JIMMA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

JIMMA INSTITUTE OF TECHNOLOGY

FACULTY OF CIVIL AND ENVIRONMENTAL ENGINEERING

HIGHWAY ENGINEERING STREAM

Evaluation of the Distresses and the Performance of Asphalt Pavement Based on Pavement Condition Index and Engineering Properties of Layers: A Case Study of Alaba-Sodo Road

By

Asrat Dansa

A Thesis submitted to the School of Graduate Studies of Jimma University in partial fulfillment of the requirements for the Degree of Master of Science in Civil Engineering (Highway Engineering Stream)

MAY, 2017

JIMMA, ETHIOPIA

JIMMA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

JIMMA INSTITUTE OF TECHNOLOGY

FACULTY OF CIVIL AND ENVIRONMENTAL ENGINEERING

HIGHWAY ENGINEERING STREAM

Evaluation of the Distresses and the Performance of Asphalt Pavement Based on Pavement Condition Index and Engineering Properties of Layers: A Case Study of Alaba-Sodo Road

By

Asrat Dansa

APPROVED BY BOARD OF EXAMINERS

1. Eng. Elmer C. Agon (Asst.Prof) _____________ ____/_____/________

   Main advisor Signature Date

2. Eng. Murad Mohammed (PhD. Fellow) _____________ ____/_____/________

   Co-advisor Signature Date

3. Dr. Bikila Teklu (PhD) _____________ ____/_____/________

   External Examiner Signature Date

4. Eng. Desalegn Ayele (MSc) _____________ ____/_____/________

   Internal Examiner Signature Date

5. Eng. Tarekegn Kumela (MSc) _____________ ____/_____/________

   Chairperson Signature Date

ACKNOWLEDGEMENT

First of all, I would like to thank God for helping me in all my ways. I want to express my deepest gratitude to my advisors, Eng. Elmer C. Agon (Asst.Prof) and Eng. Murad Mohammed (PhD. Fellow) for their invaluable guidance, patience, kindness, and respect to supports and comments for the preparation of this academic thesis.

I would also like to thank you to Eng. Adimasu, Ethiopia Roads Authority South Region Team Leader, Eng. Minilik, Ethiopia Roads Authority Sodo District Maintenance Team Leader, and their teams for providing me the necessary document and advice for the study, without their help it could be very difficult to materialize this study. Moreover, all laboratory assistant workers, whose love and support made this work possible.

I would also like to thank The Federal Democratic Republic of Ethiopia, Ethiopian Road Authority (ERA) Sodo District for permitting to take samples along the road and giving me service car to transport samples.

Next, my appreciation goes to Ethiopian road Authority and Jimma University for providing the opportunity to attend master's degree in Highway Engineering.

Last but not the least, I would like to say thank you to my family and friends for their continuous support from the beginning of my study of this program.

ABSTRACT

Generally, complex distress is the result for pavement deterioration like pavement cracking, deformation of the pavement structure, and disintegration of materials. This deterioration may begin as soon as the road is open to traffic. In Ethiopia, pavement distress is the main problem of roads affecting the socio-economic development of the nation. Alaba-Sodo road, which is located in SNNPRS, is one of the roads affected by distresses. The main objective of this study is to evaluate the distresses and the performance of Alaba-Sodo asphalt pavement based on pavement condition index and engineering properties of the layers.

Along the Alaba-Sodo road, 60 sample units of defective locations were selected using systematic random sampling technique. For the selected samples, the PCI values were used to evaluate the performance of the road. In addition, Laboratory experiments were conducted to check the compliance of the engineering property of the pavement materials with ERA specifications. Based on the PCI values, four sub-base, base and sub-grade samples were taken for laboratory experiment. Out of these, 2 were taken from location which were in a very poor condition, 1 from poor condition and 1 from where there is no distress. The major tests such as CBR test, Compaction Test, Atterberge limit test, and Sieve Analysis were conducted for each sample. The thicknesses of the layers in these points were also measured so as to check whether or not they are as per the design.

From the pavement condition survey, it was found that the PCI values range from 25- 79. Among the 60 sample units, 11.67% were very good, 16.67% were good, 43.33% were fair, 26.67% were poor, and 1.67% was very poor. Except for the sub-base, all of the layers satisfy the ERA specification requirements. Moreover, the thicknesses were found to be as per the design.

In conclusion, most of the sections of the road are in fair and poor conditions and the cause of the premature distress of the road can be the poor quality of sub-base material. Therefore, to avoid further destruction of the road, more detailed tests on the structural capacity of the pavement should be made and based on the result of those tests, remedial measures should be made urgently.

Key word: *Pavement Condition Index (PCI), Pavement Condition Rating (PCR), Pavement defects, Type and Severity level of distress.*

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

ACRONYMS

AASHTO	American Association of State of Highway and Transportation Officials
AC	Asphalt Concrete
ASTM	American Society for Testing and Material
BC	Before Christ
CDV	Corrected Deduct Value
CBR	California Bearing Ratio
DS	Design Standard
DV	Deduct Value
ERA	Ethiopia Road Authority
FDRE	Federal Democratic Republic of Ethiopia
G.C	Gregorian Calendar
H	High
JiT	Jimma institute of Technology
KIPPRA	Kenya Institute for Public Policy Research and Analysis
Km	Kilo meter
L	Low
LL	Liquid Limit
M	Medium
MDD	Maximum Dry Density
OMC	Optimum Moisture Content
PCI	Pavement Condition Index
PCR	Pavement Condition Rating
PCS	Pavement Condition Survey
PI	Plastic Index
PL	Plastic Limit
SNNPRS	Southern Nations, Nationalities, and Peoples Regional State
USCS	Unified Soil Classification System

CHAPTER ONE

INTRODUCTION

1.1 Background

Maintained road is the basic infrastructure that has a significant contribution in development of any country. For rapid economic, industrial, and cultural growth of any country, a good performance system of road transportation is very essential. Roads, to satisfy their intended purpose, must be constructed safe, easy, economical, Soundly maintained before any major defects occurred, environment friendly, and must full fill the needs of the society (Maharaj, 1996).

A well-designed and constructed road network plays an important role in transporting people and other agricultural and industrial products to any direction with in short period of time. The importance of roads increases as the area of the country increases, especially in the absence of other means of transport such as railways and waterways, which is often occurred in developing countries (Wilson, 2001).

The government of the Federal Democratic Republic of Ethiopia (FDRE) as part of its Road Sector Development Program intends to adopt improved construction and maintenance policies as a major element of road management. Besides, to routine maintenance of the road networks, periodic maintenance intervention is found essential at certain time intervals and the rehabilitation is very significant to reduce the deterioration of roads due to normal service to traffic and to restore the road to improved condition of service. With this aim, the Alaba – Sodo two lanes flexible bituminous-surfaced DS4 road is one of the upgraded road from the existing asphalt surface treatment road to an asphalt concrete standard road. The rehabilitation of the road to well accessible asphalt standard is considered to have a crucial role in developing and integrating the regional economy while this road would been a two lane-paved carriageway, 7.0m wide with 1.5m wide gravel shoulders on each side. The road segment passing through minor town sections would have a 3.5m additional parking lane and 2.5m walkway on either side of the 7.00m carriageway and that of the segment through major towns would have a 2m median at center of the road in addition to that of the minor town section. Moreover, its stretch is 68 km long and operation was started in 2010 (Engineering report, 2006).

Also, the deterioration process starts directly after opening the road to traffic. This process starts very slowly so that it may not be noticeable, and over time it accelerates at faster rates. To ensure the risk of premature deterioration is minimized, it is necessary to use the best practice method in planning, design, construction, and maintenance of the road. This can be achieved by evaluating or examining pavements that have failed prematurely, with the focus being on determining the causes of failure so that it can be prevented in the future. The greater understanding of pavement failures that could be gained from detailed investigations could be valuable in reducing the costs associated with pavement failures in the future (ASTM D6433, 1999).

Obviously, most pavement deterioration results from different factors such as environment, quality of materials, construction techniques, and/ or structural causes and so on. However, it is important to try to distinguish between from these factors in order to select the most effective maintenance and/ or rehabilitation techniques (Ali-Mohamed, 2011).

To address the objectives of this research, the researcher required procedures starting from preliminary site visit, detail pavement condition survey, collection of material samples for laboratory tests to determine the engineering properties of pavement layer, in addition to the primary data the researcher collects relevant secondary data and analysis of the data’s was performed. Finally, evaluation of results with available specification and standard had been under taken to evaluate the pavement condition and its performance based ASTM D6433, ERA, and AASHTO Manuals.

1.2 Statement of the Problem

Generally, complex distress is a result of pavement deterioration as pavement cracks through fatigue under repeated loadings; deformation of the pavement structure through shearing; and disintegration of materials when mechanical or chemical bonds are broken through weathering, infiltration and increased loading. Underground conditions, structures, traffic characteristics, and environmental contexts all have a tremendous impact on the performance of highway pavements (Ali-Mohamed, 2011).

In Ethiopia, the pavement distress is the leading problem that affects economic, industrial, and cultural growth of the country, performance system of road transportation, the accessibility, users comfort and national/ social development of the country. When taking one example, Jimma to Addis Ababa road, it is one of severely distressed road in Ethiopia, is probably distressed by a combination of different factors such as traffic load, environment, initial design, and quality of construction.

When considering the case of Alaba-Sodo road pavement, it has different problems. First, the road is characterized by heavy deformation with extreme alligator/fatigue cracks, potholes, block cracking, rutting and others cracks and disintegration. These causes traffic hazards, taking long time for travel, affecting economic, industrial and cultural growth of the country. Movement of agricultural products and performance system of road transportation is directly affected also. The accessibility, users comfort and national/ social development of country and increasing vehicle operating cost are directly and indirectly affected by road distress. Second, the road is a main hub and serves as a business route to cash crops cultivated in the area like coffee, banana, mango, eucalyptus, cotton, maize, sugarcane, other edible, non-edible, other different crops and also a promising fishing industry among other things. Third, the road from Alaba to Hulbarege is newly constructed alternative road to Addis Ababa-Alaba that creates new traffic volume on this study road. Fourth, the traffic flow from upgrading Durame to Mazoria road is also creating additional volume to Alaba – Sodo road stretch. Fifth, the road serves as a major route to one of the most well-known tourist attraction area in the region. Sixth, the road may also serve as an alternate potential route to the neighboring country of Kenya. Therefore, this research was important in order to evaluate the pavement condition and its performance in terms of Pavement Condition Index (PCI) and Engineering properties of the pavement layers of Alaba-Sodo road pavement.

1.3 Objectives

1.3.1 General Objectives

The general objective of this research was to evaluate the distresses and the performance of Alaba-Sodo asphalt pavement based on pavement condition index and engineering properties of the road layers.

1.3.2 Specific Objectives

The specific objectives of this research are;

- ✓ To determine the types, severity and density of the pavement distress based on visual inspection and field measurement.
- ✓ To determine the pavement condition or performance based on PCI.
- ✓ To check the effect of engineering properties of pavement layers on pavement distress.
- ✓ To provide remedial measures for the affected road stretches.

1.4 Research Questions

The research question will be as follows:

- ✓ How to determine the types, severity and density of pavement distress based on visual inspection and field measurement?
- ✓ How to determine the performance of pavement based on PCI?
- ✓ How the engineering properties of the pavement layers affect the pavement performance?
- ✓ What are the remedial measures for the severely affected road stretches?

1.5 Scope of the Study

This study focuses on the evaluation and estimation of asphalt pavement road condition through condition surveys using the Pavement Condition Index (PCI) method by following ASTM D6433 standard, and determination of the engineering properties of the road layers to check its effect on pavement distress. The engineering properties of materials are determined by carrying out different tests on samples of materials in a laboratory. Tests for the govern of engineering properties like Atterberg Limits, Gradation, Soil Classification, Proctor Test, and CBR Test were performed. To achieve this objective, the SNNPRS of the road part that is Alaba-Sodo road stretch was used.

1.6 Significance of the Study

The study is mainly focused on investigating the performance of the Alaba-Sodo road. In understanding this, the ERA (any concerned bodies) can undergo the remedial measures on locations along the road which are defective. Due to this, the socio-economic development of the study area as well as the nation would be accelerated.

Furthermore, this paper can also be one of the local input for current highway engineering knowledge of the country. Therefore, different universities or researchers can use it as a reference material for future studies on both the study area and other locations.

1.7 Limitation of the Study

The major limitations of the study were lack of the necessary laboratory and field equipment in the district that enable the researcher to conduct the necessary pavement material tests and pavement condition surveys. Regarding the laboratory equipment, though there are some important equipment, they are not sufficient to make comprehensive tests. Whereas regarding the pavement condition survey, there was no standard surveying equipment such as straight edge, hand odometer, automated inspection equipment, and others in the district.

CHAPTER TWO

LITERATURE REVIEW

2.0 Review of Performance of Flexible Pavement and Construction Materials

Pavement is the most common part of the transportation infrastructure and is built to provide a safe and comfortable ride for the public. To maintain a pavement system with an acceptable ride quality different types of data to be collected with evaluation of a current pavement condition, and checking of the pavement layers' effect on pavement distress. The evaluations include pavement condition survey, and road construction materials tests, surface roughness survey, surface deflection survey and surface friction (skid resistance) survey. From all of these methods of evaluation only the first two methods were performed in this research and the remained methods were not performed due to lack of instrument and resource (Ali-Mohamed, 2011).

2.1 Evaluation of Flexible Pavement Performance

Pavement performance evaluation is an important activity in the maintenance and rehabilitation works. It includes evaluation of existing distresses, road roughness, structural adequacy, traffic analysis, material testing and study of drainage condition but in this research only evaluate the existing distresses and performance of pavement based PCI and pavement layer properties. And this section also deals with types of bituminous surfaces, types and causes of distresses. Photographs all taken during pavement condition surveys and material sampling from different location are incorporated for illustration.

2.1.1 Meaning of Pavement

A pavement is a layered structure that has sufficient total thickness and internal strength to carry expected traffic loads, and distribute them over the subgrade soil without overstressing. Pavements have adequate properties to prevent or minimize the penetration or internal accumulation of moisture. It also has a surface that is reasonably smooth and skid resistant at the same time, as well as, reasonably resistant to wear, distortion and deterioration by vehicle loads and weather. To maintain a pavement system with an acceptable ride quality. The increased volume of traffic, load, and environmental conditions are factors that have created enormous amounts of wear and tear on the highway systems. More research is needed to determine the influences of these factors upon pavement performance (Feng and Dar, 2009).

2.1.2 Flexible (Bituminous Pavements)

A flexible pavement is one, which has low flexural strength, and the load is largely transmitted to the subgrade soil through the lateral distribution of stresses with increasing depth. The pavement thickness is designed such that the stresses on the subgrade soil are kept within its bearing capacity and the subgrade is prevented from excessive deformation. The strength and smoothness of flexible pavement structure depends to a large extent on the deformation of the subgrade soil (ERA, 2002).

A typical flexible or bituminous pavement structure consists of the following pavement courses: sub-grade, sub-base, base, and bituminous wearing surface. The wearing surface is the uppermost layer of the pavement structure. In a flexible pavement, it is a mixture of bituminous binder material and aggregate. The binder and aggregate may be mixed in a central plant or mixed in place on the road and referred to as hot or cold mixes. The wearing surface has four principal functions. These are; protection of the base from abrasive effects of traffic, load distribution to the underlying layers of pavement structure, prevention of surface water from penetrating into the base and sub-grade, and provision of a smooth riding surface for traffic.

The base and sub-base are made using different materials designated the upper and lower base or sub-base. Where the soil is considered to be very weak, a capping layer may also be introduced between the sub-base and the soil foundation. This may be of an inferior type of sub-base material, or it may be the upper part of the soil improved by some form of stabilization (e.g. with lime or cement). The soil immediately below the sub base (or capping layer) is generally referred to as the sub grade and the surface of the sub grade is termed the formation level (ERA, 2002).

Therefore, when the pavement is constructed and its deterioration process starts directly after opening the road to traffic. This process starts very slowly so that it may not be noticeable, and over time it accelerates at faster rates. To ensure the risk of premature deterioration is minimized, it is necessary to use the best practice method in planning, design, construction, and maintenance of the road. This can be achieved by evaluating or examining pavements that have failed prematurely, with the focus being on determining the causes of failure so that it can be prevented in the future. The greater understanding of pavement failures that could be gained from detailed investigations could be valuable in reducing the costs associated with pavement failures in the future (ASTM D6433, 1999).

Pavement deterioration is a result of complex distress as pavement cracking through fatigue under repeated loadings and environmental cycles; deformation of the pavement structure through shearing; and disintegration of materials when mechanical or chemical bonds are broken through weathering, infiltration, or loading. Underground conditions, structures, traffic characteristics, and environmental contexts all have a tremendous impact on the performance of highway pavements (Ali-Mohamed, 2011).

2.1.3 The Cause of Failure

The important things for a useful evaluation of pavement performance are identifying different types of pavement distress and linking them to a cause. Understanding the cause for current conditions is extremely essential in selecting an appropriate maintenance or rehabilitation technique. However, the major causes of pavement distresses and deterioration can be grouped in to three categories. The first is due to overloading that includes excessive gross loads, high repletion of loads and high tire pressure. Second climatic environmental conditions may cause surface irregularities and structural weaknesses on the pavement. For example, volume change of soil due to wetting and drying resulting from improper drainage may be the prime cause of pavement distress. A third causes may be disintegration of the paving materials due to method of construction and quality of construction material. Use of contaminated aggregate and inadequate construction supervisor are also factors that may aggravate pavement distress. Lack of maintenance will further aggravate pavement distress. Pavement distresses and deterioration usually occurs from the above-mentioned factors (Ali-Mohamed, 2011).

Those several factors are responsible for the degradation of pavements over time, affecting the service life of the pavement. The initial design of the pavement, based on anticipated traffic volumes and loads, is a major factor influencing its life. Cumulative traffic volume, especially truck traffic, is another major factor in the life of pavements. Finally, environmental factors such as moisture infiltration into the supporting base, and heat and cold cycles, affect how well the subsurface is able to support the pavement. The routine maintenance effort applied to a pavement also affects pavement life (Lavin, 2003).

2.2 Methods for Evaluation of Distresses and Performance of Pavement

The methods for evaluation of the distresses and performance of pavement can be grouped in to two broad categories; which are non-destructive and destructive survey methods**.** The non-destructive survey includes pavement condition survey, roughness survey and deflection survey. The destructive survey includes the DCP survey and test pit excavation. From all of these methods of evaluation only the pavement condition survey and test pit excavation methods were performed in this research and the remained methods were not performed due to lack of instrument and resource. Details of the two survey methods (pavement condition survey and test pit excavation for laboratory test methods) are discussed as follow:

2.2.1 Pavement Distress Survey

Distress survey can be performed manually, or automated equipment may be used. In this research case the manual survey method was performed. But in either case, the surface of the pavement is viewed and evaluation is made to determine the types of distress, the severity and the quantity of distress present on the pavement surface.

The type of distress tells us what type of damage has developed; the severity tells how bad the damage is; and the quantity gives us the extent of the type and severity of damage that is present. All three of these factors are required to get a full picture of the damage that has developed on the pavement surface for evaluation of a pavement's performance, and the causes of poor performance in either structural or functional modes. Various types and degrees or severity of distress are measured during the condition surveys and are used to determine the type and timing of maintenance, rehabilitation, and reconstruction.

The key for maintenance program is priority developing pavement distress information in a detailed manner. Therefore, pavement engineers have long recognized the importance of distress information in quantifying the quality of pavements in order to obtain an overall assessment of pavement conditions for a road network, it is often necessary to combine individual distress data to form the composite index called pavement condition index (PCI). PCI summarizes the condition of each pavement segment (Huang, 2004).

A complete survey of the selected sites is performed using Pavement Condition Index (PCI) method, PCI values range from zero (very poor) to 100 (excellent) and which assesses the present pavement surface condition based on specific criteria (Ali-Mohamed, 2011). In this procedure deduct values are assigned to certain observed distress types, according to their density and severity, and then subtracted from a perfect score to give the Pavement Condition Index (PCI) value and the pavement rating. The procedure consists of six steps, which are summarized below as follows (Ali-Mohamed, 2011): (1) The inspection unit inspects the target highways using a distress identification guide, and the approximate amount of each distress type/severity combination is recorded as a percentage by dividing the distress type/severity combination quantities by the total area of the segment and multiplying by 100. (2) The deduction values for each distress type/severity combination are determined from special deduct curves. The PCI procedure uses a set of "deduct curves" to calculate the numerical impact of each distress type/severity combination on the overall PCI. (3) The number of distress type/severity combinations with deducts values larger than 2 are counted. The obtained q-value is used later in the calculations to correct the curves because research found that if occurrences of small deduct values are included, the final value would be too small, or over estimated.

(4) The total deduct value is computed by summing all the deduct values for the distress type/severity combinations. (5) When multiple distress type/severity combinations are present, the deduct units must be corrected as more distress type/severity combinations occur in the same inspection unit, they have less and less impact. To account for this nonlinearity, the total deduction and q-values are used with correction curves to determine the corrected deduct value. (6) The corrected deduct value is subtracted from 100 to determine the inspection unit PCI in percentage.

2.2.2 Laboratory Tests

To determine the classification and engineering properties of the construction materials different tests should be conducted for subgrade, subbase and base course materials of the pavement. They includes Atterberg limit test, Sieve Analysis, Compaction test and California Bearing Ratio Test.

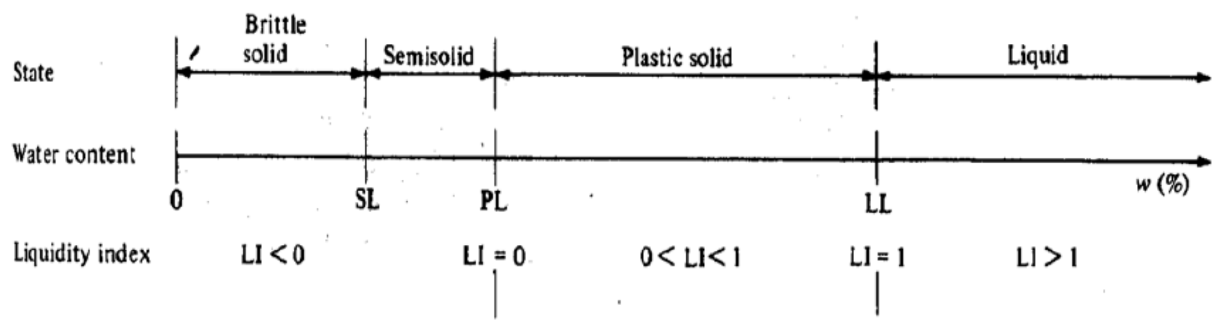
2.2.2.1 Atterberg Limit

This lab test is performed to determine the plastic and liquid limits of a fine-grained material and then determine the value of plasticity of the material and which are based on the moisture content of the material. The water contents corresponding to the transition from one state to another are termed as Atterberg Limits. The three Atterberg limits, which are liquid limit, plastic limit, and shrinkage limits are the boundary between each of the two consecutive states of the soil-water phases. The liquid limit and plastic limit test are performed only on that portion of a soil, which passes the 425mm (No. 40) Sieve (SABA Manual, 2002).

Liquid Limit (AASHTO T89): The liquid limit (LL) is the water content, expressed in percent, at which the soil changes from a liquid state to a plastic state and water content at which the soil part cut using standard groove closes for about a distance of 13cm (1/2 in) at 25 blows of the liquid limit machine (Casagrande Apparatus). The liquid limit of a soil highly depends upon the clay mineral present. The conventional liquid limit test is carried out in accordance of test procedures of AASHTO T 89 or ASTM D 4318 (SABA Manual, 2002).

Plastic Limit (AASHTO T90): The plastic limit (PL) is the water content, expressed in percentage, below which the soil stops behaving as a plastic material and it begin to crumble when rolled into a thread of soil of 3.0mm diameter. The conventional plastic limit test is carried out as per the procedure of AASHTO T 90 or ASTM D 4318 (SABA Manual, 2002).

Plasticity Index (PI = LL - PL): is the numerical difference between the liquid and plastic limits. Thus, it indicates the range of moisture content over which the soil remains deformable (in plastic state).



Generally, soils having **high values of liquid limit and plasticity index are poor as sub-grades/engineering materials**. Soils that cannot be rolled to a thread at any water content are termed as *Non-Plastic* (NP).

2.2.2.2 Grain Size Analysis

This test is performed to determine the percentage of different grain sizes contained within a soil. The grain size analysis or gradation is measured in the laboratory using two tests: a mechanical sieve analysis for the sand and coarser fraction (soil particles larger than 0.075(0.063)mm), and a hydrometer test for the silt and finer clay material (smaller particles) (AASHTO T27/T11 or ASTM D) (Structural Pavement Design lecture note, 2011/12).

2.2.2.3 Compaction Test

This laboratory test is a standard method of compaction using a standard amount of comp active effort to produce a soil density against which site density values can be compared. The original test involved compacting the soil in three approximately equal layers in a standard mould, using a 2.5kg hammer falling through a height of 305mm (standard compaction test). However, with the advent of heavier compaction equipment, greater densities were now achievable in the field. A modified version of the test was developed to allow the application of greater compactive effort (and achieve greater density) – i.e. compacting the soil of the same height in five approximately equal layers using a 4.5kg hammer falling through 457mm height (modified or heavy compaction test). Material or soil compaction tests are performed using disturbed soil sample. The tests are done in the laboratory according to AASHTO T-99 (Standard proctor test) for subgrade and AASHTO T-180 (Modified Proctor Test) is used for sub-base and base materials (SABA Manual, 2002).

The soil sample is first air dried and sieved (usually through the 4.75-mm (No.4) sieve or 19mm sieve), mixed thoroughly with water and then compacted in layers. The mass of the compacted sample is measured (W), and a small sample taken to measure the corresponding moisture content (w). More water is then added to the soil, and the procedure repeated until the dry density obtained decreases. Comparison of standard and modified compaction tests is given in the following Table 2.1;

Table 2-1: Standard and Modified compaction tests (SPD lecture note, 2011/12)

Items	Standard Compaction Test (AASHTO T99)	Modified (Heavy) Compaction Test (AASHTO T180)
Diameter of mould (mm)	101.6/152.4	101.6/152.4
Height of sample (mm)	117	117
Number of lifts (layers)	3	5
Number of blows per lift	25/56	25/56
Weight of Hammer	2.5kg	4.5kg
Diameter of end face of hammer (mm)	51	51
Free fall height (mm)	305	457
Net volume of mould (cm3)	944/2124	944/2124

Note: larger diameter mould (152.5mm) is used for gravelly soils (soils with a significant amount of gravel).

The moisture content at which maximum dry density is obtained is known as **optimum moisture content (OMC)**. At moisture content higher than the OMC, the air and water in the soil mass tend to keep particles apart and prevent compaction. The dry density at higher moisture contents than OMC, thus, decreases and the total voids increase.

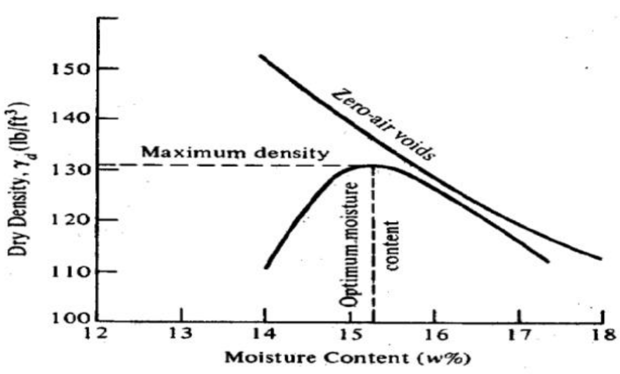


Figure 2-1:Typical compaction curve (Structural Pavement Design lecture note, 2011/12)

The bulk density of the soil for each trial is obtained by dividing the weight of the soil by the total volume $\gamma_b = \frac{W}{V}$. The dry density of the soil is determined by:

$$\gamma_d = \frac{\gamma_b}{[1 + w]}$$

2.2.2.4 Subgrade Soil Classification

The purpose of soil classification system is to group soils with similar properties or attributes. As a means of obtaining general behavior, soils are systematically categorized on the basis of some common characteristics obtained from visual description and laboratory tests. Various soil classification systems are in use throughout the world in different areas of study. The most commonly used classification systems for highway purposes are the American Association of State Highway and Transportation Officials (AASHTO) Classification System and the Unified Soil Classification System (USCS). These classification systems only help engineers to predict how the soil will behave if used as a sub-grade or sub-base material.

AASHTO Classification System

The AASHTO Classification System (ASTM D-3242, AASHTO M 145) was developed from the results of extensive research conducted by the Bureau of Public Roads, now known as the Federal Highway Administration of the United States. The system has been described by AASHTO as a means for determining the relative quality of soils for use in embankments, sub-grades, sub-bases, and bases. The system is based on the following three soil properties: (1) Particle-size distribution, (2) Liquid Limit, and (3) Plasticity Index (Murthy, V.N.S).

Either AASHTO T88, or ASTM D 1140 will be used to determine the particle size distribution of soils or soil-aggregate mixtures as a basis for classification.

Classification procedure: With the required data in mind (i.e. the results of Particle-size distribution, Liquid Limit, and Plasticity Index), proceed from left to right in the chart. A process of elimination will find the correct group. The first group from the left consistent with the test data is the correct classification. In this system of classification, soils are categorized into seven groups, A-1 through A-7, with several subgroups, as shown below. The A-7 group is subdivided into A-7-5 or *A-l-6* depending on the plasticity index, Ip.

For A-7-5, *Ip* < LL – 30 , ForA-7-6, Ip> LL-30.

Table 2-2:AASHTO Classification System (Murthy, V.N.S.)

General classification	Granular Materials (35 percent or less of total sample passing No. 200)							Silt-clay Materials (More than 35 percent of total sample passing No. 200)			
Group classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve analysis percent passing											
No. 10	50 max										
No. 40	30 max	50 max	51 min								
No. 200	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing No. 40											
Liquid limit				40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity Index	6 max		N.P.	10 max	10 max	11 min	11 max	10 max	10 max	11 min	11 min
Usual types of significant constituent materials	Stone fragments—gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General rating as subgrade	Excellent to good					Fair to poor					

Source: Murthy V.N.S: (Geotechnical Engineering book).

2.2.2.5 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 50mm diameter to penetrate a pavement component material at 1.25mm/minute. The loads for 2.54mm and 5.08mm were recorded. This load is expressed as a percentage of standard load value at a respective deformation level to obtain CBR value (SABA Manual, 2002).

The different layers such as subgrade, sub-base, and base course materials strength are classified according to the CBR values. In addition to the AASHTO Soil Classification tests, the 4 days soaked CBR tests are performed on the material samples to determine the highway construction material shear strengths. The CBR number is obtained as the ratio of the unit load (in KN/m2) required to effect a certain depth of penetration of the penetration piston in to a compacted specimen of material at some water content and density to the standard unit load. This is required to obtain the same depth of penetration on a standard sample of crushed stone. The resulting load-penetration curve is compared with that obtained for a standard crushed rock material, which is considered an excellent base course material (ERA, 2002).

Formula to calculate CBR $\Rightarrow$ $CBR\ (\%) = 100\ (\frac{x}{y})$

Where: x = material resistance or the unit load on the piston (pressure) for 2.54mm (0.1") or 5.08mm (0.2") of penetration

y = standard unit load (pressure) for well graded crushed stone

= for 2.54mm penetration = 6.9MPa (1000psi)

= for 5.08mm penetration = 10.3MPa (1500psi)

The results from the laboratory tests combined with the relevant pavement condition survey provide an evaluation of pavement performance to be made.

2.3 Types of Flexible Pavement Distress

Pavement distresses are those defects may visible on the pavement surface and those are possible to identify and evaluating or determining their severity and extents. They are symptoms, indicating some problem or phenomenon of pavement deterioration such as cracks, potholes, patches and ruts. It is necessary to have clear understanding of type of pavements distress before discussing the different methods of evaluation, checking of the pavement layers effect on distresses and treatment selection. The type and severity of distress a pavement has can provide great insight into what its future maintenance and/or rehabilitation needs will be. The distress is generally described in terms of severity, extent and distress type. However, the distress identification and measurement procedures may slightly vary from agency to agency (Luo, 2005).

The four major categories of common asphalt pavement surface distresses are cracking, surface deformation, disintegration, and surface defects. The most common types of cracking are Fatigue cracking, Longitudinal cracking, Transverse cracking, Block cracking, Slippage cracking, Edge cracking. Pavement surface 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. The basic types of surface deformation are Rutting, Corrugation, Shoving, Depressions. Disintegration is 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. Surface defects are related to problems in the surface layer. The most common types of surface distress are Raveling, Bleeding and Polishing (Gupta, et al., 1999).

