 Blows	
			BEFORE	AFTER	BEFORE	AFTER	BEFORE	AFTER
MOLD NUMBER			10		30		65	
WEIGHT OF SOIL + MOLD	(g)	W_1	10650	12040	12090	13240	13030	13040
WEIGHT OF MOLD	(g)	W_2	6480	6450	7500	7470	7440	7410
VOLUME OF MOLD	(Cm^3)	V	2461	2461	2461	2461	2461	2461
WEIGHT OF WET SOIL	(g)	$W_3 = W_1 - W_2$	4170	5590	4590	5770	5590	5630
WET DENSITY OF SOIL	(g/cm3)	$W_d = (W_3/V)$	1.69	2.27	1.87	2.34	2.27	2.29
DRY DENSITY OF SOIL	(g/cm3)	$D_d = W_d/(100+m)*100$	1.65	1.88	1.82	2.15	2.23	2.24

MOISTURE DETERMINATION

SOAKING CONDITION			10 Blows			30 Blows			65 Blows		
			BEFORE	AFTER		BEFORE	AFTER		BEFORE	AFTER	
				TOP 1in.	AVG.		TOP 1in.	AVG.		TOP 1in.	AVG.
CONTAINER NUMBER			H	M		G	A2		B3	B4	
WET SOIL + CONTAINER	(g)	a	270.0	293.3		317.5	310.6		302.3	262.3	
DRY SOIL + CONTAINER	(g)	b	263.0	246.6		310.4	287.5		297.2	257.3	
WEIGHT OF CONTAINER	(g)	c	25.3	25.3		25.4	26.9		27.6	25.9	
WEIGHT OF WATER	(g)	d = a - b	7.0	46.7		7.1	23.1		5.1	5.0	
WEIGHT OF DRY SOIL	(g)	e = b - c	237.75	221.34		284.97	260.6		269.64	231.37	
MOISTURE CONTENT	(%)	m =(d/e)*100	2.9	21.1		2.5	8.9		1.9	2.2	
AVG. MOIST. CONTENT	**(:**										

PENETRATION TEST DATA

PENETRATION	10 Blows				30 Blows				65 Blows			
(mm)	DIAL RDG	LOAD (kn)	COR.LOAD (kn)	CBR %	DIAL RDG	LOAD (kn)	COR.LOAD (kn))	CBR %	DIAL RDG	LOAD (kn)	CORLOAD (kn))	CBR %
0	0	0.000			0	0			0	0		
0.64	120	1.462			120	1.46			265	3.23		
1.27	160	1.949			240	2.92			640	7.80		
1.96	232	2.826			370	4.51			1080	13.15		
2.54	280	3.414	3.41	25.57	490	5.97	5.97	44.7	1500	18.27	18.27	136.9
3.18	352	4.287			670	8.16			1950	23.75		
3.81	422	5.140			780	9.50			2420	29.48		
4.45	484	5.900			940	11.45			2750	33.50		
5.08	570	6.943	6.94	34.83	1100	13.40	13.40	67.2	3000	36.54	36.54	182.7
7.62	598	7.284			1250	15.23						
10.16												
12.7												

SWELL			
No. OF BLOWS	10	30	65
RDG (BEFORE SOAKING)	0.00	0.00	0.00
RDG (AFTER) SOAKING)	0.08	0.08	0.05
PERCENT SWELL	0.07	0.07	0.04
AVERAGE PERCENT SWELL : %	0.06		

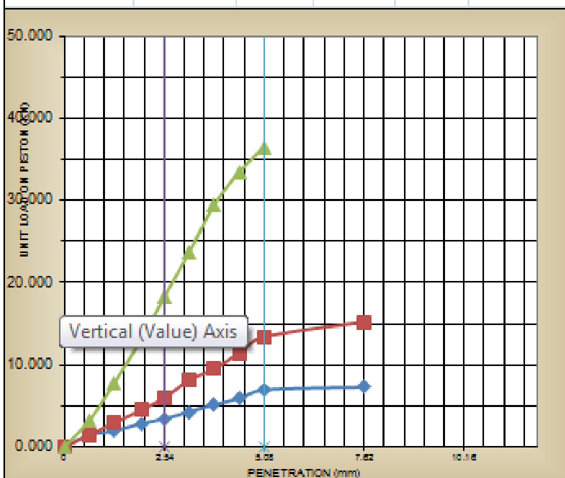
RING FACTOR	
12.18	N/Divis.

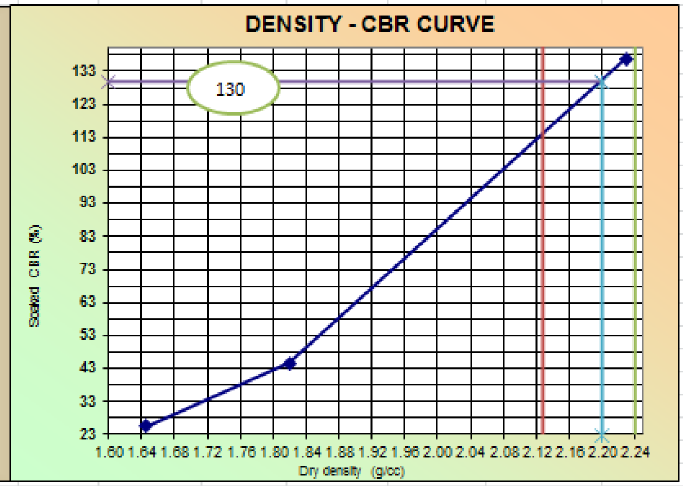
MDD (gm/cc)	2.24
OMC %	2.2
95 % of MDD	2.13

BLOWS	LOAD (KN)		CBR(%)		SWELL
	2.54mm	5.08mm	2.54mm	5.08mm	%
10	3.41	6.94	25.6	34.8	0.07
30	5.97	13.40	44.7	67.2	0.07
65	18.27	36.54	136.9	182.7	0.04

DRY DENSITY Vs SOCKED C.B.R.

No # OF BLOWS	10	30	65
DRY DENSITY	1.65	1.82	2.23
SOCKED C.B.R.	25.6	44.7	136.9





MODIFIED PROCTOR : T 180	
MDD (g/cc) :	2.24
OMC (%) :	2.24
95% of MDD(g/cc)	2.13
CBR@ 95% of MDD	**130.0**

Appendix E: CBR analysis base course sample result at section 2

CALIFORNIA BEARING RATIO TEST

TEST METHOD : AASHTO T-193

REPRESENTED SECTION	SECTION 2	DATE SAMPLED	2/6/07/16
MATERIAL DESCRIPTION	Creshed Aggregate	DATE AND TIME SOAKED	6/7/16
PUROPSE	**Basecourse**	DATE AND TIME TESTED	10/7/2016

DENSITY DETERMINATION

SOAKING CONDITION		10 Blows		30 Blows		65 Blows	
		BEFORE	AFTER	BEFORE	AFTER	BEFORE	AFTER
MOLD NUMBER		M-8		Z-5		B-7	
WEIGHT OF SOIL + MOLD (g)	W_1	11680	12070	12930	13270	13060	13470
WEIGHT OF MOLD (g)	W_2	6510	6510	7530	7530	7470	7470
VOLUME OF MOLD (Cm^3)	V	2461	2461	2461	2461	2461	2461
WEIGHT OF WET SOIL (g)	$W_3 = W_1 - W_2$	5170	5560	5400	5740	5590	6000
WET DENSITY OF SOIL (g/cm3)	$W_d = (W_3/V)$	2.10	2.26	2.19	2.33	2.27	2.44
DRY DENSITY OF SOIL (g/cm3)	$D_d = W_d/(100+m)*100$	2.04	1.83	2.14	2.13	2.22	2.38

MOISTURE DETERMINATION

SOAKING CONDITION		10 Blows			30 Blows			65 Blows		
		BEFORE	AFTER		BEFORE	AFTER		BEFORE	AFTER	
			TOP 1 in.	AVG.		TOP 1 in.	AVG.		TOP 1 in.	AVG.
CONTAINER NUMBER		E	Y		CH	H		A3	A4	
WET SOIL + CONTAINER (g)	a	250.0	273.7		297.5	290.6		283.0	242.8	
DRY SOIL + CONTAINER (g)	b	243.3	226.6		290.4	267.5		277.2	237.3	
WEIGHT OF CONTAINER (g)	c	25.3	25.3		25.7	26.9		27.6	25.9	
WEIGHT OF WATER (g)	d = a - b	6.7	47.1		7.0	23.1		5.8	5.5	
WEIGHT OF DRY SOIL (g)	e = b - c	218.03	201.34		264.71	240.55		249.63	211.32	
MOISTURE CONTENT (%)	m =(d/e)*100	3.1	23.4		2.7	9.6		2.3	2.6	

PENETRATION TEST DATA

PENETRATION	10 Blows				30 Blows				65 Blows			
(mm)	DIAL RDG	LOAD (kn)	COR. LOAD (kn)	CBR %	DIAL RDG	LOAD (kn)	COR. LOAD (kn))	CBR %	DIAL RDG	LOAD (kn)	COR LOAD (kn))	CBR %
0	0	0.000			0	0			0	0		
0.64	80	0.974			110	1.34			243	2.96		
1.27	150	1.827			206	2.51			560	6.82		
1.96	200	2.436			367	4.47			987	12.02		
2.54	253	3.082	3.08	23.08	436	5.31	5.31	39.8	1120	13.64	13.64	102.2
3.18	321	3.910			539	6.57			1482	18.05		
3.81	356	4.336			651	7.93			2014	24.53		
4.45	446	5.432			714	8.70			2147	26.15		
5.08	532	6.480	6.48	32.51	863	10.51	10.51	52.7	2639	32.14	32.14	160.7
7.62	568	6.918			1028	12.52			3569	43.47		
10.16												
12.7												

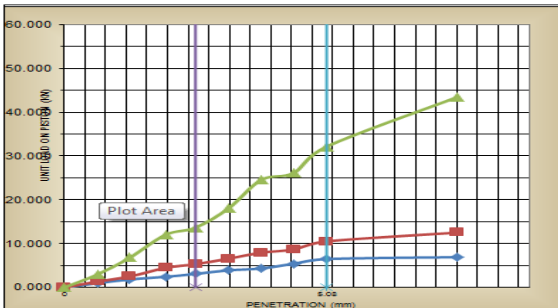
CALIFORNIA BEARING RATIO TEST
TEST MATHOD : AASHTO T-193

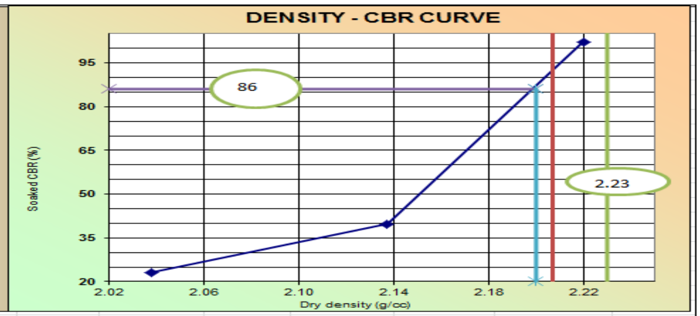
REPRESENTED SECTION	SECTION 2	DATE SAMPLED	02/07/16
MATERIAL DESCRIPTION	Creshed Aggregate	DATE AND TIME SOAKED	06/07/16
PUROPSE	Basecourse	DATE AND TIME TESTED	10/7/2016

BLOWS	LOAD (KN)		CBR(%)	
	2.54mm	5.08mm	2.54mm	5.08mm
10	3.08	6.48	23.1	32.5
30	5.31	10.51	39.8	52.7
65	13.64	32.14	102.2	160.7

DRY DENSITY Vs SOCKED C.B.R.

No # OF BLOWS	10	30	65
DRY DENSITY	2.04	2.14	2.22
SOCKED C.B.R.	23.1	39.8	102.2





MODIFIED PROCTOR : T 180	
MDD (g/cc) :	2.23
OMC (%) :	3.40
95% of MDD(g/cc)	2.21
CBR@ 95% of MDD	**86**

2.071

Appendix E: CBR analysis base course sample result at section1 3

CALIFORNIA BEARING RATIO TEST

TEST METHOD : AASHTO T-193

REPRESENTED SECTION	SECTION 3	DATE SAMPLED	12/07/16
MATERIAL DESCRIPTION	Creshed Aggregate	DATE AND TIME SOAKED	16/07/2016
PUROPSE	**Basecourse**	DATE AND TIME TESTED	20/07/16

DENSITY DETERMINATION

SOAKING CONDITION			10 Blows		30 Blows		65 Blows	
			BEFORE	AFTER	BEFORE	AFTER	BEFORE	AFTER
MOLD NUMBER			M-8		Z-5		B-7	
WEIGHT OF SOIL + MOLD	(g)	W_1	11480	11670	12530	12870	12660	13070
WEIGHT OF MOLD	(g)	W_2	6510	6510	7530	7530	7470	7470
VOLUME OF MOLD	(Cm³)	V	2461	2461	2461	2461	2461	2461
WEIGHT OF WET SOIL	(g)	$W_3 = W_1 - W_2$	4970	5160	5000	5340	5190	5600
WET DENSITY OF SOIL	(g/cm3)	$W_d = (W_3/V)$	2.02	2.10	2.03	2.17	2.11	2.28
DRY DENSITY OF SOIL	(g/cm3)	$D_d = W_d/(100+m)*100$	1.96	1.98	1.98	2.04	2.06	2.22

MOISTURE DETERMINATION

SOAKING CONDITION			10 Blows			30 Blows			65 Blows		
			BEFORE	AFTER		BEFORE	AFTER		BEFORE	AFTER	
				TOP 1 in.	AVG.		TOP 1 in.	AVG.		TOP 1 in.	AVG.
CONTAINER NUMBER			E	Y		CH	H		A3	A4	
WET SOIL + CONTAINER	(g)	a	250.0	273.7		297.5	290.6		283.0	242.8	
DRY SOIL + CONTAINER	(g)	b	243.3	260.0		290.4	275.0		277.2	237.3	
WEIGHT OF CONTAINER	(g)	c	25.3	25.3		25.7	26.9		27.6	25.9	
WEIGHT OF WATER	(g)	d = a - b	6.7	13.7		7.0	15.6		5.8	5.5	
WEIGHT OF DRY SOIL	(g)	e = b - c	218.03	234.74		264.71	248.1		249.63	211.32	
MOISTURE CONTENT	(%)	m =(d/e)*100	3.1	5.8		2.7	6.3		2.3	2.6	

PENETRATION TEST DATA

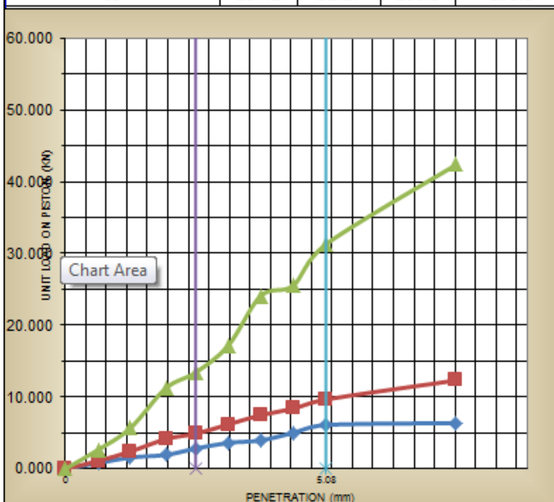
PENETRATION	10 Blows				30 Blows				65 Blows			
(mm)	DIAL RDG	LOAD (kn)	COR. LOAD (kn)	CBR %	DIAL RDG	LOAD (kn)	COR. LOAD (kn))	CBR %	DIAL RDG	LOAD (kn)	COR LOAD (kn))	CBR %
0	0	0.000			0	0			0	0		
0.64	65	0.792			90	1.10			218	2.66		
1.27	130	1.583			198	2.41			467	5.69		
1.96	160	1.949			341	4.15			923	11.24		
2.54	232	2.826	2.83	21.17	402	4.90	4.90	36.7	1102	13.42	13.42	100.5
3.18	298	3.630			508	6.19			1412	17.20		
3.81	325	3.959			614	7.48			1978	24.09		
4.45	413	5.030			689	8.39			2098	25.55		
5.08	501	6.102	6.10	30.62	793	9.66	9.66	48.5	2567	31.27	31.27	156.3
7.62	523	6.370			1011	12.31			3478	42.36		
10.16												
12.7												

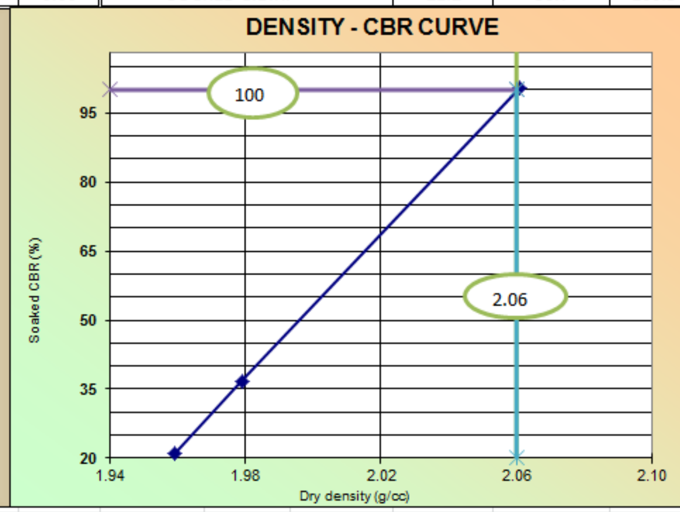
RING FACTOR		
12.18 N/Divis.	MDD (gm/cc)	2.15
	OMC %	5.0
	95 % of MDD	2.04

BLOWS	LOAD (KN)		CBR(%)		SWELL
	2.54mm	5.08mm	2.54mm	5.08mm	%
10	2.83	6.10	21.2	30.6	0.00
30	4.90	9.66	36.7	48.5	0.00
65	13.42	31.27	100.5	156.3	0.00

DRY DENSITY Vs SOCKED C.B.R.

No # OF BLOWS	10	30	65
DRY DENSITY	1.96	1.98	2.06
SOCKED C.B.R.	21.2	36.7	100.5

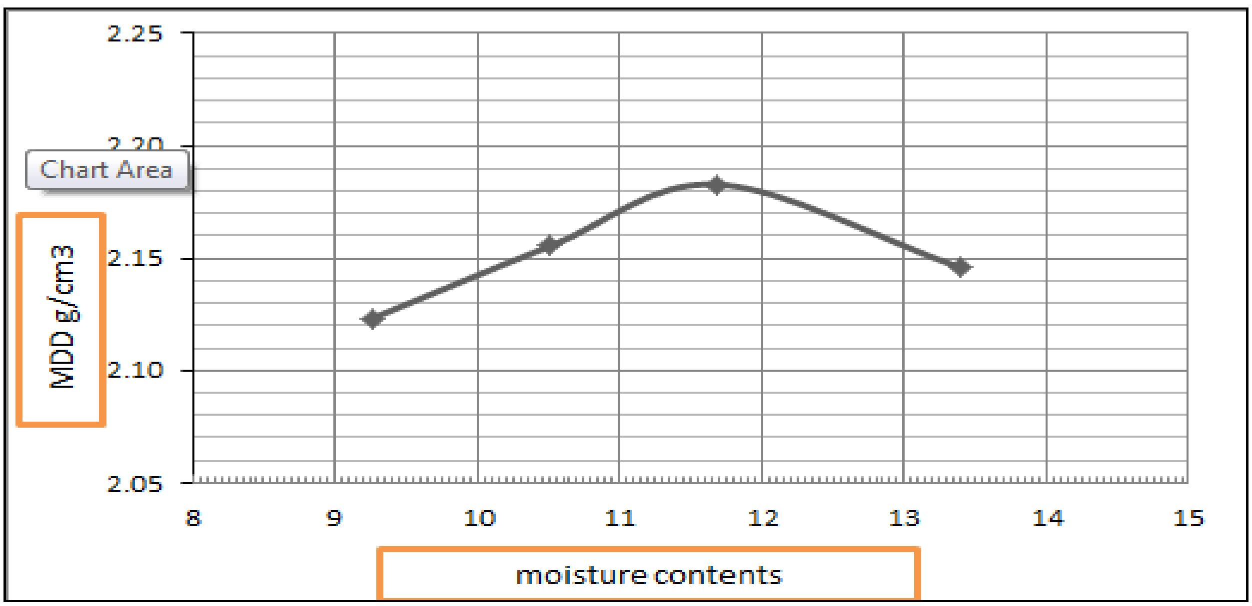




MDD (g/cc) :	2.06
OMC (%) :	5.00
95% of MDD(g/cc)	2.21
CBR@ 95% of MDD	**100**

Appendix F: Compaction curve of Natural subbsae at section 1

REPRESENT SECTION : SECTION 1				Date Sampeld 14/06/16			
description: Ntural subbase material				Date Tested:- 18/06/2016			
PURPOSE:Proposed for Sub-base material							
Trial No.	1	2	3	4			
Weight of Mould + Wet soil (g)	12229	12361	12480	12470			
Weight of Mould (g)	7304	7304	7304	7304			
Weight of Wet soil (g)	4925	5057	5176	5166			
Volume of Mould (cc)	2123	2123	2123	2123			
Wet density (g / cm3)	2.32	2.38	2.44	2.43			
Moisture Content Determination							**NMC**
Moisture can	**A-6**	**A-5**	**A-2**	**A-8**			**A-3**
Weight of Wet soil + cont. (g)	370.5	406.0	619.0	530.0			415.0
Weight of Dry soil + cont. (g)	343.0	371.0	558.0	472.0			400.0
Weight of Container (g)	46.0	38.0	36.0	39.0			37.0
Weight of water (moisture) (g)	27.5	35.0	61.0	58.0			23.0
Weight of Dry soil (g)	297.0	333.0	522.0	433.0			272.0
Moisture content (%)	9.26	10.51	11.69	13.39			8.50
Dry Density (g / cm3)	2.12	2.16	2.18	2.15			
	MDD 2.18 g/cc		**OMC 11.69 %**				