In another way the American Standard for Testing of Materials (ASTM) grouped distresses identified during condition surveys in to three major categories of possible causes. These are load associated distresses, climate (durability) associated distresses and drainage (moisture) associated distresses. In load associated distresses, alligator cracking, edge cracking, potholes, rutting, slippage and cracking are grouped. Block cracking, weathering and raveling and shoving are grouped in climate/ durability associated distress. Finally in the drainage associated distress, lane/ shoulder drop off and depression are grouped (ASTM D6433, 1999).

2.3.1 Different Types of Cracks

2.3.1.1 Alligator or Fatigue Cracking

Alligator cracks are interconnected cracks forming a series of small blocks resembling an alligator skin. The lengths of the cracked pieces are usually less than 15cms on the longest side. In some cases, alligator cracking is caused by fatigue failure of the asphalt concrete surface under repeated traffic loading, by excessive deflection of the surface over unstable sub grade or lower courses of the structure. The unstable support is usually the result of saturation of the bases or subgrade. Although the affected areas in most cases are not large, occurring principally in traffic lanes, occasionally, will cover entire sections of pavements. Cracking begins at the bottom of the asphalt surface (or stabilized base) where tensile stress and strain are highest under a wheel load. The cracks propagate to the surface initially as a series of parallel longitudinal cracks. After repeated traffic loading, the cracks connect, forming many sided, sharp angled pieces that develop a pattern resembling chicken wire or the skin of an alligator (ASTM D6433, 2007).

There are three severity level of alligator cracks. These are low level, medium level and high level. Low level of intensities (L) are those having fine, longitudinal hairline cracks running parallel to each other with one or only a few interconnecting cracks. The cracks are not spalled. And average level of intensity (M) are those with further development of light alligator cracks into a pattern or network of cracks that may be lightly spalled. In higher level of intensity (H), network or pattern cracking has progressed so that the pieces are well defined and spalled at the edges. Some of the pieces may rock under traffic. Potholes of all sizes are recorded as high severity alligator cracking (ASTM D6433, 2007).

Alligator cracks are measured in square meter of surface area. The major difficulty in measuring this type of distress is that two or three levels of severity often exist within one distressed area. If these portions can be easily distinguished from each other, they should be measured and recorded separately, however, if the different levels of severity cannot be divided easily, the entire area should be rated at the highest severity level present (ASTM D6433, 2007).

The causes of alligator cracks can be damaged layer of asphalt concrete as a result of damage to the substrate due to repeated traffic loads, instability of the foundation layer of asphalt case or layer under the foundation because of the drop surface, double layer foundation stone, making it unable to land resulting from excessive loads of traffic, aging of asphalt materials by the time. Insufficient thickness of the pavement and poor drainage in the base layers and under the foundation (GTC, 1998).

2.3.1.2 Block cracking

Blocks are interconnected cracks that divide the pavement into approximately rectangular pieces. The blocks may range in size from approximately (3 by 3cm) to (3 by 3m). Block cracking usually indicates that the asphalt has hardened significantly. Block cracking normally occurs over a large proportion of pavement area, but sometimes will occur only in non-traffic areas. This type of distress differs from alligator cracking in that alligator cracks from smaller, many sided pieces with sharp angles. Also, unlike block cracks, alligator cracks are caused by repeated traffic loadings, and found only in traffic areas (wheel paths) (ASTM D6433, 2007).

The classification of the low severity level, network cracks must provide either non-filled cracks (non-filled) offer less than (10 mm). Or cracks filled with insulation any offer was in acceptable condition. In the medium level of intensity (M), the classification of moderate cracks network must provide either of three different cases. These are width of cracks more than 10 mm and less than 75 mm, cracks introduced less than or equal to 75 mm and surrounded by light random badly broken and cracks filled with any offer and is surrounded by light random badly broken. Higher level of intensity (H) is for the classification of high intensity of the cracks network that are either of three cases. These are any cracks filled or not filled with badly broken surrounded by random high or medium severity, showing unfilled cracks greater than 75mm and cracks introduced about 100 mm and surrounded by very badly broken and broken (ASTM D6433, 2007).

Block cracking is measured in square meter of surface area. It usually occurs at one severity level in a given pattern section; however, any areas of the pavement section having distinctly different levels of severity should be measured and recorded separately (ASTM D6433, 2007).

2.3.1.3 Longitudinal and Transverse Cracks

Longitudinal cracks are parallel to the pavement’s center-line or laydown direction. They may be adjacent to the pavement edge. Transverse cracking occurs predominantly perpendicular to the pavement centerline. It can occur anywhere within the lane (ASTM D6433, 2007).

Longitudinal and transverse cracks are measured in linear meter. The length and severity of each crack should be recorded after identification. If the crack does not have the same severity level along its entire length, each portion of the crack having a different severity level should be recorded separately. If a bump or sag occurs at a crack, it is also recorded as a distortion (ASTM D6433, 2007).

The possible cases of this type of crack are a poorly constructed paving lane joint, shrinkage of the surface due to low temperatures or hardening of the asphalt and daily temperatures cycling, a reflective crack caused by joints and cracks beneath the surface course and decreased support or thickness near the edge of the pavement (GTC, 1998).

2.3.1.4 Slippage Cracks (Sliding Cracks)

Slippage cracks are crescent- or half-moon shaped cracks having two ends pointed away from the direction of traffic. They are produced when braking or turning wheels cause the pavement surface to slide and deform. This usually occurs when there is a low-strength surface mix or poor bond between the surface and next layer of pavement structure (ASTM D6433, 2007).

In low level of intensity (l) of this kind of cracking the width of cracks must be less than 10 mm. In medium level of intensity (m) either average width of cracks must be between 11-40 mm or break the average in the surrounding area cracks happened to here and / or that the region is surrounded by secondary badly broken. In higher level of intensity (h) either the average width of cracks greater than 40 mm or the area around the cracks has broken into pieces easy removal (ASTM D6433, 2007).

Sliding cracks can be measured by the area affected cracks sliding surface box. Density is calculated by dividing the area affected by the defect on the total area of the section scanned.

2.3.1.5 Edge Cracking

Edge cracking is crack in the side is parallel to the edge of the pavement and away from a distance ranging between 0.3-0.5 meters from the edge, and extends these cracks longitudinal and transverse direction and branching towards the shoulders. And increasing the cracks as a result of side-load traffic is classified as the area enclosed between the part and the edge of pavement as volatile if there has been a break (ASTM D6433, 2007).

The low level of intensity (l) of this crack is a shallow surface cracks that does not cause breaks and loss of materials on the pavement. In medium level of intensity (m) moderate cracks are classified when they contain break and loss of materials in the length of up to 10% of the length of the paving of the area affected. The higher level of intensity (h) is a deep and many cracks and contains break and loss of materials in the length of more than 10% of the length of the paving of the area affected (ASTM D6433, 2007).

Surface cracks measured longitudinal profiles for each level of severity alone. Measured by the area affected by the defect length of the affected area multiplied by one meter, and the defect density is calculated by dividing the area affected by the total area of the section scanned multiplied by one hundred (ASTM D6433, 2007).

2.3.1.6 Shoulder/lane drop-off

Lane/shoulder drop-off is a difference in elevation between the pavement edge and the shoulder. This distress is caused by either shoulder erosion or settlement, or by building up the roadway (i.e., overlay) without correcting the shoulder height (ASTM D6433, 2007).

Severity Levels: L—The difference in elevation between the pavement edge and shoulder is > 25 mm and< 50 mm, M—The difference in elevation is > 50 mm and < 100 mm and H—The difference in elevation is > 100 mm and Lane/shoulder drop-off is measured in linear meters.

2.3.2 Pavement Deformation

2.3.2.1 Crawl or Shoving

Crawl or longitudinal movement of the offset is localized to the area of the surface of the road towards the traffic generated as a result of motor traffic loads, when the pavement layer drive traffic it generates a short, high waves on the surface layer of pavement. This occurs in the defect sites intersections (and slower acceleration) and before traffic signals where to stop and start of a movement or in areas adjoining cement concrete layer with a layer of asphalt floppy (ASTM D6433, 2007).

The low level of intensity (l) of this failure is the level that affects the quality of a simple command. The medium level of intensity (m) is the level that affects the average level of quality leadership. Higher level of intensity (h): is the level which severely affect the quality of leadership (ASTM D6433, 2007).

Surface creep is measured by the box of the affected area for each level of severity, but when it happens to crawl sites recorded as patching only. Density is calculated by dividing the area affected by the defect on the total area of the section scanned multiplied by one hundred (ASTM D6433, 2007).

This type of distress can be caused due to Shear stresses generated by the movement of vehicles on sites with steep slope or at the intersections of traffic signals, Poor stability pavement surface because of the increased proportion of asphalt or increase the proportion of the soft material in the mix or the use of rubble circular shape or Stability of weak layers under the foundation stone and the foundation is reflected on the surface of pavement (ASTM D6433, 2007).

2.3.2.2 Rutting

A rut is a surface depression in the wheel paths. Pavement uplift may occur along the sides of the rut, but in many instances, ruts are noticeable only after a rainfall, when the wheel paths are filled with water. Rutting stems from a permanent deformation in any of the pavement layers or sugared, usually caused by consolidated or lateral movement of the materials due to traffic loads. Significant rutting can lead to major structural failure of the pavement (ASTM D6433, 2007).

The Low level of intensity (L) of this distress has average depth for this level between 6 - 13 mm. In Medium level of intensity (M) the average depth of between 14-25 mm. In Higher level of intensity (H) the average depth of rutting at this level more than 25 mm

Rutting is measured in square meter of surface area. The rut depth is determined by laying a (3m) straight edge across the rut and measuring its depth (ASTM D6433, 2007).

Contribute to poor materials or poor design materials mixture in compression classes, in addition to inadequate father during implementation, the smoothness of asphalt mix, the softer substrate material as a result of water leakage or shock frames (Studded tires), pavement thickness are all causes of rutting(ASTM D6433, 2007).

2.3.2.3 Swell:

Swell is the localized upward displacement of a pavement due to the upheaval of the sub-grade or some portion of the pavement structure. Swell or heave is commonly caused by infiltration of moisture into an expansive-type soil (ASTM D6433, 2007).

2.3.2.4 Depression

Depressions are localized pavement surface areas with elevations slightly lower than those of the surrounding pavement. In many instances, light depressions are not noticeable until after a rain, when ponding water creates a “birdbath” area; on dry pavement, depressions can be spotted by looking for stains caused by ponding water. Depressions are created by settlement of the foundation soil or are a result of improper construction. Depressions cause some roughness, and when deep enough or filled with water, can cause hydroplaning (ASTM D6433, 2007).

In low level of intensity (l) it is noted that level depression in the areas of spots, and have a mild impact on the quality of leadership and can cause ups and downs of the car at high speeds. Ranges from a maximum depth of depression between 13 - 25 mm in the case of low-intensity. In medium level of intensity (m) it is noted that the defect easily at this level and moderately affect the quality of leadership, where a depression the rise and fall of a car at high speeds. Estimated depth of this level of intensity between 25 - 50 mm. Higher level of intensity (h) of depression can be seen easily and is severely affecting the quality of leadership, causing vibration and clear of the car at high speeds, and greater depth of the decline is more than 50 mm (ASTM D6433, 2007).

Depressions are measured in (square meters) of surface area. The maximum depth of the depression determines the level of severity. This depth can be measured by placing a (3-meters) straight edge across the depressed area and measuring the maximum depth in (millimeters). Depressions larger than (3 meters) across must be measured by either visual estimation or direct measurement when filled with water (ASTM D6433, 2007).

The possible causes can be either of three cases. These are depression occurs due to falling base layers or arise during construction, basis because of the drop, as a result of excess loads, which is pressing basis or because of the decline that occurs during the immediate implementation of the rate of movement of the upper lower classes. Inadequate compaction of hardcore sand and the inability of the substrate to withstand loads of reasons depression and traffic loads, temperature, materials, and disadvantages of implementation are all factors that contribute to the emergence depression and accelerate the deployment (GTC, 1998).

2.3.3 Disintegration

2.3.3.1 Potholes

Are usually Basin drilling diameters of about 750 mm, as have aspects of vertical near the top of the pit, which occur on the road surface and vary in depth and breadth. If there are cracks alligator drilling because of high intensity should be defined and not digging flying (Weathering). Potholes are bowl-shaped holes of various sizes occurring in the pavement surface (ASTM D6433, 1999).

The severity level of potholes can be described in Table 2.3.

Table 2-3:Severity levels of potholes (GTC, 1998)

Maximum depth (mm)	Median diameter (mm)		
	100-200	201-450	451-750
13-25	Low	Low	Medium
26-50	Low	Medium	Higher
More than 50	Medium	Medium`	Higher

The measurement of pot holes is made by taking the depth and the diameter of each potholes. If a hole more than (750) is scaled mm surface area and then divided by (0.5) half a meter box to find the equivalent number of craters, but if the depth of excavation is less than 25 mm are considered moderate, and high intensity in the case of depth of more than 25 mm.

Potholes may happen in either of different cases. These are; break the surface of the pavement as a result of alligator cracks, turn the place of the surface layer of paving, the presence of moisture and do accelerate the movement from the emergence of drilling.

2.3.4 Surface defects

2.3.4.1 Patching

A patch is an area of pavement which has been replaced with new material to repair the existing pavement. A patch is considered a defect no matter how well it is performing (a patched area or adjacent area usually does not perform as well as an original pavement section) (ASTM D6433, 2007).

The low level of intensity (l) of this distress is the level that affects the quality of a simple command and where the patching in good condition. In medium level of intensity (m), patch is moderately deteriorated and ride quality is rated as medium severity. Higher level of intensity (h) is the level where patch is badly deteriorated and ride quality is rated as high severity. Patch needs replacement (maintenance needs to be immediate) (ASTM D6433, 2007).

If a hole more than (750) is scaled mm Surface Area and then divided by (0.5) half a meter box to find the equivalent number of craters, but if the depth of excavation is less than 25 mm are considered moderate, and high intensity in the case of depth of more than 25 mm (ASTM D6433, 2007).

Possible causes of patching include a defect patching traffic loads, not controlling the quality of materials or poor implementation of re-filling and the poor operation of asphalt (ASTM D6433, 2007).

2.3.4.2 Raveling &Weathering

Weathering and raveling are the wearing away of the pavement surface due to a loss of asphalt or tar binder and dislodged aggregate particles. These distresses indicate either that the asphalt binder has hardened appreciably or that a poor-quality mixture is present. In addition, raveling may be caused by certain types of traffic, for example, tracked vehicles. Softening of the surface and dislodging of the aggregates due to oil spillage also are included under raveling (ASTM D6433, 2007).

In the low level of severity (l) of this distress, aggregate or binder has started to wear away. In some areas, the surface is starting to pit. In the case of oil spillage, the oil stain can be seen, but the surface is hard and cannot be penetrated with a coin. In moderate level of severity (m), aggregate or binder has worn away. The surface texture is moderately rough and pitted. In the case of oil spillage, the surface is soft and can be penetrated with a coin. In higher level of severity (h), aggregate or binder has been worn away considerably. The surface texture is very rough and severely pitted. The pitted areas are less than 10 mm (4 in.) In diameter and less than 13 mm (1⁄2 in.) Deep; pitted areas larger than this are counted as potholes. In the case of oil spillage, the asphalt binder has lost its binding effect and the aggregate has become loose (ASTM D6433, 2007).

Weathering and raveling are measured in square meters (square feet) of surface area (ASTM D6433, 2007).

2.4 Road Construction Materials

The materials used for construction of the flexible pavement road structure can be consisted of a prepared roadbed or subgrade material, underlying layers of sub-base, base, and surface course.

2.4.1 Subgrade Soils

2.4.1.1 General

The subgrade is the undermost layer of a pavement and it is the natural ground, graded and compacted, foundation, on which the vehicle load and the weight of the pavement layers finally rest and also as such is one the main concerns of a pavement design (Emer, T. Q 2016). “It is an in situ or a layer of selected material compacted to the desirable density near the optimum moisture content. It is graded into a proper shape, properly drained, and compacted to receive the pavement layers” (Yosef, 2012).

“The performance of a pavement depends on the quality of its subgrade materials. As the foundation for the pavement’s upper layers, the subgrade layer play a key role in mitigating the detrimental effects of climate and the static and dynamic stresses generated by traffic. Therefore, building a stable subgrade and a properly drained sub-base is vital for constructing an effective and long lasting pavement system” (ERA, 2002).

“And the design of the various pavement layers is very much dependent on the strength of the sub grade soil over which they are going to be laid. Sub grade strength is also depending on the basic properties of the soil. Although a pavement’s wearing course is important component of a road, the success or failure of a pavement is dependent upon the underlying subgrade material upon which the pavement structure is built. Thus, the subgrade must be able to support the loads transmitted from the pavement structure without undergoing excessive settlement" (ERA, 2002).

Subgrade strength is mostly expressed in terms of CBR (California Bearing Ratio). Weaker sub grade essentially requires thicker layers whereas stronger sub grade goes well with thinner pavement layers. The pavement and the sub grade mutually must sustain the traffic volume. The Ethiopian Road Authority scaling the exact design strategies of the pavement layers based upon the sub grade strength. The sub grade is always subjected to change in its moisture content due to rainfall, capillary action, overflow, or rise of water table (ERA, 2002).

The routine tests normally carried out on sub-grade soils include:

- ✓ *Soil classification and index tests*: gradation, Atterberg limits, moisture content;
- ✓ *Compaction and Strength tests*: compaction test (standard or modified), CBR test
- ✓ *Field tests (on existing road)*: field density and moisture content, DCP test

While conducting the soil testing, reference is frequently made to the standard methods of testing such as ERA Standard, British Standard (BS), American Society for Testing and Materials (ASTM), and American Association of State Highway and Transportation Officials (AASHTO). It should be noted that when actually performing tests it is of the utmost importance that the specified standards be followed precisely, as small differences in the testing procedure may have a noticeable influence on the test result obtained.

Its performance generally depends on its load bearing capacity, moisture content and volume changes. Moreover, its load bearing capacity depends on the degree of compaction, moisture content and soil type. Hence, the relationships among the strength, density and moisture content.

2.4.4.2 ERA Standard Requirements for Suitable Subgrade Material:

a) Californian Bearing Ratio (CBR)

The fill material or subgrade shall have a minimum soaked Californian Bearing Ratio (CBR) of not less than 4% and a swell value of not more than 1.5% (with two surcharge rings) when determined in accordance with AASHTO T-193.

The Californian Bearing Ratio (CBR) shall be determined at a density of 95% of the maximum dry density determined in accordance with the requirements of AASHTO T-180 method D.

b) Liquid Limit and Plasticity Index

The fill material shall have a liquid limit not exceeding 60% and a plasticity index not exceeding 30 when determined in accordance with the requirements of AASHTO T-89 and T-90.

Unsuitable Material consists of:

- ✓ Clay material having a Liquid Limit (LL) exceeding 60; or a Plasticity Index (PI) exceeding 30; or CBR value less than 3% at 95% of modified AASHTO compaction (AASHTO method T-180) after 4 days soaking; or a swell value of more than 3% (with two surcharge rings) when determined in accordance with AASHTO T-193 at 95% of modified AASHTO compaction.

2.4.2 Granular Pavement Materials

2.4.2.1 General

Granular or unbound pavement material is one of the most important components of a flexible pavement structure that is for use as base course, sub-base, capping, and selected subgrade layers. 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 (ERA, 2002).

2.4.2.2 Properties of Unbound Pavement Materials

Granular materials are generally used in road pavements as base, sub-base and as part of asphalt concrete courses, which includes crushed rock aggregates obtained from hard rock sources, natural gravels, gravel-sand-soil mixtures either as dug or semi-processed. Which mean that screening, crushing of oversized stones, mixing with other materials (mechanical stabilization) and other artificial or modified materials and which are as important a component of roads as the surface composition and foundations. As a base course, they play a structurally important role, or subjected to severe loading, especially on medium and low volume roads, stable, resistance to abrasion, resistance to penetration of water, Capillary properties to replace moisture lost by surface evaporation upon the addition of wearing course requirement change. As a sub-base, they protect the soil, and act as a working platform and an insulating layer against frost action (Structural Pavement Design lecture note, 2011/12) (ERA, 2002).

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 2.3.

Table 2-4: Properties of unbound materials

Code	Description	Summary of Specification
GB1	Fresh, crushed rock	Dense graded, un weathered crushed Stone, non-plastic parent fines
GB 2	Crushed weathered rock, gravel or Boulders	Dense grading, PI<6, soil or parent Fines; PP<60
GB 3	Natural coarsely graded granular material, Including processed and modified gravels	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%

Source: ERA Manual, 2002

Notes:

a. These specifications are sometimes modified according to site conditions, material type and principal use.
b. GB = Granular base course, PP = Plasticity Product = PI x (percent passing 0.075mm sieve), GS = Granular sub-base, GC = Granular capping Layer.

Different standard methods of design specify materials of construction differently considering the traffic load, locally available materials, and environmental conditions. The following describes the requirements set for different unbound pavement materials for base and subbase courses as specified in ERA pavement design manual (2002).

2.4.2.3 ERA Standard Material Requirements for Sub-Base:

Sub-base Course Materials

The sub-base is an important load spreading layer which enables traffic stresses to be reduced to acceptable levels on the sub-grade. The selection of sub-base materials depends on the design function of the layer and the anticipated moisture regime, both in service and at construction. The quality of sub-base is very important for the useful life of the road and can outlive the life of the surface, which can be scrapped off and after checking that the sub-base is still in good condition, a new layer can be applied. According to the ERA Pavement Design Manual the requirements to use as a sub-base material is discussed below a bearing capacity.

Gravel material to be used for sub-base shall be obtained from approved sources in borrow areas, cuttings or existing pavement layers and shall conform to requirements specified herein.

a. Plasticity Index

All sub-base materials shall have a maximum Plasticity Index of 6 or 12, and when determined in accordance with AASHTO T-90.

b. Particle Size Distribution and Shape

The sub-base material shall comply with one of the gradings shown in Table 2.8. It shall be well-graded with a smooth continuous grading within the limits shown. The material shall have a smooth continuous grading within the limits for grading A, B or C given below.

Tests to determine whether the material complies with the specified grading requirements shall be conducted after the material has been mixed and spread on the road.

Table 2-5: Typical Particle Size Distribution for Sub-bases (GS)

Test Sieve (mm)	**Percentage by mass of total aggregate passing test sieve (%)**			
	A	**B**	**C**	**D**
63	100	-		-
50	90-100	100	100	-
37.5			80-100	
25	51-80	55-85		100
20			60-100	
9.5	-	40-70		51-85
5			30-100	
4.75	35-70	30-60		35-65
2.0	-	20-51		25-51
1.18			17-75	
0.425	-	10-30		15-30
0.3			9-50	
0.075	5-15	5-25	5-25	5-15

Source: ERA Manual, 2002

The complete sub-base shall contain no material having a maximum dimension exceeding two-thirds of the compacted layer thickness.

c. Californian Bearing Ratio (CBR)

The minimum soaked Californian Bearing Ratio (CBR) shall be 30% when determined in accordance with the requirements of AASHTO T-193. The Californian Bearing Ratio (CBR) shall be determined at a density of 95% of the maximum dry density when determined in accordance with the requirements of AASHTO T-180 method D.

d. Compaction requirements

The minimum in- situ dry density of sub-base material shall be as specified hereinafter for the layers in terms of a percentage of modified AASHTO density. (i) 95% or 97% as required for material not chemically stabilized. (ii) 95% or 96 % as required for chemically stabilized material.

2.4.3.4 ERA Standard Material Requirements for Base:

Base Course Materials (Graded crushed aggregate (GB1)):

This material is produced by crushing fresh, quarried rock 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. The rock used for crushed aggregates should be hard and durable. Laboratory and field experiences have shown that crushed particles have, in general, more stability than rounded materials due to primarily to added grain interlock. In addition, crushed materials possess high coefficient of permeability. In constructing a crushed stone base course, the aim should be to achieve maximum impermeability compatible with good compaction and high stability under traffic (Structural Pavement Design lecture note, 2011/12).

a. Plasticity Index

The fine fraction of a GB1 material shall be non-plastic or shall have a maximum Plasticity Index of 6 when determined in accordance with AASHTO T-90.

b. Grading

The combined grading of the material shall be a smooth continuous curve falling within the grading limits shown in Table 2.5 when determined in accordance with the requirements of AASHTO T-27. The mass of material passing the 0.075mm sieve shall be determined in accordance with the requirements of AASHTO T-11.

Table 2-6: Grading limits for graded crushed stone base course materials

Test sieve (mm)	**Percentage by mass of total aggregate passing test sieve**		
	Nominal maximum particle size		
	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	5-12	5-12	5-12

Source: ERA Manual, 2002

c. Californian Bearing Ratio (CBR)

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 200 mm. Crushed stone base courses constructed with proper care with GB1 materials described above should have CBR values in excess of 100 per cent.

2.5 Pavement Management Systems (PMS)

Pavement management system (PMS) is described as: “a set of tools or methods that can assist decision makers in finding cost-effective strategies for providing, evaluating, and maintaining pavement in a serviceable condition” (AASHTO, 1990). Based on the above description PMS can address three questions. These are; ‘What maintenance and rehabilitation (M&R) strategies should be the most cost effective?’, ‘Where (which pavement segments) are M&R treatments required? ‘and ‘When would be the most suitable time (condition) to plan a treatment?’ (Ann.et al., 2000).

2.6 Assessment of Pavement Condition

Pavement condition assessment includes collecting and analyzing pavement performance data (i.e., cracking, rutting, faulting, structural capacity, surface characteristics) for determining individual or overall indicators of pavement condition. The inspection method is designed to allow the calculation of a composite rating index called the pavement condition index. The PCI scale is shown in (Table 2.9). The distress types, severity levels, and methods of estimating quantities are keyed to the deduct curves presented in the area **(**Cline et al, 2002).

The key to a useful evaluation is identifying different types of pavement distress and linking them to a cause. Understanding the cause for current conditions is important in selecting an appropriate maintenance or rehabilitation technique (Walker, 2002).

2.6.1 Pavement Condition Index (PCI)

The detailed field inspections categorize and quantify the pavement distresses and deterioration that are mentioned above section. These deficiencies are entered into the PMS program that calculates a Pavement Condition index (PCI) for each road section. PCI values range from zero (very poor) to 100 (excellent) (Weil, 2009).

2.6.2 Pavement Condition Rating

The pavement condition rating is a description of pavement condition through rating scale ranges as a function of the PCI value that varies from 100 (excellent condition) to 0 (failed) as shown in Table 2.9. Most pavements will deteriorate through the phases listed in the rating scale. The time it takes to go from excellent condition (100) to complete failure (0) depends largely on the quality of the original construction and the amount of heavy traffic loading (Donald, 2013).

Table 2-7: Pavement condition ratings and pavement condition index ranges

Pavement condition rating	**Pavement condition index**
Excellent	86-100
Very good	71-85
Good	56-70
Fair	41-55
Poor	26-40
Very poor	11-25
Failed	0-11

Source: (Donald W, 2013)

The PCI is a quick method of comparing the overall condition of pavement and magnitude of rehabilitation needs. The following figure shows how pavement condition typically deteriorates over time. The new pavement holds its good condition for a long period, but once it begins to fail; its condition drops rapidly (Weil, 2009).

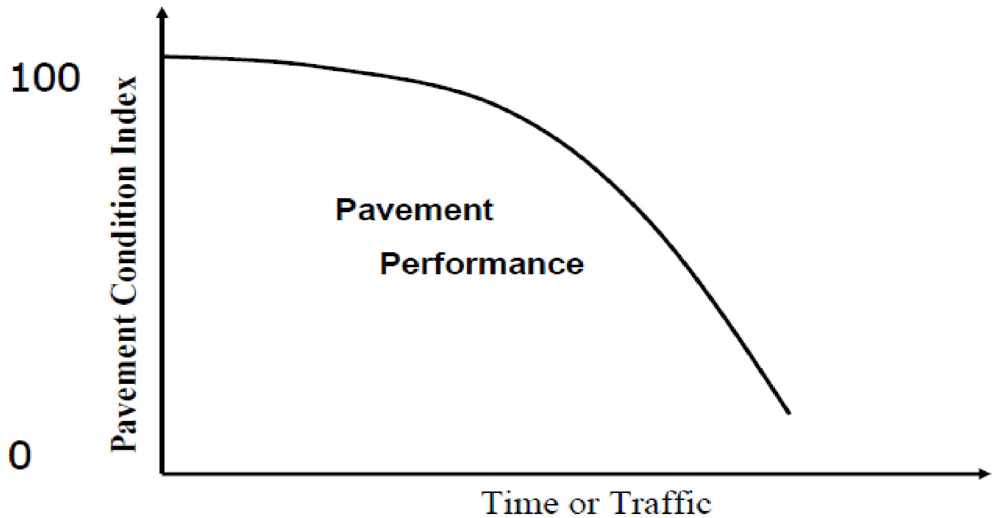


Figure 2-2: Relationship between pavement condition and time (Donald, 2013).

2.6.3 Definition of Pavement Condition

A. **Excellent:** Pavement is new construction. Nothing would improve the roadway at this time (Bashir, 2006).

B. **Very Good:** Pavement structure is stable, with no cracking, no patching, and no deformation evident. Roadways in this category are usually fairly new. Riding qualities are excellent. Nothing would improve the roadway at this time.

C. **Good:** Pavement structure is stable, little cracking and no deformation evident. Little maintenance would improve the roadway at this time.

D. **Fair:** Pavement structure is generally stable with minor areas of structural weakness evident. Cracking is easier to detect. The pavement may be patched but not excessively. Although riding qualities are good, deformation is more pronounced and easily noticed.

E. **Poor:** Areas of instability, marked evidence of structural deficiency, large crack patterns (alligator) heavy and numerous patches, deformation very noticeable. Riding qualities are range from acceptable to poor.

F. **Very Poor:** Pavement is in extremely deteriorated condition. Numerous areas of instability, Majority of section is showing structural deficiency. Riding quality is unacceptable (probably should slow down).

G. **Failed:** Pavement structure is failed, with cracking and deformation evident. Roadways in this category are usually failed and reconstruction at this time.

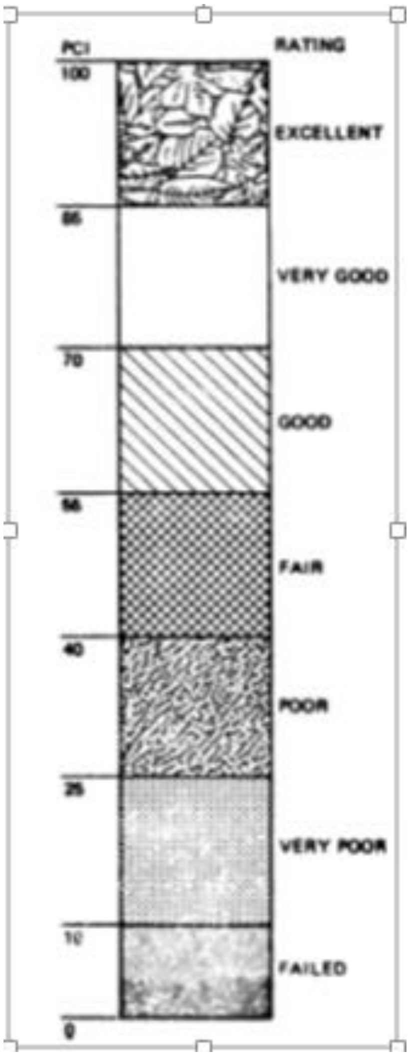


Figure 2-3: Pavement Condition Index (PCI) and Rating Scale (ASTM D 6433, 1999)

In addition to indicating the surface condition of a road, a given rating also includes a recommendation for needed maintenance or repair. This feature of the rating system facilitates its use and enhances its value as a tool in ongoing road maintenance (Donald, 2013).

2.7 Maintenance and Rehabilitation Treatment

Road pavement maintenance is one of the important components of the entire road system and is work performed from time to time to keep a pavement, under normal conditions of traffic and forces of nature, as nearly as possible in its as-constructed condition. The maintenance operations involve the assessment of road condition, diagnosis of the problem and adopting the most appropriate maintenance steps. Even if the highway is well designed and constructed, they require maintenance, the extent of which will depend on several factors such as the traffic system to which the pavement is subjected; Climate; the structure of the pavement; the quality of construction; the frequency and extent of inspection performed, both during construction and during maintenance; engineering talent involved, maintenance practices; discipline; and money. In order to carry out design of pavement rehabilitation, the existing pavement condition must be evaluated. Such an evaluation usually involves the assessment of the existing pavement surface distress, roughness, rutting. Pavement condition index (PCI) used as a parameter to track pavement distress over time and apply maintenance (Flaherty, 1991).

2.7.1 Maintenance Programs

Pavement maintenance programmers are required to perform the task of preserving, repairing and restoring different damaged elements of a pavement system to its designed or accepted standard. In general, maintenance of pavements is similar in concept to maintenance of one's home or car. If one doesn't want to repair his leaking roof, it will be further damaged and cause destruction in the house. If one does not want to timely change oil filter, he will certainly pay for the engine in a short time. The same is true to pavement maintenance. The longer one waits to maintain a pavement, the more it will cost to repair (ERA, 2002).