Appendix F: Compaction curve of Natural subbsae at section 2

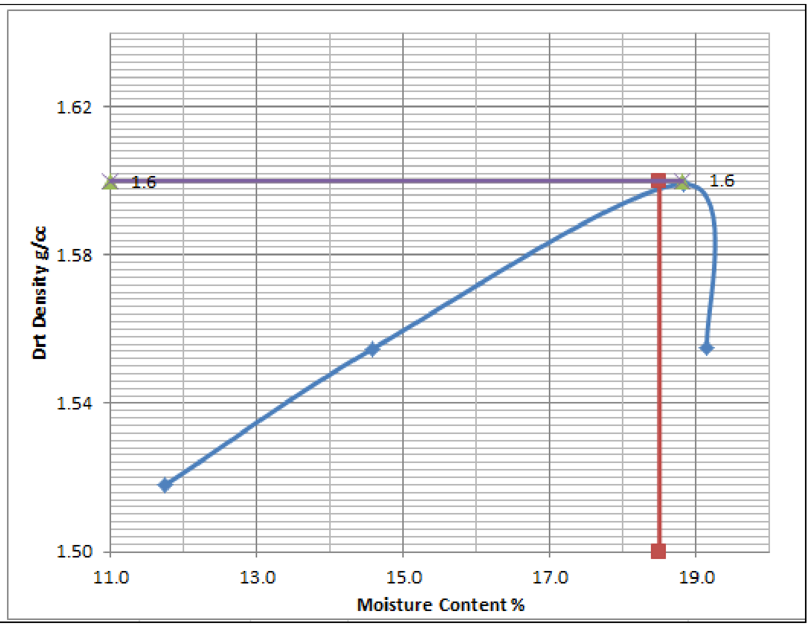
MOISTURE DENSITY RELATIONSHIP OF SOIL
TEST METHOD : AASHTO T-180 METHOD D

REPRESENTING SECTION	section 2	DATE SAMPLED	13/06/2016
MATERIAL DESCRIBTION	Granular material	DATE TESTED	16/06/2016
PURPOSE	Natural Subbase		

DENSITY	TRIAL NUMBER		1	2	3	4	5		
	WEIGHT OF SAMPLE	(g)	5000	5000	5000	5000			
	WATER ADDED	litre	0.0	200.0	400.0	600.0			
	WEIGHT OF SOIL + MOLD (g)		10,160	10,340	10,590	10,490			
	WEIGHT OF MOLD (g)		6590	6590	6590	6590			
	WEIGHT OF SOIL (g)		3570	3750	4000	3900			
	VOLUME OF MOLD (cc)		2105	2105	2105	2105			
	Wet DENSITY OF SOIL (g/cc)		1.70	1.78	1.90	1.85			NMC

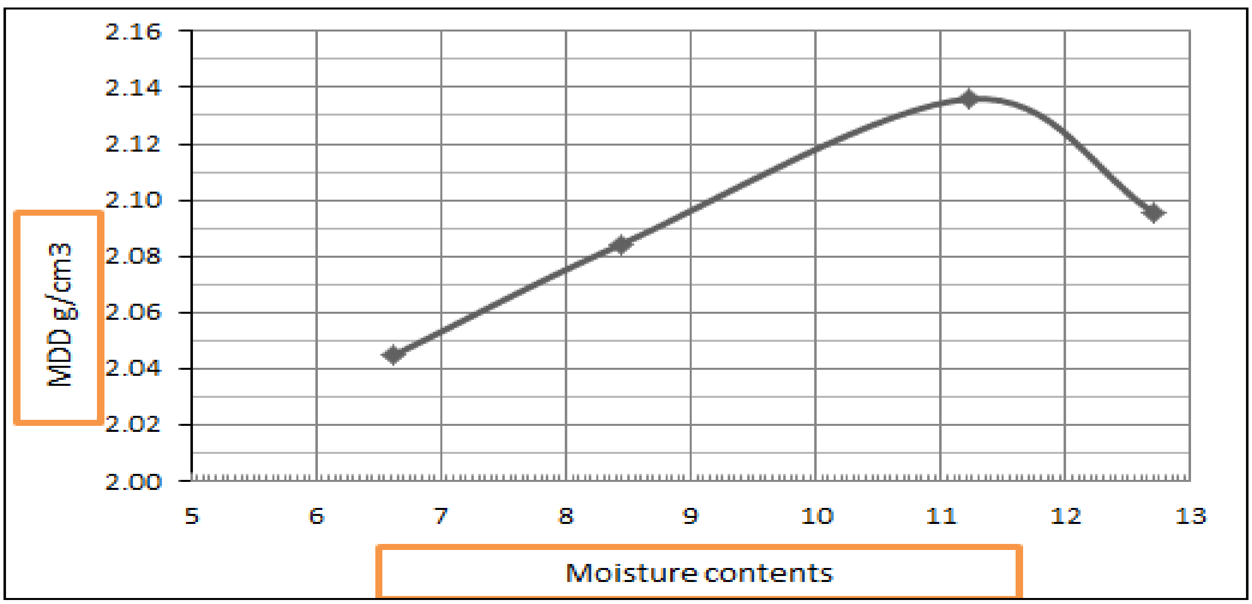
MOISTURE	CONTAINER NUMBER	w	b	c	f			d
	WET SOIL + CONTAINER (g)	205.9	203	154	158.7			187.8
	DRY SOIL + CONTAINER (g)	187.1	180.5	133.7	137.6			176.2
	WEIGHT OF WATER (g)	18.80	22.50	20.30	21.10			11.6
	WEIGHT OF CONTAINER (g)	26.9	26.2	25.9	27.4			26.1
	WEIGHT OF DRY SOIL (g)	160.2	154.3	107.8	110.2			150.1
	MOISTURE CONTENT (%)	11.7	14.6	18.8	19.15			7.7
	DRY DENSITY OF SOIL (g/cc)	1.52	1.55	1.60	1.55			

MDD (gm/cc) :	1.60
OMC (%) :	18.8



Appendix F: Compaction curve of Natural sub base at section 3

REPRESENT SECTION : SECTION 3					Date Sampeld 24/06/16		
description: Ntural sub baase material					Date Tested:- 29/06/2016		
PURPOSE MOISTURE CONTENT DETERMINATION							
Trial No.	1	2	3	4			
Weight of Mould + Wet soil (g)	11932	12102	12348	12318			
Weight of Mould (g)	7304	7304	7304	7304			
Weight of Wet soil (g)	4628	4798	5044	5014			
Volume of Mould (cc)	2123	2123	2123	2123			
Wet density (g / cm3)	2.18	2.26	2.38	2.36			
Moisture Content Determination							**NMC**
Moisture can	**A-7**	**A-3**	**A-6**	**A-8**			**A-1**
Weight of Wet soil + cont. (g)	328.0	383.0	352.0	322.0			419.0
Weight of Dry soil + cont. (g)	310.0	356.0	321.0	290.0			405.0
Weight of Container (g)	38.0	36.0	45.0	38.0			40.0
Weight of water (moisture) (g)	18.0	27.0	31.0	32.0			23.0
Weight of Dry soil (g)	272.0	320.0	276.0	252.0			272.0
Moisture content (%)	6.62	8.44	11.23	12.70			8.50
Dry Density (g / cm3)	2.04	2.08	2.14	2.10			
	MDD 2.14 g/cc		**OMC 11.23 %**				



Appendix G: CBR Analysis of natural subgrade sample at section 1

CALIFORNIA BEARING RATIO WORK SHEET

TEST METHOD (AASHTO T-193)

REPRESENTED SECTION :SECTION 1				DATE SAMPLED:	**14/06/2016**
PURPOSE	**Proposed for Sub-base**			DATE SOAKED:	**18/06/16**
MDD (g/cc)	**2.18**	OMC (%)	**11.69**	DATE TESTED:	**22/06/16**

DENSITY DETERMINATION

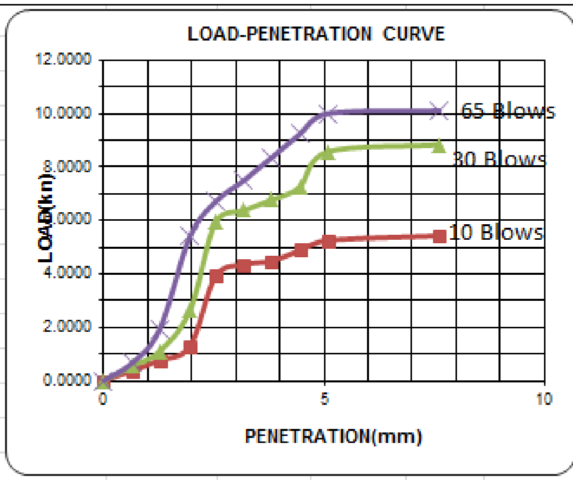
SOAKING CONDITION	10 Blows		30 Blows		65 Blows	
	BEFORE	AFTER	BEFORE	AFTER	BEFORE	AFTER
MOLD NUMBER	A17		M8		12	
WEIGHT OF SOIL + MOLD (gm)	11603	11732	11942	11991	12032	12106
WEIGHT OF MOLD (gm)	6840	6840	6973	6973	6807	6807
WEIGHT OF SOIL (gm)	4763	4892	4969	5018	5225	5299
VOLUME OF MOLD (cc)	2123	2123	2123	2123	2123	2123
WET DENSITY OF SOIL (g/cc)	2.24	2.30	2.34	2.36	2.46	2.50
DRY DENSITY OF SOIL (g/cc)	1.99	2.06	2.08	2.12	2.19	2.26

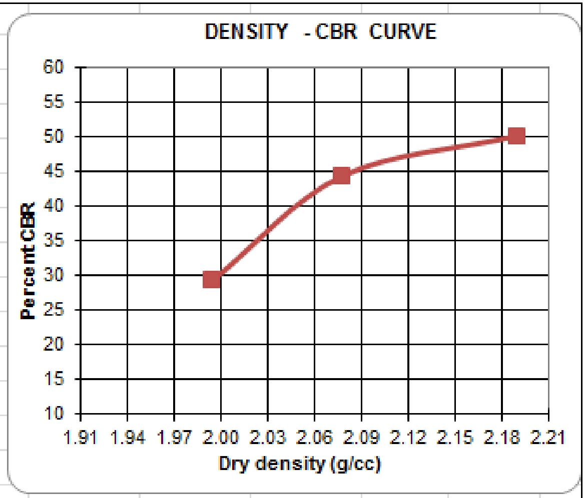
MOISTURE DETERMINATION

SOAKING CONDITION	10 Blows			30 Blows			65 Blows		
	BEFORE	AFTER		BEFORE	AFTER		BEFORE	AFTER	
		TOP 1 in.	AVG.		TOP 1 in.	AVG.		TOP 1 in.	AVG.
CONTAINER NUMBER	A15	A7		A5	A5		A8	A3	
WET SOIL + CONTAINER (gm)	439	486		431	460		429	396	
DRY SOIL + CONTAINER (gm)	396	439		387	416		386	362	
WEIGHT OF WATER (gm)	43.0	47.0		44.0	44.0		43.0	34.0	
WEIGHT OF CONTAINER (gm)	52	39		39	38		39	37	
WEIGHT OF DRY SOIL (gm)	344	400		348	378		347	325	
MOISTURE CONTENT (%)	12.50	11.75		12.64	11.64		12.39	10.46	

PENETRATION TEST DATA												
PENETRATION	10 Blows				30 Blows				65 Blows			
(mm)	DIAL RDG	LOAD (kn)	COR. LOAD (kn)	CBR %	DIAL RDG	LOAD (kn)	COR. LOAD (k	CBR %	DIAL RDG	LOAD (kn)	COR LOAD (k	CBR %
0	0	0.0000			0	0			0	0		
0.64	17	0.3735			25	0.5493			30	0.6591		
1.27	35	0.7690			50	1.0985			87	1.9114		
1.96	59	1.2962			120	2.6364			246	5.4046		
2.54	178	3.9107	4	29	270	5.9319	6	44	305	6.7009	7	50
3.18	198	4.3501			290	6.3713			341	7.4918		
3.81	204	4.4819			308	6.7668			380	8.3486		
4.45	222	4.8773			330	7.2501			420	9.2274		
5.08	239	5.2508	5	26	390	8.5683	9	43	455	9.9964	10	50
7.62	247	5.4266			402	8.8319			460	10.1062		
10.16												
12.7												

SWELL			
No. OF BLOWS	10	30	65
RDG (BEFORE SOAKING)	15.45	10.95	13.92
RDG (AFTER) SOAKING)	15.90	11.30	14.12
PERCENT SWELL	0.39	0.30	0.17
AVERAGE PERCENT SWELL :		0.29	





Blow	LOAD (KN)		CBR(%)		Swell %
	2.54mm	5.08mm	2.54mm	5.08mm	
10	4	5	29	26	0.39
30	6	9	44	43	0.30
65	7	10	50	50	0.17

Blow	Dry density	CBR%
10	1.99	29
30	2.08	44
65	2.19	50
CBR at 95%		37

Appendix G: CBR Analysis of natural subgrade sample at section 2

CALIFORNIA BEARING RATIO TEST TEST METHOD : AASHTO T-193			
REPRESENTED SECTION	SECTION 2	DATE SAMPLED	13/06/2016
MATERIAL DESCRIPTION	Granular material	DATE AND TIME SOAKED	22/06/2016
PUROPSE	**Natural Subbase**	DATE AND TIME TESTED	26/06/2016

DENSITY DETERMINATION

SOAKING CONDITION		10 Blows		30 Blows		65 Blows	
		BEFORE	AFTER	BEFORE	AFTER	BEFORE	AFTER
MOLD NUMBER		10		30		A	
WEIGHT OF SOIL + MOLD (g)	W_1	10420	11750	11260	11540	12190	12420
WEIGHT OF MOLD (g)	W_2	7510	7510	6810	6810	7450	7450
VOLUME OF MOLD (Cm³)	V	2266	2266	2266	2266	2266	2266
WEIGHT OF WET SOIL (g)	$W_3 = W_1 - W_2$	2910	4240	4450	4730	4740	4970
WET DENSITY OF SOIL (g/cm3)	$W_d = (W_3/V)$	1.28	1.87	1.96	2.09	2.09	2.19
DRY DENSITY OF SOIL (g/cm3)	$D_d = W_d/(100+m)*100$	1.08	1.37	1.63	1.63	1.71	1.74

SOAKING CONDITION		10 Blows			30 Blows			65 Blows		
		BEFORE	AFTER		BEFORE	AFTER		BEFORE	AFTER	
			TOP 1 in.	AVG.		TOP 1 in.	AVG.		TOP 1 in.	AVG.
CONTAINER NUMBER		X	D		A	B3		B	A	
WET SOIL + CONTAINER (g)	a	185.4	190.8		180.6	170.0		188.6	190.9	
DRY SOIL + CONTAINER (g)	b	159.7	146.3		154.0	138.4		159.2	157.2	
WEIGHT OF CONTAINER (g)	c	25.8	25.8		25.7	25.2		26.1	27.6	
WEIGHT OF WATER (g)	d = a - b	25.7	44.5		26.6	31.6		29.4	33.7	
WEIGHT OF DRY SOIL (g)	e = b - c	133.9	120.5		128.3	113.2		133.1	129.6	
MOISTURE CONTENT (%)	m =(d/e)*100	19.2	36.9		20.7	27.9		22.1	26.0	

PENETRATION TEST DATA

PENETRATION	10 Blows				30 Blows				65 Blows			
(mm)	DIAL RDG	LOAD (kn)	COR. LOAD (kn)	CBR %	DIAL RDG	LOAD (kn)	COR. LOAD (kn))	CBR %	DIAL RDG	LOAD (kn)	COR LOAD (kn))	CBR %
0	0	0.000			0	0			0	0		
0.64	200	2.436			221	2.69			254	3.09		
1.27	212	2.582			234	2.85			270	3.29		
1.96	224	2.728			249	3.03			294	3.58		
2.54	236	2.874	2.87	21.53	268	3.26	3.26	24.5	321	3.91	3.91	29.3
3.18	250	3.045			278	3.39			344	4.19		
3.81	264	3.216			294	3.58			362	4.41		
4.45	270	3.289			302	3.68			380	4.63		
5.08	292	3.557	3.56	17.85	316	3.85	3.85	19.3	398	4.85	4.85	24.2
7.62	314	3.825			327	3.98			410	4.99		
10.16	326.0	3.971			340.0	4.141			422.0	5.14		
12.7												

SWELL			
No. OF BLOWS	10	30	65
RDG (BEFORE SOAKING)	0.00	0.00	0.00
RDG (AFTER) SOAKING)	0.76	1.18	0.86
PERCENT SWELL	0.65	1.01	0.74
AVERAGE PERCENT SWELL : %	0.80		

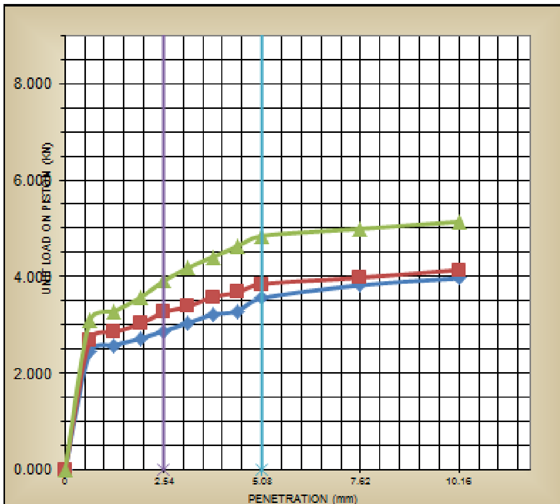
RING FACTOR
12.18 N/Divis.

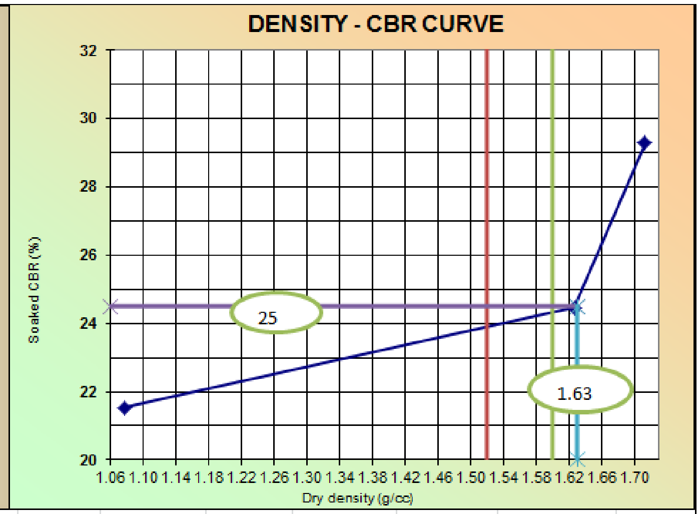
MDD (gm/cc)	1.60
OMC %	18.8
95 % of MDD	1.52

CALIFORNIA BEARING RATIO TEST TEST MATHOD : AASHTO T-193			
REPRESENTED SECTION	SECTION 2	DATE SAMPLED	13/07/16
MATERIAL DESCRIPTION	Granular material	DATE AND TIME SOAKED	22/07/16
PUROPSE	Natural Subbase	DATE AND TIME TESTED	26/07/16