There is no uniform terminology with regard to definitions concerning pavement maintenances and rehabilitation. It varies from country to country and even from authority to authority as well. Maintenance programs can be categorized into routine, periodic and extraordinary maintenances. Each can be discussed in a little more detail as follows (ERA, 2002):

a) Routine Maintenance

A routing maintenance program comprises different activities that are to be carried out as frequently as required in order to ensure serviceability at all times. This kind of maintenance includes three major activities. These are; Clearing roadway pavement, ditches, drains, signs, and safety barriers, etc., as well as grass cutting and tree pruning, Repair of minor damage to pavement, drainage system as well as any urgent repairs to restore disrupted traffic movement and maintenances during rainy season such as provision of turn out ditch form storm water, clearing of mud and debris etc.

b) Periodic Maintenance

A periodic maintenance includes operations to be carried out under a long term program within the design period of the pavement. These operations can be divided into two main groups. These are renovation of the wearing surface of the existing pavement that become worn or damaged; e.g. resealing or surface dressing of existing asphalt road and restoration of drainage systems, road markings and ancillary items.

c) Extraordinary Maintenance

Extraordinary maintenance consists of activities necessary to restore highly distressed pavements to their original design requirement. The tasks include strengthening and or/reconstruction of a pavement structure which has severely deteriorated (e.g. overlays) and activities to protect roads against external agents (such as slope stabilization, retaining structures & flood control measures).

Maintenance Priorities

It is also very important to allocate the limited resources available for the maintenance purpose in such a way that it satisfies objectives and maintenance polices of the roads authority. The basic approaches to determine priorities for pavement maintenance are Urgent maintenance- such as emergency repairs to pavements that are cut, removal of debris and other foreign objects, Routine drainage maintenance; ditch cleaning and deepening, cleaning bridges and culverts, backfilling scoured areas, constructing check dams and etc., Routine maintenance of pavement- such as patching, sealing and repairing of road furniture and Periodic maintenance- such as resurfacing (Flaherty, 1991).

As indicated above the routine drainage maintenance should get more priority than the routine maintenance on pavements as repairing pavement surface defect caused by drainage problem is wastage of resource unless the drainage is first corrected (Flaherty, 1991).

2.7.2 Types of Maintenance Treatments Technique

Crack repair with sealing: A localized treatment method used to prevent water and debris from entering a crack, which might include routing to clean the entire crack and to create a reservoir to hold the sealant. It is only effective for a few years and must be repeated. However, this treatment is very effective at prolonging the pavement life (Ann. et al., 2000).

Crack filling: Differs from crack sealing mainly in the preparation given to the crack prior to treatment and the type of sealant used. Crack filling is most often reserved for more worn pavements with wider, more random cracking (Ann. et al., 2000).

Full-depth crack repair: A localized treatment method to repair cracks that are too deteriorated to benefit from sealing. Secondary cracking requires the reestablishment of the underlying base materials. Involves milling a trench centered over an existing crack, placing hot-mix asphalt (HMA) into the reservoir in one or more lifts, and compacting to achieve density (Ann. et al., 2000).

Seal coat: A seal coat is an application of asphalt followed immediately with an aggregate cover. Applications with two layers are referred to as a double chip seal. Rapid-setting asphalt emulsions are normally used when placing a seal coat. Seal coats can waterproof the surface, provide low-severity crack sealing, and restore surface friction. Used to waterproof the surface, seal small cracks, reduce oxidation of the pavement surface, and improve friction (Ann. et al., 2000).

Double chip seal: This treatment involves the application of two single seal coats. The second coat is placed immediately after and directly over the first. Sixty percent of the total asphalt binder required is placed in the first pass, with larger aggregate. The remaining forty percent is placed in the second pass, with aggregates half as large as those placed first (Ann. et al., 2000).

Slurry seal: A slurry seal is a mixture of fine aggregate, asphalt emulsion, water, and mineral filler. The mineral filler most often used is Portland cement. Slurry seals are used to seal the existing asphalt pavement surface, slow surface raveling, seal small cracks, and improve surface friction. Slurry seals are similar to chip seals in that they use a thermal break process, requiring heat from the sun and pavement. This process takes anywhere from two to eight hours depending on the heat and humidity (Ann. et al., 2000).

Thin hot-mix overlays: Thin hot-mix asphalt (HMA) overlays are blends of aggregate and asphalt cement. Three types of HMAs (dense-graded, open-graded friction courses, and gap-graded) have been used in the United States to improve the functional (non-structural) condition of the pavement. Thicknesses typically range from 3/4 to 1-1/2 inch. These mixes are often modified with polymers to meet high performance expectations (Ann. et al., 2000).

Pothole patching: Includes using cold- and hot-asphalt mixture, spray injection methods, as well as slurry and micro-surfacing materials, to repair distress and improve ride quality (Ann. et al., 2000).

2.7.3 Maintenance Option for Different Types of Pavement Distresses

Maintenance is an essential practice in providing for the long-term performance and the esthetic appearance of an asphalt pavement. The purpose of pavement maintenance is to correct deficiencies that caused by distresses and to protect the pavement from further damage. Various degrees or levels of maintenance can be applied to all pavements, regardless of the end user (Lavin, 2003).

Maintenance suggestion for different rate scaled distresses which categories into four common asphalt pavement surface distress (Lavin, 2003):

1) Surface defects
   - ✓ Raveling, polishing
2) Surface deformation or distortion
   - ✓ Rutting and shoving
3) Cracks
   - ✓ Alligator cracks, slippage, edge Cracks, and block.
4) Disintegration
   - ✓ Patches and potholes

2.7.3.1 Maintenance of Cracking Type Distress

Maintenance methods for cracking type distresses vary depending on the type of crack, which in turn is an indicator of the cause.

1. Alligator Cracks Maintenance

The maintenance should include removing the wet material and installing appropriate drainage. Full depth asphalt patching is necessary for having a strong and dependable repair. When necessary, temporary repairs can also be made by applying skin patches or aggregate seal coats to the affected areas to avoid any further damage to the pavement (Asphalt Institute, 1983).

2. Edge Cracks

Poor drainage will aggravate edge cracking by causing settlement or yielding of the material underlying the cracked area. Hence improving drainage by installing under drain is necessary, in those areas where drainage facilities do not exist, before repairing the cracked surface.

3. **Slippage Cracks**

The proper method of repairing a slippage crack is by removing the damaged surface layer and patching the area with plant mixed asphalt material after applying a light tack coat.

2.7.3.2 Maintenance of Distortion Type Distresses

Distortion types distresses are characterized by a change of the pavement surface form its original shape. Lack of proper compaction, excessive fines in the mix, too much asphalt, swelling of underlying courses, or settlements are major causes of distortion. Distortion takes a number of different forms: grooves or ruts, showing corrugations, depression, and upheaval.

The type of distortion and its cause must be determined before the correct remedy can be applied.

a. **Ruts**

It is necessary to repair such distresses by applying light tack coat and spreading asphalt concrete in the channel since rutting can be aggravated and lead to major structural failures and hydroplaning.

b. **Corrugations and Shoving**

Corrugations in a thick asphalt surface can be repaired using a pavement planning machine and Shoved areas can be repaired using deep patch like for the alligator cracking.

c. **Depressions**

Two or more layers of asphalt are required in the repair of deep depression. Filling the area by following the contour of depression is mostly mistakenly done. The correct way to repair a deep depression is to begin in the deepest part of the depression and place a thin layer, the surface of which, when compacted, will be parallel to the original pavement surface. Successive layers can be placed in the same manner.

2.7.3.3 Maintenance of Disintegrating Type Distresses

Disintegration type distresses are characterized by the breaking up of a pavement into small loose fragments. This includes the dislodging of aggregate particles. If not repaired at its early stage, it can progress until the pavement requires complete reconstruction.

The two common types of early stage disintegration are potholing and raveling. Repair ranges from simple seals to deep patches.

1. Potholes

For best results, all materials for filling potholes must meet appropriate and approved standards. Proper preparation and backfilling are very important. This can be done using asphalt cutter, jackhammer, chisel and other hand tools. The sides of cut surface have to be vertical and base. The base materials should be replaced with equal or better material than that removed or with bituminous material. The hole should be primed before placing and compaction of the bituminous material. It is advisable to overfill the bituminous material by around 40% of the pavement thickness to allow for compaction. (Technical Manual, 1995).

2. Raveling

Raveling surfaces usually require a surface treatment that can be looked upon as corrective or preventive maintenance. The type of surface treatment can be selected depending on the extent of damage and nature of traffic.

CHAPTER THREE

RESEARCH METHODOLOGY

3.1 Location of the Study Area

The study was conducted in southwestern part of Ethiopia and it connects the capital Addis Ababa with Southern Nations, Nationalities and Peoples Regional State, SNNPRS. Two zones within the state, namely Kambata-Alaba-Tembaro, and Wolaita are connected by the road. It starts 4.5km away from the Alaba town (Alaba town is located 309km from Addis Ababa and can be reached by branching off to the right at Shashmene town from the main Addis - Awassa Road). The exact study location start is the Bilate River Bridge and terminates in Sodo Town. The total length of the route is 68km. The geographic positions of the total road falls between 7° 18' N latitude to 38° 05' E longitude and 6° 02' N latitude 37° 33' E longitude. The location map of the study road is shown below (Engineering Design Report, 2006):

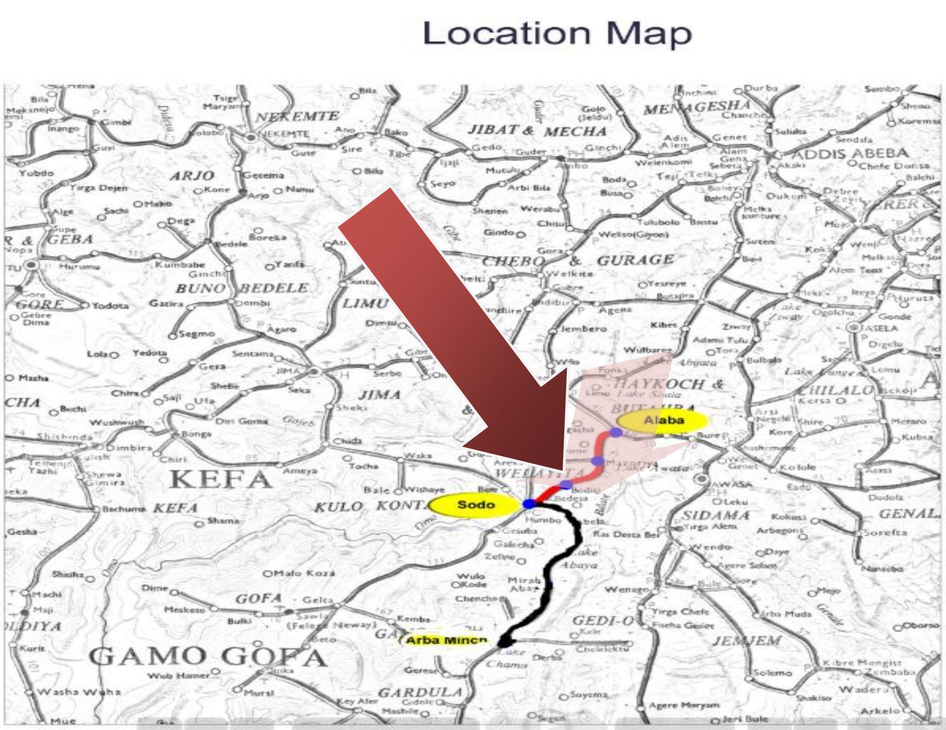


Figure 3-1: The Alaba – Sodo road stretch (Engineering Report, 2006)

3.2 Study Design

The research had two types of study designs. The first was Survey type of Descriptive study. In this frame, both quantitative and qualitative data types were collected and analyzed used to describe the condition of the road and to measure the level of pavement failures quantitatively under sections studied. The qualitative data that were used to describe the condition of the road are those data were collected in the preliminary visit stage of the data collection. The quantitative data were those that were collected in filed measurement or pavement condition survey stage.

The second type of the study design was comparative (case-control) study, in which the researcher investigated the case of the distresses on the road section in relation to the material qualities of each pavement layers of the road sections. As discussed in Section 3.4.4, different experiments were conducted in the laboratory on the materials taken from locations of the distressed points and none distressed point to comparison and to check or to find the case of the distresses in relation to pavement material quality in light of ERA material specification.

- Qualitative
  - Good or bad condition of the road (during preliminary visit)
  - Data for type of distress identification
- Quantitative
  - Measuring data; severity level and extent of distress
  - Material quality data's (from lab results)
  - Laboratory experiment: to check the quality of materials in the road

3.3 Population

The pavement distress and pavement layer materials existing within the range of study area which covers distance of 68 km from Alaba-Sodo along the main roads were considered as the population for this research.

3.4 Sample size and Sampling Technique

3.4.1 Sample size and Sampling Technique for PCI

3.4.1.1 Selection of the Sample Site for PCI

In order to achieve the objectives of this research, primarily the pavement was divided into sections that were divided into sample units due to preliminary site visit with visual observations of a pavement section for study. According to ASTM D6433, each section was divided into sample units with homogenous or equal length of 200m. The type and severity of pavement distress was assessed by visual inspection of the pavement sample units. The quantity of the distress was measured as described in Appendix A. The distress data were used to calculate the PCI for each sample unit. The PCI of the pavement section was determined based on the PCI of the inspected sample units within the section. Therefore, road was sectioned in five different sections as shown in Table 3.1. Out of the five sections, the three sections (Alaba-Mazoria, Shone-Buge and Buge-Boditi) were selected for detail study by considering the following major criteria's:

- ✓ Because of preliminary site visit, the two selected sections were identified as more distressed than other sections and the one was selected for comparison.
- ✓ By consideration of current pavement condition, travel distance and available time, resource, money, and manpower, the decision was made to conduct condition survey only within the three sections of road stretch. These selected sections also having greater length than others, and so that more representing the whole road than others

The number of these divided sample units or homogeneous test blocks of the all divide up sections were shown in Table 3.2 with value 200m length according to ASTM D6433 Manual.

Table 3-1: Study road sections with pavement condition survey need during preliminary site visiting

No. of section	Station (km)	Start town/village	End town/village (length)	Pavement condition survey
1	0+000– 20+000	Alaba	Mazoria=20km	Need detail survey
2	20+000– 6+100	Mazoria	Shone=6.1km	Need detail survey
3	26+100-33+400	Shone	Buge=7.3km	Need detail survey
4	33+400-50+400	Buge	Boditi =17km	Relatively good condition than others
5	50+400-68+000	Boditi	Sodo =17.6km	Relatively good condition than others

Table 3-2:The number of sample units in each divided sections

No. of section	Station (km)	Start-End (length)	Total number of sample units	Remark
1	0+000 –20+000	Alaba-Mazoria =20km	20,000/200 =100	Have been surveyed
2	20+000– 26+100	Mazoria-Shone =6.1km	6,100/200 =30.5	Not selected for survey
3	26+100– 33+400	Shone-Buge =7.3km	7,300/200 =36.5	Have been surveyed
4	33+400– 50+400	Buge-Boditi =17km	17,000/200 =85	Have been surveyed
5	50+400– 68+000	Boditi-Sodo =17.6km	17,600/200= 88	Not selected for survey

Note: The total number of sample units of the pavement sections were used to determine the minimum number of sample units (n) that were surveyed within given sections of the pavement selected for inspection and which were also inspected to determine the average PCI of the sections.

3.4.1.2 Sampling Technique for PCI

According to ASTM D6433 Manual, this study have been used a systematic random sampling techniques were to determine the number of sample or sample units of the pavement sections selected for inspection and which were also inspected to determine the average PCI of the sections. The minimum number of sample units (n) that were surveyed within given sections to obtain a statistically adequate estimate (95 % confidence) of the PCI of the sections were calculated using the following formula and rounding n to the next highest whole number (see Equ.1).

$$n = \frac{Ns^2}{\left(\left(\frac{e^2}{4}\right)(N-1) + s^2\right)} \text{ ....Equ.1}$$

where:

e = acceptable error in estimating the section PCI; commonly, e = ±5 PCI points;

s = standard deviation of the PCI from one sample unit to another within the section. When performed the initial inspection the standard deviation was assumed to be ten for AC pavements. This assumption should been checked as described below after PCI values were determined. For subsequent inspections, the standard deviation from the preceding inspection should been used to determine n; and,

N = total number of sample units in the section.

- ✓ If obtaining the 95 % confidence level is critical, the adequacy of the number of sample units surveyed must be confirmed. The number of sample units was estimated based on an assumed standard deviation. Calculate the actual standard deviation (s) as follows (see Equ.2):

$$s = \left(\sum_{i=1}^{n}(PCI_i - PCI_s)^2/(n-1)\right)^{1/2} \text{ ....Equ.2}$$

where:

PCI_i = PCI of surveyed sample units i,

$PCI_s s$ = PCI of section (mean PCI of surveyed sample units), and

n = total number of sample units surveyed.

- ✓ Calculate the revised minimum number of sample units (Equ.1) to be surveyed by using the calculated standard deviation (Equ.2). If the revised number of sample units to be surveyed is greater than the number of sample units already surveyed, select and survey additional random sample units. These sample units should be spaced evenly across the section. Repeat the process of checking the revised number of sample units and surveying additional random sample units until the total number of sample units surveyed equals or exceeds the minimum required sample units (n) in Equ.1, using the actual total sample standard deviation.

- ✓ Once the number of sample units to be inspected has been determined, compute the spacing interval of the units using systematic random sampling. Therefore, the samples within different sections were spaced equally throughout the sections with the first sample selected at random.

The spacing interval (i) of the units to be sampled was calculated by the following formula rounded to the next lowest whole number:

$$i = \frac{N}{n} \qquad \text{.......Equ.3}$$

where: N = total number of sample units in the section, and

n = number of sample units to be inspected.

The first sample unit to be inspected was selected at random from sample units 1 through i. The sample units within the sections that were successive increases of the interval i after the first randomly selected unit also were inspected.

Therefore, the next steps such as the determination of PCI value for each sample unit and average PCI value for each section were continued after determinations of sample units of each section.

3.4.2 Sample size and Sampling technique for Laboratory test

3.4.2.1 Selection of the Sample Site and Sampling technique for Lab tests

The engineering properties of materials are determined by carrying out different tests on samples of materials in a laboratory. The site selections for collecting samples for laboratory tests were performed based on pavement condition index (PCI) values or PCR. Those location which were taken as very poor, poor, and no distressed pavement condition and the distressed location taken as with dominant alligator cracking and rutting types of distresses. Which were also taken from four different sections. Two samples from Alaba-Mazoria (those were from poor and very poor surface condition, from stations 7+600 - 7+800 km and 13+600 - 13+800km). One sample from Buge-

Boditi section (this was from poor surface condition, from station 44+200 - 44+400 km), and one also from Boditi-Sodo section (this was from no distressed surface condition, from station 67+926km) to compare the results and check the effect of pavement layers properties on distresses.

For the indicative evaluation of distressed pavement sections at this road pavement level, it occasionally becomes necessary to conduct destructive testing by removing portions of the pavement to identify the layers where problems are occurring and why. This also gives an opportunity to collect material samples from different pavement layers and subgrade. Subsequently, the pavement is repaired or replaced. Destructive testing techniques include pit excavation from different layers for different tests such as CBR, particle size distribution, proctor, soil classification and atterberge limit testing. The process also involves removing samples of the various layers, examining the samples in the field by measuring thickness, and then testing them in the laboratory by using the sampling technique as per ASTM C702: Mechanical splitter method was used in this study. Only a limited number of samples are taken from field and tested in the laboratory because of the time and cost involved in the destructive testing.

Figure 3-2: Sample splitting in laboratory for tests

3.4.3 Pavement Condition Survey Procedures

The PCI provides an objective and rational basis for determining maintenance and repair needs and priorities. Continuous monitoring of the PCI is used to establish the rate of pavement deterioration, which permits early identification of major rehabilitation needs. The Pavement Condition Index (PCI) is determined by measuring pavement distress with a numerical indicator based on a scale of 0 to 100. The following pavement condition ranking is given to PCI values: 0-10 (failed); 11-25 (very poor); 26-40 (poor); 41-55 (fair); 56-70 (good); 71-85 (very good); 86 - 100 (excellent) (Seiler, 2009) as cited by (Ali-Mohamed, 2011). For each distress measured, there are deduct values depending upon the nature of the distress, its severity and quantity. The deduct values are summed, adjusted to take into account the total number of distresses identified, and then subtracted from 100 to give the PCI index for the pavement (ASTM D6433, 2007).

The condition survey procedure offers a method for identifying pavement distress types and defining the levels of severity and extent associated with each distress. The pavement condition survey was made using commonly used recording formats (see Table 3.2) and guidelines for determining pavement condition that involves observing and recording the presence of specific types and severities of defects or distresses on the pavement surface.

The procedure followed for visual inspection of the road pavement:

- ✓ Record distresses walking along the test road sections.
- ✓ Measure distresses using proper parameters such as fatigue cracking(sq.mt), bleeding (sq.mt), corrugation (sq.mt), depression (sq.mt), longitudinal and transverse cracking (linear.mt), patch deterioration(sq.mt), potholes (number), raveling and weathering (sq.mt), rutting(sq.mt), slippage cracking (sq.mt), and swell (sq.mt).
- ✓ Log the results on the data collection form (ASTM D6433, 2007).

The following pieces of equipment were utilized to carry out pavement condition survey: straight edges, measuring tape and camera.

Figure 3-3: Shows the pavement condition surveys on subsection Alaba to Mazoria.

Figure 3-4: Shows the pavement condition surveys on sub section road Buge to Boditi.

Figure 3-5: Shows the condition surveys and Measurement of thickness of pavement layers on sub section road Buge to Boditi.

The identified distress was quantified and recorded using the following estimators: (ASTM D6433, 2007):

- ✓ **Distress type** - identify types of physical distress existing in the pavement. The distress types are categorized according to their casual mechanisms (i.e. functional or structural).
- ✓ **Distress severity** - estimating the distress items in three damage levels i.e. low (L), medium (M) and high (H) severity. This assessment helps to estimate degree of deterioration.
- ✓ **Distress extent** - Denote relative area (percentage of the road section) affected by each combination of distress type and severity.

Table 3-3: Flexible Pavement Condition Survey Data Sheet for Sample Unit (ASTM D6433, 199)

ASPHALT SURFACE ROADS CONDITION SURVEY DATA SHEET FOR SAMPLE UNIT	**200m**
Branch: ***Alaba-Sodo*** Sub-Section: ____________ Station: __________ Surveyed By: **Asrat** No. of Sample: ______ Date:______________	**7m** Carriageway width **N** Direction of survey →

	1 Alligator/Fatigue cracking 2 Bleeding 3 Block cracking 4 Bumps and sags 5 corrugation 6 Depression 7 Edge cracking 8 Reflection cracking 9 Lane shoulder drop 10 Longitudinal & Transverse 11 Patching &Utility patch 12 Polished Aggregate 13 Potholes 14 Rutting 15 Railroad crossing 16 Shoving 17 Slippage 18 Swell 19 Raveling &Weathering

DISTRESS SEVERITY	**QUANTITY**											**TOTAL**	**DENSITY**	**DEDUCT VALUE**

3.4.4 Laboratory Test Procedure

3.4.4.1 Atterberg Limit

These tests were conducted for all selected station samples to determine the value of plasticity of the soil and also which were based on the moisture content of the soil. The water contents corresponding to the transition from one state to another are termed as Atterberg Limits. In this study the liquid limit and plastic limit test were performed only on that portion of a soil which passes the 0.425mm (No. 40) Sieve (SABA Manual, 2002).

Test Procedure for Liquid Limit:

The liquid limit (LL) is the water content, at which the soil changes from a liquid state to a plastic state and principally it is defined as the water content at which the soil part cut using standard groove closes for about a distance of 13cm (1/2 in) at 25 blows of the liquid limit machine (Casagrande Apparatus).

This test was carried out in accordance of test procedures of AASHTO T 89 (SABA Manual, 2002).

Test Procedure for Plastic Limit:

The plastic limit (PL) is the water content, expressed in percentage, below which the soil stops behaving as a plastic material and it begin to crumble when rolled into a thread of soil of 3.0mm diameter.

This test was carried out as per the procedure of AASHTO T 90 (SABA Manual, 2002).

Figure 3-6: Laboratory test for Atterberg limit determination

3.4.4.2 Grain Size Analysis

In this study the grain size analysis was performed for different materials such as subgrade (for classification), subbase, and base course materials to determine the classification and the proportion of materials with their different grain sizes.

The grain size analysis or gradation is measured in the laboratory using two tests: a mechanical sieve analysis for the sand and coarser fraction, and a hydrometer test for the silt and finer clay material. But in this study only a mechanical sieve analysis was used due to lack of a hydrometer equipment in have been executed laboratory. The combined grading of the material shall be a smooth continuous curve falling within the grading limits.

Therefore, a mechanical sieve analysis was carried out in accordance of test procedures of AASHTOT-27/ASTM C 136 for sub-base & base course material and AASHTO Soil Classification System method used for sub-grade soil classification (as per AASHTO M 145) (SABA Manual, 2002).

Figure 3-7: Laboratory test for grain size determination

3.4.4.3 Compaction test/ Proctor test

This laboratory test was performed to determine the relationship between the moisture content and the dry density of a soil for a specified comp active effort. A material was mixed with water to form samples at various moisture contents ranging from the dry state to wet state. Material or soil compaction tests were performed using disturbed soil sample.

The tests were done in the laboratory accordance of test procedures of AASHTO T -99 (Standard proctor test) for subgrade and AASHTO T-180 (Modified Proctor Test) for sub-base and base materials.

In standard proctor test (AASHTO T99): Each layer is compacted by 25 blows of a rammer weighing 2.5kg, which is allowed to drop freely from a height of 30.5cm at each blow. After compaction of three layers, the soil is trimmed to the top of the mold. The mold with its content is removed from the base plate and weighed (ERA Manual, 2002).

In-modified proctor test (AASHTO T180): Each layer is compacted by 56 blows of a modified rammer weighing 4.54kg, which is allowed to drop freely from a height of 45.7cm at each blow. After compaction of five layers, the soil is trimmed to the top of the mold. The mold with its content is removed from the base plate and weighed (ERA Manual, 2002).

Figure 3-8: Laboratory test for MDD and OMC determination

3.4.4.4 Soil Classification

Soil Classification is the arrangement of soils into different group in order to that the soils in a particular group would have similar behavior. The method of classification used in this study was the AASHTO Classification System method. Which is useful for classifying soils for highways (SABA Manual, 2002).

Therefore, the classification only for subgrade materials was carried out in accordance of classification procedures of AASHTO M 145.

Figure 3-9: Laboratory test for soil classification determination

3.4.4.5 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 (SABA Manual, 2002).

The different layers such as subgrade, sub-base, and base/course materials strength are classified according to the CBR values. In this research also covers the determination of the CBR of pavement subgrade, subbase, and base/course materials from laboratory compacted specimens. In addition to the AASHTO soil classification tests, the 4 days soaked CBR tests were performed on the material samples to determine the highway construction material shear strengths. The results from the laboratory tests combined with the relevant pavement condition survey provide an evaluation of pavement performance to be made. Therefore, the test was done in the laboratory accordance of test procedures AASHTO T-193.

Figure 3-10: Laboratory test for CBR value determination

Figure 3-11: Repairing and replacing for excavated pavement layers at section Alaba-Mazoria

3.5 Study variables

3.5.1 Dependent variables

- ✓ Pavement distress and performance evaluation

3.5.2 Independent variables

- ✓ Distress levels based on PCI (types, severities, extent, sample area and quantity)
- ✓ Engineering properties of the pavement layers

3.6 Data collection equipment

a) **For pavement condition index**:
   1. Data Sheets: for recording the following information:
      - Date, location, section, sample unit size, slab number and size, distress types, severity levels, quantities, and names of surveyors, As show in Table 3.2.
   2. Digital Camera
   3. Hand measuring tape (50m and 10m)
   4. Straight edge
   5. Safety equipment

b) **For laboratory works:**
   1. Sampling bags
   2. Shovel and other Excavating tools
   3. All laboratory tests apparatus's
   4. Sample transporting cars

3.7 Data Collection Process

Before starting any data collection, formal letter was gotten from JIT and an official permission was obtained from Ethiopian Road Authority Sodo District Office. Then the data collection process was started with including field visual inspection (preliminary survey), detail pavement condition survey with field measurements and laboratory tests were conducted. Quantitative as well as qualitative data types were been collected and analyzed to determine the existing pavement condition and to measure the level of pavement failures quantitatively. Condition surveying, direct field measurement, determining of the engineering properties of pavement layers (due to laboratory tests) and some of secondary data were been the main sources for quantitative data and the preliminary survey was also a source for qualitative data. Those all the laboratory-tested data's were utilized based on the necessary input parameters for the analysis by comparing with ERA specification manuals.

The different type of data was collected for determination of PCI value from the three sections, out five sections of the road with in an interval length (sample unit length) 200m by width of 7.0m which selected randomly for the study of 68 Km and the data (type and severity levels of distress) was recorded on the data sheet for PCI determination. The data was collected using visual survey method by the researcher, assistant and daily labour. And for the section which show very poor, poor and no distress pavement condition rating a laboratory tests were conducted for subgrade, subbase and base course materials.

3.8 Data processing and Analysis

The field work and laboratory data was processed and analysis using Microsoft office which included with the table, graph, figure, and discussion format to achieve the objectives of this study.

- ✓ For PCI: simple descriptive statistical method (mean, min-, max-, standard deviation), Manuals (ASTM D6433, 2007), Excel, graphs and tables were used.
- ✓ For Laboratory Tests: Manuals, Excel graphs and Tables were used.

3.8.1 Pavement Condition Data Analysis

In this research, the pavement condition data analyzed as with involves the following five steps with its typical example of one sample unit (ASTM D6433, 2007):

1) Inspect sample units. Determine distress types, severity levels and measure density: Add up the total quantity of each distress type at each severity level, and record them in the "Total Severities" section. The units for the quantities may be either in square feet (square meters), or number of occurrences, depending on the distress type.
2) Divide the total quantity of each distress type/severity level by the total area of the sample unit and multiply by 100 to obtain the percent density of each distress type and severity.
3) Determine the deduct value (DV) for each distress type and severity level combination from the distress deduct value curves in ASTM D 6433, as show in figure 3.12 deduct value curves for asphalt for Alligator Cracking.

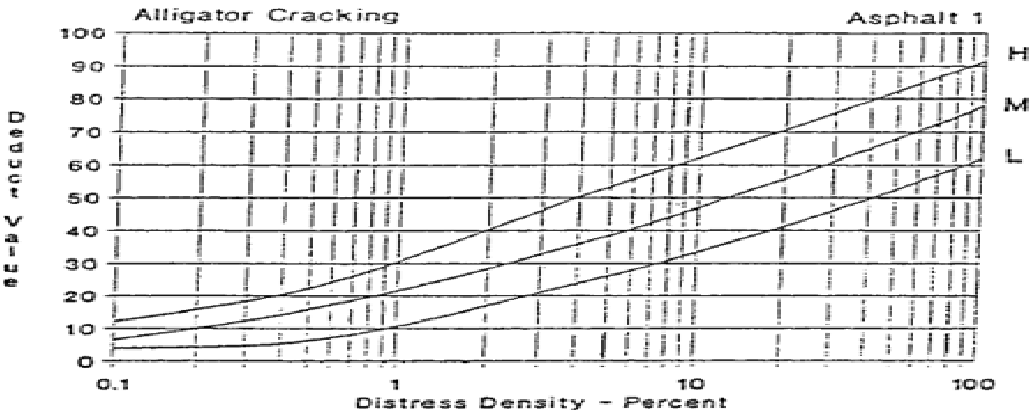


Figure 3-12: Deduct values for Alligator cracking (ASTM D 6433)

4) Determine the maximum Corrected Deduct Value (CDV).
5) The following procedure must be used to determine the maximum CDV:
   a. If none or only one individual deduct value is greater than two, the total value is used in place of the maximum CDV in determining the PCI; otherwise, maximum CDV must be determined using the procedure described in below b-e.
   b. List the individual deduct values in descending order.

      For example, in Table 3.4 this will be 38, 30, 23.5, 10.5, 8, and 5.2.
   c. Determine the allowable number of deducts, **m**, from Figure 3.13 or using the following formula from ASTM D 6433:

      m= 1+ (9/98) (100-HDV) ≤ 10

      Where: **m** = allowable number of deducts including fractions (must be less than or equal to 10).

      HDV=highest individual deduct value.