BLOWS	LOAD (KN)		CBR(%)		SWELL %
	2.54mm	5.08mm	2.54mm	5.08mm	
10	2.87	3.56	21.5	17.8	0.65
30	3.26	3.85	24.5	19.3	1.01
65	3.91	4.85	29.3	24.2	0.74

DRY DENSITY Vs SOCKED C.B.R.			
No # OF BLOWS	10	30	65
DRY DENSITY	1.08	1.63	1.71
SOCKED C.B.R.	21.5	24.5	29.3





MODIFIED PROCTOR : T 180	
MDD (g/cc) :	1.60
OMC (%) :	18.80
95% of MDD(g/cc)	1.52
CBR@ 95% of MDD	**25**

Appendix G: CBR Analysis of natural subgrade sample at section 3

CALIFORNIA BEARING RATIO WORK SHEET

TEST METHOD (AASHTO T-193)

REPRESENTED SECTION	SECTION 3			DATE SAMPLED:	13/6/2013
PURPOSE	Proposed for Sub-base			DATE SOAKED:	18/06/2013
MDD (g/cc)	2.14	OMC (%)	11.23	DATE TESTED:	22/06/2013

DENSITY DETERMINATION

SOAKING CONDITION	10 Blows		30 Blows		65 Blows	
	BEFORE	AFTER	BEFORE	AFTER	BEFORE	AFTER
MOLD NUMBER	HC-5		A-17		HC-11	
WEIGHT OF SOIL + MOLD (gm)	11088	11232	11700	11891	12126	12306
WEIGHT OF MOLD (gm)	6782	6782	6858	6858	6840	6840
WEIGHT OF SOIL (gm)	4306	4450	4842	5033	5286	5466
VOLUME OF MOLD (cc)	2123	2123	2123	2123	2123	2123
WET DENSITY OF SOIL (g/cc)	2.03	2.10	2.28	2.37	2.49	2.57
DRY DENSITY OF SOIL (g/cc)	1.87	1.85	2.09	2.12	2.26	2.31

MOISTURE DETERMINATION

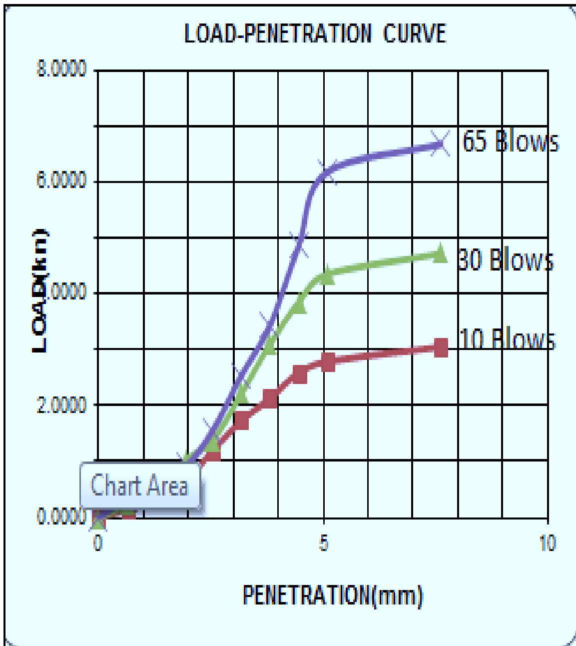
SOAKING CONDITION	10 Blows			30 Blows			65 Blows		
	BEFORE	AFTER		BEFORE	AFTER		BEFORE	AFTER	
		TOP 1 in.	AVG.		TOP 1 in.	AVG.		TOP 1 in.	AVG.
CONTAINER NUMBER	A-6	D-7		A-8	C-2		A-1	C-6	
WET SOIL + CONTAINER (gm)	453	450		375	483		405	344	
DRY SOIL + CONTAINER (gm)	422	405		347	436		371	316	
WEIGHT OF WATER (gm)	31.0	45.0		28.0	47.0		34.0	28.0	
WEIGHT OF CONTAINER (gm)	46	67		39	46		40	71	
WEIGHT OF DRY SOIL (gm)	376	338		308	390		331	245	
MOISTURE CONTENT (%)	8.24	13.31		9.09	12.05		10.27	11.43	

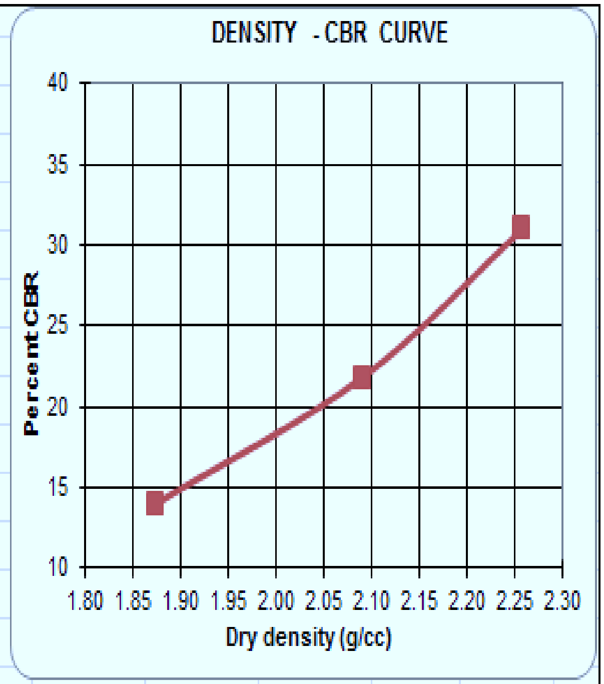
PENETRATION TEST DATA

PENETRATION	10 Blows				30 Blows				65 Blows			
(mm)	DIAL RDG	LOAD (kn)	COR. LOAD (kn))	CBR %	DIAL RDG	LOAD (kn)	COR. LOAD (k	CBR %	DIAL RDG	LOAD (kn)	COR LOAD (k	CBR %
0	0	0.0000			0	0			0	0		
0.64	7	0.1538			10	0.2197			15	0.3296		
1.27	15	0.3296			17	0.3735			20	0.4394		
1.96	31	0.6811			47	1.0326			42	0.9227		
2.54	55	1.2084	1	9	63	1.3841	1	10	71	1.5599	2	12
3.18	79	1.7356			101	2.2190			116	2.5485		
3.81	97	2.1311			141	3.0978			156	3.4273		
4.45	118	2.5925			175	3.8448			221	4.8554		
5.08	127	2.7902	3	14	198	4.3501	4	22	281	6.1736	6	31
7.62	139	3.0538			215	4.7236			304	6.6789		
10.16												
12.7												

SWELL

No. OF BLOWS	10	30	65
RDG (BEFORE SOAKING)	15.83	15.53	5.70
RDG (AFTER) SOAKING)	15.96	15.66	5.74
PERCENT SWELL	0.11	0.11	0.03
AVERAGE PERCENT SWELL :		0.09	





Blow	LOAD (KN)		CBR(%)		Swell %
	2.54mm	5.08mm	2.54mm	5.08mm	
10	1	3	9	14	0.11
30	1	4	10	22	0.11
65	2	6	12	31	0.03

Blow	Dry density	CBR%
10	1.87	14
30	2.09	22
65	2.26	31
CBR at 95%		19.5

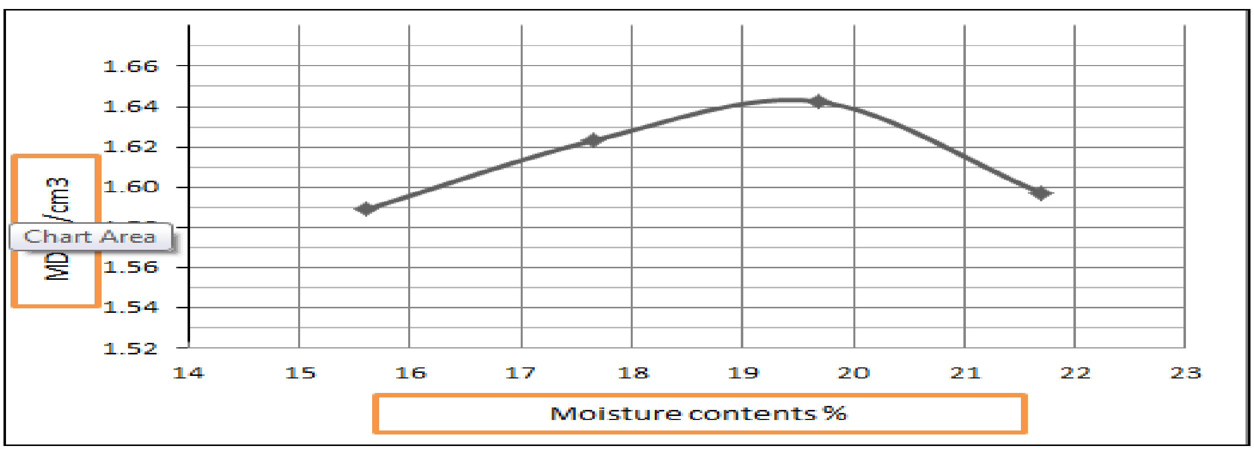
Appendix H: Compaction curve of the subgrade sample at section 2

REPRESENT SECTION: SECTION 2 — Date Sampeld 11//07/2016

DESCRIPTION: red Clay Soil — Date Tested:- 16/07/2016

PURPOSE :material Classificati subgrade

Trial No.	1	2	3	4			
Weight of Mould + Wet soil (g)	11204	11359	11477	11430			
Weight of Mould (g)	7304	7304	7304	7304			
Weight of Wet soil (g)	3900	4055	4173	4126			
Volume of Mould (cc)	2123	2123	2123	2123			
Wet density (g / cm3)	1.84	1.91	1.97	1.94			
Moisture Content Determination							**NMC**
Moisture can	**C6**	**A5**	**A7**	**A4**			**A9**
Weight of Wet soil + cont. (g)	383.0	392.0	337.0	369.0			596.0
Weight of Dry soil + cont. (g)	341.0	339.0	288.0	310.0			543.0
Weight of Container (g)	72.0	39.0	39.0	38.0			105.0
Weight of water (moisture) (g)	42.0	53.0	49.0	59.0			53.0
Weight of Dry soil (g)	269.0	300.0	249.0	272.0			438.0
Moisture content (%)	15.61	17.67	19.68	21.69			12.10
Dry Density (g / cm3)	1.59	1.62	1.64	1.60			
	MDD=1.64g/cc			**OMC=19.5%**			



Appendix H: Compaction curve of the subgrade sample at section 3

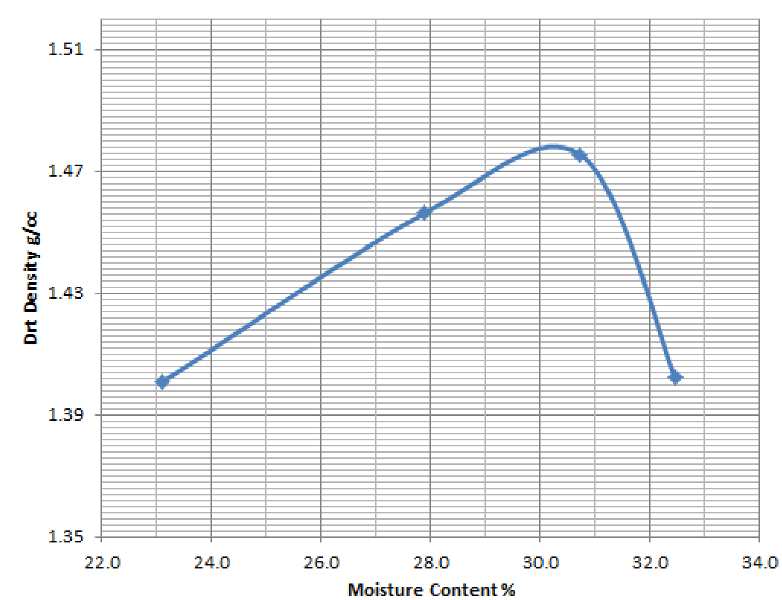
MOISTURE DENSITY RELATIONSHIP OF SOIL
TEST METHOD : AASHTO T-180 METHOD D

REPRESENTING SECTION	SECTION 1	DATE SAMPLED	12/06/16
MATERIAL DESCRIBTION	Red clay soil	DATE TESTED	17/06/2016
PURPOSE	subgrade		

DENSITY	TRIAL NUMBER	1	2	3	4	5		
	WEIGHT OF SAMPLE (g)	6000	6000	6000	6000			
	WATER ADDED litre	500.0	700.0	900.0	1100.0			
	WEIGHT OF SOIL + MOLD (g)	10,220	10,510	10,650	10,500			
	WEIGHT OF MOLD (g)	6590	6590	6590	6590			
	WEIGHT OF SOIL (g)	3630	3920	4060	3910			
	VOLUME OF MOLD (cc)	2105	2105	2105	2105			
	Wet DENSITY OF SOIL (g/cc)	1.72	1.86	1.93	1.86		NMC	

MOISTURE	CONTAINER NUMBER	A4	G	F	X			44
	WET SOIL + CONTAINER (g)	248.3	233.4	207.9	198.9			205.6
	DRY SOIL + CONTAINER (g)	206.5	188.5	165.5	157			185.0
	WEIGHT OF WATER (g)	41.80	44.90	42.40	41.90			20.6
	WEIGHT OF CONTAINER (g)	25.6	27.5	27.5	27.9			27.4
	WEIGHT OF DRY SOIL (g)	180.9	161.0	138.0	129.1			157.6
	MOISTURE CONTENT (%)	23.1	27.9	30.7	32.46			13.1
	DRY DENSITY OF SOIL (g/cc)	1.40	1.46	1.48	1.40		13.1	

MDD (gm/cc) :	1.48
OMC (%) :	30.7



Appendix I :CBR analysis result sample of the subgrade section 1

TEST METHOD (AASHTO T-193)

REPRESENTED SECTION: SECTION 2				DATE SAMPLED:	27/06/2016
PURPOSE	material classification	Subgrade		DATE SOAKED:	1/7/2016
MDD (g/cc)	1.64	OMC (%)	19.5	DATE TESTED:	4/7/2016

DENSITY DETERMINATION

SOAKING CONDITION	10 Blows		30 Blows		65 Blows	
	BEFORE	AFTER	BEFORE	AFTER	BEFORE	AFTER
MOLD NUMBER	12		HC10		HC5	
WEIGHT OF SOIL + MOLD (gm)	10355	10955	10688	10901	10900	11363
WEIGHT OF MOLD (gm)	6868	6868	6831	6831	6796	6796
WEIGHT OF SOIL (gm)	3487	4087	3857	4070	4104	4567
VOLUME OF MOLD (cc)	2123	2123	2123	2123	2123	2123
WET DENSITY OF SOIL (g/cc)	1.64	1.93	1.82	1.92	1.93	2.15
DRY DENSITY OF SOIL (g/cc)	1.40	1.42	1.55	1.47	1.65	1.68

MOISTURE DETERMINATION

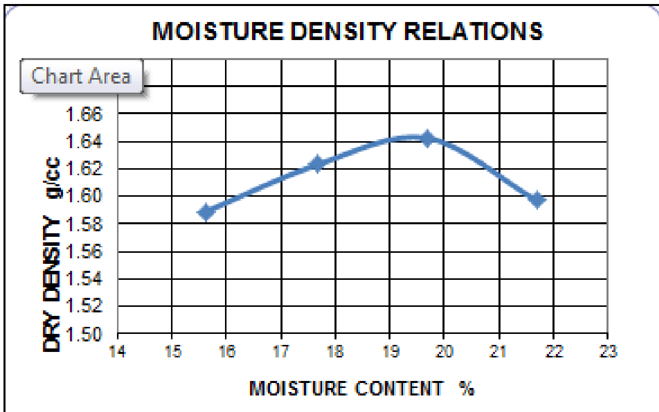
SOAKING CONDITION	10 Blows			30 Blows			65 Blows		
	BEFORE	AFTER		BEFORE	AFTER		BEFORE	AFTER	
		TOP 1 in.	AVG.		TOP 1 in.	AVG.		TOP 1 in.	AVG.
CONTAINER NUMBER	A7	C1		A13	D1		D7	D6	
WET SOIL + CONTAINER (gm)	314	346		642	372		476	368	
DRY SOIL + CONTAINER (gm)	273	274		593	302		418	308	
WEIGHT OF WATER (gm)	41.0	72.0		49.0	70.0		58.0	60.0	
WEIGHT OF CONTAINER (gm)	39	71		313	75		74	69	
WEIGHT OF DRY SOIL (gm)	234	203		280	227		344	214	
MOISTURE CONTENT (%)	17.52	35.47		17.50	30.84		16.86	28.04	

PENETRATION TEST DATA

ENETRATIO	10 Blows				30 Blows				65 Blows			
(mm)	DIAL RDG	LOAD (kn	COR. LOAD	**CBR %**	DIAL RDG	LOAD (	COR. LOA	**CBR %**	DIAL RDG	LOAD (k	COR LOA	**CBR %**
0	0	0.0000			0	0			0	0		
0.64	2	0.0439			3	0.0659			7	0.1538		
1.27	4	0.0879			6	0.1318			10	0.2197		
1.96	9	0.1977			13	0.2856			14	0.3076		
2.54	25	0.5493	1	4	27	0.5932	1	4	28	0.6152	1	5
3.18	27	0.5932			29	0.6371			31	0.6811		
3.81	30	0.6591			31	0.6811			33	0.7250		
4.45	32	0.7030			37	0.8129			39	0.8568		
5.08	37	0.8129	1	4	40	0.8788	1	4	42	0.9227	1	5
7.62	39	0.8568			42	0.9227			46	1.0106		
10.16												
12.7												

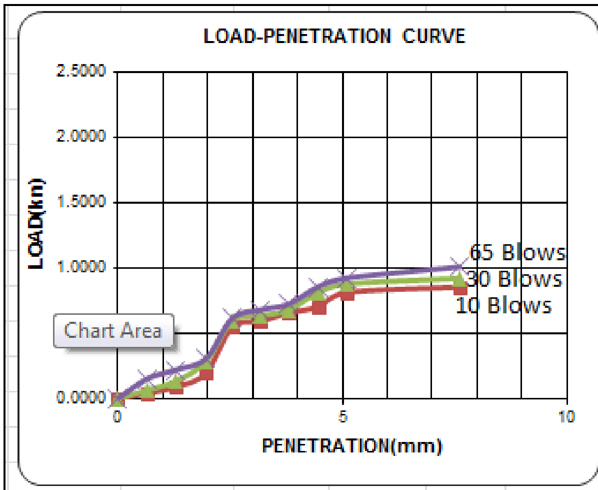
SWELL

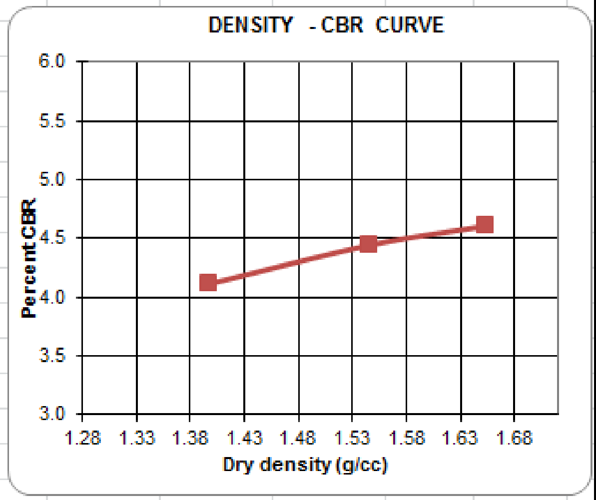
No. OF BLOWS	10	30	65
RDG (BEFORE SOAKING	15.62	4.71	17.48
RDG (AFTER) SOAKING)	20.83	9.52	21.33
PERCENT SWELL	4.48	4.13	3.31
AVERAGE PERCENT SWELL :		3.97	



MODIFIED PROCTOR : T 180	
MDD (g/cc) :	1.64
OMC (%) :	19.5
95 % of MDD(g/cc)	1.56

	Before Soaking		After Soaking	
Blows	DD (g/cc)	Moisture (%)	DD (g/cc)	Moisture (%)
10	1.40	17.52	1.42	35.47
30	1.55	17.50	1.47	30.84
65	1.65	16.86	1.68	28.04





Blow	LOAD (KN)		CBR(%)		Swell %
	2.54mm	5.08mm	2.54mm	5.08mm	
10	1	1	4.1	4.1	4.48
30	1	1	4.4	4.4	4.13
65	1	1	4.6	4.6	3.31