For the example in Table 3.4,

$$m = 1 + \left(\frac{9}{98}\right) * (100 - HDV) = 1 + \left(\frac{9}{98}\right) * (100 - 38) \ = 6.7 < 10 \text{ OK!!}$$

d. The number of individual deduct values is reduced to the m largest deduct values, including the fractional part when the m value is greater than number deduct values. But, if less than m deduct values are available, all of the deduct values are used. For the example in Table 3.5, the values are 38, 30, 23.5, 10.5, 8, 5.2 directly used in iteration table.

e. Determine maximum CDV iteratively, as shown in Table 3.5.

   i. Determine total the deduct value by summing individual deduct values. The total deduct value is obtained by adding the individual deduct values in d, that is, 111.5.

   ii. Determine q as the number of deducts with a value greater than 2.0. For example, in Table 3.5, q=6.

   iii. Determine the CDV from total deduct value and q by looking up the appropriate correction curve for AC pavements in figure 3.14.

   iv. Reduce the smallest individual deduct value greater than 2.0 to 2.0 and repeat i – iv until q=1.

   v. Maximum CDV is the largest of the CDVs.

6) Calculate PCI by subtracting the maximum CDV from 100:

For the example, Max CDV = **61**; PCI = 100 – MaxCDV = **39**, and Rating = **Poor**

7) Table 3.5 shows a summary of PCI calculation for the example AC pavement data in Table 3.4. A blank PCI calculation form is included in Table 3.3.

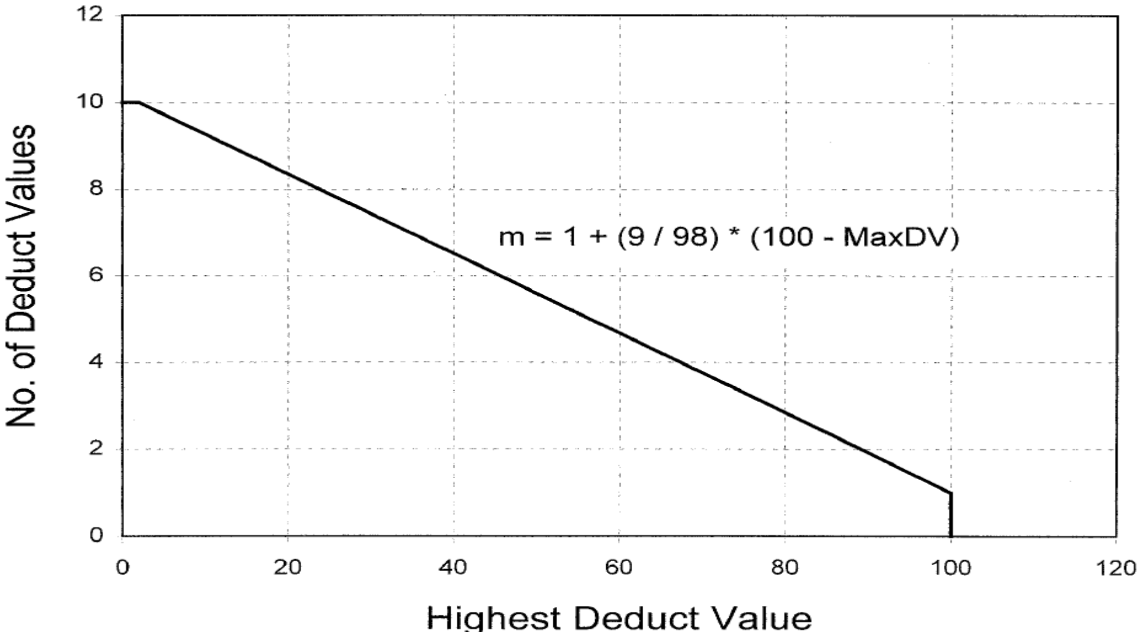


Figure 3-13: Adjustment of Number of Deduct Values (ASTM standard D 6433)

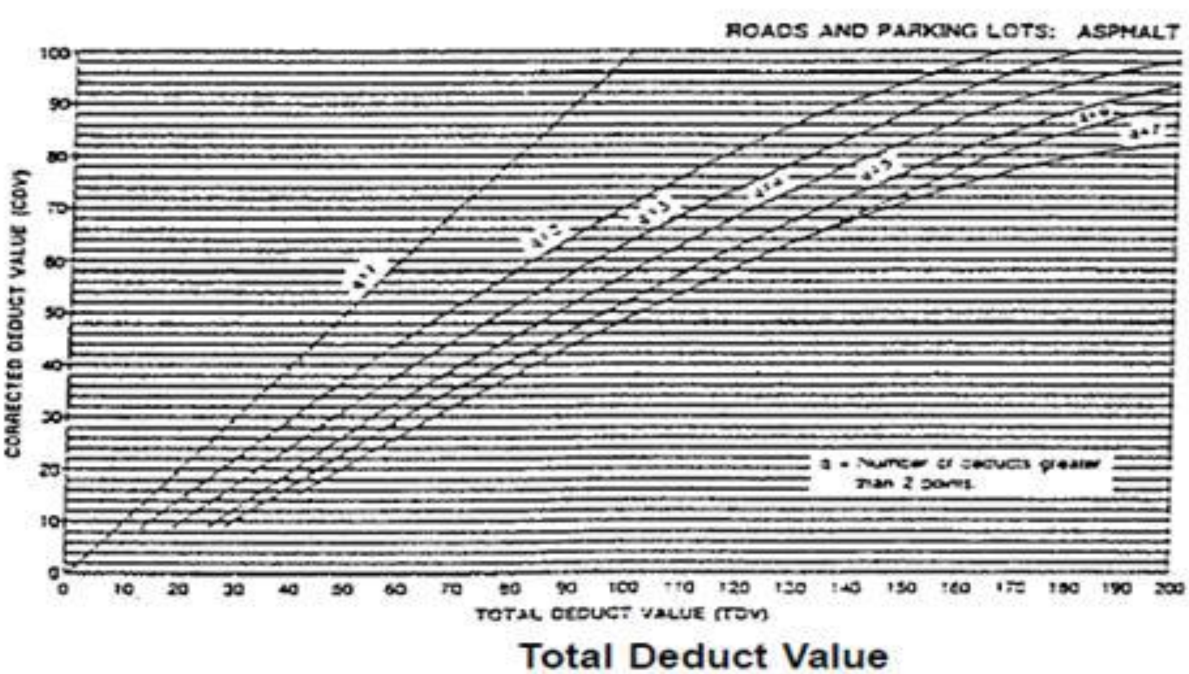


Figure 3-14: Corrected deduct values (ASTM standard D 6433)

Table 3-4:Example of a Flexible Pavement Condition Survey Data Sheet

ASPHALT SURFACE ROADS CONDITION SURVEY DATA SHEET FOR SAMPLE UNIT	**200m** Carriageway width **7m**
Branch: **Alaba-Sodo** Station Interval: **27+900-28+100** Section: **Shone-Buge** No. of sample: **4th** Surveyed By: **Asrat** Date: July 18, 2016	Direction of survey → N

	1 Alligator/Fatigue cracking	6 Depression	11 Patching &Utility patch	16 Shoving
	2 Bleeding	7 Edge cracking	12 Polished Aggregate	17 Slippage
	3 Block cracking	8 Reflection cracking	13 Potholes	18 Swell
	4 Bumps and sags	9 Lane shoulder drop	14 Rutting	19 Raveling &Weathering
	5 corrugation	10 Longitudinal & Transverse	15 Railroad crossing	

DISTRESS SEVERITY	**QUANTITY**												**TOTAL**	**DENSITY**	**DEDUCT VALUE**
1L	3*8	2.7*6	1.4*6.6	1.6*6.4									59.68	4.26	23.5
1M	2.2*6	1.8*5.6											23.28	1.66	30
1H	2.6*10.6												27.56	1.97	38
14L	1.2*11.1	1.8*5.6											23.4	1.67	10.5
9M	23.3	18.5	33										74.8	5.34	5.2
9H	27.5	13.5	28										69	4.93	8

$$m = 1 + \left(\frac{9}{98}\right) * (100 - HDV) = 1 + \left(\frac{9}{98}\right) * (100 - 38) = 6.7 < 10 \text{ OK!!}$$

Descending order for deduct values: 38, 30, 23.5, 10.5, 8, 5.2

If less than m deduct values are available, all of the deduct values are used. (m=6.7>6)

Table 3-5:Calculation of CDV value—Flexible Pavement (ASTM D 6433)

#	Deduct value						Total	q	CDV
1	38	30	23.5	10.5	8	5.2	111.5	6	58
2	38	30	23.5	10.5	8	2	112	5	57.5
3	38	30	23.5	10.5	2	2	106	4	58.5
4	38	30	23.5	2	2	2	97.5	3	61
5	38	30	2	2	2	2	76	2	52
6	38	2	2	2	2	2	48	1	48

Max CDV = **61**

PCI =**100 – Max CDV**

PCI = **39**

Rating = **Poor**

3.8.2 Laboratory Data Analysis

In order to achieve the objectives of this research, it was first necessary to analysis different data's such as pavement condition data and laboratory tests data. Therefore, adequate samples were collected and labeled immediately from each station for every pavement layer, after the field test or measurement was performed. The representative samples collected from more distressed (with dominant alligator cracking and rutting distress types) and no distressed surface condition, which were also from the base, subbase and subgrade of pavement layer thickness. Immediately after extracting samples, these were transported to ERA Sodo District laboratory 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 and to checking whether it effects on the pavement distresses or not. The necessary tests were conducted for all the samples and the summary of the results are presented in a tabulated form in next chapter of this research. The tests were processed and analyzed according to AASHTO, ASTM, and ERA specification by following the procedures discussed on the soil laboratory manual by SABA, 2002. The laboratory data analyses with their results are discussed in chapter four of this research and some are attached in Appendix C up to Appendix F.

Figure 3-15: Extracting the existing pavement for Sampling of subgrade soil, base course, and sub base materials at Station 67+926 (Sodo Town).

Figure 3-16: Sampling of subgrade soil, base course, and sub base materials from existing pavement at Station 67+926 (Sodo Town).

Figure 3-17: Measurement of total depth of excavated pit and transporting it from the stations 13+600-13+800 and 67+926.

CHAPTER FOUR

RESULTS AND DISCUSSIONS

4.1 Pavement Condition Survey

One of the key components of an effective pavement management system is an accurate assessment of the condition of the existing pavement roads. Therefore, the pavement condition survey was made on selected test sections by following ASTM D6433 Manual methods as discussed in chapters 3 to evaluate the distresses and the performance of asphalt pavement based on pavement condition index and engineering properties of layers. This was done by dividing road pavement into different sections. Each section was divided into different sample units. The main objective of the pavement condition survey for this study was to evaluate the state of the existing pavement distresses and performance and that of the subgrade, subbase and base course by inspecting the physical conditions of the existing pavement. Before the beginning of the detailed pavement evaluation, the entire road length was visually assessed and an attempt was made to identify the current condition of the road and the types of distresses occurred on the road prism.

Table 4.1 to Table 4.3 show that each result of pavement condition rating along the different selected road sections those are Alaba-Sodo, Shone-Buge, and Buge-Boditi sections. Some detail sample calculation of selected sample unit of surveyed one’s with their values are shown in Appendix A and the deduct value curves for each distress types are also shown in Appendix B.

The type and severity of s distress was assessed by visual inspection of the pavement sample units and the quantity of each distress was measured. Detail results of the pavement condition survey are as shown below:

1. Calculated pavement condition index(PCI) on section-1(Alaba-Mazoria section):

The pavement condition survey for this test section road is carried out from Alaba – Mazoria, which is 20 Km length.

- ✓ A total of fourteen (14) preliminary homogeneous test blocks were taken from the assumed standard deviation value from PCI field inspection and seventeen (17) additional test blocks were taken from actual standard deviation value with 200m length from the total number of sample units, i.e. 14+17 = 31 sample units were surveyed, which were from the total of one hundred (100) homogeneous test blocks.
- ✓ From the surveyed sample units, the minimum and maximum block PCI values are 25 and 76 respectively.
- ✓ The weighted average PCI value (i.e. considering area of pavement) for this Alaba-Mazoria test road section is 47.96%, which can be rated as fair pavement surface.
- ✓ The detail PCI values and PCR for each surveyed blocks are as shown in Table 4.1 and Table 4.3.

2. Calculated pavement condition index (PCI) on section-3 test road (Shone-Buge):

- ✓ This test section road covers from Shone-Buge and the pavement condition survey is carried out on the whole chainage of 7.3 Km length.
- ✓ A total of Twelve (12) preliminary homogenous test blocks were taken from the assumed standard deviation value from PCI field inspection and two (2) additional test blocks were taken from actual standard deviation value having with the same length (which is 200m) are incorporated in this test road, i.e.12+2 = 14 sample units were surveyed and they selected from the total number of sample units of the section (total number sample units for this section are 36).
- ✓ From the surveyed sample units, the minimum and maximum block PCI values recorded in this test road are 33.5 and 76 respectively.
- ✓ The weighted average PCI value for test Road No.2 is 47.42%, which can be rated as fair pavement surface.
- ✓ The detail PCI values and PCR for each surveyed blocks are as shown in Table 4.1 and table 4.3.

3. Calculated Pavement Condition index (PCI) on section-4 test road (Buge-Boditi):

- ✓ This study road covers section from Buge-Boditi. A total of 17Km length of section the pavement condition survey was performed within this section.
- ✓ A total of Fourteen (14) preliminary homogeneous test blocks were taken from the assumed standard deviation value from PCI field inspection and one (1) additional test block was taken from actual standard deviation value with 200m length, i.e. 14+1 =15 sample units were surveyed from the total number of sample units, which were eighty five (85) homogeneous test blocks.
- ✓ From the surveyed sample units, the minimum and maximum block PCI values are 31% and 72% respectively.
- ✓ The weighted average PCI value for test road No.3 is 48.96%, which can be rated as fair road surface.
- ✓ The detail PCI values and PCR for each surveyed blocks are as shown in Table 4.2and Table 4.3.

Table 4-1: Summary of Preliminary PCI and PCR values for Alaba-Mazoria and Shone-Buge sections (from assumed standard deviation value)

Alaba-Mazoria				Shone-Buge		
Sample unit No.	**Station No. (0+000-20+000) (20km)**	**PCI Value**	**PCR**	**Station No. (26+100-33+400) (7.3km)**	**PCI Value**	**PCR**
1	0+600-0+800	29.5	Poor	26+100-26+300	52	Fair
2	2+000-2+200	40	Poor	26+700-26+900	59	Good
3	3+400-3+600	32	Poor	27+300-27+500	42	Fair
4	4+800-5+000	52	Fair	27+900-28+100	39	Poor
5	6+200-6+400	33	Poor	28+500-28+700	42.5	Fair
6	7+600-7+800	26	Poor	29+100-29+300	41	Fair
7	9+000-9+200	28	Poor	29+700-29+900	57	Good
8	10+400-10+600	47	Fair	30+300-30+500	76	Very Good
9	11+800-12+000	74	Very Good	31+900-31+100	41.5	Fair
10	13+200-13+400	59	Good	31+500-31+700	37.5	Poor
11	14+600-14+800	58	Good	32+100-32+300	33.5	Poor
12	16+000-16+200	48	Fair	32+700-32+900	42	Fair
13	17+400-17+600	29	Poor			
14	18+800-19+000	73	Very Good			

Table 4.1 show the values of PCI that determined from the assumed standard deviation and they also used for determination of the value of actual standard deviation for the calculation of additional PCI values. Majority of the section is poor and fair condition and very few are in good and very good condition. For the reason that both the PCI

values determined from assumed standard deviation and actual standard deviation jointly indicates the whole pavement condition of the section.

Table 4-2: Summary of Preliminary PCI and PCR values for Buge to Boditi section (from assumed standard deviation value)

Buge-Boditi			
Sample unit No.	**Sample unit station (33+400-50+400)(17km)**	**PCI Value**	**PCR**
1	33+400 - 33+600	47	Fair
2	34+600 - 34+800	50	Fair
3	35+800 - 36+000	46	Fair
4	37+000 - 37+200	56	Good
5	38+200 - 38+400	52	Fair
6	39+400 - 39+600	72	Very Good
7	40+600 - 40+800	34	Poor
8	41+800 - 42+000	58	Good
9	43+000 - 43+200	48	Fair
10	44+200 - 44+400	31	Poor
11	45+400 - 45+600	47.5	Fair
12	46+600 - 46+800	39	Poor
13	47+800 - 48+000	44	Fair
14	49+000 - 49+200	56	Good

The section shown in Table 4.2 were selected using assumed and actual standard deviation value and check using Pavement Condition Index. It shows the condition of the road from Buge - Boditi road stretch, majority of the section is in fair condition and very few are in good and very good condition.

Table 4-3: Summary of Additional PCI and PCR Values for Three Surveyed Sections (from actual standard deviation value)

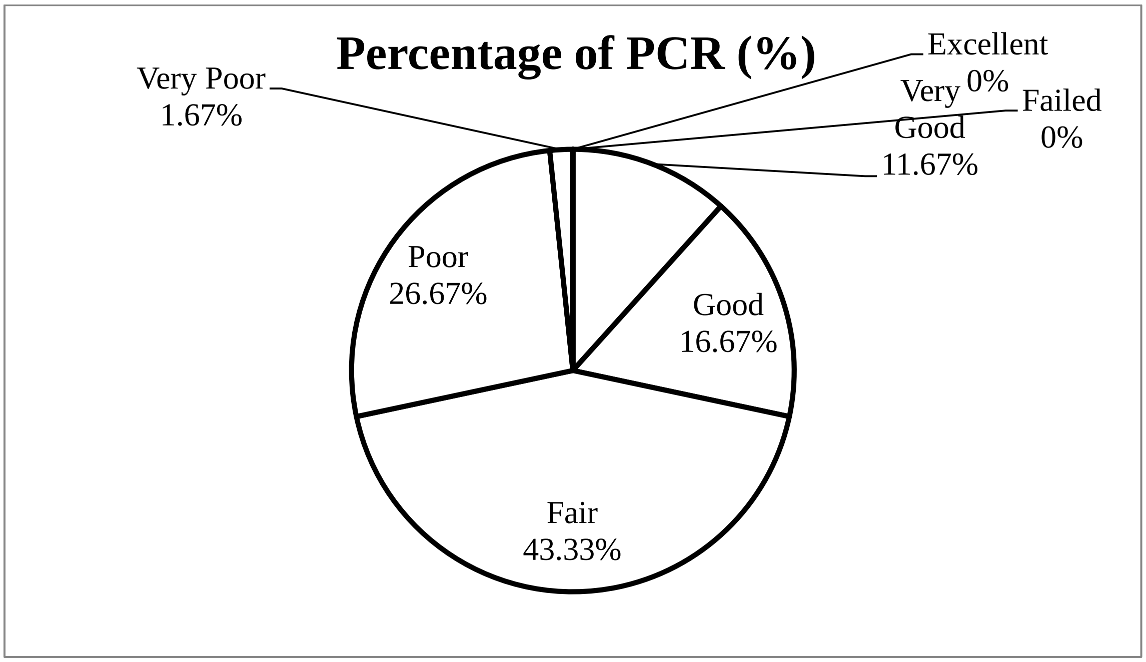
Alaba-Mazoria				**Shone-Buge**		
Sample unit No.	**Station No. (0+000-20+000) (20km)**	**PCI Value**	**PCR**	**Station No. (26+100-33+400) (7.3km)**	**PCI Value**	**PCR**
1	0+400-0+600	38	Poor	26+300-26+500	47	Fair
2	1+600-1+800	48	Fair	29+900-30+100	54	Fair
3	2+800-3+000	48	Fair			
4	4+000-4+200	33.5	Poor			
5	5+200-5+400	73	Very Good			
6	6+400-6+600	34	Poor	**Buge-Boditi (33+400-50+400) = (17km)**		
7	7+800-8+000	58	Good	34+200-34+400	54	Fair
8	8+800-9+000	48	Fair			
9	10+000-10+200	56.6	Good			
10	11+200-11+400	48	Fair			
11	12+400-12+600	41	Fair			
12	13+600-13+800	25	Very Poor			
13	14+800-15+000	72	Very Good			
14	16+200+16+400	76	Very Good			
15	17+200-17+400	57	Good			
16	18+400-18+600	54.4	Fair			
17	19+600-19+800	48	Fair			

Table 4.3 show the values of additional PCI values that determined from the actual SD of the sections and It was observed and check that the road condition is mostly fair. In this table also, it was found out that in the road section 13+600 to 13+800 the condition was very poor which need full attention from the concerned bodies.

Table 4-4: Percentage of pavement condition rating

PCR	Total Number of PCR on the three surveyed sections	Percentage of PCR (%)
Excellent	0	0
Very Good	7	11.67
Good	10	16.67
Fair	26	43.33
Poor	16	26.67
Very Poor	1	1.67
Failed	0	0

By the similar manner the above Table 4.4 results discussed in chart format shown as below:



Therefore, based on these percentage values of PCI survey shown in Table 4.4 or above chart, the majority of the distressed sections has fair and poor surface condition.

Finally, the above all results and discussion are only about the three selected sections out the total five sections of the study and in conclusion, the remained two sections also almost same condition with the surveyed three sections and implies the whole 68km road rated as fair condition and they need suitable maintenance works.

Table 4-5: PCI calculation sample for Alaba to Mazoria section at 12+400-12+600

ASPHALT SURFACE ROADS CONDITION SURVEY DATA SHEET FOR SAMPLE UNIT	**200m**
Branch: **Alaba-Sodo** Station Interval: 12+400-12+600 No. of sample: **11th** Surveyed By: **Asrat** Date: July 25, 2016	**7m** Carriageway width Direction of survey → N

1 Alligator/Fatigue cracking; 2 Bleeding; 3 Block cracking; 4 Bumps and sags; 5 corrugation; 6 Depression; 7 Edge cracking; 8 Reflection cracking; 9 Lane shoulder drop; 10 Longitudinal & Transverse; 11 Patching &Utility patch; 12 Polished Aggregate; 13 Potholes; 14 Rutting; 15 Railroad crossing; 16 Shoving; 17 Slippage; 18 Swell; 19 Raveling &Weathering

DISTRESS SEVERITY	**QUANTITY**												**TOTAL**	**DENSITY**	**DEDUCT VALUE**
1M	2.6*14.6	1.8*7.4	2.4*14.4										85.84	6.13	40.2
14L	2.8*18.3												51.24	3.66	18.3
7M	3.1	2.4											5.5	0.39	5
9M	14.4	21.3	25										60.7	4.34	5.2
13M	3												3	0.21	10
13H	4												4	0.29	30.4

$m = 1 + \left(\frac{9}{98}\right) * (100 - HDV) = 1 + \left(\frac{9}{98}\right) * (100 - 40.2) = 6.5 < 10$ OK!!

40.2, 30.4, 18.3, 10, 5.2,5

If less than m deduct values are available, all of the deduct values are used. (m=6.5>6)

Table 4-6: Maximum CDV determination sample

#	Deduct value						Total	q	CDV
1	40.2	30.4	18.3	10	5.2	5	109.1	6	53
2	40.2	30.4	18.3	10	5.2	2	106.1	5	54
3	40.2	30.4	18.3	10	2	2	102.9	4	58
4	40.2	30.4	18.3	2	2	2	94.9	3	59
5	40.2	30.4	2	2	2	2	78.6	2	56
6	40.2	2	2	2	2	2	50.2	1	50

Max CDV = **59**

PCI =100 – Max CDV

PCI = **41**

Rating = Fair

These the above Table 4.5 and Table 4.6 shown the detail sample calculation for a sample unit of Alaba-Mazoria at 12+400-12+600 and the PCI value 41 and PCR can be rated as fair road surface.

4.3 Laboratory Test Results and Discussion for Poor and Very Poor PCR

The engineering properties of materials were determined by carrying out different tests on samples of materials in a laboratory. Tests to determine the engineering properties such as Atterberg Limits, gradation, Soil Classification, Compaction, and CBR were performed in this research.

The site selections for collecting samples for laboratory tests were performed based on pavement condition index (PCI) values or PCR. Those location which were taken as very poor, poor (with dominant alligator cracking and rutting distress types), and no distressed pavement condition. Which were taken from four different sections. Two samples from Alaba-Mazoria section (those were from poor and very poor surface condition, from stations 7+600-7+800km and 13+600-13+800km). One sample from Buge-Boditi section (this was from poor surface condition, stations 44+200 - 44+400 km), and one also from Boditi-Sodo section (this was from no distressed surface condition, from station 67+926km) to evaluate or check the effect of pavement layers properties on distresses. Enough samples were collected from each pavement layer to perform the necessary tests. Samples were collected and labeled immediately after the field measurement is performed.

The representative samples collected during detailed field investigation were carried to ERA Sodo District Laboratory, and the following tests were undertaken. These tests are Atterberg limit test, Proctor (Compaction) test, Sieves analysis, Soil Classification, and CBR test of each pavement layers (subgrade, subbase, and base course) to understand the general behavior of the road materials and to observe or check whether the laboratory results affect pavement distresses or not. So here in this chapter, the results and discussion of different type of tests are covered as follow:

4.3.1 Atterberg limit test results with discussion

The Atterberg limits were based on the moisture content of the material. Based on their mode of formation and mineralogical composition different materials respond differently for the same moisture content. The purpose of Atterberg Limit test is to determine the plastic and liquid limits of a fine-grained material.

4.3.1.1 Atterberg limit test results for subgrade soils:

The plasticity of the subgrade soils of different sections are shown in the table below and the some data analysis are attached in Appendix C.

Table 4-7: Results of Atterberg limit tests for Sub-grade Soils

Station Between	PCR	Subgrade Soil		
		LL	PL	PI
7+600 - 7+800 **Alaba-Mazoria**	Poor	43.02	28.34	15
13+600 - 13+800 **Alaba-Mazoria**	Very Poor	45.16	25.66	20
44+200 - 44+400 **Buge-Boditi**	Poor	42.54	24.5	18
67+926(**Boditi-Sodo)**	From not distressed	45.08	22.84	22

According to ERA Manual, 2002, the soils with PI values less than 25% and LL< 50 are suitable subgrade materials. So that all station test results show suitable subgrade materials for very poor and poor pavement condition rating (PCR) sections (existing with dominant alligator cracking and rutting distress types) and the test also was performed for no distressed section and it resulted as the same to all other sections (distressed sections) of subgrade soils.

4.3.1.2 Atterberg limit tests for Sub-base materials (red ash):

The plasticity of the sub-base materials of different sections are shown in the table below and some data analysis are attached in Appendix C.

Table 4-8: Results of Atterberg limit tests for Sub-base Soils

Station Between	PCR	Subbase		
		LL	PL	PI
7+600 - 7+800 **Alaba-Mazoria**	Poor	39.7	24.5	15
13+600 - 13+800 **Alaba-Mazoria**	Very Poor	39.08	20.49	19
44+200 - 44+400 **Buge-Boditi**	Poor	40.12	24.5	16
67+926(**Boditi-Sodo)**	No distress	40.01	26.39	14

According to ERA Manual, 2002, all suitable sub-base materials shall have a maximum Plasticity Index of 6 or 12 and when determined in accordance with AASHTO T-90. Therefore, all the results of these tests show that the red ash (subbase materials) have been constructed were not suitable subbase materials for very poor or poor pavement condition rating (PCR) sections (existing with dominant alligator cracking and rutting distress types).

4.3.1.3 Atterberg limit tests for Base Course material:

According to ERA manual, 2002, the fine fraction of a GB1 material shall be non-plastic or shall have a maximum Plasticity Index of 6 when determined in accordance with AASHTO T-90. Therefore, these test values shows that the materials have been constructed were fulfilled the minimum requirement of ERA specification.

The plasticity of the base materials of different sections are shown in the table below and some data analysis are attached in Appendix C.

Table 4-9: Results of Atterberg limits for base course

Station Between	PCR	Base Course		
		LL	PL	PI
7+600 - 7+800 **Alaba to Mazoria**	Poor	24	23	1
13+600 - 13+800 **Alaba to Mazoria**	Very Poor	24	22	2
44+200 - 44+400 **Buge - Boditi**	Poor	24	20	4
67+926(**Boditi-Sodo)**	No distress	25	23	2

4.3.2 Sieve Analysis

This test is performed to determine the percentage of different grain sizes contained within a soil or aggregate and it is used to determine the textural classification of soils (i.e., gravel, sand, silty clay, etc.) which in turn is useful in evaluating the engineering characteristics such as permeability, strength, and swelling potential.

Therefore, a mechanical sieve analysis was carried out in accordance of test procedures of AASHTOT-27/ASTM C-136 for sub-base & base course material and AASHTO Soil Classification System method used for sub-grade soil classification (as per AASHTO M 145).

4.3.2.1 Sieve Analysis for Subbase Materials

At the whole stretch of the road, red ash material was used as a sub-base materials, and this test was also performed on these materials and the results shows for the samples have been tested were out of Minimum and Maximum range limit of ERA specification. Which implies those have been constructed were not uniformly graded at all. The detail results are attached in appendix D**.**

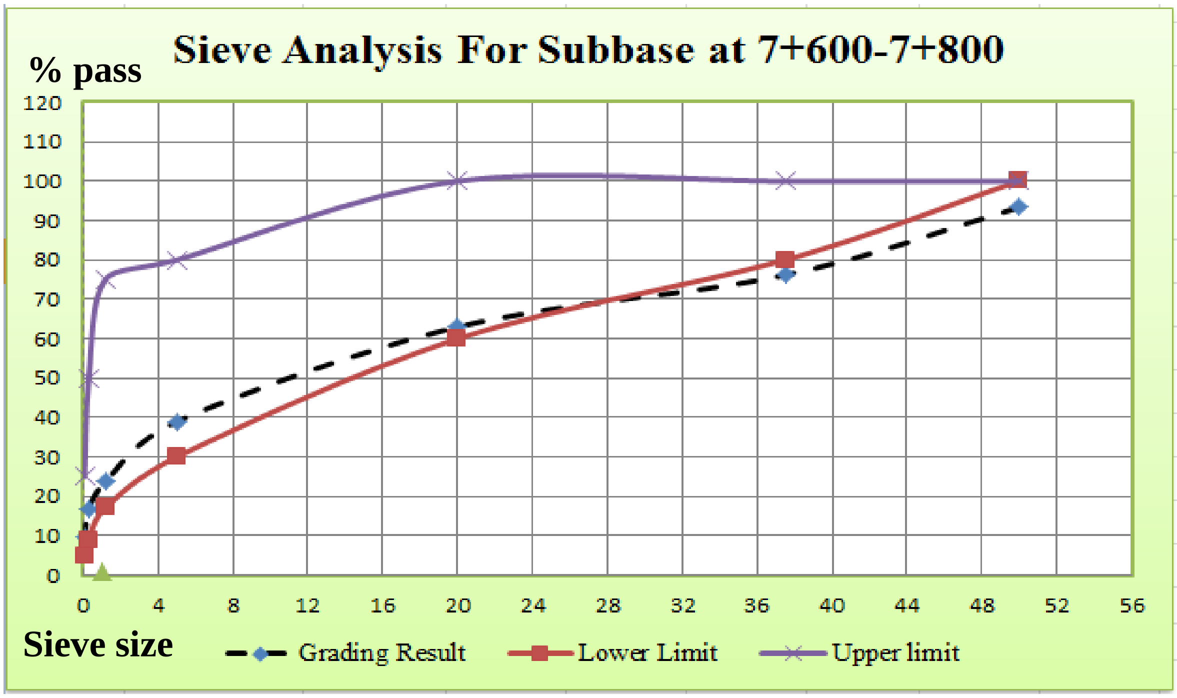


Figure 4-1:Sieve analysis result for sub-base material

The gradation results for all remained stations (13+600-13+800, 44+200-44+400, and 67+926) are approximately the same with the above station result. Therefore, these values indicate that the cause of pavement failure for very poor and poor pavement condition rating sections (existing with dominant alligator cracking and rutting distress types) was the subbase material. Some detail results are attached in appendix D.

4.3.2.2 Sieve Analysis for Base Course:

To attain well graded base course material, the alternative gradation limits are very important, which depending grain particle size and shape conditions. Therefore, in this road crushed rock have been used as a base course material and different tests were performed on it, one of these tests was a sieve analysis of base course materials. However, all base course materials must have a particle size distribution and particle shape which provide high mechanical stability and should contain sufficient fines (amount of material passing the 0.425 mm sieve) to produce a dense material when compacted.

The graph with table below shows the base course sieves analysis according to AASHTO standard for 13+600-13+600 station. Some detail results are attached in appendix D.