Blow	Dry density	CBR%
10	1.40	4.1
30	1.55	4.4
65	1.65	4.6
CBR at 95%		**4.4**

Appendix J: Compaction curve sample result of the capping at section 1

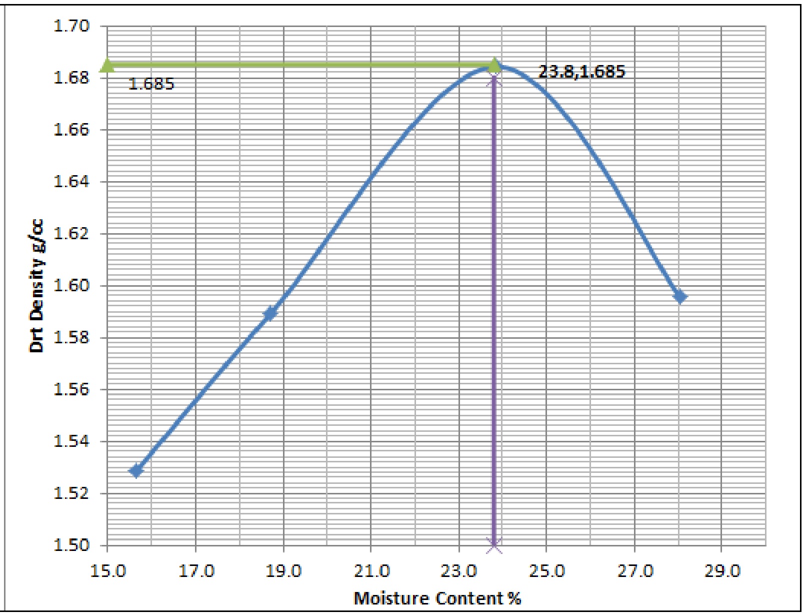
MOISTURE DENSITY RELATIONSHIP OF SOIL
TEST METHOD : AASHTO T-180 METHOD D

REPRESENTING SECTION	section 1	DATE SAMPLED	13/06/2016
MATERIAL DESCRIBTION	Gravel material	DATE TESTED	17/06/2016
PURPOSE	Capping		

DENSITY	TRIAL NUMBER	1	2	3	4	5		
	WEIGHT OF SAMPLE (g)	6000	6000	6000	6000			
	WATER ADDED litre	400.0	600.0	800.0	1000.0			
	WEIGHT OF SOIL + MOLD (g)	10,310	10,560	10,980	10,890			
	WEIGHT OF MOLD (g)	6590	6590	6590	6590			
	WEIGHT OF SOIL (g)	3720	3970	4390	4300			
	VOLUME OF MOLD (cc)	2105	2105	2105	2105			
	Wet DENSITY OF SOIL (g/cc)	1.77	1.89	2.09	2.04			NMC

MOISTURE	CONTAINER NUMBER	A4	CH	A1	B3			A2
	WET SOIL + CONTAINER (g)	272.6	251	206.6	198.8			279.4
	DRY SOIL + CONTAINER (g)	239.5	215.6	171.4	160.8			259.3
	WEIGHT OF WATER (g)	33.10	35.40	35.20	38.00			20.1
	WEIGHT OF CONTAINER (g)	27.8	26.3	23.7	25.3			25.8
	WEIGHT OF DRY SOIL (g)	211.7	189.3	147.7	135.5			233.5
	MOISTURE CONTENT (%)	15.6	18.7	23.8	28.04			8.6
	DRY DENSITY OF SOIL (g/c)	1.53	1.59	1.68	1.60			8.6

MDD (gm/cc) :	1.685	
OMC (%) :	23.83	



Appendix K: CBR analysis capping sample result at section 1

CALIFORNIA BEARING RATIO TEST TEST METHOD : AASHTO T-193			
REPRESENTED SECTION	SECTION 1	DATE SAMPLED	13/06/2016
MATERIAL DESCRIPTION	Gravel material	DATE AND TIME SOAKED	17/06/2016
PUROPSE	**Capping**	DATE AND TIME TESTED	21/06/2016

DENSITY DETERMINATION

SOAKING CONDITION			10 Blows		30 Blows		65 Blows	
			BEFORE	AFTER	BEFORE	AFTER	BEFORE	AFTER
MOLD NUMBER			M-8		Z-5		B-7	
WEIGHT OF SOIL + MOLD	(g)	W_1	11930	13170	11440	12200	11670	12250
WEIGHT OF MOLD	(g)	W_2	7990	7990	6960	6960	6810	6810
VOLUME OF MOLD	(Cr	V	2266	2266	2266	2266	2266	2266
WEIGHT OF WET SOIL	(g)	$W_3 = W_1 - W_2$	3940	5180	4480	5240	4860	5440
WET DENSITY OF SOIL	(g/c	$W_d = (W_3/V)$	1.74	2.29	1.98	2.31	2.14	2.40
DRY DENSITY OF SOIL	(g/c	$D_d = W_d/(100+m)*100$	1.40	1.81	1.62	1.83	1.74	1.90

MOISTURE DETERMINATION

SOAKING CONDITION		10 Blows			30 Blows			65 Blows		
		BEFORE	AFTER		BEFORE	AFTER		BEFORE	AFTER	
			TOP 1 in.	AVG.		TOP 1 in.	AVG.		TOP 1 in.	
CONTAINER NUMBER		A4	E		CH	F		B3	A4	
WET SOIL + CONTAINEI	a	199.1	212.7		213.1	214.1		205.1	243.4	
DRY SOIL + CONTAINER	b	166.1	173.7		179.1	174.9		170.8	198.8	
WEIGHT OF CONTAINER	c	27.7	25.1		26.2	26.5		25.2	27.8	
WEIGHT OF WATER	d = a - b	33.0	39.0		34.0	39.2		34.3	44.6	
WEIGHT OF DRY SOIL	e = b - c	138.4	148.6		152.9	148.4		145.6	171	
MOISTURE CONTENT	m =(d/e)*100	23.8	26.2		22.2	26.4		23.6	26.1	

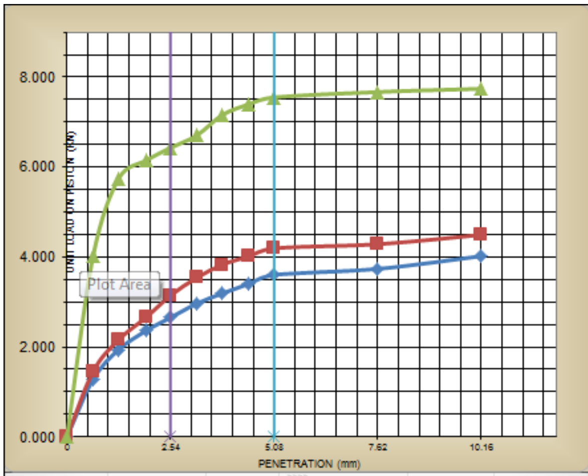
PENETRATION TEST DATA

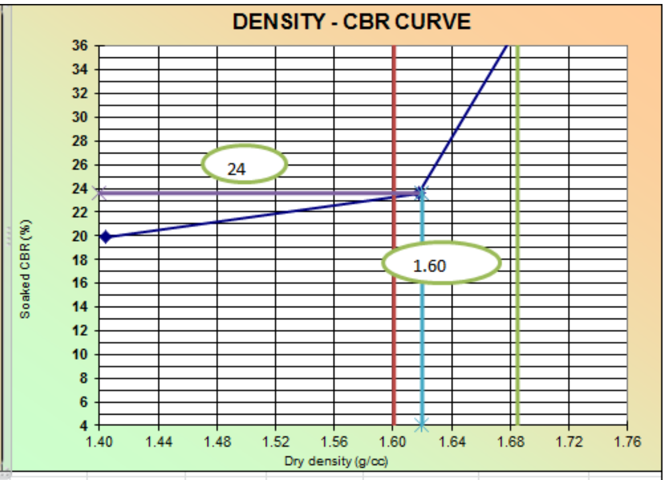
ENETRATION	10 Blows				30 Blows				65 Blows			
(mm)	DIAL RDG	LOAD (k	COR. LOAD	**CBR %**	DIAL RDG	LOAD (k	COR. LOAD (	**CBR %**	DIAL RDG	LOAD (kn)	COR LOAD (kr	**CBR %**
0	0	0.000			0	0			0	0		
0.64	280	1.277			318	1.45			880	4.01		
1.27	426	1.943			474	2.16			1260	5.75		
1.96	520	2.371			586	2.67			1350	6.16		
2.54	582	2.654	2.65	19.88	690	3.15	3.15	23.6	1410	6.43	6.43	48.2
3.18	650	2.964			776	3.54			1470	6.70		
3.81	700	3.192			840	3.83			1570	7.16		
4.45	746	3.402			880	4.01			1622	7.40		
5.08	790	3.602	3.60	18.08	920	4.20	4.20	21.0	1656	7.55	7.55	37.8
7.62	820	3.739			940	4.29			1682	7.67		
10.16	882.0	4.022			986.0	4.496			1698.0	7.74		
12.7												

SWELL				RING FACTOR		
No. OF BLOWS	**10**	**30**	**65**	4.56 N/Divis.	MDD (gm/cc)	1.69
RDG (BEFORE SOAKING)	0.00	0.00	0.00		OMC %	23.8
RDG (AFTER) SOAKING)	1.68	0.18	0.59		95 % of MDD	1.60
PERCENT SWELL	1.44	0.15	0.51			
AVERAGE PERCENT SWEL	0.70					

BLOWS	LOAD (KN)		CBR(%)		SWELL %
	2.54mm	5.08mm	2.54mm	5.08mm	
10	2.65	3.60	19.9	18.1	1.44
30	3.15	4.20	23.6	21.0	0.15
65	6.43	7.55	48.2	37.8	0.51

DRY DENSITY Vs SOCKED C.B.R.			
No # OF BLOWS	10	30	65
DRY DENSITY	1.40	1.62	1.74
SOCKED C.B.R.	19.9	23.6	48.2





MODIFIED PROCTOR : T 180	
MDD (g/cc) :	1.685
OMC (%) :	23.83
95% of MDD(g/cc)	1.60
CBR@ 95% of MDD	**24.0**

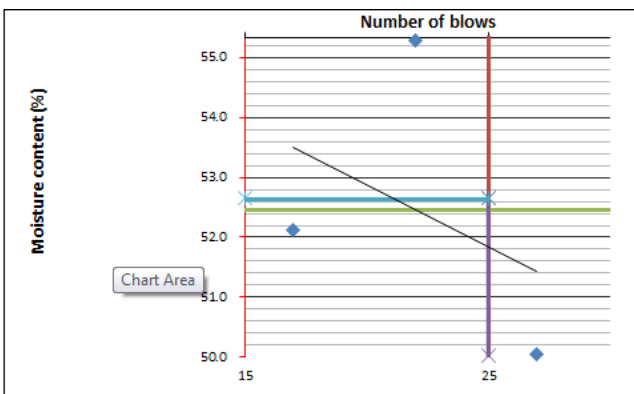
APPENDIX L: Analysis of atterberg's limit of subgrade sample results