Table 4-10: Sieve analysis result for base course material at station between 7+600-7+800

Station	**13+600-13+800**			Sampling Date:			**August 26,2016**				
Sample taken	**From Existing Road**			Test date :			**August 30, 2016**				
Material Type	**Base Course**			Sample No:			**Two**				
Visual Discription	**Crushed Aggregate**			Test condition:			**Dry**				
Representing sectior	**Alaba-Mazoria**										
PARTICLE SIZE DISTRIBUTION BY SIEVING TEST METHODS: AASHTO 27/11											
Before	7807			Before	7771						
Trial one				**Trial two**							
Sieve Size (mm)	**Wt. of Sample Retained (g)**	**% Retaind**	**% Pass**	**Sieve Size (mm)**	**Wt. of Sample Retained (g)**	**% Retaind**	**% Pass**	Average % retained	**Average % pass**	**Lower limit**	**Upper limit**
63.00	0.00		**100.00**	**63.00**	**0.00**		**100.00**				
50.00	0.00	-	**100.00**	**50.00**	0	-	**100.00**	0.00	**100.00**	**100**	**100**
37.5	964	12.35	**87.65**	37.5	575	7.40	**93**	9.87	**90.13**	80	100
20.0	1202	15.40	**72.26**	20.0	1984	25.53	**67**	20.46	**69.66**	60	100
5.0	2894	37.07	**35.19**	5.0	2467	31.75	**35**	34.41	**35.26**	30	80
1.18	965	12.36	**22.83**	1.18	846	10.89	**24.4**	11.62	**23.63**	17	75
0.30	982	12.58	**10.25**	0.30	960	12.35	**12.1**	12.47	**11.17**	9	50
0.075	398	5.10	**5.15**	0.075	489	6.29	**5.8**	5.70	**5.47**	5	25
Pan	402	5.15		Pan	450	5.79					
Total	7807			Total	7771						

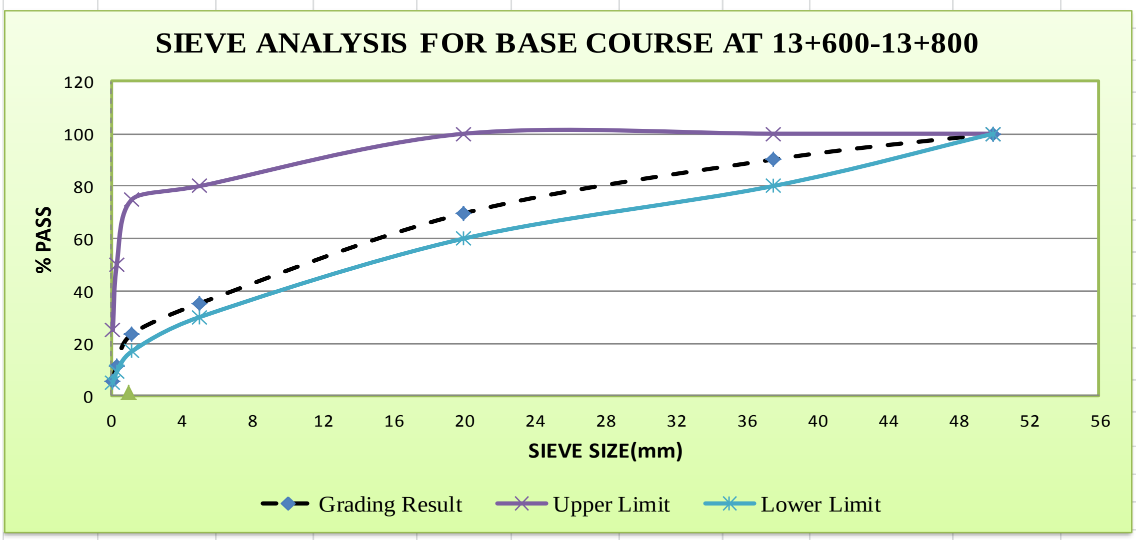


Figure 4-2:Gradation result graph for base course material at between 13+600-13+800

Moreover, the gradation results for all remained stations (7+600-7+800, 44+200-44+400, and 67+926) base course materials were approximately the same with above station result. Therefore, the test results of all stations base course materials that have been constructed were fulfilled the minimum requirement of the ERA standard, i.e. the materials were uniformly graded at all. Some additional detail results are attached in appendix D.

4.3.3 Soil Classification for Subgrade:

The sieve analysis for subgrade is widely used in classification of subgrade soils and its procedure for classifying soils into seven groups based on laboratory determination of particle-size distribution, liquid limit, and plasticity index. AASHTO Soil Classification System and the Unified Soil Classification System usually used for highway construction. But in this study the method of classification used was the AASHTO Soil Classification System. In this system the sieve sizes for test which have been used are 2mm, No. 40 (425-μm), No.200 (75-μm) to determine the categories of soil. The Table 4.10 below shows the subgrade soil classification according to AASHTO Classification System.

Table 4-11: Particle-size distribution with classification of subgrade at station 7+600 - 7+800km

Station: 7+600 - 7+800	**AASHTO Classification:**		***A-7-6***
Alaba to Mazoria	**Wt of Before Washing (gm)**		500
Sieve sizes(mm)	Wt Retained(gm)	% Retained(gm)	Cum.% Pass
2	35	7	93.0
0.425	56.0	11.2	81.8
0.075	70.0	14	67.8
pan	339.0		
Total	500		

These soils lead with general rating of subgrade soil as fair to poor as a sub-grade material and they were classified by ASSHTO under the A-7-6 category which indicate that usual types of major essential materials was clayey.

Moreover, the Table 4.12 below shows the soil classification of subgrade soil in all stations according to AASHTO standard.

Table 4-12: Results of subgrade soils classification

Station Between	PCR	Subgrade Soil				AASHTO Soil Classification System
		LL	PL	PI	%pass #200 Sieve	
7+600 - 7+800 **Alaba-Mazoria**	Poor	43.02	28.34	15	68	A – 7 – 6
13+600 - 13+800 **Alaba-Mazoria**	Very Poor	45.16	25.66	20	78.4	A – 7 – 6
44+200 - 44+400 **Buge-Boditi**	Poor	42.54	24.5	18	73	A – 7 – 6
67+926 (**Boditi-Sodo)**	From Not distressed	45.08	22.84	22	81	A – 7 – 6

These deal for usual types of significant constituent materials implies for all station of subgrade materials were on group of clayey soils and they were classified by ASSHTO under A-7-6 category which showed that general rating of a soil fair to poor as a sub-grade material.

4.3.3 Compaction test

The purposes for this test are to determine the maximum dry density attainable under specified nominal compaction energy for a given material and the (optimum) moisture content corresponding to this density. Material or soil compaction tests were conducted in the laboratory according to AASHTO T-99 (Standard Proctor Test) for subgrade and AASHTO T-180 (Modified Proctor Test) for subbase and base course material by using disturbed samples which taken from the existing road. Therefore, these test results with their discussion for different layers are described as follow:

4.3.3.1 Compaction Test Results for Subgrade Soils:

Table 4-13: Results of Compaction Test for Different Stations

Station Between	PCR	Subgrade Compaction test values	
		MDD	OMC
7+600 - 7+800 **Alaba to Mazoria**	Poor	1.54	32
13+600 - 13+800 **Alaba to Mazoria**	Very Poor	1.55	32.5
44+200 - 44+400 **Buge- Boditi**	Poor	1.58	29
67+926 **(Boditi-Sodo)**	From not distressed	1.57	32

These results for subgrade soils that were tested with standard proctor test and their samples compacted in three layers in a mold by a hammer in accordance with specified nominal compaction energy. So the dry density was determined based on the moisture content and the unit weight of compacted soil. The water content at which this dry density occurs was termed as the optimum moisture content (OMC). They also used the graph of moisture content verses dry density to determine their maximum values by graph reading. The results of MDD and OMC are given in the above Table 4.13 and some data analysis is shown in Appendix E.

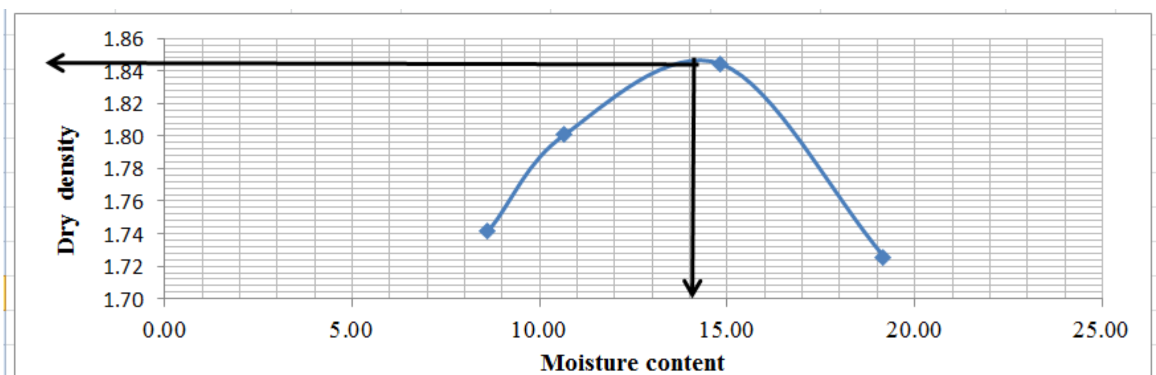
4.3.3.2 Compaction Test for Sub-Base Material

The results of MDD and OMC of subbase (red ash) material samples are tabulated below, and which were determined from moisture-density relationship graphs. The material samples are compacted in five layers in a mold by a hammer in accordance with specified nominal compaction energy. The tests were done in the laboratory according to AASHTO T-180(Modified proctor test).

Table 4-14: Results of compaction test for sub-base of different stations

Station Between	PCR	Subbase compaction test values	
		MDD	**OMC**
7+600 - 7+800 **Alaba to Mazoria**	Poor	1.89	12.4
13+600 - 13+800 **Alaba to Mazoria**	Very Poor	1.842	14
44+200 - 44+400 **Buge- Boditi**	Poor	1.86	14
67+926 **(Boditi-Sodo)**	No distress	1.85	15

Dry density is determined based on the moisture content and the unit weight of compacted soil. The water content at which this dry density occurs is termed as the optimum moisture content (OMC) and they also determined by reading from graph. Some detail data analysis are shown in Appendix E.



MDD :	**1.842**	**g/cm3**
OMC :	**14**	**%**

Figure 4-3: Compaction test result for subbase material at station 13+600-13+800

4.3.3.3 Compaction Test for Base Course materials

Compaction test for base course material of different stations were done in the laboratory according to AASHTO T-180 (Modified proctor test) and the samples prepared and compacted in five layers in a mold by a hammer in accordance with specified nominal compaction energy. Dry density is determined based on the moisture content and the unit weight of compacted base course material. The water content at which this dry density occurs is termed as the optimum moisture content (OMC).

Table 4-15: Results of compaction test for Base course of different stations

Station Between	PCR	Base course Compaction test values	
		MDD	OMC
7+600 - 7+800 **Alaba to Mazoria**	Poor	2.31	4.45
13+600 - 13+800 **Alaba to Mazoria**	Very Poor	2.31	4.5
44+200 - 44+400 **Buge- Boditi**	Poor	2.33	4.1
67+926 **(Boditi-Sodo)**	No distress	2.34	3.6

The results of MDD and OMC are given in the above Table 4.15 and some data analysis are shown in Appendix E.

4.3.4 California Bearing Ratio Test

The different layers such as subgrade, sub-base, and base course materials strength are classified according to the CBR values. This test technique also covers the determination of the CBR of pavement subgrade, subbase, and base course materials from laboratory compacted specimens. The CBR test procedures for subgrade material are similar to that of the subbase or base course material procedures, but the main difference are the number of layers and the weight of the hammer i.e. the subgrade sample molded in 3 layers with 2.5kg hammer and the subbase or base course materials molded in 5 layers with 4.5kg hammer. Therefore, the test results for different pavement layers are discussed here in as below:

4.3.4.1 CBR Test for Subgrade Soils:

The strength of the road subgrade for pavements is commonly assessed in terms of CBR and this is dependent on the type of soil, its density, and its moisture content. The results of the CBR tests in Table 4.16 show that samples from all selected stations (7+600 - 7+800, 13+600 - 13+800, 44+200 - 44+400, and 67+926) (existing with dominant alligator cracking and rutting distress types) have CBR values greater than 5%. According to the ERA specification, these samples indicates as good subgrade materials, or good existing subgrade and suitable borrow fill materials. The percent swell test results also are found below 1% which is an indication of less expansiveness of the soil, which is very good as a subgrade material.

Table 4-16: Results of CBR test for all subgrade materials

Station	**PCR**	**Sub grade CBR value**	**% Swell**
7+600 - 7+800 **Alaba to Mazoria**	Poor	7.6	0.45
13+600 - 13+800 **Alaba to Mazoria**	Very Poor	8.5	0.25
44+200 - 44+400 **Buge- Boditi**	Poor	9	0.25
67+926 **Boditi-Sodo**	No distress	11	0.22

Therefore, these values implies that the cause of pavement failure for very poor and poor pavement condition rating was not the subgrade soil.

4.3.4.2 CBR Test for Sub-base Materials

The CBR test for subbase materials was done in laboratory for different stations (7+600 - 7+800, 13+600 -13+800, 44+200-44+400, and 67+926) (existing with dominant alligator cracking and rutting distress types). In ERA standard, the minimum soaked CBR for sub base material shall be 30% when determined in accordance with the requirements of AASHTO T-193. The CBR shall be determined at a density of 95% of the maximum dry density when determined in accordance with the requirements of AASHTO T-180 method D. However, these tests shows the results of the CBR test of the samples from all selected stations have values less than the minimum requirement of ERA standard for subbase materials (30%) i.e. their values are less than 30%.

Table 4-17: Result of California bearing ratio test for all sub base materials

Station	**PCR**	**Sub base CBR value**	**% Swell**
7+600-7+800 **Alaba to Mazoria**	Poor	24	0.72
13+600-13+800 **Alaba-Mazoria**	Very Poor	25.5	0.69
44+200 - 44+400 **Buge-Boditi**	Poor	26	0.21
67+926 **Boditi-Sodo**	No distress	25	0.2

Therefore, these values indicate that the cause of pavement failure for very poor and poor pavement condition rating sections (existing with dominant alligator cracking and rutting distress types) can be the subbase material. The results of the CBR are shown in the above Table 4.17 and some detail analysis are attached in Appendix F.

4.3.4.3 CBR Test for Base Courses

The test results of the CBR for all selected sample stations are shown in the Table 4.18 below and some detail analysis is attached in Appendix F.

Table 4-18: Result of California bearing ratio test for all base course materials

Station	PCR	Base course CBR value	% Swell
7+600 - 7+800 **Alaba-Mazoria**	Poor	116	0
13+600 - 13+800 **Alaba-Mazoria**	Very Poor	116	0
44+200 - 44+400 **Buge-Boditi**	Poor	115	0
67+926 **Sodo Town**	No distress	116	0

In the whole stretch of the road, the crushed stone used as a base course materials which constructed with good quality and the results of the CBR test show that the samples from all stations having the CBR values greater than 100%. According to ERA standard these samples implies that the crushed stone have been used is a good base course materials and suitable for good performance of pavement road because their CBR values are greater than 100%. All of the percent swell tests results are 0%, which is an indication of no expansiveness of the material.

Therefore, the above CBR values indicate that the cause of pavement failure for very poor and poor pavement condition rating was not the base course material.

Table 4-19:Summary for Laboratory test results of subgrade material

No	Test Performed	Station	Standard Test Method	Accepted Criteria	Test Results	Pass (√) or Fail (X)
1	Plasticity Index	7+600-7+800	AASHTO T89&T90	<30	28.34	√
		13+600-13+800			25.66	√
		44+200-44+400			24.5	√
		67+926			22.84	√
2	Proctor Test (MDD/OMC)	7+600-7+800	AASHTO T99	N/A	1.54/32	N/A
		13+600-13+800			1.55/32.5	
		44+200-44+400			1.58/29	
		67+926			1.57/32	

3	CBR Test	7+600-7+800	AASHTO T-193	>4%	7.6	√
		13+600-13+800			8.5	√
		44+200-44+400			9	√
		67+926			11	√
4	Swell	7+600-7+800	AASHTO T-193	<1.5%	0.45	√
		13+600-13+800			0.25	√
		44+200-44+400			0.25	√
		67+926			0.22	√
5	Soil Classification	7+600-7+800	AASHTO M 145	AASHTO Classification System	A-7-6	N/A
		13+600-13+800			A-7-6	
		44+200-44+400			A-7-6	
		67+926			A-7-6	

Due to the above results in Table 4.19 and PCI data, this material have been constructed was in good quality and not a cause for distresses.

Table 4-20: Summary for Laboratory test results of subbase materials

No	Test Performed	Station	Standard Test Method	Accepted Criteria	Test Results	Pass (√)/ Fail (X)
1	Grading	7+600-7+800	AASHTO T27/T11	ERA Specification Table 5401/1	See the attached grading results in Appendix	X
		13+600-13+800				X
		44+200-44+400				X
		67+926				X
2	Plasticity Index	7+600-7+800	AASHTO T89&T90	<6 or12	24.5	X
		13+600-13+800			20.49	X
		44+200-44+400			24.5	X
		67+926			26.39	X
3	Proctor Test (MDD/OMC)	7+600-7+800	AASHTO T180	N/A	1.89/12.4	N/A
		13+600-13+800			1.86/14	
		44+200-44+400			1.86/14	
		67+926			1.85/15	
4	CBR Test	7+600-7+800	AASHTO T-193	>30%	24	X
		13+600-13+800			25.5	X
		44+200-44+400			26	X
		67+926			25	X

Due to the above results in Table 4.20 and PCI data, this material can be a cause for distresses and it needs further investigation with urgent remedies.

Table 4-21:Summary for Laboratory test results of base course material

No	Test Performed	Station	Standard Test Method	Accepted Criteria	Test Results	Pass (√)/ Fail (X)
1	Grading	7+600-7+800	AASHTO T27/T11	ERA Specification Table 5200/1	See the attached grading results in Appendix	√
		13+600-13+800				√
		44+200-44+400				√
		67+926				√
2	Plasticity Index	7+600-7+800	AASHTO T89&T90	Non-plastic or <6	1	√
		13+600-13+800			2	√
		44+200-44+400			4	√
		67+926			2	√
3	Proctor Test (MDD/OMC)	7+600-7+800	AASHTO T180	N/A	2.31/4.45	N/A
		13+600-13+800			2.31/4.5	
		44+200-44+400			2.33/4.1	
		67+926			2.34/3.6	
4	CBR Test	7+600-7+800	AASHTO T-193	>100%	116	√
		13+600-13+800			116	√
		44+200-44+400			115	√
		67+926			116	√

Due to the above results in Table 4.20 and PCI data, this base course material have been constructed was in good quality and not a cause for distresses.

In general, based on PCI values and a laboratory investigation for very poor and poor pavement condition rating sections (existing with dominant alligator cracking and rutting distress types) show that the tests with according to ERA and AASHTO Manuals and the PCI values with according to ASTM D6433, the quality of base and subgrade materials of the road are fulfilled the minimum requirement of ERA standard and they are not the causes of distress. Whereas, the sub-base material is not fulfilled the minimum requirement of ERA standard and it can be a cause for the failures. And also according to PCI finding most of the deterioration in road caused by repeated traffic load factors because of the PCI values, the alligator cracks and rutting are more dominant distress than other surveyed distresses on selected sections.

4.4 Maintenance & Repair Alternatives For Pavement Distresses

The pavement maintenance in general consists of all the routine repair tasks necessary to keep the pavement, under normal conditions of traffic and normal forces of nature, as nearly as possible in its as-constructed condition. All distress types with their severity level and their corresponding method of maintenance and repair is as shown below (Department of the Army (TM-5-624), 1995).

The following Tables shows maintenance option for cracking, surface deformation, disintegration, and surface defects with their severity level.

Table 4-22: Maintenance suggestion for Cracking

Pavement distress	Severity level	Maintenance Option
Alligator cracking	Low	Seal Coat
	Medium	Seal coat or Patching
	High	Thin hot-mix Overlay
Block cracking	Medium	Chip seal, seal coat or Thin hot-mix Overlay
Edge cracking	Low	Seal coat
	Medium	Patching
	High	Patching
Long & travs cracking	Low	Clean and Seal
	Medium	Clean and Seal or Full-depth crack Repair
	High	Full-depth crack Repair

Table 4-23: Maintenance suggestion for surface deformation

Pavement distress	Severity level	Maintenance Option
Shoving	Medium	Thin hot-mix Overlay
	High	Thin hot-mix Overlay
Depression	Low	Patching
Rutting	Low	Slurry Seal, Patching
	Medium	Slurry seal, Patching, or Thin hot-mix overlay
	High	Patching, or Thin hot-mix overlay
Swell	Low/Medium	Thin hot-mix overlay

Table 4-24: Maintenance suggestion for disintegration

Pavement distress	Severity level	Maintenance option
Potholes	Low	Patching
	Medium	Patching
	High	Patching
Raveling	Low	Crack sealing/ chip sealing
	Medium	Thin overlay
	High	Thin overlay

From the whole stretch of the road, the selected road sections condition needs full attention from the concerned bodies because the section indicates that the different failure types such as dominant alligator cracking, surface deformation (secondly dominant, rutting), disintegration and surface defects were existing. For Cracking, use one of this technique Clean and Seal, Full-depth crack Repair, seal coat, or thin hot-mix Overlay and for surface treatment technique such as seal coat, double chip seal, slurry seal, or thin hot-mix overlay by observing level of severity. And the sections with various sizes of potholes should be patched with good quality of asphalt. Certain types of traffic (tracked vehicles), Poor quality mixture, and segregation of the mix during construction are possible causes for Raveling. If the cause is superficial, a surface treatment will solve the problem. If poor drainage is causing a stripping problem, the drainage should be corrected.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

The pavement condition survey along the selected road sections shows that the different failure types such as alligator cracking, edge cracking, potholes, rutting, slippage cracking, block cracking, weathering and raveling, shoving, lane/ shoulder drop off, and depression were existing. From these the alligator cracking and rutting types of distress are majorly dominate types of distress along the stretch. The results of the selected road sections that are evaluated shows the PCI value range (25-79). From the field work pavement condition survey was collected as 0% Excellent, 11.67% Very Good, 16.67% Good, 43.33% fair, 26.67 % poor, 1.67% very poor, 0% failed. From which 46.33% of road sections are on fair condition, 26.67% of road sections are on poor condition, which indicates both condition dominate the whole road section. The average PCI values for selected Alaba-Mazoria, Shone-Buge, and Buge-Boditi are 47.96%, 47.42% and 48.96% respectively and these values shows that the selected road sections under fair surface condition. According to PCI finding most of the deterioration in road caused by repeated traffic load and subbase material factors because PCI value the alligator cracks and rutting are more dominant distress than other surveyed distresses on selected sections. To further improve continuous pavement investigation and monitoring are needed.

The PCI values implies the pavement performance condition of the total road length (68km) is under fair surface condition, that is the result of the three selected sections which weighted average values categorized under fair surface condition. And the remaining unselected two sections also concluded as similar condition i.e. which are under fair surface condition.

A laboratory investigation for very poor and poor pavement condition rating sections (existing with dominant alligator cracking and rutting distress types) show that the tests with according to ERA and AASHTO Manuals, the quality of base and subgrade materials of the road are fulfilled the minimum requirement of ERA standard and they are not the causes of distress. Whereas, the sub-base material is not fulfilled the minimum requirement of ERA standard and it can be a cause for the failures.

The road sections condition were full of dominant cracking, surface deformation, surface defects, and disintegration. Therefore, these failures can be maintained with observing level of severity as per maintenance option, in section 4.4. For Cracking, use one of this technique Clean and Seal, Full-depth crack Repair, seal coat, or thin hot-mix Overlay, for surface deformation use one this technique slurry seal, patching, or thin hot-mix overlay and for surface disintegration ad defects use one of this technique chip sealing, patching or thin overlay.

5.2 Recommendation

Lastly, the following recommendations may be considered:

- ✓ Accordingly urgent maintenances should be made by the ERA or any concerned entity.
- ✓ The additional traffic volume generated from Alaba-Hulbarege, which is from Sodo–Hosana–Addis Ababa, and durame-mazoria on Alaba–Sodo road stretch should be known or counted and included in the maintenance work.
- ✓ Periodic inspection is necessary to provide current and useful evaluation data. It is recommended that ratings be updated every year and periodic pavement maintenance practices should be employed to reduce aging of pavement failure.
- ✓ For quickly inspection and accurate data, new technology developments have produced a methodology that can quickly inspect roads and streets by using automated inspection equipment. The automated system has the ability to assess the condition of the pavement and use the resulting data to create and populate a database. This can be conducted at the same cost or less than manual survey procedures and the surveys become safer and less labor intensive. Therefore, it is recommended to consider using automated survey techniques to reduce labor needs and increase safety of any personnel that may conduct the surveys.
- ✓ The provision of necessary equipment for maintenance work in order to raise the level of efficiency of maintenance is also recommended.
- ✓ No detail material tests of the existing pavement structure are carried out under this research. Hence, it is also advisable to perform some additional non-destructive survey such as roughness survey and deflection survey and the destructive survey includes the DCP survey and other laboratory tests for aggregate such as LAA, ACV and for asphalt concrete such as ductility, marshal test, and others to further check of the structural capacity of the pavement.
- ✓ While maintaining, usage of efficient technics should be given due consideration

REFERENCE

AASHTO, (1986). Guide for design of pavement structures, American Association of State Highway and Transportation Officials.

AASHTO, (1993). Standard Specification for Transportation materials and methods of sampling and testing (Twenty first edition 2001 (Part 1)).

AASHTO, (1993). Standard Specification for Transportation materials and methods of sampling and testing (Twenty first edition 2001 (Part 2)).

Ali-Mohamed, A. Z. (2011). Evaluation Pavement Distresses Using Pavement Condition Index. A Thesis presented to Post Graduate University of Diponegoro, Semarang.

Ann, M., Johnson, P.E. (2000). Best Practices Handbook on Asphalt Pavement Maintenance, Centre for Transportation Studies, University of Minnesota.

Arora, K.R., Soil Mechanics and Foundation Engineering, Re-print Standard Publishers Distributer, Nai Sarak, Delhi, 2004.

Asphalt Institute, "Asphalt in Pavement Maintenance," Manual Series No.16 (MS-16), March 1983.

ASTM D 6433-07, (2007) Standard Practice for Roads and Parking Lots Pavement Condition Index Surveys, American Society for Testing and Materials.

ASTM D 6433-99, (1999) Standard Practice for Roads and Parking Lots Pavement Condition Index Surveys, American Society for Testing and Materials.

Barry, R.C.,(2006). Geotechnical Aspects of Pavements; Manual. U.S Department of Transport Federal Highway Administration, publication No. FHWA NHI-05-037.

Bashir, M. (2006). Effect of thermal cracking and environmental condition on asphalt Pavement, Master Dissertation, Al-mergheb University, Khoms.

Cline, D.G., Shahin, M., Burkhalter, J. (2002). Automated Data Collection For Pavement Condition Index Survey.

Department of the Army, Technical Manual (TM-5-624); Maintenance and Repair of Surface Defects, 1995.

Donald, W. (2013) (Donald Walker, former T.I.C. Director). " Pavement Surface Evaluation and Rating (PASER) Manual,".

Emer, T.Q. (2016). Pavement design and management lecture note. Jimma Institute of Technology.

Engineering Design Report for Alaba to Sodo Road Project, 2006.

ERA, (2002). Ethiopian Roads Authority Standard Manuals, Pavement Rehabilitation and Asphalt Overlay Manual.

ERA Specification, (2002). Geometric Design Manual. Federal Democratic Republic of Ethiopia. Ethiopia Road Authority.

ERA, (2002). Ethiopian Roads Authority Standard Manuals, Pavement Design Manual: Flexible Pavements and Gravel Roads, Volume I.

Feng Hand Dar.H,(2009). Effects of surface preparation, thickness, and material on asphalt pavement overlay transverse crack propagation, NRC Research Press, Can. J. Civ. Eng. Vol. 36. P 1411.

Fiker, A. (2005). Pavement Distresses on Addis Ababa City Arterial Roads, Causes and Maintenance Options. A Thesis Presented to the School of Graduate Studies Addis Ababa University Faculty of Technology.

Gary, M., Hao, Z., Qinghin, C. (2009). Stochastic Modeling for Pavement Warranty cost Estimation, Journal of construction engineering and management.135.

GTC (Geotechnical Testing Center), (1998), guide of pavement defects and proposals for maintenance, Riyadh – KSA.

Gupta, K.A., Adlinge, S.S. (1998). Pavement Deterioration and its Causes. Journal of Mechanical & Civil Engineering. Shivaji University, India.

Huang, Y.H.(2004). “Pavement Analysis and Design,” Prentice Hall, New jersey.

Lavin, P.G, (2003). Asphalt Pavement, Taylor & Francis e-Library, London &New York.

Krishna, R. Engineering Properties of Soils Manual Based on Laboratory Testing, UIC

Luo, Z. (2005). Flexible Pavement Condition Model Using Clusterwise Regression and Mechanistic-Empirical Procedure for Fatigue Cracking Modeling, Ph.D Dissertation, The University of Toledo, December.

Maharaj, M. (1996). National Transport Policy White Paper, Republic of South Africa. http://www.gov.za/documents/National-transport-policy-white-paper

Murthy, V.N.S. Geotechnical Engineering: Principles and Practices of Soil Mechanics and Foundation Engineering. New York. P71-72

SABA Engineering PLC, (2002). Central Material Testing Laboratory; Manual Of Soil Testing, Part 1, Addis Ababa.

Structural Pavement Design lecture note, (2011/12). Hawassa University IOT department of Civil and Urban Engineering.

Walker, D. (2002). Pavement Surface Evaluation and Rating, University of Wisconsin–Madison.

Weil, R. (2009). State of the Streets in San Carlos, Report to Council, California.

Wilson, S.K., Wasike, (2001). Road Infrastructure Policies in Kenya: Historical Trends and Current Challenges, KIPPRA Working Paper No.1.

Yosef, (2012). Highway Engineering-II lecture note. Arbaminch University, Institute of Technology

APPENDIX A: SOME OF PAVEMENT CONDITION INDEX (PCI) CALCULATION

ASPHALT SURFACE ROADS CONDITION SURVEY DATA SHEET FOR SAMPLE UNIT

Branch: **Alaba-Sodo** Station Interval: 4+800 – 5+000 Section: Alaba-Mazoria

No. of sample: **4th** Surveyed By: **Asrat** Date: July 15, 2016

200m

7m Carriageway width

Direction of survey → N

1 Alligator/Fatigue cracking
2 Bleeding
3 Block cracking
4 Bumps and sags
5 corrugation
6 Depression
7 Edge cracking
8 Reflection cracking
9 Lane shoulder drop
10 Longitudinal & Transverse
11 Patching &Utility patch
12 Polished Aggregate
13 Potholes
14 Rutting
15 Railroad crossing
16 Shoving
17 Slippage
18 Swell
19 Raveling &Weathering

DISTRESS SEVERITY	QUANTITY												TOTAL	DENSITY	DEDUCT VALUE
1M	3.5*7	2.7*8	3.5*7.8	3.5*6.2									95.1	6.79	28
1H	3*8.5	3.5*6.8	3.5*5										66.8	4.77	35.5
14L	3.5*7	1.2*8											34.1	2.44	16
6L	2.5*5.6												14	1	4

$m = 1 + \left(\frac{9}{98}\right) * (100 - HDV) = 1 + \left(\frac{9}{98}\right) * (100 - 35.5) = 6.9 < 10$ OK!!

35.5, 28, 16, 4

If less than m deduct values are available, all of the deduct values are used. (m=6.9>4)

#	Deduct value						Total	q	CDV
1	35.5	28	16	4			83.5	4	47.5
2	35.5	28	16	2			81.5	3	48
3	35.5	28	2	2			67.5	2	47.5
4	35.5	2	2	2			41.5	1	42
5									
6									

Max CDV = **48**

PCI = 100 – Max CDV

PCI = **52**

Rating = Fair

ASPHALT SURFACE ROADS CONDITION SURVEY DATA SHEET FOR SAMPLE UNIT

Branch: **Alaba-Sodo** Station Interval: 12+400-12+600
No. of sample: **11th** Surveyed By: **Asrat** Date: July 15, 2016

200m
Carriageway width
7m
Direction of survey → N

1 Alligator/Fatigue cracking; 6 Depression; 11 Patching &Utility patch; 16 Shoving
2 Bleeding; 7 Edge cracking; 12 Polished Aggregate; 17 Slippage
3 Block cracking; 8 Reflection cracking; 13 Potholes; 18 Swell
4 Bumps and sags; 9 Lane shoulder drop; 14 Rutting; 19 Raveling &Weathering
5 corrugation; 10 Longitudinal & Transverse; 15 Railroad crossing

DISTRESS SEVERITY	QUANTITY												TOTAL	DENSITY	DEDUCT VALUE
1M	2.6*14.6	1.8*7.4	2.4*14.4										85.84	6.13	40.2
14L	2.8*18.3												51.24	3.66	18.3
7M	3.1	2.4											5.5	0.39	5
9M	14.4	21.3	25										60.7	4.34	5.2
13M	3												3	0.21	10
13H	4												4	0.29	30.4

$m = 1 + \left(\frac{9}{98}\right) * (100 - HDV) = 1 + \left(\frac{9}{98}\right) * (100 - 40.2) = 6.5 < 10$ OK!!

40.2, 30.4, 18.3, 10, 5.2,5

If less than m deduct values are available, all of the deduct values are used. (m=6.5>6)

#	Deduct value						Total	q	CDV
1	40.2	30.4	18.3	10	5.2	5	109.1	6	53
2	40.2	30.4	18.3	10	5.2	2	106.1	5	54
3	40.2	30.4	18.3	10	2	2	102.9	4	58
4	40.2	30.4	18.3	2	2	2	94.9	3	59
5	40.2	30.4	2	2	2	2	78.6	2	56
6	40.2	2	2	2	2	2	50.2	1	50

Max CDV = **59**

PCI = 100 – Max CDV

PCI = **41**

Rating = Fair

ASPHALT SURFACE ROADS CONDITION SURVEY DATA SHEET FOR SAMPLE UNIT	**200m** 7m Carriageway width Direction of survey → N
Branch: **Alaba-Sodo** Station Interval: 13+600-13+800 Section: **Alaba-Mazoria** No. of sample: add-**12th** Surveyed By: **Asrat** Date: July 17, 2016	

1 Alligator/Fatigue cracking, 2 Bleeding, 3 Block cracking, 4 Bumps and sags, 5 corrugation, 6 Depression, 7 Edge cracking, 8 Reflection cracking, 9 Lane shoulder drop, 10 Longitudinal & Transverse, 11 Patching &Utility patch, 12 Polished Aggregate, 13 Potholes, 14 Rutting, 15 Railroad crossing, 16 Shoving, 17 Slippage, 18 Swell, 19 Raveling &Weathering

DISTRESS SEVERITY	QUANTITY											TOTAL	DENSITY	DEDUCT VALUE
1M	3.2*14.6	4.6*16	3.5*12.5									164.07	11.72	46.5
1L	3.5*12.4											43.4	3.10	22.5
14L	3.2*23.7	1.4*9										88.44	6.32	23
14H	3.2*10.5											33.6	2.4	38
13M	3											3	0.21	10.2

$m = 1 + \left(\frac{9}{98}\right) * (100 - HDV) = 1 + \left(\frac{9}{98}\right) * (100 - 46.5)$ = 5.9 < 10 OK!!

46.5, 38, 23, 22.5, 10.2

If less than m deduct values are available, all of the deduct values are used. (m=5.9>5)

#	Deduct value						Total	q	CDV
1	46.5	38	23	22.5	10.2		140.2	5	72
2	46.5	38	23	22.5	2		132	4	75
3	46.5	38	23	22.5	2		111.5	3	71
4	46.5	38	2	2	2		90.5	2	65
5	46.5	2	2	2	2		54.5	1	54
6									

Max CDV = **75**

PCI =100 – Max CDV

PCI = **25**

Rating = Very Poor

ASPHALT SURFACE ROADS CONDITION SURVEY DATA SHEET FOR SAMPLE UNIT	**200m** Carriageway width **7m** Direction of survey → N
Branch: **Alaba-Sodo**Station Interval:16+200-16+400 Section:**Alaba-Mazoria** No. of sample: add-**14th** Surveyed By: **Asrat** Date: July 17, 2016	

	1 Alligator/Fatigue cracking, 2 Bleeding, 3 Block cracking, 4 Bumps and sags, 5 corrugation, 6 Depression, 7 Edge cracking, 8 Reflection cracking, 9 Lane shoulder drop, 10 Longitudinal & Transverse, 11 Patching &Utility patch, 12 Polished Aggregate, 13 Potholes, 14 Rutting, 15 Railroad crossing, 16 Shoving, 17 Slippage, 18 Swell, 19 Raveling &Weathering

DISTRESS SEVERITY	**QUANTITY**											**TOTAL**	**DENSITY**	**DEDUCT VALUE**
1L	2.8*10.6	1.4*6.4										38.64	2.76	19.2
1M	2.3*10.3											23.69	1.69	14.5

$m = 1 + \left(\frac{9}{98}\right) * (100 - HDV) = 1 + \left(\frac{9}{98}\right) * (100 - 19.2)$ = 8.4 < 10 OK!!

19.2, 14.5

If less than m deduct values are available, all of the deduct values are used. (m=5.9>5)

#	Deduct value							Total	q	CDV
1	19.2	14.5						33.7	2	24
2	19.2	2						21.2	1	22
3										

Max CDV = **24** PCI =100 – Max CDV = 76 Rating = Very Good

APPENDIX B: DEDUCT VALUE CURVE FOR ASPHALT CONCRETE PAVEMENT

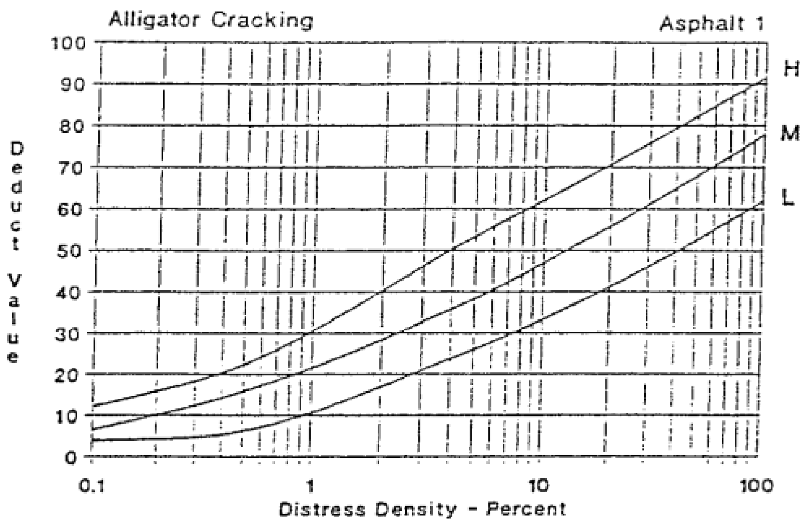


Fig: Alligator Cracking

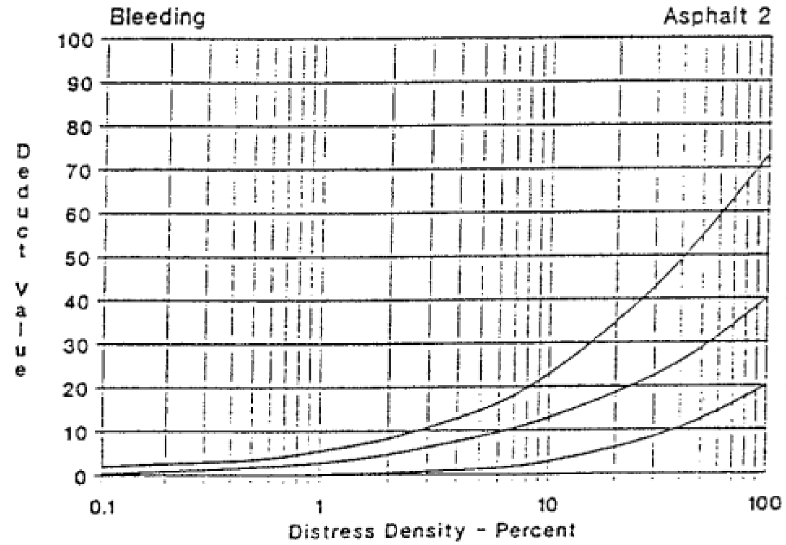


Fig: Bleeding

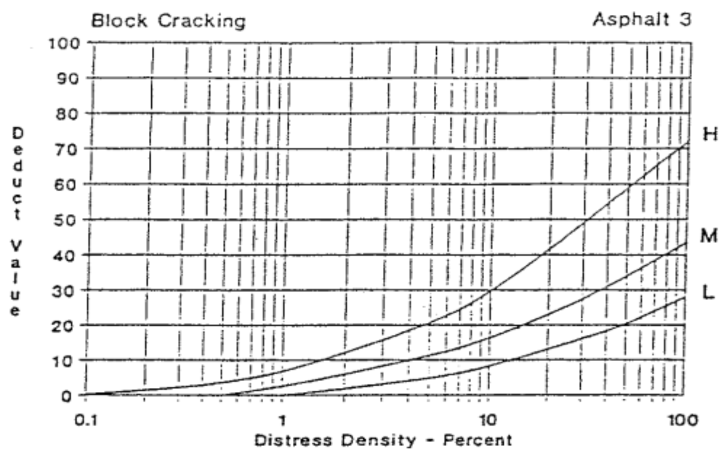


Fig: Block Cracking

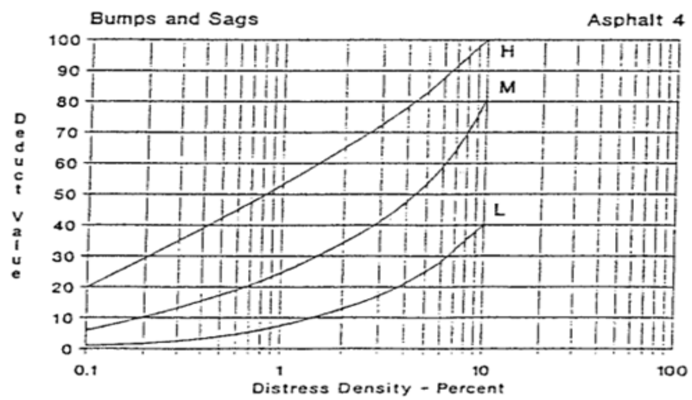


Fig: Bumps and Sags

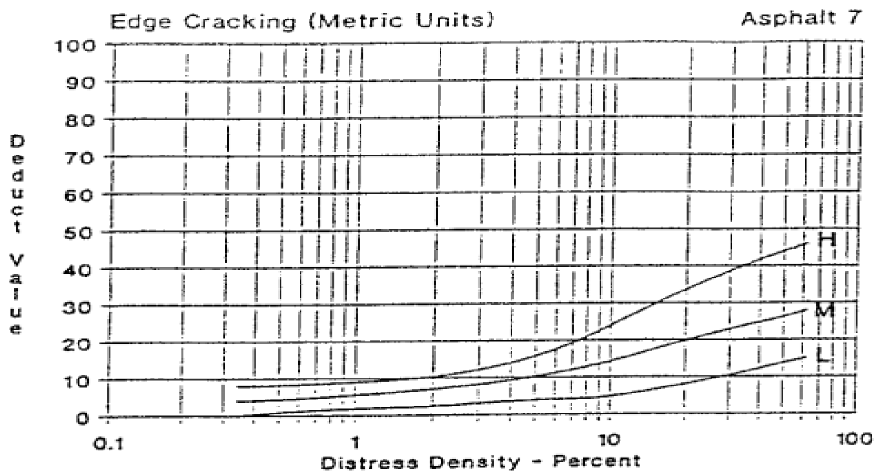


Fig: Edge Cracking (metric units)

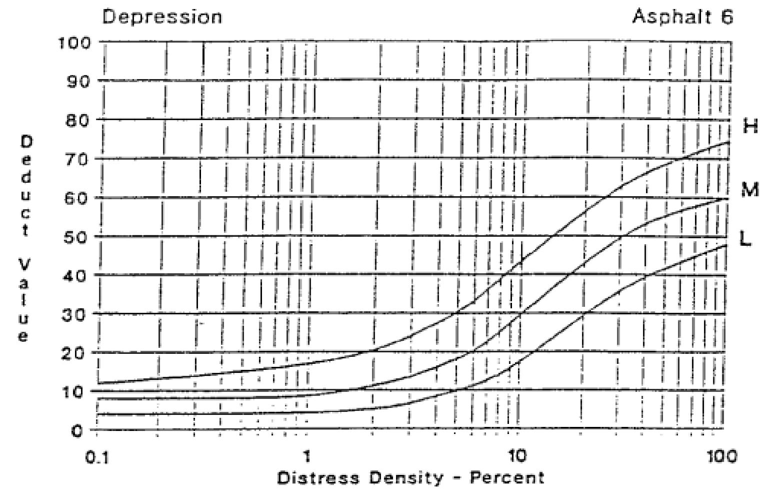


Fig: Depression

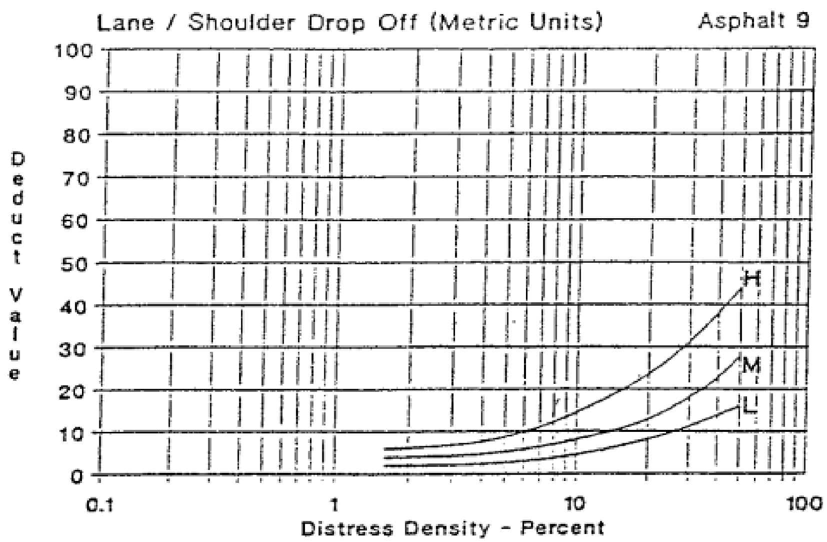


Fig: Lane/Shoulder Drop-Off (metric units)

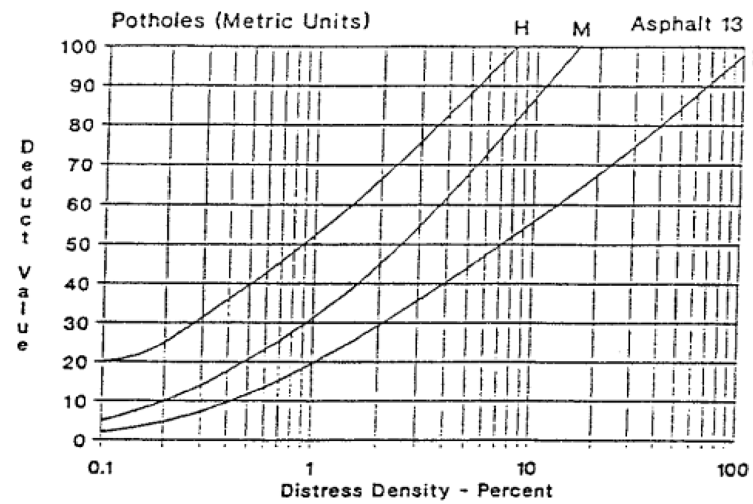


Fig: Potholes (metric units)

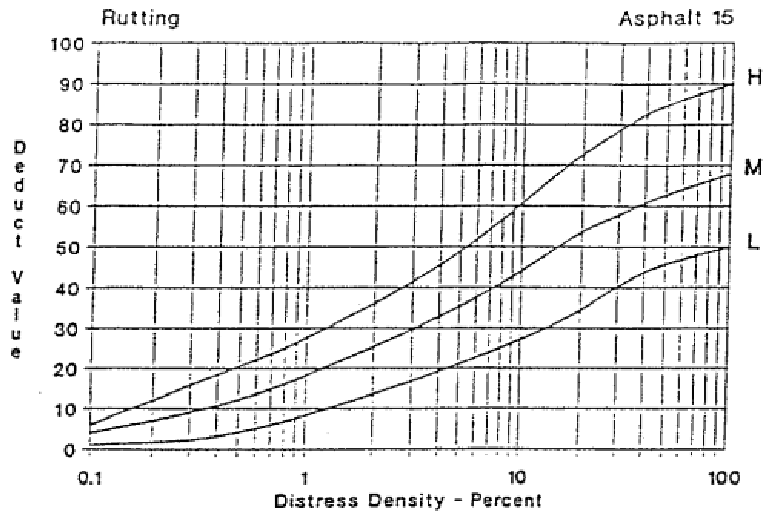


Fig: Rutting

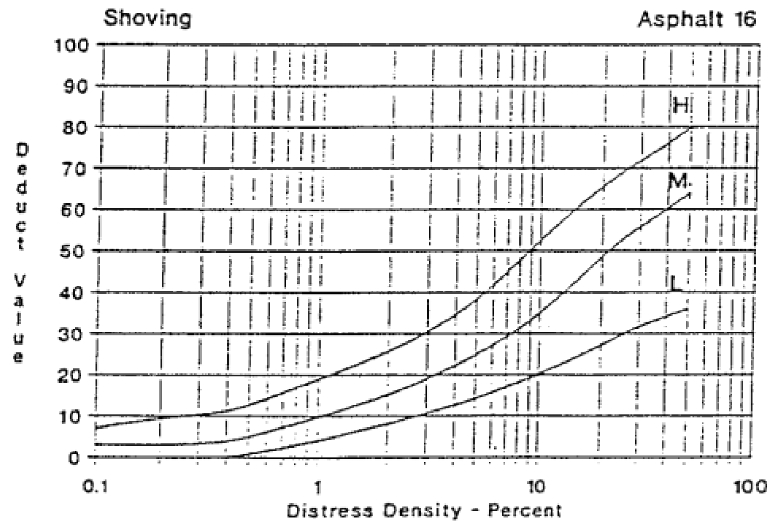


Fig: Shoving

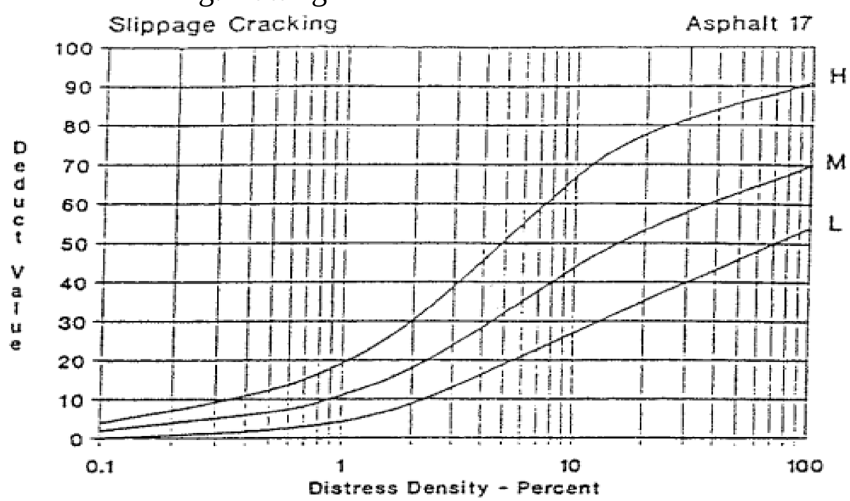


Fig: Slippage Cracking

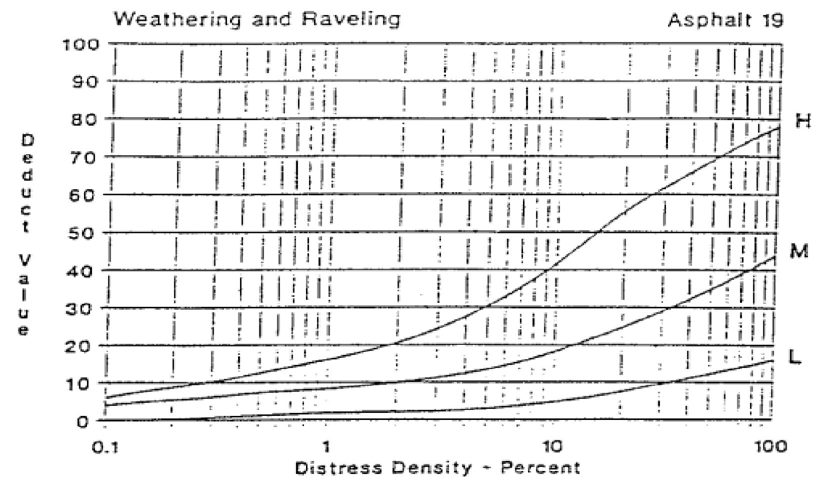


Fig: Weathering and Raveling

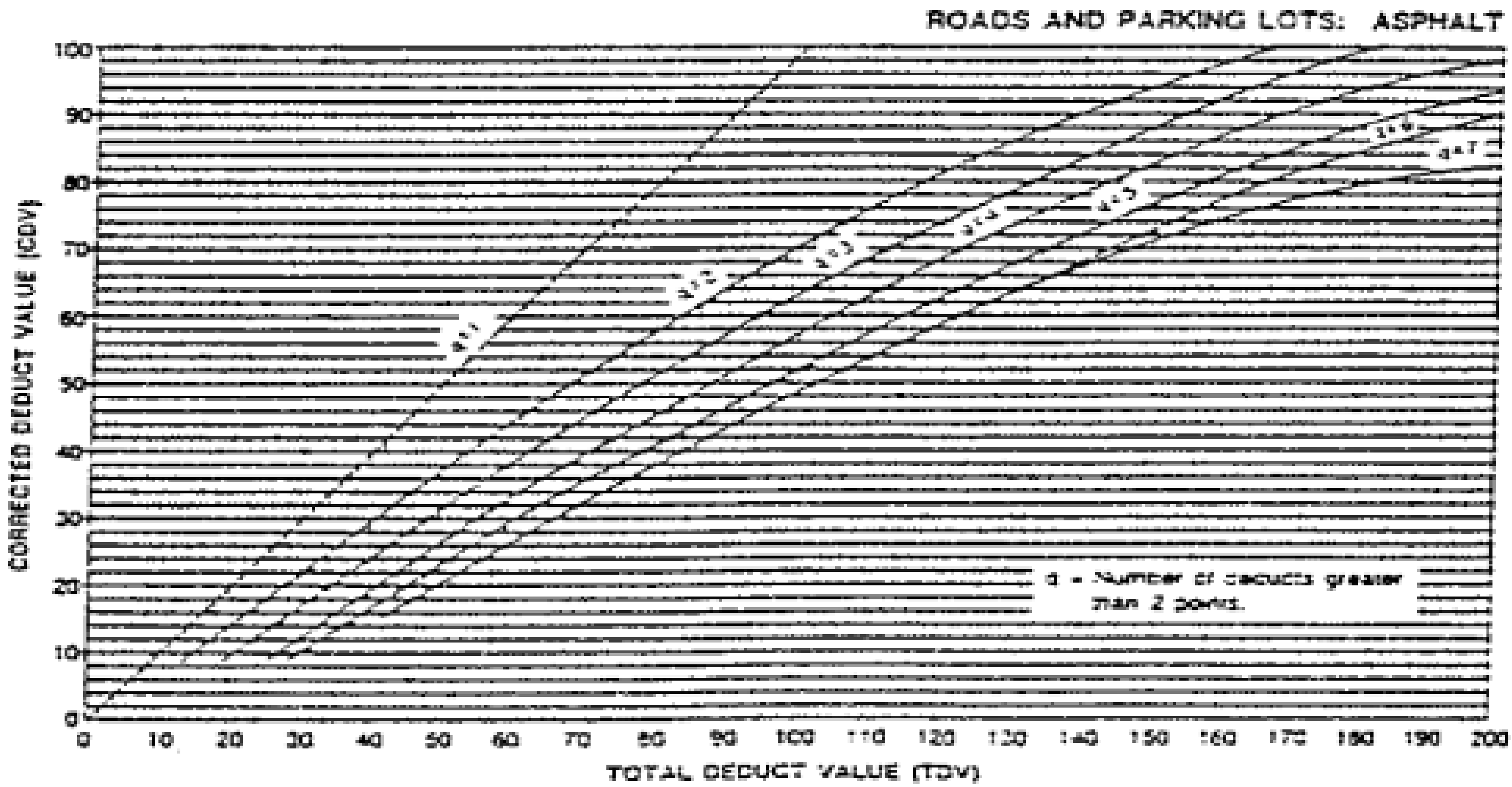


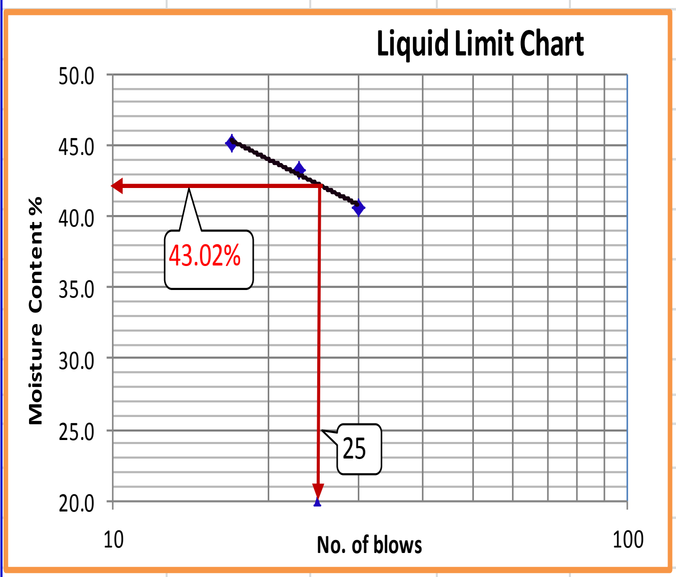
Fig: Total Deduct Value

APPENDIX C: SOME OF ATTERBERG'S LIMIT DETERMINATION

SOIL CONSISTENCY TEST RESULT (TEST METHOD: AASHTO T-89, & T-90)

Sample Station	7+600-7+800	Sample Date	August 26,2016
Representing Section	Alaba-Mazoria	Test Date	August 29,2016
Material Source	Existing Road	Sample No	one
Sample Type	Subgrade soil		
Visual description	Brown Soil	Depth	

	Liquid Limit				Plastic Limit	
Container Number	B	7	C		A	W
No. of Blows	17	23	30		_	_
Wt. of Container + Wet Soil (g) = (W_1)	57.3	57.7	57.6		19.8	21.4
Wt. of Container + Dry Soil (g) = (W_2)	45.6	46.2	46.5		19.3	20.8
Wt. of Container (g) = (W_3)	19.7	19.6	19.2		17.6	18.6
Weight of Moisture (g) = (W_1 - W_2) = A	11.7	11.5	11.1		0.5	0.6
Weight of Dry Soil (g) = (W_2 - W_3) = B	25.9	26.6	27.3		1.7	2.2
Moisture Content (%) = (A / B)x 100	45.2	43.2	40.7		29.4	27.3
Av.			43.02	AV. Plas. Lim.		28.34

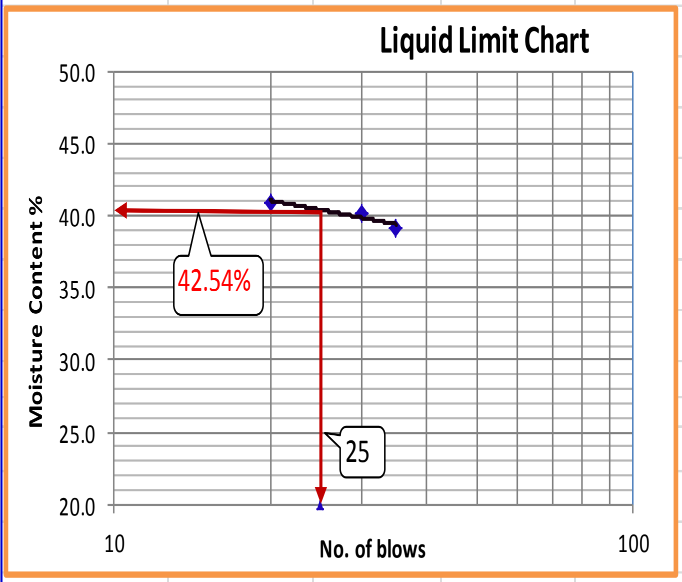


Liquid limit %	LL	43.02
Plastic limit %	PL	28.34
Plasticity index %	(LL-PL)=PI	15

SOIL CONSISTENCY TEST RESULT (TEST METHOD: AASHTO T-89, & T-90)

Sample Station	44+200-44+400	Sample Date	
Representing Section	Buge-Boditi	Test Date	
Material Source	Existing Road	Sample No	
Sample Type	Subbase/ Red ash	Test No	
Visual description	Natural gravel	Depth	

	Liquid Limit				Plastic Limit	
Container Number	N1	3	T		U1	W
No. of Blows	20	30	35		_	_
Wt. of Container + Wet Soil (g) = (W_1)	53.8	54.1	53.7		20.1	21.7
Wt. of Container + Dry Soil (g) = (W_2)	43.6	44.0	43.4		19.6	21.1
Wt. of Container (g) = (W_3)	18.7	18.9	17.1		17.6	18.6
Weight of Moisture (g) = (W_1 - W_2) = A	10.2	10.1	10.3		0.5	0.6
Weight of Dry Soil (g) = (W_2 - W_3) = B	24.9	25.1	26.3		2.0	2.5
Moisture Content (%) = (A / B)x 100	41.0	40.2	39.2		25.0	24.0
Av.			40.12	AV. Plas. Lim.		24.50

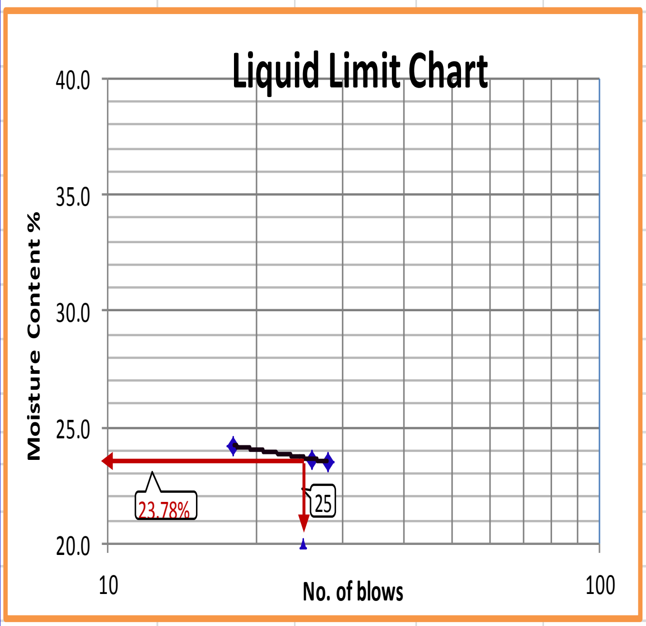


Liquid limit %	LL	40.12
Plastic limit %	PL	24.50
Plasticity index %	(LL-PL)=PI	16

SOIL CONSISTENCY TEST RESULT (TEST METHOD: AASHTO T-89, & T-90)

Sample Station	7+600-7+800	Sample Date:	
Representing Section	Alaba-Mazoria	Test Date:	
Material Source	Existing Road	Sample No : One	
Sample Type	Base course		
Visual description	Crushed Aggregate	Depth	

	Liquid Limit				Plastic Limit	
Container Number	N	B	6		34	w
No. of Blows	18	26	28		–	–
Wt. of Container + Wet Soil (g) = (W_1)	51.6	56.7	53.6		23.5	22.3
Wt. of Container + Dry Soil (g) = (W_2)	44.8	49.5	47.2		22.8	21.6
Wt. of Container (g) = (W_3)	16.7	19.0	20.0		19.7	18.6
Weight of Moisture (g) = (W_1 - W_2) = A	6.8	7.2	6.4		0.7	0.7
Weight of Dry Soil (g) = (W_2 - W_3) = B	28.1	30.5	27.2		3.1	3.0
Moisture Content (%) = (A / B)x 100	24.2	23.6	23.5		22.6	23.3
				AV. Plas. Lim.	23	

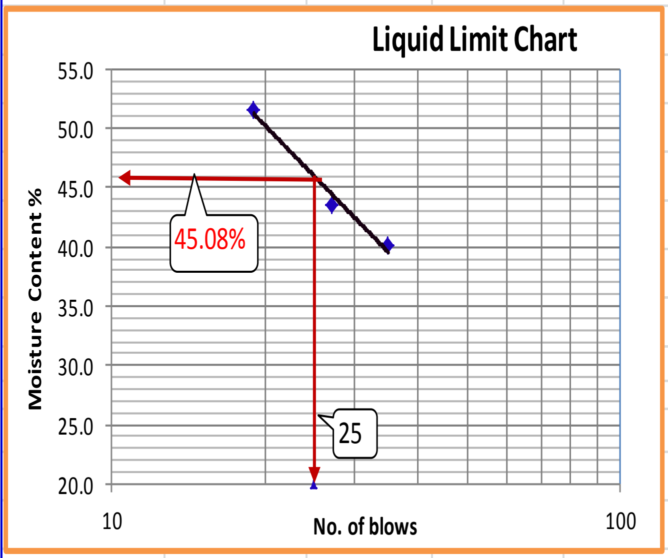


Liquid limit %	LL	23.78
Plastic limit %	PL	23
Plasticity index %	(LL-PL)=PI	1

SOIL CONSISTENCY TEST RESULT (TEST METHOD: AASHTO T-89, & T-90)

Sample Station	67+926	Sample Date	
Representing Section	Boditi- Sodo	Test Date	
Material Source	Existing Road	Sample No: Four	
Sample Type	Subgrade		
Visual description	Light Brown	Depth	

	Liquid Limit				Plastic Limit	
Container Number	C	B	A		4	7
No. of Blows	19	27	35		_	_
Wt. of Container + Wet Soil (g) = (W_1)	52.1	52.0	51.5		16.0	22.6
Wt. of Container + Dry Soil (g) = (W_2)	40.9	42.2	41.8		15.4	22.0
Wt. of Container (g) = (W_3)	19.2	19.7	17.6		12.5	19.6
Weight of Moisture (g) = (W_1 - W_2) = A	11.2	9.8	9.7		0.6	0.6
Weight of Dry Soil (g) = (W_2 - W_3) = B	21.7	22.5	24.2		2.9	2.4
Moisture Content (%) = (A / B)x 100	51.6	43.6	40.1		20.7	25.0
Av.			45.08	AV. Plas. Lim.		22.84



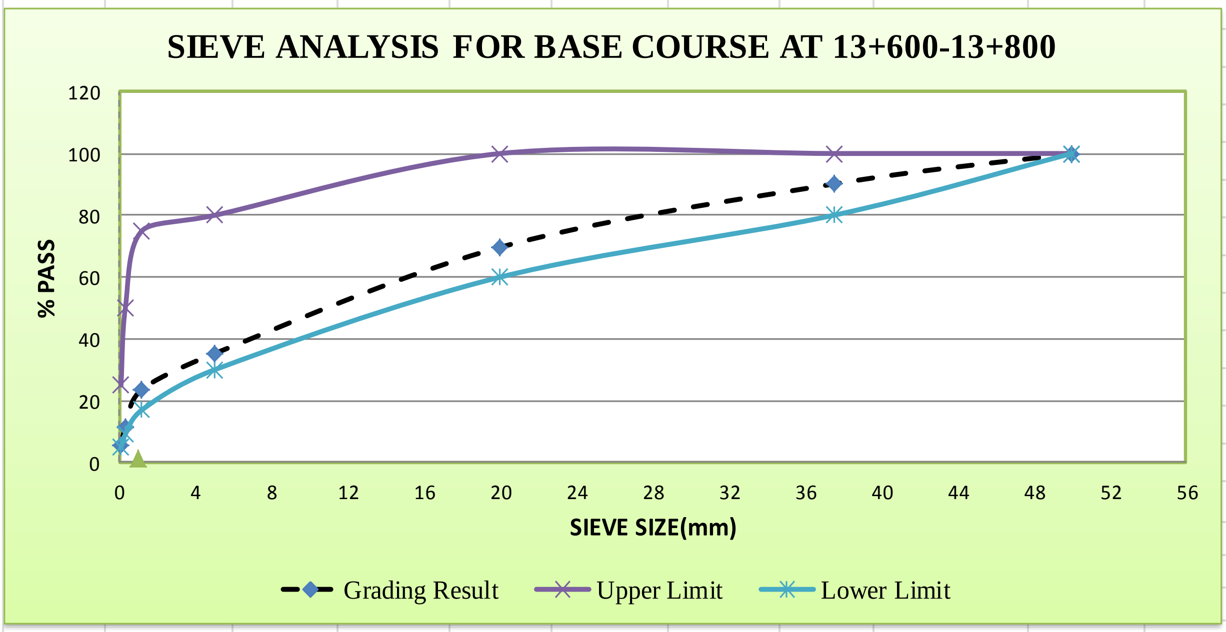
Liquid limit %	LL	45.08
Plastic limit %	PL	22.84
Plasticity index %	(LL-PL)=PI	22

APPENDIX D: DETERMINATION OF GRAIN SIZE DISTRIBUTION

Station	**13+600-13+800**	Sampling Date:	**August 26,2016**
Sample taken	**From Existing Road**	Test date :	**August 30, 2016**
Material Type	**Base Course**	Sample No:	**Two**
Visual Discription	**Crushed Aggregate**	Test condition:	**Dry**
Representing sectior	**Alaba-Mazoria**		

PARTICLE SIZE DISTRIBUTION BY SIEVING TEST METHODS: AASHTO 27/11

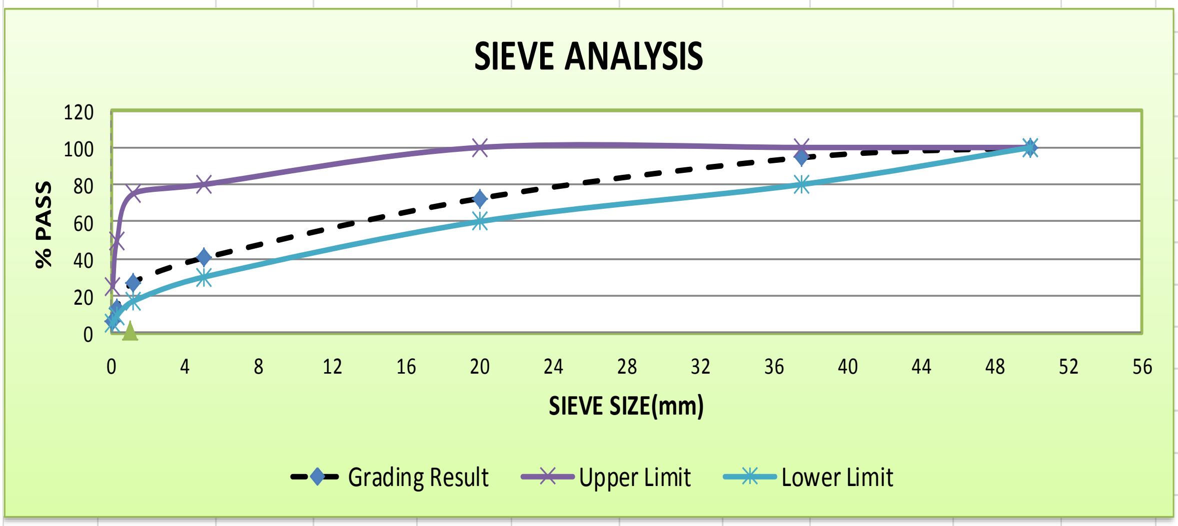
Trial one				Trial two							
Before	7807			Before	7771						
Sieve Size (mm)	**Wt. of Sample Retained (g)**	**% Retaind**	**% Pass**	**Sieve Size (mm)**	**Wt. of Sample Retained (g)**	**% Retaind**	**% Pass**	Average % retained	**Average % pass**	**Lower limit**	**Upper limit**
63.00	0.00		**100.00**	**63.00**	**0.00**		**100.00**				
50.00	0.00	-	**100.00**	**50.00**	0	-	**100.00**	0.00	**100.00**	**100**	**100**
37.5	964	12.35	**87.65**	37.5	575	7.40	**93**	9.87	**90.13**	80	100
20.0	1202	15.40	**72.26**	20.0	1984	25.53	**67**	20.46	**69.66**	60	100
5.0	2894	37.07	**35.19**	5.0	2467	31.75	**35**	34.41	**35.26**	30	80
1.18	965	12.36	**22.83**	1.18	846	10.89	**24.4**	11.62	**23.63**	17	75
0.30	982	12.58	**10.25**	0.30	960	12.35	**12.1**	12.47	**11.17**	9	50
0.075	398	5.10	**5.15**	0.075	489	6.29	**5.8**	5.70	**5.47**	5	25
Pan	402	5.15		Pan	450	5.79					
Total	7807			Total	7771						



Station	67+926	**Sampling Date:**	August 26,2016
Sample taken	From Existing Road	**Test date :**	August 30, 2016
Material Type	Base Course	**Sample No:**	Two
Visual Discription	Crushed Aggregate	**Test condition:**	Dry
Representing secti	Alaba-Mazoria		

PARTICLE SIZE DISTRIBUTION BY SIEVING TEST METHODS: AASHTO 27/11

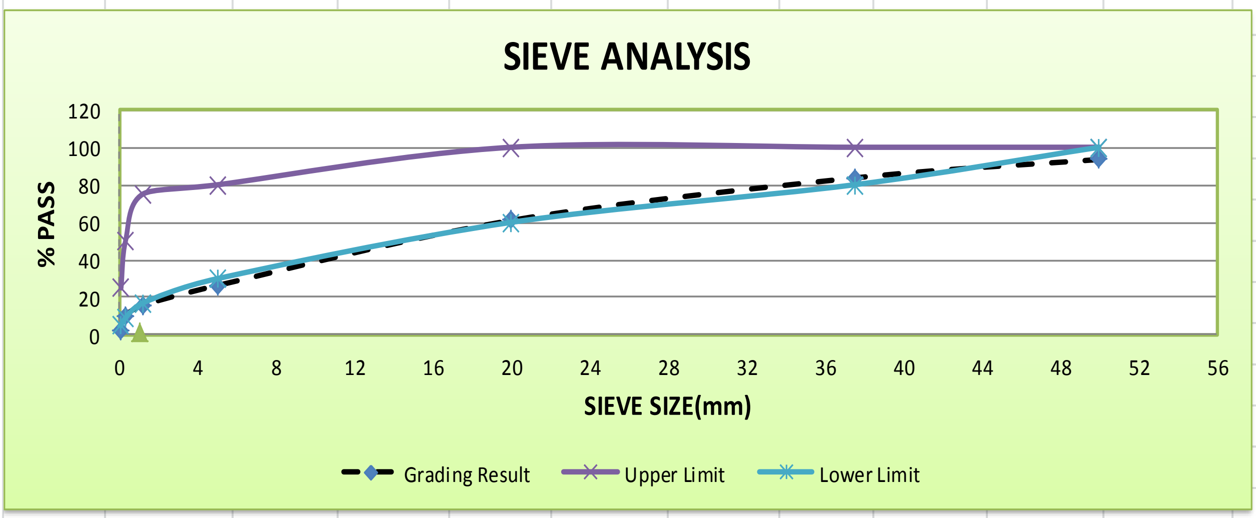
Before	8399			Before	7852						
Trial one				**Trial two**							
Sieve Size (mm)	**Wt. of Sample Retained (g)**	**% Retaind**	**% Pass**	**Sieve Size (mm)**	**Wt. of Sample Retained (g)**	**% Retaind**	**% Pass**	Average % retained	**Average % pass**	**Lower limit**	**Upper limit**
63.00	0.00		**100.00**	**63.00**	**0.00**		**100.00**				
50.00	0.00	-	**100.00**	**50.00**	0	-	**100.00**	0.00	**100.00**	**100**	**100**
37.5	664	7.91	**92.09**	37.5	246	3.13	**97**	5.52	**94.48**	80	100
20.0	1546	18.41	**73.69**	20.0	2013	25.64	**71**	22.02	**72.46**	60	100
5.0	2724	32.43	**41.25**	5.0	2504	31.89	**39**	32.16	**40.30**	30	80
1.18	1142	13.60	**27.66**	1.18	1001	12.75	**26.6**	13.17	**27.13**	17	75
0.30	1247	14.85	**12.81**	0.30	984	12.53	**14.1**	13.69	**13.44**	9	50
0.075	678	8.07	**4.74**	0.075	583	7.42	**6.6**	7.75	**5.69**	5	25
Pan	398	4.74		Pan	521	6.64					
Total	8399			Total	7852						



Station	67+926	**Sampling Date:**	26-Aug-16
Sample taken	From Existing Road	**Test date :**	30-Aug-16
Material Type	Subbase	**Sample No:**	Two
Visual Discription	Red ash	**Test condition :**	Dry
Representing section	Boditi-Sodo Town		

PARTICLE SIZE DISTRIBUTION BY SIEVING TEST METHODS: AASHTO 27/AASHTO 11

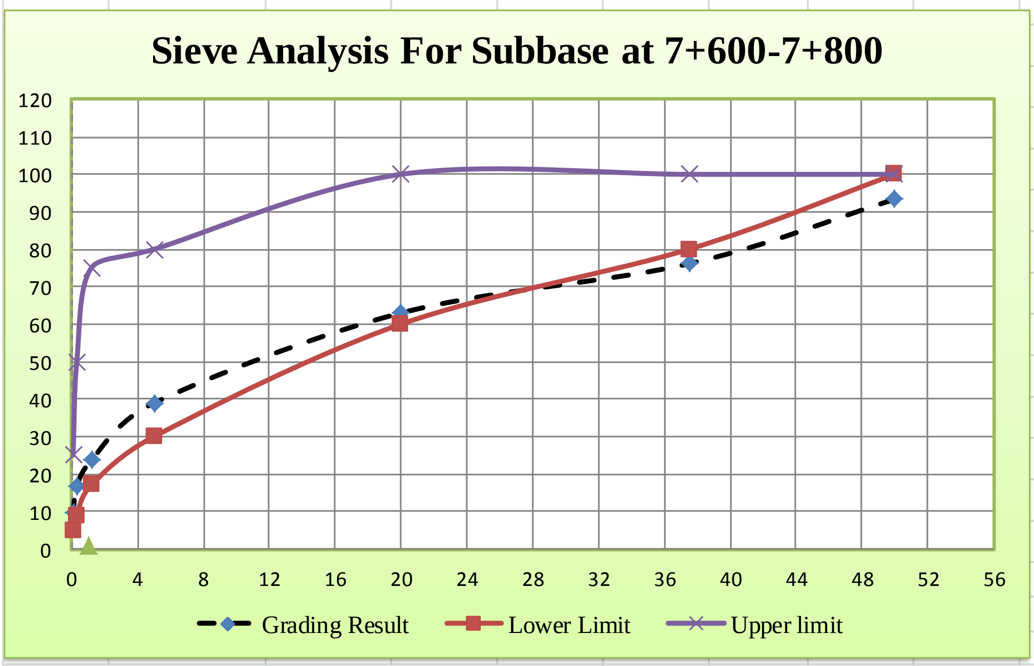
Before	10388			Before	10891						
Trial one				**Trial two**							
Sieve Size (mm)	**Wt. of Sample Retained (g)**	**% Retaind**	**% Pass**	**Sieve Size (mm)**	**Wt. of Sample Retained (g)**	**% Retaind**	**% Pass**	Average % retained	Average % pass	**Lower limit**	**Upper limit**
63.00	0		**100.00**	**63.00**	**0**						
50.00	546	13.02	**86.98**	**50.00**	446	10.58	**100.00**	11.80	93.49	**100**	**100**
37.5	1006	9.68	**77.30**	37.5	1097	10.07	90	9.88	83.61	80	100
20.0	2345	22.57	**54.73**	20.0	2465	22.63	67	22.60	61.01	60	100
5.0	3645	35.09	**19.64**	5.0	3729	34.24	33	34.66	26.35	30	80
1.18	1234	11.88	**7.76**	1.18	1010	9.27	23.8	10.58	15.77	17	75
0.30	546	5.26	**2.50**	0.30	645	5.92	17.9	5.59	10.18	9	50
0.075	754	7.26	**-4.76**	0.075	956	8.78	9.1	8.02	2.16	5	25
Pan	312	3.00		Pan	543	4.99					
Total	10388			Total	10891						



Sample Station	7+600-7+800	Sample Date	
Representing Section	Alaba-Mazoria	Test Date	
Material Source	Existing Road	Sample No	
Sample Type	Subbase/ Red ash	Test No	
Visual description	Natural gravel		

PARTICLE SIZE DISTRIBUTION BY SIEVING TEST METHODS: AASHTO 27/AASHTO 11

Befor	9970			Befor	9920						
Trial one				trial two							
Sieve Size (mm)	Wt. of Sample Retained (g)	% Retaind	% Pass	Sieve Size (mm)	Wt. of Sample Retained (g)	% Retaind	% Pass	Average % retained	Average % pass	Lower limit	Upper limit
63.00	0.00		100.00	63.00	0.00	0.00	100.00				
50.00	568.00	5.70	94.30	50.00	759	7.65	92.35	6.67	93.33	100	100
37.5	1564	15.69	78.62	37.5	1853	18.68	73.67	17.18	76.14	80	100
20.0	1195	11.99	66.63	20.0	1421	14.32	59.34	13.16	62.99	60	100
5.0	2534	25.42	41.21	5.0	2246	22.64	36.70	24.03	38.96	30	80
1.18	1597	16.02	25.20	1.18	1456	14.68	22.03	15.35	23.61	17	75
0.30	683	6.85	18.35	0.30	685	6.91	15.12	6.88	16.73	9	50
0.075	865	8.68	9.67	0.075	543	5.47	9.65	7.07	9.66	5	25
Pan	964	9.67		Pan	957	9.65					
Total	9970			Total	9920						



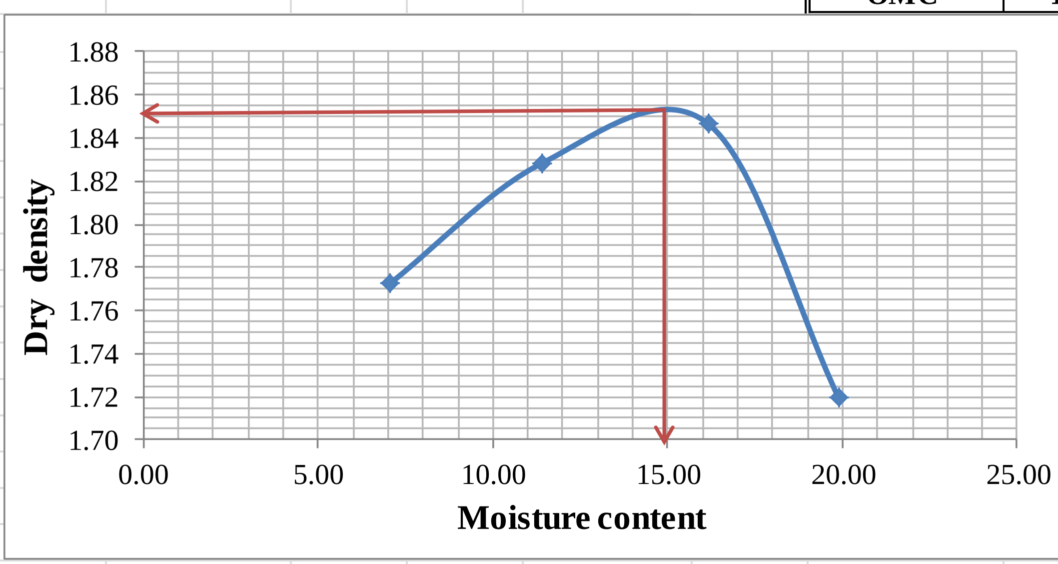
APPENDIX E: SOME OF COMPACTION TESTS

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

Sample Station	**67+926**		Sampling Date	**3/9/2016**
Materail Type	**Subbase**		Testing Date	**12/9/2016**
Visual Discription	**Res ash**		Sample Taken	**Existing road**
Representing section	**Boditi-Sodo**		Sample No	**Four**

Trail	1	2	3	4		
Weight of Mould + Wet soil (g)	9120	9405	9625	9455		
Weight of Mould (g)	5250	5250	5250	5250		
Weight of Wet soil (g)	3870	4155	4375	4205		
Volume of Mould (mm3)	2039.7	2039.7	2039.7	2039.7		
Wet density (g / cm3)	1.90	2.04	2.14	2.06		

Moisture can	R	W	S1	A		NMC
Weight of Wet soil + cont. (g)	500.0	500.0	**500**	**500**		**500**
Weight of Dry soil + cont. (g)	473.0	458.0	443.0	432.0		478.0
Weight of Container (g)	90.0	90.0	90.0	90.0		90.0
Weight of water (moisture) (g)	27.0	42.0	57.0	68.0		22.0
Weight of Dry soil (g)	383.0	368.0	353.0	342.0		388.0
Moisture content (%)	7.05	11.41	16.15	19.88		5.67
Dry Density (g / cm3)	1.77	1.83	1.85	1.72		
	MDD	**1.85**				
				OMC	**15.00**	

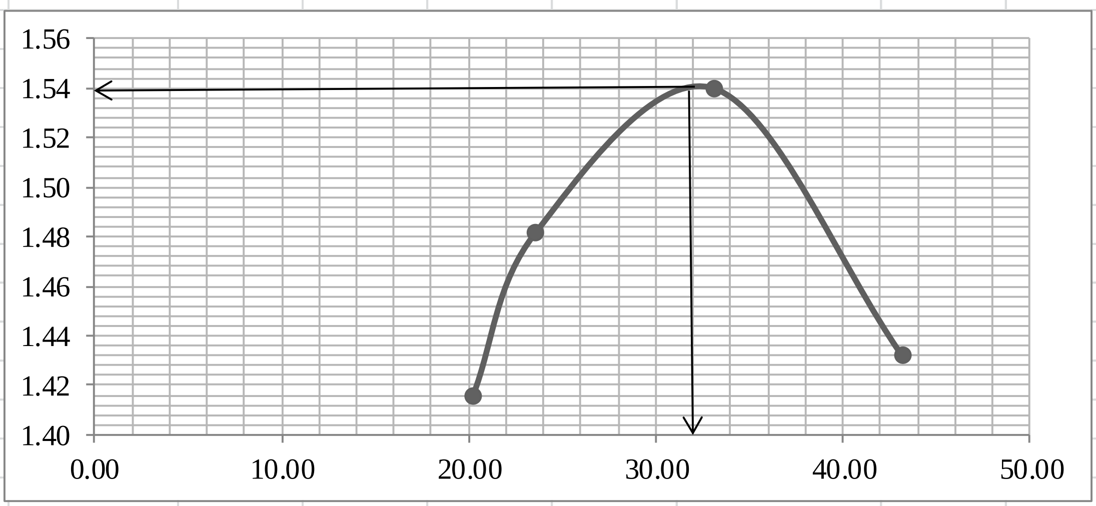


MDD :	**1.85**	**g/cm3**
OMC :	**15**	**%**

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

Sample Station	**7+600-7+800**		Sampling Date	
Materail Type	**Subgrade**		Testing Date	
Visual Discription	**Brown Soil**		Sample Taken	**Existing road**
Representing section	**Alaba - Mazoria**		Sample No	**One**

Trail	**1**	**2**	**3**			
Weight of Mould + Wet soil (g)	4980	5100	5305	5085		
Weight of Mould (g)	3390	3390	3390	3390		
Weight of Wet soil (g)	1590	1710	1915	1695		
Volume of Mould (mm3)	934.15	934.15	934.15	934.15		
Wet density (g / cm3)	1.70	1.83	2.05	1.81		
Moisture can	**A**	**10**	**8**			**NMC**
Weight of Wet soil + cont. (g)	301	300.0	300.0	**300**		**300**
Weight of Dry soil + cont. (g)	265.5	259.0	244.0	233.5		282.0
Weight of Container (g)	90.0	85.0	90.0	90.0		90.0
Weight of water (moisture) (g)	35.5	41.0	56.0	66.5		18.0
Weight of Dry soil (g)	175.5	174.0	169.0	154.0		192.0
Moisture content (%)	20.23	23.56	33.14	43.18		9.38
Dry Density (g / cm3)	1.42	1.48	1.54	1.43		
	MDD	**1.55**				
				OMC	**32.00**	



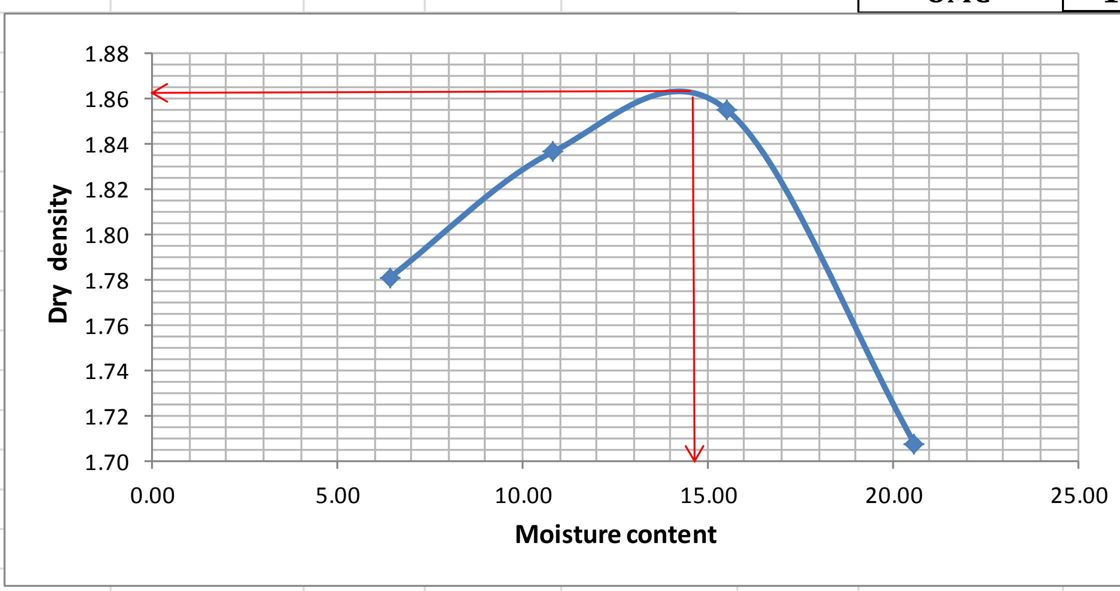
MDD :	**1.55**	**g/cm3**
OMC :	**32**	**%**

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

Sample Station	**13+600 - 13+800**		Sampling Date	
Materail Type	**Sub base**		Testing Date	
Visual Discription	**RED ASH**		Sample Taken	**Existing road**
Representing section	**Alaba - Mazoria**		Sample No	**Two**

Compaction test for Subbase material, station 13+600-13+800

Trail	1	2	3	4		
Weight of Mould + Wet soil (g)	9115	9400	9620	9450		
Weight of Mould (g)	5250	5250	5250	5250		
Weight of Wet soil (g)	3865	4150	4370	4200		
Volume of Mould (mm3)	2039.7	2039.7	2039.7	2039.7		
Wet density (g / cm3)	1.89	2.03	2.14	2.06		
Moisture can	**Z**	**W**	**10**	**A**		**NMC**
Weight of Wet soil + cont. (g)	500.0	500.0	**500**	**500**		**500**
Weight of Dry soil + cont. (g)	475.0	460.0	445.0	430.0		476.0
Weight of Container (g)	85.0	90.0	90.0	90.0		90.0
Weight of water (moisture) (g)	25.0	40.0	55.0	70.0		24.0
Weight of Dry soil (g)	390.0	370.0	355.0	340.0		386.0
Moisture content (%)	6.41	10.81	15.49	20.59		6.22
Dry Density (g / cm3)	1.78	1.84	1.86	1.71		
	MDD	**1.86**				
				OMC	**14.50**	

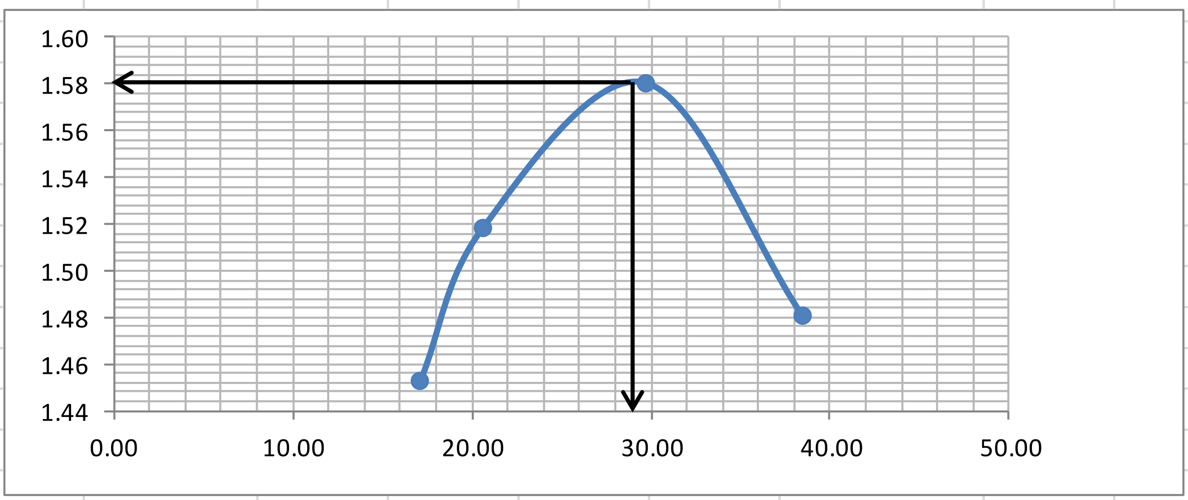


MDD :	**1.86**	**g/cm3**
OMC :	**14.5**	**%**

MOISTURE DENSITY RELATIONSHIP TEST (TEST METHOD: AASHTO T-99)

Sample Station	**44+20-44+400**		Sampling Date	**3/9/2016**
Materail Type	**Subgrade Soil**		Testing Date	**17/9/2016**
Visual Discription	**Brown Soil**		Sample Taken	**Existing road**
Representing section	**Buge- Boditi**		Sample No	**Three**

Trail	**1**	**2**	**3**	**4**		
Weight of Mould + Wet soil (g)	4990	5110	5315	5095		
Weight of Mould (g)	3400	3400	3400	3400		
Weight of Wet soil (g)	1590	1710	1915	1695		
Volume of Mould (mm3)	934.15	934.15	934.15	934.15		
Wet density (g / cm3)	1.70	1.83	2.05	1.81		
Moisture can	**B**	**2A**	**T**	**R**		**NMC**
Weight of Wet soil + cont. (g)	300	300.0	300.0	**300**		**300**
Weight of Dry soil + cont. (g)	265	260.0	245.0	235.0		280.0
Weight of Container (g)	90.0	90.0	90.0	90.0		90.0
Weight of water (moisture) (g)	30.0	35.0	50.5	59.6		20.0
Weight of Dry soil (g)	175.0	170.0	170.0	155.0		190.0
Moisture content (%)	17.14	20.59	29.71	38.45		10.53
Dry Density (g / cm3)	1.45	1.52	1.58	1.48		
	MDD	**1.58**				
				OMC	**29.00**	

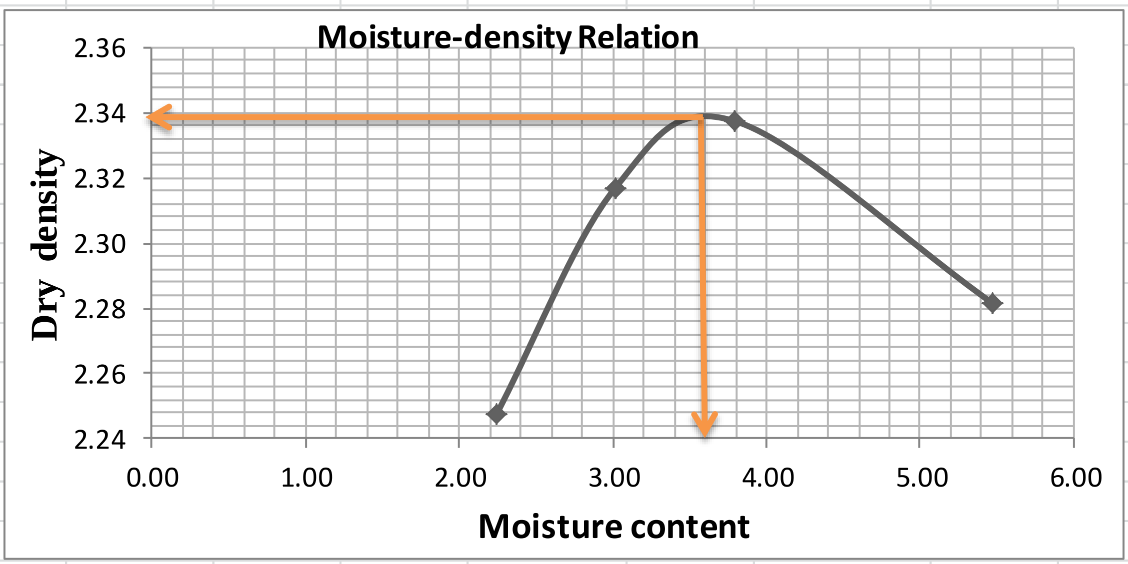


MDD :	**1.58**	**g/cm3**
OMC :	**29**	**%**

MOISTURE DENSITY RELATIONSHIP TEST (TEST METHOD: AASHTO T-180 METHOD D)

Sample Station	**67+926**		Sampling Date	**3/9/2016**
Materail Type	**Base Course**		Testing Date	**13/9/2016**
Visual Discription	**Crushed Aggrega**		Sample Taken	**Existing road**
Representing sectio	**Boditi-Sodo**		Sample No	**Four**

Trail	1	2	3	4		
Weight of Mould + Wet soil (g)	9920	10100	10180	10140		
Weight of Mould (g)	5255	5255	5255	5255		
Weight of Wet soil (g)	4665	4845	4925	4885		
Volume of Mould (mm3)	2030	2030	2030	2030		
Wet density (g / cm3)	2.30	2.39	2.43	2.41		
Moisture can	**6**	**3**	**10**	**8**		
Weight of Wet soil + cont. (g)	500.0	500.0	500.0	500.0		**NMC**
Weight of Dry soil + cont. (g)	491.0	488.0	485.0	479.0		**510**
Weight of Container (g)	90.0	90.0	90.0	95.0		505.0
Weight of water (moisture) (g)	9.0	12.0	15.0	21.0		90.0
Weight of Dry soil (g)	401.0	398.0	395.0	384.0		5.0
Moisture content (%)	2.24	3.02	3.80	5.47		415.0
Dry Density (g / cm3)	2.25	2.32	2.34	2.28		1.20
	MDD	**2.34**				
				OMC	**3.60**	



MDD :	**2.34**	**g/cm3**
OMC :	**3.6**	**%**

APPENDIX F: SOME OF CALIFORNIA BEARING RATIO TESTS

CALIFORNIA BEARING RATIO WORK SHEET (TEST METHOD: AASHTO T-193)

Sample Station	**67+926**	Sample Date	**3-Sep-16**
Representing Section	**Boditi-Sodo**	Socked Date	**3-Nov-16**
Material Type	**Subbase**	Penetration Date	**7-Nov-16**
Visual Discription	**Red ash**	Sample No	**Four**
MDD (g/cc)	**1.85**	OMC (%)	**15**

D E N S I T Y   D E T E R M I N A T I O N

SOAKING CONDITION	10 Blows		30 Blows		65 Blows	
	BEFORE	AFTER	BEFORE	AFTER	BEFORE	AFTER
MOLD NUMBER	T		H		K	
WEIGHT OF SOIL + MOLD (gm)	10525	10745	10805	10925	10850	10925
WEIGHT OF MOLD (gm)	6575	6575	6525	6525	6440	6440
WEIGHT OF SOIL (gm)	3950	4170	4280	4400	4410	4485
VOLUME OF MOLD (cc)	2103.85	2103.9	2103.85	2103.9	2103.9	2103.9
WET DENSITY OF SOIL (g/cc)	1.88	1.98	2.03	2.09	2.10	2.13
DRY DENSITY OF SOIL (g/cc)	1.68	1.58	1.84	1.70	1.92	1.81

M O I S T U R E   D E T E R M I N A T I O N

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	12			M			TH		
WET SOIL + CONTAINER (g	510	385	447.5	510	385	447.5	510	385	447.5
DRY SOIL + CONTAINER (gm	465	325	395	470	330	400	475	340	407.5
WEIGHT OF WATER (gm)	45.0	60.0	52.5	40.0	55.0	47.5	35.0	45.0	40
WEIGHT OF CONTAINER (gr	90	90	90	90	90	90	85	85	85
WEIGHT OF DRY SOIL (gm)	375	235	305	380	240	310	390	255	322.5
MOISTURE CONTENT (%)	12.00	25.53	18.77	10.53	22.92	16.72	8.97	17.65	13.31

Ring Factor	**21.21**

P E N E T R A T I O N   T E S T   D A T A

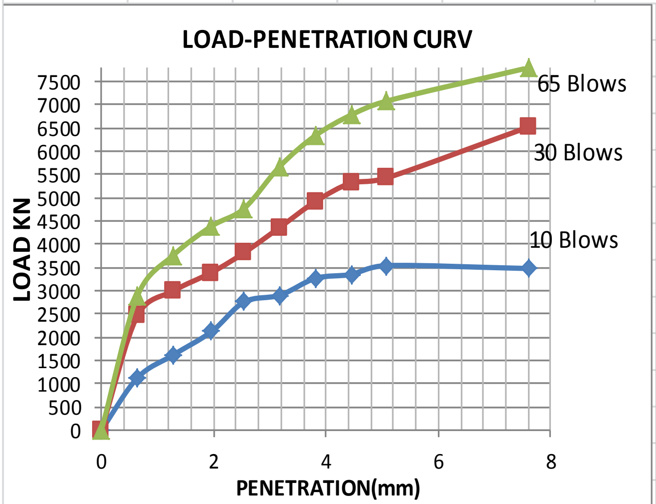
PENETRATION (mm)	10 Blows				30 Blows				65 Blows			
	DIAL RDG	LOAD (kn	P.SI	**CBR %**	DIAL RDG	LOAD (I	P.SI	**CBR %**	DIAL RDG	LOAD (I	P.SI	**CBR %**
0	0	0			0	0			0	0		
0.64	53	1124.13	0.58		116	2460.36	1.27		136	2884.56	1.49	
1.27	76	1611.96	0.83		141	2990.61	1.55		177	3754.17	1.94	
1.96	100	2121	1.10		160	3393.60	1.75		207	4390.47	2.27	
2.54	130	2757.3	1.42	21	180	3817.80	1.97	29	225	4772.25	2.47	36
3.18	137	2905.77	1.50		206	4369.26	2.26		268	5684.28	2.94	
3.81	154	3266.34	1.69		232	4920.72	2.54		299	6341.79	3.28	
4.45	158	3351.18	1.73		251	5323.71	2.75		320	6787.20	3.51	
5.08	167	3542.07	1.83	18	256	5429.76	2.81	27	334	7084.14	3.66	35
7.62	164	3478.44	1.80		308	6532.68	3.38		368	7805.28	4.03	
10.16												
12.7												

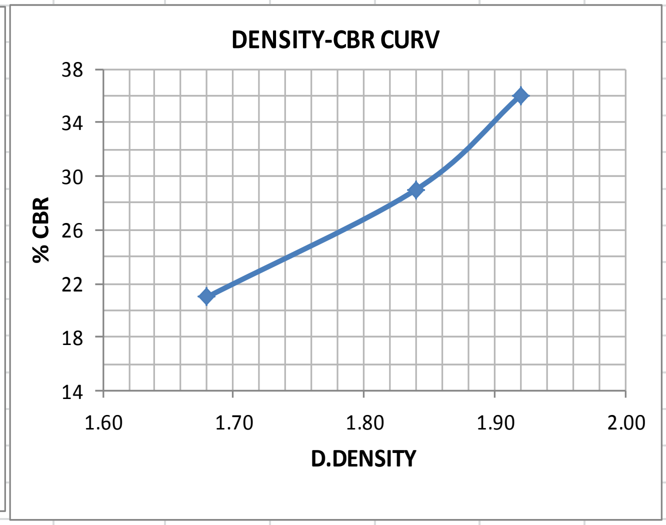
SWELL

No. OF BLOWS	**10**	**30**	**65**
RDG (BEFORE SOAKING)	6.9	12.0	3.8
RDG (AFTER) SOAKING)	6.5	11.8	3.7
PERCENT SWELL	0.3	0.2	0.1
AVERAGE PERCENT SWELL	0.20		

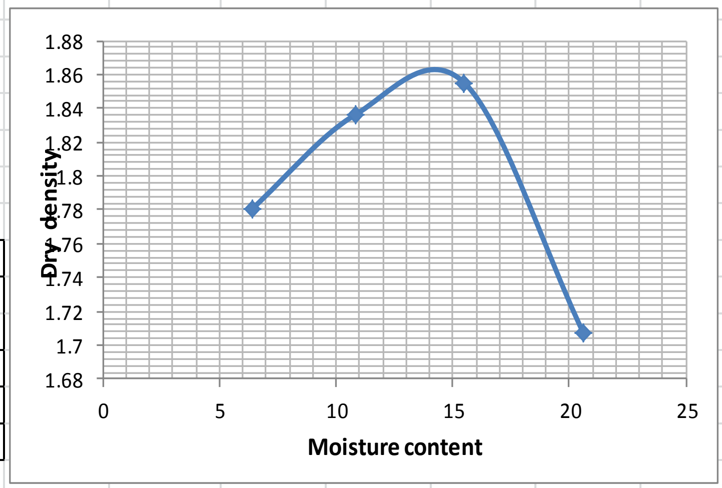
CALIFORNIA BEARING RATIO WORK SHEET (TEST METHOD: AASHTO T-193)

Sample Station	**67+926**			Sample Date	**3-Sep-16**
Representing Section	**Boditi-Sodo**			Socked Date	**3-Nov-16**
Material Type	**Subbase**			Penetration Date	**7-Nov-16**
Visual Discription	**Red ash**			Sample No	**Four**
MDD (g/cc)	**1.85**			OMC (%)	**15**





MODIFIED PROCTOR : T 180	
MDD (g/cc)	1.85
OMC (%)	14
95 % of MDD(g/cc)	1.76



Blows	Before Soaking		After Soaking	
	DD (g/cc)	Moisture (%)	DD (g/cc)	Moisture (%)
10	1.68	12.00	1.58	25.53
30	1.84	10.53	1.70	22.92
65	1.92	8.97	1.81	17.65

Blow	LOAD (KN)		CBR(%)		Swell
	2.54mm	5.08mm	2.54mm	5.08mm	%
10	2757.30	3542.07	20	18	0.30
30	3817.80	5429.76	27	27	0.20
65	4772.25	7084.14	36	36	0.10

Blow	Dry densit	CBR%
10	1.68	21
30	1.84	29
65	1.92	36
CBR at 95%		**25**

CALIFORNIA BEARING RATIO WORK SHEET (TEST METHOD AASHTO T-193)

Sample Station	7+600-7+800	Sample Date	
Representing Section	Alaba-Mazoria	Socked Date	
Material Type	Subgrade	Penetration Date	
Visual Discription	Brown Soil	Sample No	
MDD (g/cc)	1.54	OMC (%)	32

DENSITY DETERMINATION

SOAKING CONDITION	10 Blows		30 Blows		65 Blows	
	BEFORE	AFTER	BEFORE	AFTER	BEFORE	AFTER
MOLD NUMBER	N		A		T	
WEIGHT OF SOIL + MOLD (gm)	10380	10400	10280	10450	10350	10435
WEIGHT OF MOLD (gm)	6595	6595	6520	6520	6425	6425
WEIGHT OF SOIL (gm)	3785	3805	3760	3930	3925	4010
VOLUME OF MOLD (cc)	2103.85	2103.85	2103.85	2103.85	2103.85	2103.85
WET DENSITY OF SOIL (g/cc)	1.80	1.81	1.79	1.87	1.87	1.91
DRY DENSITY OF SOIL (g/cc)	1.42	1.40	1.45	1.47	1.55	1.53

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	10			11			W		
WET SOIL + CONTAINER (gm)	300	395		300	395		300	395	
DRY SOIL + CONTAINER (gm)	255	325		260	330		265	335	
WEIGHT OF WATER (gm)	45.0	70.0	57.5	40.0	65.0	52.5	35.0	60.0	47.5
WEIGHT OF CONTAINER (gm)	85	85	85	90	90	90	90	90	90
WEIGHT OF DRY SOIL (gm)	170	240	205	170	240	205	175	245	210
MOISTURE CONTENT (%)	26.47	29.17	27.82	23.53	27.08	25.31	20.00	24.49	22.24

Ring Factor	**21.21**

PENETRATION TEST DATA

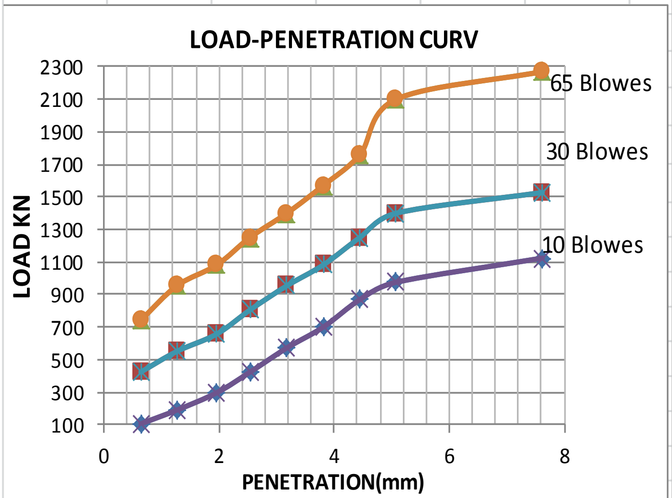
PENETRATION	10 Blows				30 Blows				65 Blows			
(mm)	DIAL RDG	LOAD (kn	P.SI	CBR %	DIAL RDG	LOAD (kn	P.SI	CBR %	DIAL RDG	LOAD (kn	P.SI	CBR %
	0	0			0	0			0	0		
0.64	5	106.05	0.05		20	424.20	0.22		35	742.35	0.38	
1.27	9	190.89	0.10		26	551.46	0.28		45	954.45	0.49	
1.96	14	296.94	0.15		31	657.51	0.34		51	1081.71	0.56	
2.54	20	424.2	0.22	3	38	805.98	0.42	6	59	1251.39	0.65	9
3.18	27	572.67	0.30		45	954.45	0.49		66	1399.86	0.72	
3.81	33	699.93	0.36		51	1081.71	0.56		74	1569.54	0.81	
4.45	41	869.61	0.45		59	1251.39	0.65		83	1760.43	0.91	
5.08	46	975.66	0.50	5	66	1399.86	0.72	7	99	2099.79	1.08	10
7.62	53	1124.13	0.58		72	1527.12	0.79		107	2269.47	1.17	
10.16												
12.7												

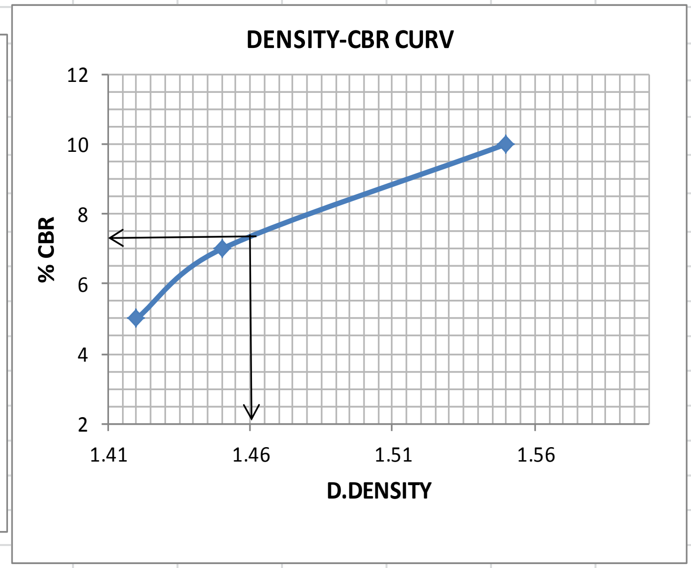
SWELL

No. OF BLOWS	**10**	**30**	**65**
RDG (BEFORE SOAKING)	11.5	7.4	3.2
RDG (AFTER) SOAKING)	12.6	7.8	3.4
PERCENT SWELL	0.9	0.3	0.2
AVERAGE PERCENT SWELL :	0.45		

CALIFORNIA BEARING RATIO WORK SHEET (TEST METHOD AASHTO T-193)

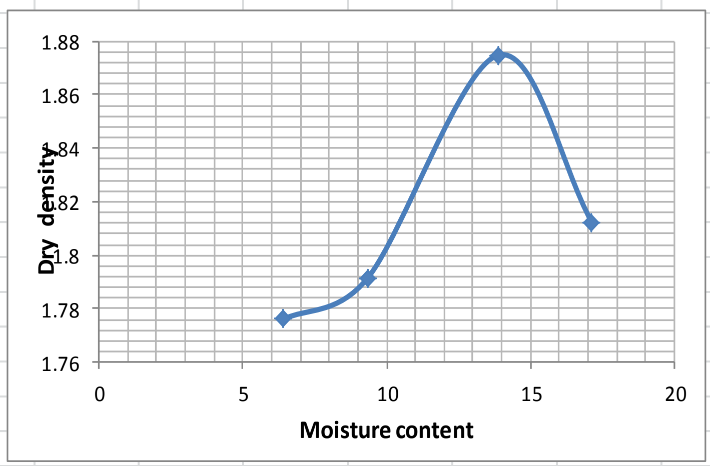
Sample Station	**7+600-7+800**			Sample Date	
Representing Sectio	**Alaba-Mazoria**			Socked Date	
Material Type	**Subgrade**			Penetration Date	
Visual Discription	**Brown**			Sample No	
MDD (g/cc)	**1.54**			OMC (%)	32





MODIFIED PROCTOR : T 180	
MDD (g/cc)	1.54
OMC (%)	32
95 % of MDD(g/cc)	1.46

Blows	Before Soaking		After Soaking	
	DD (g/cc)	Moisture (%)	DD (g/cc)	Moisture (%)
10	1.42	26.47	1.41	29.17
30	1.45	23.53	1.47	27.08
65	1.55	20.00	1.53	24.49



Blow	LOAD (KN)		CBR(%)		Swell %
	2.54mm	5.08mm	2.54mm	5.08mm	
10	424.20	975.66	3	5	0.90
30	678.72	1272.60	6	7	0.30
65	1251.39	2079.99	9	10	0.20

Blow	Dry density	CBR%
10	1.42	5
30	1.45	7
65	1.55	10
CBR at 95%		**7.6**

CALIFORNIA BEARING RATIO WORK SHEET (TEST METHOD: AASHTO T-193)

Sample Station	13+600-13+800	Sample Date	26-Aug-16
Representing Section	Alaba-Mazoria	Socked Date	8-Oct-16
Material Type	Subbase	Penetration Date	12-Oct-16
Visual Discription	Red ash	Sample No	Two
MDD (g/cc)	1.86	OMC (%)	14.5

DENSITY DETERMINATION

SOAKING CONDITION	10 Blows		30 Blows		65 Blows	
	BEFORE	AFTER	BEFORE	AFTER	BEFORE	AFTER
MOLD NUMBER	N		A		T	
WEIGHT OF SOIL + MOLD (gm)	10515	10735	10795	10915	10840	10915
WEIGHT OF MOLD (gm)	6570	6570	6520	6520	6435	6435
WEIGHT OF SOIL (gm)	3945	4165	4275	4395	4405	4480
VOLUME OF MOLD (cc)	2103.85	2103.85	2103.85	2103.85	2103.85	2103.85
WET DENSITY OF SOIL (g/cc)	1.88	1.98	2.03	2.09	2.09	2.13
DRY DENSITY OF SOIL (g/cc)	1.67	1.60	1.81	1.73	1.87	1.80

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	11			A			10		
WET SOIL + CONTAINER (gm)	500	375	437.5	500	375	437.5	500	375	437.5
DRY SOIL + CONTAINER (gm)	455	320	387.5	455	325	390	455	330	392.5
WEIGHT OF WATER (gm)	45.0	55.0	50	45.0	50.0	47.5	45.0	45.0	45
WEIGHT OF CONTAINER (gm)	90	90	90	85	85	85	85	85	85
WEIGHT OF DRY SOIL (gm)	365	230	297.5	370	240	305	370	245	307.5
MOISTURE CONTENT (%)	12.33	23.91	18.12	12.16	20.83	16.50	12.16	18.37	15.26

Ring Factor	**21.21**

PENETRATION TEST DATA

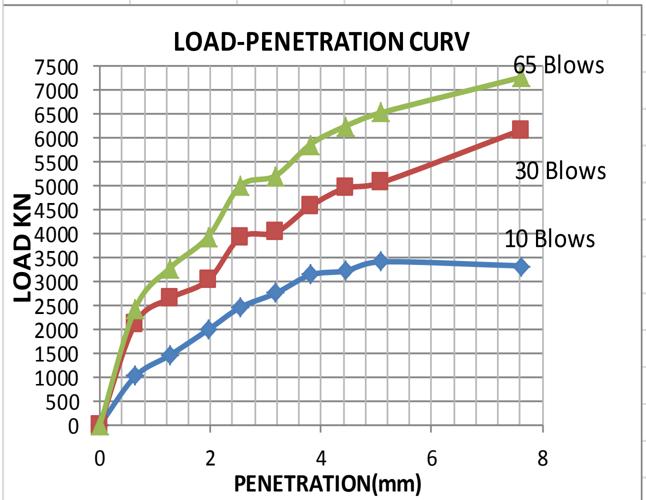
PENETRATION (mm)	10 Blows DIAL RDG	LOAD (kn)	P.SI	**CBR %**	30 Blows DIAL RDG	LOAD (kn	P.SI	**CBR %**	65 Blows DIAL RDG	LOAD (kn	P.SI	**CBR %**
0	0	0			0	0			0	0		
0.64	48	1018.08	0.53		100	2121.00	1.10		115	2439.15	1.26	
1.27	69	1463.49	0.76		125	2651.25	1.37		155	3287.55	1.70	
1.96	94	1993.74	1.03		143	3033.03	1.57		186	3945.06	2.04	
2.54	116	2460.36	1.27	18	185	3923.85	2.03	29	236	5005.56	2.59	38
3.18	130	2757.3	1.42		190	4029.90	2.08		246	5217.66	2.70	
3.81	148	3139.08	1.62		216	4581.36	2.37		277	5875.17	3.04	
4.45	152	3223.92	1.67		234	4963.14	2.56		295	6256.95	3.23	
5.08	161	3414.81	1.76	17	239	5069.19	2.62	25	308	6532.68	3.38	33
7.62	157	3329.97	1.72		291	6172.11	3.19		343	7275.03	3.76	
10.16												
12.7												

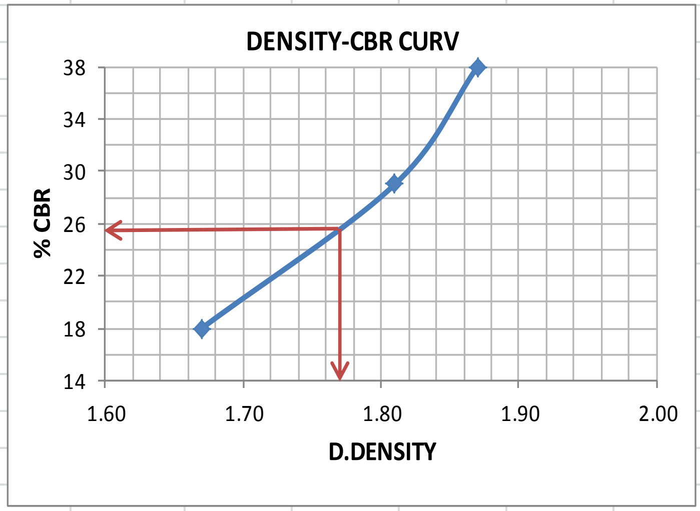
SWELL

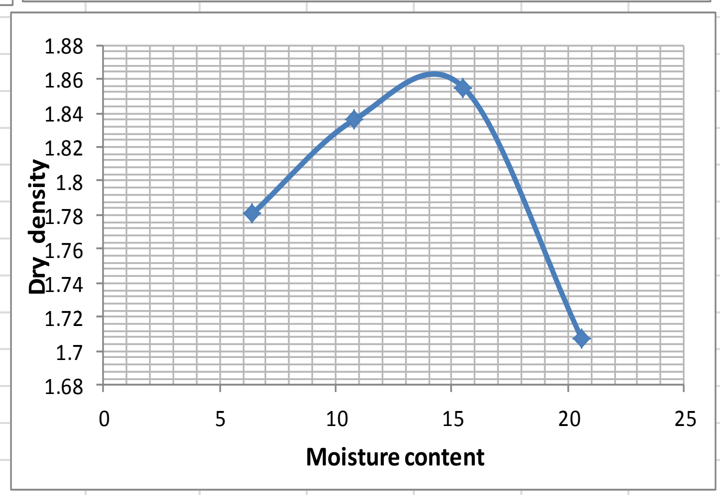
No. OF BLOWS	**10**	**30**	**65**
RDG (BEFORE SOAKING)	6.6	11.9	3.1
RDG (AFTER) SOAKING)	5.9	11.0	2.2
PERCENT SWELL	0.6	0.7	0.8
AVERAGE PERCENT SWELL :	0.69		

CALIFORNIA BEARING RATIO WORK SHEET (TEST METHOD: AASHTO T-193)

Sample Station	13+600-13+800	Sample Date	26-Aug-16
Representing Section	Alaba-Mazoria	Socked Date	8-Oct-16
Material Type	Subbase	Penetration Date	12-Oct-16
Visual Discription	Red ash	Sample No	Two
MDD (g/cc)	1.86	OMC (%)	14.5







MODIFIED PROCTOR : T 180	
MDD (g/cc)	1.86
OMC (%)	14.5
95 % of MDD(g/cc)	1.77

Blows	Before Soaking		After Soaking	
	DD (g/cc)	Moisture (%)	DD (g/cc)	Moisture (%)
10	1.67	12.33	1.60	23.91
30	1.81	12.16	1.73	20.83
65	1.87	12.16	1.80	18.37

Blow	LOAD (KN)		CBR(%)		Swell %
	2.54mm	5.08mm	2.54mm	5.08mm	
10	2460.36	3414.81	18	17	0.60
30	3923.85	5069.19	29	25	0.70
65	4899.51	6405.42	38	33	0.80

Blow	Dry density	CBR%
10	1.67	18
30	1.81	29
65	1.87	38
CBR at 95%		25.5

CALIFORNIA BEARING RATIO WORK SHEET (TEST METHOD: AASHTO T-193)

Sample Station	**44+200-44+400**	Sample Date	**26-Aug-16**
Representing Section	**Buge-Boditi**	Socked Date	**26-Oct-16**
Material Type	**Base Course**	Penetration Date	**30-Oct-16**
Visual Discription	**Crushed Aggregate**	Sample No	**Three**
MDD (g/cc)	**2.33**	OMC (%)	**4.1**

DENSITY DETERMINATION

SOAKING CONDITION	10 Blows		30 Blows		65 Blows	
	BEFORE	AFTER	BEFORE	AFTER	BEFORE	AFTER
MOLD NUMBER	P1		J8		N1	
WEIGHT OF SOIL + MOLD (gm)	10700	10870	11305	11485	11620	11730
WEIGHT OF MOLD (gm)	6450	6450	6580	6580	6520	6520
WEIGHT OF SOIL (gm)	4250	4420	4725	4905	5100	5210
VOLUME OF MOLD (cc)	2140	2140	2140	2140	2140	2140
WET DENSITY OF SOIL (g/cc)	1.99	2.07	2.21	2.29	2.38	2.43
DRY DENSITY OF SOIL (g/cc)	1.91	1.86	2.15	2.11	2.35	2.29

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	11			6			10		
WET SOIL + CONTAINER (gm)	500	600	550	500	600	550	500	600	550
DRY SOIL + CONTAINER (gm)	485	550	517.5	490	560	525	495	570	532.5
WEIGHT OF WATER (gm)	15.0	50.0	32.5	10	40	25	5	30	17.5
WEIGHT OF CONTAINER (gm)	90	90	90	90	90	90	90	91	90.5
WEIGHT OF DRY SOIL (gm)	395	460	427.5	400	470	435	405	479	442
MOISTURE CONTENT (%)	3.80	10.87	7.60	2.50	8.51	5.5	1.23	6.26	3.75

Ring Factor =	**21.21**	**N/div**

PENETRATION TEST DATA

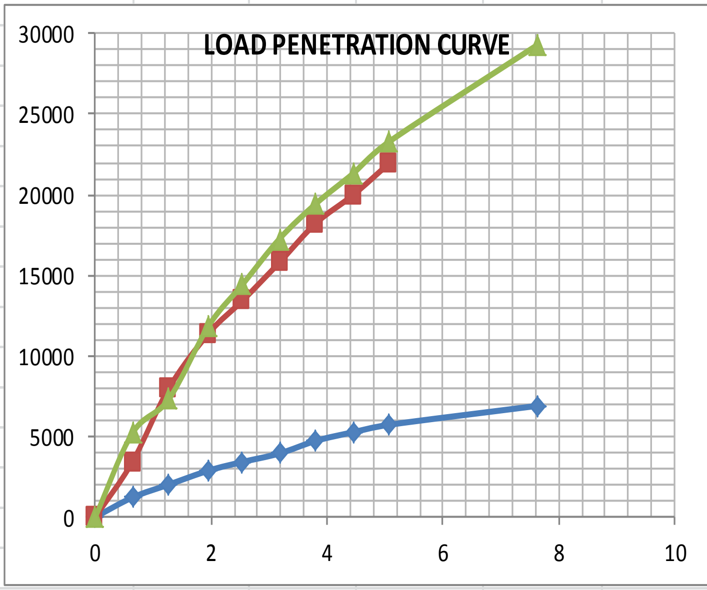
PENETRATION	**10 Blows**				**30 Blows**				**65 Blows**			
(mm)	DIAL RDG	LOAD (kn)	P.SI	**CBR %**	DIAL RDG	LOAD (kn	P.SI	**CBR %**	DIAL RDG	LOAD (kn	P.SI	**CBR %**
0	0	0				0				0		
0.64	57	1208.97	0.6		161	3414.81	1.74		245	5196.45	2.65	
1.27	95	2014.95	1.0		377	7996.17	4.07		345	7317.45	3.73	
1.96	136	2884.56	1.5		538	11410.98	5.81		560	11877.6	6.05	
2.54	161	3414.81	1.7	25	633	13425.93	6.84	99	680	14422.8	7.35	107
3.18	186	3945.06	2.0		745	15801.45	8.05		814	17264.94	8.80	
3.81	223	4729.83	2.4		860	18240.6	9.29		917	19449.57	9.91	
4.45	248	5260.08	2.7		942	19979.82	10.18		1005	21316.05	10.86	
5.08	270	5726.7	2.9	28	1036	21973.56	11.20	108	1099	23309.79	11.88	115
7.62	325	6893.25	3.5		1167	24752.07	12.61		1380	29269.80	14.91	
10.16												
12.7												

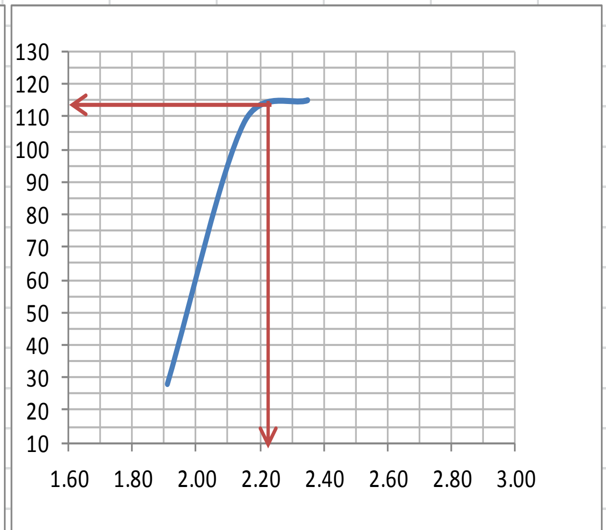
SWELL

No. OF BLOWS	**10**	**30**	**65**
RDG (BEFORE SOAKING)(mm)	0.00	0.00	0.00
RDG (AFTER) SOAKING)(mm)	0.00	0.00	0.00
PERCENT SWELL/mm	0.00	0.00	0.00
AVERAGE PERCENT SWELL	0.00		

CALIFORNIA BEARING RATIO WORK SHEET (TEST METHOD: AASHTO T-193)

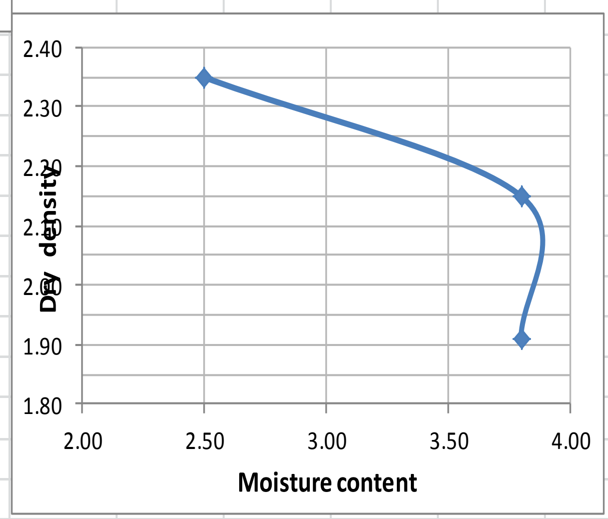
Sample Station	44+200-44+400	Sample Date	26-Aug-16
Representing Section	Buge-Boditi	Socked Date	26-Oct-16
Material Type	Base Course	Penetration Date	30-Oct-16
Visual Discription	Crushed Aggregate	Sample No	Three
MDD (g/cc)	2.33	OMC (%)	4.1





MODIFIED PROCTOR : T 180	
MDD (g/cc)	2.33
OMC (%)	4.4
95 % of MDD(g/cc)	2.21

Blows	Before Soaking		After Soaking	
	DD (g/cc)	Moisture (%)	DD (g/cc)	Moisture (%)
10	1.91	3.80	1.86	10.87
30	2.15	3.80	2.50	8.51
65	2.35	2.50	1.23	6.26



Blow	LOAD (KN)		CBR(%)		Swell %
	2.54mm	5.08mm	2.54mm	5.08mm	
10	3414.81	5726.70	25	28	0.00
30	13425.93	21973.56	99	108	0.00
65	14422.80	23309.79	107	115	0.00

Blow	Dry density	CBR%
10	1.91	28
30	2.15	108
65	2.35	115
CBR at 95%		**115**