REPRESENTED SECTION	SECTION 1	DATE SAMPLED	06/06/16
MATERIAL DESCRIPTION	Red clay soil	DATE TESTED	10/06/16
TYPE OF LAYER	subgrade		

DETERMINATION OF LIQUID LIMIT & PLASTIC LIMIT OF SOIL

TEST METHOD : AASHTO T89

Determination		*Liquid Limit*				*Plastic Limit*		
Number of blows		*27*	*22*	*17*				
Test	No	1	2	3		1	2	
Container	No	B	A1	1		E	E5	
Wt. of container + wet soil,	(g)	54.38	56.66	52.52		10.63	9.81	
Wt. of container + dry soil,	(g)	43.40	43.45	41.36		10.08	9.24	
Wt. of container,	(g)	21.45	19.55	19.94		8.28	7.36	
Wt. of water,	(g)	10.98	13.21	11.16		0.55	0.57	
Wt. of dry soil,	(g)	21.95	23.90	21.42		1.80	1.88	
Moisture container,	(%)	*50.0*	*55.3*	*52.1*		*30.6*	*30.3*	
Average	(%)		52.47			30.4		



Initial length of specimen (L_1)______

Length of dried specimen (L_2)______

Linear shrinkage = $\left(\frac{(L_1 - L_2)x100}{L_1}\right)$

If LL or PL cannot be determined

use PI = 2.13 x LS =

Remark

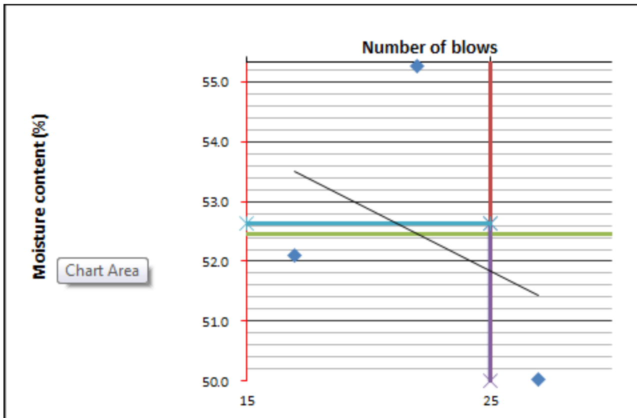
Determination of (PI)		*(LL - PL)*		**Grading**			
	-	LL	56	ASTM SIEVE NO	10	40	200
	-	PL	30	Dim. mm	2.00	0.425	0.075
	PI	26		% passing	97.6	72.8	36.9
Plastic Product		**943.5**		**AASHTO Soil Classification**	**A-7-5**		

REPRESENTED SECTION	SECTION 1	DATE SAMPLED	19/07/2016
MATERIAL DESCRIPTION	Red clay soil	DATE TESTED	24/07/2016

DETERMINATION OF LIQUID LIMIT & PLASTIC LIMIT OF SOIL

TEST METHOD : AASHTO T89

Determination		*Liquid Limit*				*Plastic Limit*		
Number of blows		*27*	*22*	*17*				
Test	No	1	2	3		1	2	
Container	No	B	A1	1		E	E5	
Wt. of container + wet soil,	(g)	54.38	56.66	52.52		10.63	9.81	
Wt. of container + dry soil,	(g)	43.40	43.45	41.36		10.08	9.24	
Wt. of container,	(g)	21.45	19.55	19.94		8.28	7.36	
Wt. of water,	(g)	10.98	13.21	11.16		0.55	0.57	
Wt. of dry soil,	(g)	21.95	23.90	21.42		1.80	1.88	
Moisture container,	(%)	*50.0*	*55.3*	*52.1*		*30.6*	*30.3*	
Average	(%)		52.47			30.4		



Linear shrinkage

Initial length of specimen (L_1)______

Length of dried specimen (L_2)______

Linear shrinkage = $\left(\frac{(L_1 - L_2)x100}{L_1}\right)$

If LL or PL cannot be determined

use PI = 2.13 x LS =

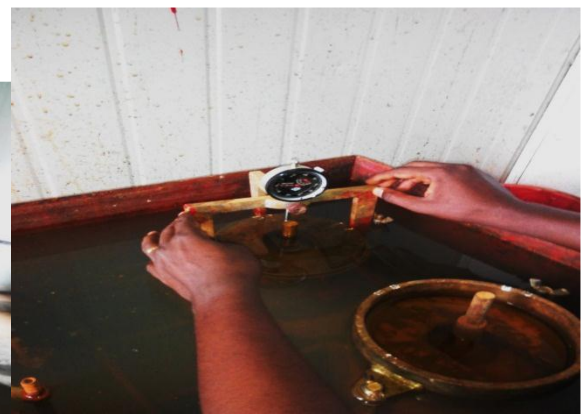
Remark

Determination of (PI)	*(LL - PL)*			Grading			
	-	LL	53	ASTM SIEVE NO	10	40	200
	-	PL	30	Dim. mm	2.00	0.425	0.075
	PI	22		% passing	#REF!	#REF!	34.0
Plastic Product		754.9		AASHTO Soil Classification	A-7-6		

Appendix N: Some samples of the laboratory and field investigation photos

Figures of laboratory tests



Figures of laboratory tests.

Figures of field investigations

Appendix L Observations and relevant remarks about the different deteriorations.

SECTION 1						
no.	Station	length	side	deterioration	width	area(m2)
1	130+504	+16	LHS	potholes	5.m	80
2	130+518	+5	C/L	potholes	3.5m	17.5
3	130+518	+7	LHS	potholes	3.5m	23.5
4	130+518	+3	LHS	potholes	1.m	3
5	130+523	+4	C/	potholes	1.m	4
6	130+528	+6	C/	potholes	4.m	24
7	130+532	+4	C/	potholes	6.m	24
8	130+535	+10	RHS	pumping	8.m	80
9	130+537	+2	LHS	pumping	2.m	4
10	130+537	+7	LHS	edge	3.m	21
11	130+539	+10	RHS	edge	6.m	60
12	130+542	+2	LHS	pumping and potholes	3.5m	7
13	130+544	+2	LHS	pumping and potholes	3.5m	7
14	130+547	+3	LHS	potholes	4m	12
15	130+549	+3	RHS	pothole	4.m	12
16	130+550	+1	LHS	pothole	1.m	1
17	130+553	+3	C/L	pothole	2.m	6
18	130+564	+1	C/L	pothole	1.m	1
19	130+568	+1	RHS	potholes	1.m	1
20	130+571	+10	RHS	pumping and potholes	3.5m	35
21	130+576	+2	RHS	pumping	1.5m	3
22	130+579	+10	RHS	pumping	3.5m	35
23	130+583	+20	LHS	pumping	3.5m	7
24	130+585	+2	RHS	patch	3.5m	7
25	130+585	+7	LHS	patch	3.5m	24.5
26	130+589	+10	RHS	pothole	5.m	50
27	130+593	+4	RHS	potholes	3.5m	13.5
28	130+595	+2	LHS	potholes	4m	16
29	130+597	+2	RHS	potholes	3.m	6
30	130+598	+1	RHS	pumping and potholes	3.5m	3.5
31	130+599	+2	RHS	potholes	7.m	14
32	130+601	+2	C/	potholes	3.m	6
33	130+607	+6	LHS	pumping	4.m	36
34	130+612	+5	LHS	edge	7.m	35
TOTAL AREA						**676.5m2**

SECTION 2						
35	102+030	+13	LHS	rutting	5m	65
36	102+046	+6	C/L	rutting	2.m	12
37	102+048	+1	C/L	rutting	1.m	1
38	102+053	+5	RHS	rutting	2.m	10
39	102+058	+4	C/L	rutting	0.5m	2
40	102+059	+6	F/W	rutting	7.m	48
41	102+063	+2	C/L	rutting	2.m	4
42	102+066	+2	LHS	rutting	2.m	4
43	102+071	+2.5	LHS	rutting	3.5m	8.5
44	102+076	+1	F/W	rutting	5.m	5
45	102+074	+1.8	LHS	potholes	1.m	1.8
46	102+077	+2	LHS	Pothole and rutting	2.m	4
47	102+086	+2	LHS	potholes	2.m	4
48	102+088	+1	RHS	potholes	3.m	3
49	102+098	+8	LHS	potholes	5.m	40
50	102+123	+6	LHS	potholes	4m	24
51	102+129	+3	RHS	rutting	3.m	9
52	102+134	+8	RHS	rutting	5.m	40
53	102+156	+2	RHS	rutting	1.m	2
54	102+167	+9	C/L	Potholes and rutting	3m	27
55	102+169	+2	C/L	rutting	3.m	6
56	102+178	+7	RHS	potholes	1.m	7
57	102+181	+3	C/L	Potholes and rutting	3.m	15
58	102+184	+2	C/L	potholes	3.m	6
59	102+186	+1	RHS	Potholes and rutting	3.m	3
60	102+191	+20	C/L	rutting	4.m	80
TOTAL AREA					**421.6m2**	
SECTION 3						
61	76+240	+5	C/L	edge	2.m	10
62	76+278	+20	F/W	edge	2.m	40
63	76+298	+2	F/W	edge	7.m	14

64	76+321	+6	RHS	raveling	3.5m	23.5
65	76+346	+9	LHS	raveling	3.5m	30.5
66	76+369	+11	RHS	edge	3.5m	38.5
67	76+448	+12	RHS	edge	1.m	12
68	76+448	+1	RHS	raveling	3.m	3
69	76+449	+1	RHS	edge	2.m	2
70	76+456	+5	RHS	edge	2.m	10
71	76+467	+11	LHS	raveling	3.5m	38.5
72	76+479	+10	RHS	raveling	2.m	20
73	76+488	+6	RHS	raveling	3.5m	21.5
74	76+508	+10	LHS	edge	2.m	20
75	76+528	+2	LHS	edge	3.5m	7
76	76+534	+4	LHS	edge	2.m	8
77	76+544	+3	LHS	edge	2.m	6
78	76+548	+2	RHS	raveling	3.m	6
79	76+555	+4	RHS	raveling	3.5m	14
80	76+560	+9	LHS	raveling	2.m	18
81	76+568	+10	RHS	raveling	3m	30
TOTAL AREA					**331m2**	

: