JIMMA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

JIMMA INSTITUTE OF TECHNOLOGY

FACULTY OF CIVIL AND ENVIROMENTAL ENGINEERING

HIGHWAY ENGINEERING STREAM

Effects of Subgrade and Sub-Base Material Quality for the Deterioration of Flexible Pavement

A Case Study on Jimma - Sekoru Road Segment

A Research Submitted to School of Graduate Studies in Partial Fulfillment of the Requirements for Degree of Master of Science in Civil Engineering (Highway Engineering)

By:

Dagmawi Kedir

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Jimma, Ethiopia

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Advisor:

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2018

Jimma, Ethiopia

DECLARATION

This research is my original work and has not been presented for degree in any other university.

Dagmawi Kedir ________________ ____/____/_____

Name Signature Date

This research has been submitted for examination with my approval as university supervisors.

Eng'r Elmer C. Agon ________________ _______/____/_____

Advisor Signature Date

Eng'r. Abel Tesfaye (MSc.) ________________ _____ /____/_____

Co-Advisor Signature Date

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ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
ASTM	American Society Testing Materials
CBR	California Bearing Ratio
Cu	Coefficient of Uniformity,
ERA	Ethiopian Road Authority
GI	Group Index
GSD	Grain Size Analysis
LL	Liquid Limit
MC	Moisture Content
MDD	Maximum Dry Density
OMC	Optimum Moisture Content
PI	Plasticity Index
PL	Plasticity Limit
SSIS	Soil Stabilization Index System
UCS	Unconfined Compressive Strength
US	United States

ABSTRACT

A flexible pavement structure consists of several layers of which the most bottom layers are the sub base and pavement foundation or subgrade. A pavement structure is a layer structure which supports the vehicle load on its surface and transfers, supports the vehicle load on its surface and transfers and spreads the load to the sub base and subgrade. The effects of weak subgrade and poor sub base course materials on pavement design, construction, and performance prediction are very significant. This study examined the relationship between the subgrade geotechnical data, sub base material test data with the actual performance of pavement.

The main objective of this study were evaluate the effect of subgrade and sub base material quality for the deterioration of flexible pavement. Investigating the quality of subgrade and sub base materials, studying the effect of subgrade and sub base for the flexible pavement deterioration, indicating the best remedial measures to solve deterioration problems of the road segment are the key objectives in this study. The effect of the subgrade soil types and the quality of the subbase materials were investigated to check and recommend solutions for the existed deterioration in study segment.

The representative samples were taken in the selected subgrade and subbase course layers at which the road segment have been highly damaged. The road segment have built on a subgrade layer consisted granular, white clay and black cotton soil types. The distress spots have seen on areas which have black cotton and white clay subgrade soils as checked by field observation. Thus, natural granular soil, white clay and black cotton soil were taken to check the subgrade layer quality of Jimma-Sekoru road segment. And subbase course materials were also taken to check the quality of the layer on those spot areas.

The finding were found using a series of laboratory tests such as gradation analysis, Atterberg limits, moisture density relationships and CBR tests for both subgrade samples and subbase layer materials. The liquid limit varies from 49.72 - 64.82% and plasticity index from 12.44 - 33.10%. The soaked CBR values of subgrade soil materials were between 3.65 - 15%. The minimum CBR value for subbase layer for moisture susceptible areas is 30% as per ERA but Jimma –Sekoru road segment subbase materials CBR is 15%. The results of the subgrade soils investigation showed that the road pavement structures are underlined by A-7-5 and A-7-6 category of soils according to AASHTO which showed that the soil is very poor subgrade materials. As the findings have shown, the quality of the subgrade materials are poorly graded, highly plastic, low moisture content, low dry density and below fair CBR strength according to ERA pavement design specification. The subbase materials quality is also the same as the subgrade materials based on ERA pavement design manual for Jimma-Sekoru road segment. Thus the quality of the subgrade and sub base materials bring the deterioration of Jimma-Sekoru road segment.

As the study concludes, the subgrade soil needs improvement either mechanically or by chemical methods. The white clay and the black cotton soil have to be replaced by granular materials or stabilized by normal soil and/or chemicals. The subbase layer materials have to be changed and replaced by the other quality meet material based on ERA pavement design specification. Finally ERA, ORA and Jimma zone officers have to return their eyes to this deteriorated road segment to improve its subgrade and subbase layer for special and long live service.

Key Words:- Subgrade soils, Subbase materials, road deterioration, CBR strength

CHAPTER ONE
INTRODUCTION

1.1Background

A flexible pavement structure consists of several layers of which the most bottom layer is called pavement foundation or subgrade. A pavement structure is a layer structure which supports the vehicle load on its surface and transfers supports the vehicle load on its surface and transfers and spreads the load to the subgrade without exceeding either the strength of the sub grade or the internal strength of the pavement itself.

Subgrade strength and stiffness are very important for pavement design, construction, and performance. To date, no systematic study has been performed to evaluate the effects of weak, variable subgrade conditions on pavement design, construction, and performance prediction [78]. There is a need for evaluating the effect of pavement subgrade on pavement design and performance. The strength and stiffness properties of subgrade can be expressed in terms of California Bearing Ratio, R-value, or resilient modulus.

The type of subgrade soil is largely determined by the location of the road. However, where the soils within the possible corridor for the road vary significantly in strength from place to place, it is clearly desirable to locate the pavement on the stronger soils if this does not conflict with other constraints [75].

The presence of unsuitable soils in construction sites creates significant influence on planning, structural design, construction and maintenance costs, performance and service life, especially of shallow depth engineering infrastructures where the moisture fluctuation is significant. Such soils are particularly susceptible to considerable volume changes in response to moisture content fluctuations following seasonal climatic variations. This property can cause severe damage to pavement structures unless proper measures are taken prior to construction phase. Identification of unsuitable soils and characterization of their anticipated behavior is thus an important parameter for site selection, design, and construction projects [79].

On the other hand, sub base course with thickness typically between 4 and 16 in. (100 and 405 mm) is a layer of select material between the subgrade and the base course. Sub base course provides uniform support and adds to the required structural capacity of the pavement section. The material can be gravel, crushed stone or subgrade soil stabilized with cement, fly ash or lime. The use of permeable sub base course is becoming more common to accommodate drainage of water infiltrating from the surface or to keep subsurface water from reaching the surface.

1.2 Statement of the Problem

In recent years, many of the major trunk roads of the country have experienced an increase in the severity extent of permanent deformation, fatigue cracking and ravelling over considerably visible stretches. The quality and ride ability of trunk roads is so vital in that such roads provide linkage amongst the most economically and geographically critical regions of the country.

Advanced pavement distresses including widening longitudinal cracks, side cracking, rutting, shoving and potholes have been observed on Jimma – Sekoru road segment. It may be due to pavement design or possibly the mix design, or may be due to weak and non-uniform subgrade and poor quality of sub base material. Jimma-Sekoru road was constructed in an area well known for its weak and variable subgrade. In this study, the effects of weak subgrade and poor sub base course materials on pavement design, construction, and performance prediction will be evaluate through the case study. In essence, attempts will make in this study to examine the relation between the subgrade geotechnical data, sub base material test data with the actual performance of pavement.

1.3 Research Questions

The main research questions to be answered through the research process include the following:

- What are the main properties of subgrade and sub base materials in the study area?
- How subgrade and sub base materials can be the effect of pavement deterioration in the segment?
- Which remedial measures are the best solutions to solve the segment deterioration problems?

1.4 Research Objective

1.4.1 General Objective

The main objective of the study was to evaluate the effect of subgrade and sub base material quality for the deterioration of flexible pavement.

1.4.2 Specific Objective

The specific objectives of this paper are:

- To investigate the properties of subgrade and sub base materials of the study area.
- To study the effect of subgrade and sub base for the flexible pavement deterioration in Jimma – Sekoru road.
- To find the best remedial measures to solve deterioration problems in the road segment.

1.5 Significance of the Research

Ethiopia is at the eve of developing into a middle income economy with its development being shown by the level of road network under construction. Roads are being either constructed or being improved throughout the country.

At this research, it is expected that all the research questions answered and possible solutions or recommended values for the selected road deterioration problems due to subgrade and sub base material quality have indicated. And, the researcher believes that this research have solve poor quality materials of subgrade, sub base problems of Jimma – Sekoru road segment.

The research can also serve as a reference material for those who need further investigation about pavement deterioration due to the effects of subgrade and sub base material qualities, road defects across the country and public organizations to reduce pavement defects and increase traffic safety at different road corridors.

1.6 Scope of the Study

This study have supported by different types of literatures and a series of laboratory experiments. However, the findings of the research are limited to investigate the quality of subgrade and sub base material quality. The results are also specific to test procedures that have adopted in the experimental work. Therefore, findings shall be considered indicative rather than definitive for field applications.

1.7 Limitation of the Study

The work was limited to the budget and workmanship available in the laboratory. During the subgrade soils and subbase materials sampling the ERA-Jimma district administrative officers were not interested and limit sampling areas. The sampling areas, that were permitted, were only the deteriorated spots of the segment.

1.8. Organization of the Thesis

The presentation of this thesis work is organized in Five Chapters with appendices. The first Chapter gives a brief description of the thesis background, objectives, scope and significance of the study. Chapter 2 comprises of a comprehensive literature review and important details from previous studies regarding to the study. The general methodology and test procedures followed during the process of this research have been presented in Chapter three. The fourth Chapter presents the test results obtained; analysis and discussions of results with respect to the theoretical background and findings of previous studies. Finally, conclusions and recommendations drawn from the research are presented in Chapter Fifth. The appendix includes photo of different laboratory activities.

CHAPTER TWO

LITERATURE REVIEW

2.1. Introduction

An examination of the history of pavement design reveals an evolutionary process that began with rule-of-thumb procedures and gradually evolved into empirical design equations based on experience and road test pavement performance studies. As Elliott and Thompson [1], Monismith [2], de Beer [3] stated, this evolution and transformation has been accompanied by the development of an understanding of material behavior, load-pavement distress relationships and environment interactions. Through the years, much of the development has been hampered by the complexity of the pavement structural system both in terms of its indeterminate nature and in terms of the changing and variable conditions to which it is subjected.

Major advancements in layered theory of pavements have been made since its introduction in the early 1940s. Recently very sophisticated analytical or mechanistic methods for the design of new pavements and reconstruction or strengthening of existing ones have been developed. Although these methods are theoretically sound, a gap still exists between actual pavement behavior (practice) and theory [3].

Unbound granular materials are commonly used in aggregate base– granular subbase courses in flexible pavements. The main functions of these unbound pavement foundation layers are to distribute load through aggregate interlock and protect the weak subgrade beneath; other performance needs pertinent to maintaining integrity in changing environmental conditions are also nontrivial. In the past few decades, there have been significant efforts to understand individual aggregate properties as factors influencing mechanical and hydraulic response trends of unbound aggregate materials [4-8]. Compared With aggregate type and mineralogy, properties such as aggregate shape, texture and angularity, fines content (percentage passing No. 200 sieve or smaller than 0.075 mm), plasticity index, and moisture and density conditions related to compaction and their interactions are not well understood. For example, particle size distribution or gradation is a key factor influencing not only the mechanical response behavior characterized by resilient modulus (MR), shear strength, and permanent deformation but also permeability, frost susceptibility, and susceptibility to erosion [9, 10].

2.2. Soil classification

Soil is a broad term used in engineering applications which includes all deposits of loose material on the earth's crust that are created by weathering and erosion of underlying rocks. Although weathering occurs on a geologic scale, the process is continuous and keeps the soil in constant transition. The physical, chemical, and biological processes that form soils vary widely with time, location and environmental conditions and result in a wide range of soil properties [12]. Physical weathering occurs due to temperature changes, erosion, alternate freezing and thawing and due to plant and animal activities causing disintegration of underlying rock strata whereas chemical weathering decomposes rock minerals by oxidation, reduction, hydrolysis, chelation, and carbonation. These weathering processes, individually or in combination, can create residual in-place soils or facilitate the transport of soil fractions away from the parent rocks by geologic agents like wind, water, ice or gravity. These transport processes often result in mixing of soil minerals or introducing salts or organic material of a variety of species and concentrations. Soil impacted by the presence of organics and salts, such as sulfates, can exist as remote outcrops or over large areas and often do not have clearly defined boundaries.

The soil geological profile also varies considerably with location and even within a specific soil series or association. The complexity of soils requires a disciplined yet efficient method to identify and classify them for their use as a construction material. Soil texture is defined, at least initially, by its appearance and is dependent on the size, shape and distribution of particles in the soil matrix. Soil particle sizes may vary from boulders or cobbles, roughly a meter in diameter, to very fine clay particles, roughly a few microns in diameter.

Engineering properties of coarse fractions are dependent on physical interlocking of grains and vary with the size and shape of individual particles. Finer fractions in soil have a significantly higher specific surface area and their behavior is influenced more by electro-chemical and physio-chemical aspects than particle interaction.

Among finer particles, clays exhibit varying levels of consistency and engineering behavior and demonstrate various levels of plasticity and cohesiveness in the presence of water. Silt fractions are also classified as fine-grained soils because more than 50 percent of the soil mass is smaller than 75 µm, which fits in the designation of fine-grained material according to the Unified Classification System [11]. However, the specific surface area of silt fines is several orders of magnitude larger than that of clay soil particles. This difference is part of the reason that clay particles are more reactive than silt particles. In addition, clay minerals have a unique sheet particle structure and a crystalline layer structure that is amenable to significant isomorphous substitution. As a result of the isomorphous substitution of lower valence cations for higher valence cations within the layer structure, clay mineral surfaces carry a significant negative surface charge that can attract positively charged ions and dipolar water molecules. The cumulative effect of high surface area and surface charge makes clay particles particularly reactive, especially with water, and is the root cause of the propensity of clay particles to shrink and swell depending on the availability of water.

The AASHTO (M 145) soil classification system differentiates soils, first based on particle size and secondly based on Atterberg limits. If 35 percent or more of the mass of the soil is smaller than 75 µm in diameter, then the soil is considered either a silt or clay and if less than 35 percent of particles are smaller than 75 micron sieve, then the soil is considered to be coarse-grained, either a sand or gravel. For stabilization purposes, soils can be classified into subgrade and Base course materials based on fractions passing No. 200 sieve. If 25 percent or more passes through the no.200 sieve the soil can be considered as a subgrade, and if not, they may be classified as a base course material. However, more than simple gradation impacts the definition of a subgrade or base. In order to be termed a base material, the material in question must also be targeted for use as a base layer from a structural perspective. On the other hand, an in situ coarse-grained soil with less than 25 percent fines, may be, by definition a native subgrade even though it may achieve the required classification of a base. For stabilization purposes, the soils may be differentiated into subgrade (soil) stabilization and base stabilization (coarse-grained) on the basis on the fine content index [13].

The Unified Soil Classification System (USCS), as per ASTM D2487-11, quantifies the gradation of a soil with <12% of fines with two parameters: coefficient of uniformity, C_u (D_{60}/D_{10}), and coefficient of curvature, C_c ($D^2_{30}/D_{60}*D_{10}$). Soils are considered very poorly graded when $C_u < 3$, whereas gravels and sands are deemed well graded when $C_u > 4$ and 6, respectively. C_C for well-graded soils or aggregates often ranges between 1 and 3. The definitions for gravel and sand are not unique, with USCS defining gravel as particles passing a 75-mm (3-in.) sieve and retained on 4.75-mm (No. 4) sieve and sand as particles passing a 4.75-mm (No. 4) sieve and retained on a 75-µm (No. 200) sieve. Thus, an aggregate is classified as gravel or sand (coarse aggregate or fine aggregate) depending on which proportion present is larger.

The influence of gravel (or coarse aggregate) content on the shear strength of cohesionless soil–gravel and sand–gravel mixtures has been the topic of investigation of many geotechnical researchers. According to Vallejo [15], the frictional resistance between the gravel particles controlled the shear strength of the soil–gravel and sand–gravel mixtures when the percentage by weight of gravel averaged >70%, whereas the gravel particles with an average concentration by weight of <49% basically had no control over the shear strength of the mixtures. This scientific observation could imply that the relative contents of gravel and sand particles in aggregate base–granular subbase materials may be an inherent factor controlling mixture performance mechanically or hydraulically, as supported by the findings of Sánchez-Leal [14] from studies with hot-mix asphalt.

2.3 Soil Gradation

Soil gradation is a classification of a grained soil that ranks the soil based on the different particle sizes contained in the soil. Soil gradation is an important aspect of soil mechanics and geotechnical engineering because it is an indicator of other engineering properties such as compressibility, shear strength, and hydraulic conductivity. In a design, the gradation of the in situ or on site soil often controls the design and ground water drainage of the site. A poorly graded soil will have better drainage than a well graded soil [16]. The process for grading a soil is in accordance with either the Unified Soil Classification System or the AASHTO Soil Classification System. In Highway and Road construction, the American Association of State Highway and Transportation Officials (AASHTO) soil classification System is used the results of soil grain size analysis and Atterberg limits. Soil Components are described as: Gravel, Sand, Silt and Clay. It might be reasonable to believe that the best gradation is one that produces the MDD and which is best for any road construction.

Silt and Clay Materials: >35% of total sample passing No.200 sieve. The influence of fine grain soils on stabilized pavement structures and demonstrated that an increase in angularity of crushed fines increased the values at the optimum cement and lime content. An increase in angularity in the fine aggregate also increased the void content at a given compaction level and requirement of the optimum cement or lime content.

Granular Materials: < or equal to 35% of total sample passing No.200 sieve. Coarse-grained soils are divided into two major divisions: gravels and sands. The coarse and fine soil grain size characteristics are important factors related to moisture damage. Gradation of a soil is determined by reading the grain size distribution curve produced from the results of laboratory tests on the soil. Gradation of a soil can also be determined by calculating the coefficient of uniformity, Cu, and the coefficient of curvature, Cc, of the soil and comparing the calculated values with published gradation limits [16].

2.4 SUBGRADE SOILS

The type of subgrade soil is largely determined by the location of the road. However, where the soils within the possible corridor for the road vary significantly in strength from place to place, it is clearly desirable to locate the pavement on the stronger soils if this does not conflict with other constraints. For this reason, the pavement engineer should be involved in the route corridor selection process when choices made in this regard influence the pavement structure and the construction costs [75].

Table 2. 1 AASHTO soil classification chart

General Classification	Granular Materials (35% or less passing the 0.075 mm sieve)							Silt-Clay Materials (>35% passing the 0.075 mm sieve)			
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, % passing											
2.00 mm (No. 10)	50 max	...	...	...	...	...	...	...	...	...	...
0.425 (No. 40)	30 max	50 max	51 min	...	...	...	...	...	...	...	...
0.075 (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 0.425 mm (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 min	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 a subgrade	excellent to good							fair to poor			

Note: Plasticity index of A-7-5 subgroup is equal to or less than the LL - 30. Plasticity index of A-7-6 subgroup is greater than LL - 30

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

The moisture content of the subgrade soil is governed by the local climate and the depth of the water table on the road surface [75]. According to ERA-PDM, 2002 volume 1 (Flexible pavements and gravel roads) chapter three explains details concerning subgrade materials. According to the manual the strength of the Subgrade soil is assessed by the type of soil, its density and moisture content. According to ERA manual 2002 subgrades are classified from S1 to S6 based on the California bearing ratio (CBR), and are illustrated in table below.

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

Table 2. 2 CBR range subgrade class [75]

Serial No.	Class	% CBR Range
1	S1	2
2	S2	3-4
3	S3	5-7
4	S4	8-14
5	S5	15-29
6	S6	30+

Table 2. 3 CBR range Subgrade quality [76]

Serial No.	CBR (%) Range	Subgrade Quality
1	0-3	Very poor subgrade
2	3-7	Poor to fair subgrade
3	7-20	Fair subgrade
4	20-50	Good subgrade
5	50+	Excellent subgrade

2.5 Sub base materials

The subbase is the layer of aggregate material that lies immediately below the pavement and usually consists of crushed aggregate or gravel or recycled materials. Although the terms "base" and "subbase" are sometimes used interchangeably to refer to the subsurface layers of a pavement, base course is typically used in asphalt pavements, primarily as a structural load-distributing layer, whereas the subbase layer used in concrete pavements primarily serves as a drainage layer. Aggregate subbase is typically composed of crushed rock, comprised of material capable of passing through a 1 1/2 inch screen, with component particles varying in size from 1 1/2 inch down to dust. The material can be made of virgin (newly mined) rock or of recycled asphalt and concrete.

The function of the pavement subbase is to provide drainage and stability to achieve longer service life of the pavement. Most pavement structures now incorporate subsurface layers, part of whose function is to drain away excess water that can be deleterious to the life of the pavement. However, aggregate materials for permeable bases must be carefully selected and properly constructed to provide not only permeability, but uniform stability as well. Proper construction and QC/QA testing operations can help to ensure good performance of the subbase layer. Excessive compaction can alter the gradation and create additional fines that may result in lower permeability's than determined in laboratory tests and used in the pavement system design. However, the optimization of structural contributions from high stability, versus the need to provide adequate drainage for pavement materials is still a point of debate [77].

As the granular subbase provides both bearing strength and drainage for the pavement structure, proper size, grading, shape, and durability are important attributes to the overall performance of the pavement structure. Granular subbase aggregates consist of durable particles of crushed stone or gravel capable of withstanding the effects of handling, spreading, and compacting without generation of deleterious fines. Aggregates used as subbase tend to be dense-graded with a nominal maximum size, commonly up to 1 1/2 inches. The percentage of fines (passing No. 200 sieve) in the subbase is limited to 10% for drainage and frost-susceptibility purposes [77].

- **Particle Shape:** Equi-dimensional aggregate with rough surface texture is preferred.
- **Permeability:** The fines content is usually limited to a maximum of 10% for normal pavement construction and 6% where free-draining subbase is required.
- **Plasticity:** Plastic fines can significantly reduce the load carrying capacity of subbase; plasticity index (PI) of the fines of 6 or less is required.

2.6. Review of Unbound Granular Materials Behavior

Unbound granular materials (UGMs) are extensively used in bases and subbases of flexible pavements to provide load distribution. The bearing capacity of UGMs is a result of the shear resistance of the aggregate skeleton i.e. through aggregate interlock between particles. As loading and performance requirements of pavements continually increase, a better basic understanding of the mechanical behavior of UGMs and their response to loading is essential. [17]

2.6.1. Mechanical behavior of unbound granular materials

Mechanical material properties can be measured using equipment developed at research establishments. The main requirement from the test is that the information generated is of a fundamental nature, such that a true understanding of material behavior can be obtained from it [19]. An example of a stress – strain curve taken from a monotonic loading triaxial test is shown in Figure 2.1(A). The first part of the monotonic curve forms as the applied stress level increases, until it approaches the yield stress point, after which the strain continues, even with a reduction in stress. Under repeated loading conditions (well below the failure/yield stress level), materials undergo recoverable and irrecoverable components of deformation. The permanent and recoverable components for a single load cycle are shown in Figure 2.1(B).

2.6.1.1. Resilient deformation behavior

The resilient properties of UGMs were first noted by Hveem in 1950's [18], who concluded that the deformation of such materials under transient loading is elastic in the sense that it is recoverable. The actual concept of resilient modulus was later introduced by Seed et al. [21] in characterizing the elastic response of subgrade soils and their relation to fatigue failures in asphalt pavements.

Granular materials are not truly elastic [20] but experience some non- recoverable deformation after each load application. In the case of transient loads and after the first few load applications, the increment of non- recoverable deformation is much smaller compared to the resilient/recoverable deformation, Figure 2.2.

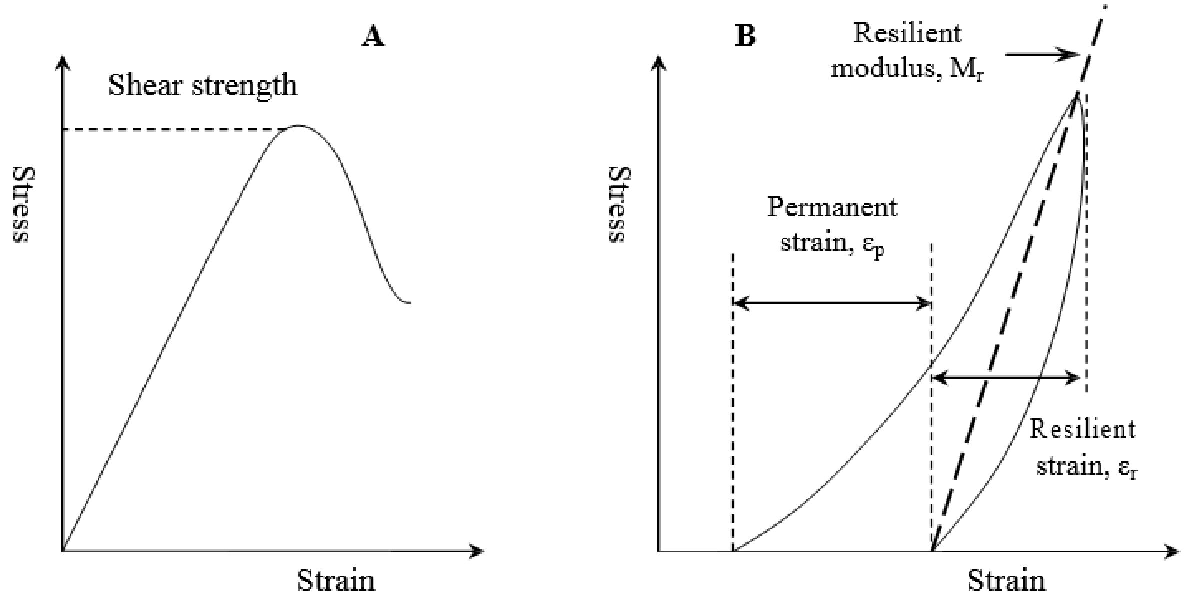


Figure 2. 1 (A) Monotonic loading to failure (B) strains in UGM during one load cycle [20]

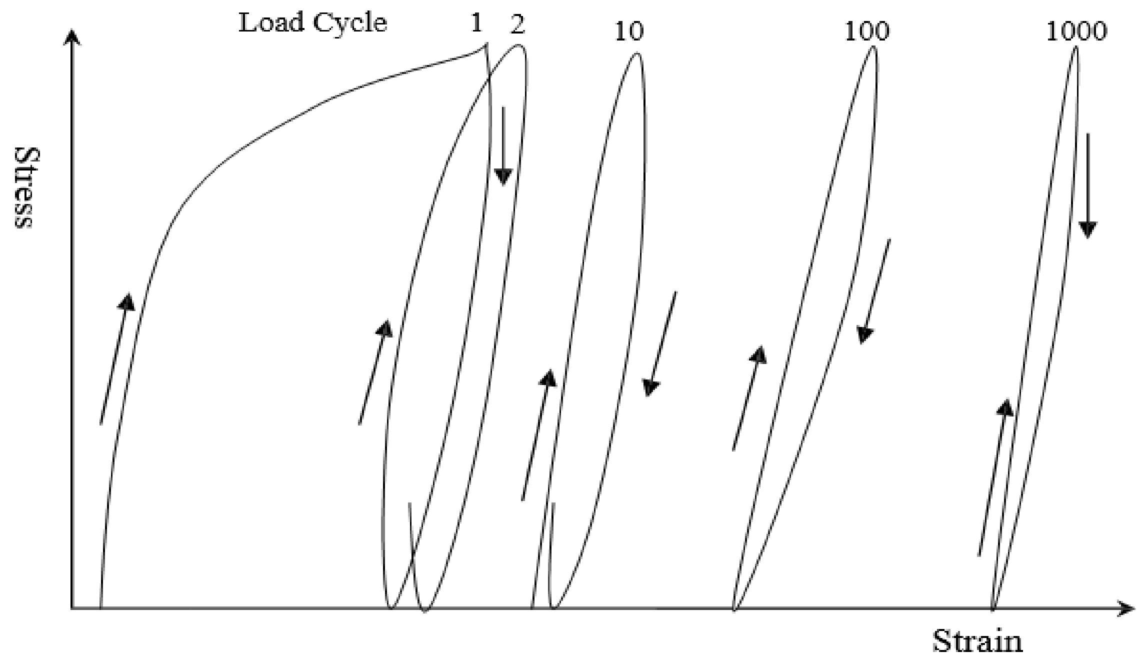


Figure 2. 2 Granular material behavior under repeated loading [22]

The term "resilient" has a precise meaning. It refers to that portion of the energy that is put into a material while it is being loaded that is completely recovered when it is unloaded [23]. This resilient behavior of granular layers is the main justification for using elastic theory to analyze their response to traffic loads.

2.6.1.2. Permanent deformation behavior

Permanent deformations represent the non-recoverable part of the deformations. Rutting is the most common damage caused by permanent deformations in UGLs. Many researchers [18, 25, 26] have reported that the accumulation rate of permanent strain under repeated loading decreases with the number of load reputations. Barksdale [26] performed a comprehensive study of the behavior of different base course materials using cyclic load triaxial tests with 10^5 load applications. He established the first well known relationship using a lognormal method between the permanent strain, ε_p, and the number of load repetitions, N, equation 2-1.

$$\epsilon_p = a + b\log N \qquad (2.1)$$

Another approach is to describe the plastic (permanent deformation) behavior of UGMs by means of the shakedown approach. The concept of shakedown in materials was originally developed to describe the deformation behavior of metal in pressure vessels under cyclic loading. Later this concept has been applied to describe the plastic behavior of UGMs under cyclic loading [24].

Werkmeister et al. [27] studied the permanent deformation behavior of UGMs using the shakedown approach and reported the cyclic load triaxial test results as either shakedown range A, B, or C, Figure 2-3. Range A refers to a plastic shakedown range, where the material after a post- compaction period becomes entirely resilient with no further permanent strain. Range B is defined as an intermediate response, or plastic creep, where the high plastic strain rate observed during the first load cycles decreases to a low, nearly constant level. Range C represents the incremental collapse where the permanent strain only increases with increasing number of load applications.

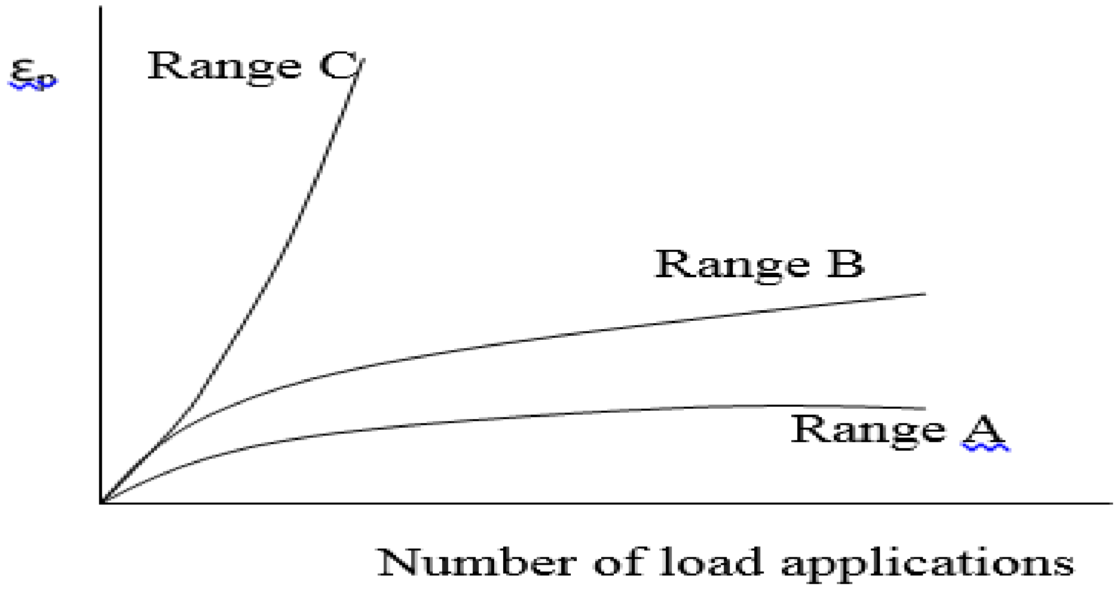


Figure 2. 3 Different types of permanent deformation behavior, depending on stress level [28, 29]

2.6.2. Granular skeleton: factors affecting deformation behavior

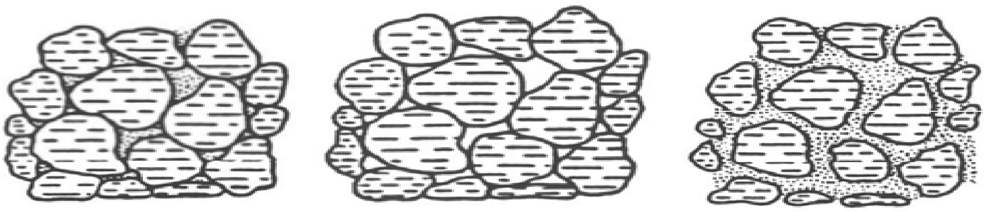
In dealing with skeletons of granular materials the following two aspects need to be considered and need to be defined clearly. Granular material properties comprise the soil/stone grain properties and the soil/stone aggregate properties [30]. The soil/stone grain properties comprise aspects such as color, particle shape and texture, gradation, mineralogical composition, plasticity characteristics, etc. which can be considered as constant for any soil/stone over the typical service life in roads. The soil/stone aggregate properties comprise structure, density, void ratio, permeability, strength, etc., which vary with changing conditions (e.g. environmental, construction, remolding and loading etc).

2.6.2.1. Soil/stone grain properties

Gradation (Particle size distribution)

The gradation of UGMs as used in pavements is critical to the performance of the pavement structure. The earliest attempts at specifying materials for roads made use of simple breakdowns of the material in terms of a size- related classification [31]. Various performance-related studies and laboratory investigations have confirmed the importance of gradation in allowing compaction to be effected with the least effort and in ensuring interlock and a tight configuration of the soil particles in service [32].

Thom and Brown [22] studied the behavior of crushed limestone-material at different grading and arrived at the conclusion that the stiffness and resistance to permanent deformation decreased with increasing fines content. This could be explained by the presence of an amount of fines that is larger than the pore spaces between the large particles and thus hinders full particle to particle contact, Figure 2.4 (c). As a result the resistance against permanent deformation and stiffness decreases.



(a) no fines (b) enough fines (c) excess fines

Figure 2. 4 Three physical states of aggregate mixtures [34]

By testing UGMs with gradations that follow the upper, middle and lower German Specification, Werkmeister [29] confidently concluded that the grading (within the limits tested) does not affect the deformation behavior of UGMs significantly. A grading closer to the lower limit should be ideally selected, to guarantee good water permeability and to avoid high moisture contents within the UGLs in a pavement construction.

Studies [35-37] have also shown that the performance of granular materials is significantly impacted by the aggregate's morphological properties, including particle shape, angularity, and surface texture. It is generally recognized that aggregates with equi-dimensional, angular shapes and rough surfaces increase the strength and durability of UGLs in pavements.

Grain shape and texture

Grain shape is established at three different scales: the global form, the scale of major surface features and the scale of surface roughness. Each scale reflects aspects of the formation history, and participates in determining the global behavior of the soil mass, from particle packing to mechanical response. Two general groups can be identified with respect to grain shapes: natural aggregates that generally exhibit rounded shapes and crushed materials in which particle edges can be very sharp. The difference between the natural aggregate having rounded grains and crushed aggregate having sharp- edged grains is most significant on the permanent deformation behavior. Crushed materials are likely to have more grain abrasion, thus high resistance to permanent deformation, than the natural aggregates, especially at high stresses [17].

Hicks and Monismith [38] reported that the resilient modulus was higher for a crushed material than for a partially crushed material regardless of the aggregate gradation. Allen and Thompson [39] and Barksdale and Itani [35] found that the resilient modulus was higher for the crushed rock, than for gravel. These were all well-graded materials. Barksdale and Itani [35] also found that the gravel was more than two times more susceptible to rutting than the crushed aggregates. In contrast Uthus et al. [40] reported that at high stress levels the resilient modulus curves for cubical crushed materials seem to level off, while rounded aggregates have much steeper curves and seem to give steadily increasing resilient moduli with increasing bulk stresses. This was also highlighted previously by Janoo and Bayer [25] who found that the angularity of aggregates had a considerable influence on the resilient modulus. Natural gravel gave higher resilient modulus properties than crushed materials prepared and tested at about the same density levels. This was explained by the ability of rounded gravel particles to better rearrange, in some cases to more stable structures.

On the other hand, Janoo [41] concluded from a laboratory study of unbound material that rounded particles caused significantly higher permanent deformations over time than angular aggregate particles when subjected to cyclic loading. In general it was found that rounded particles were able to slip easily, whereas angular materials had to overcome higher frictional forces at the contact interfaces. From this it was concluded that the angle of internal friction, and thereby the resistance against permanent deformation, increases with increasing angularity.

2.6.2.2. Soil/stone aggregate condition properties

Apart from the material's physical properties, the work of Proctor and others [32] showed that factors such as the moisture content, the magnitude and the manner in which the compactive effort was being applied, as well as the reactive support of the underlying material during the compaction process all had an important influence on the results that could be achieved.

Resilient behavior of UGMs has been found to be very sensitive to moisture content, density and stress level to which the material is exposed. Many studies have reported relationships between resilient behavior and other material properties. Thompson and Robnett [42] conducted an extensive study of resilient properties of Illinois soils. Rada and Witczak [43] presented a comprehensive evaluation of variables that influence the resilient modulus response of granular materials.

Moisture content

Water in a pavement structure has its origin from many sources; groundwater, surface water migrating through the shoulder, ditches or through cracks in the paved surface of the road. In many roads in developing countries the ditches are very shallow and in some cases the drainage system is not designed for large amounts of surface water, so due to the high intensity tropical rain the water level in the ditches may rise and penetrate into the pavement structure. Water is a polar material, which means that the molecules have a definite positive and negative direction. This makes the molecules able to combine with the minerals in the aggregate surface. Water also tends to migrate into the layer's pore system by capillary attraction if the pores are small enough, which is related to the grain size distribution of the material and the amount of fines. The water film on the surface of the grains influences the shear resistance. The occurrence of a moderate amount of moisture benefits the strength and the stress and strain behavior of UGMs. Having achieved total saturation, repeated load applications may lead to the development of positive pore water pressure. Excessive pore water pressure reduces the effective stress, resulting in diminishing deformation resistance of the material. Thus a high water content within an UGL causes a reduction in stiffness and hence deformation resistance of the layer [17].

Many researchers have studied the effect of water on the resilient modulus. Hicks and Monismith [38] reported an apparent reduction in resilient modulus with increasing water content. Barksdale and Itani [35] observed a significant decrease in resilient modulus for four materials tested upon soaking. All samples were run under drained conditions. Raad et al. [44] concluded that the effect of water on the resilient properties seemed to be most significant in well-graded materials with a high amount of fines.

Through an extensive laboratory investigation into the influence of water on sand, granular base course material and tropical laterites, Sweere [18] has reached to the conclusion that moisture has a significant role in the stiffness behavior of granular materials. Laterites containing an excess of fines, having a structure of a matrix of fines with coarse particles floating in it similar were shown to have a moisture dependent stiffness. Other laterites, with a grading closer to the Fuller-curve and thus consisting of a skeleton of coarse particles, were shown to be far less moisture dependent with respect to their stiffness. The behavior of sands in this investigation upon change in moisture content was consistent with the skeleton-type of structure; the stiffness which is mainly derived from the skeleton itself was hardly dependent on the water content.

In the same study Sweere [18] found, surprisingly, for both a fine graded and a coarse graded crushed rock material a marked moisture dependent stiffness. The stiffness of the fine graded material was more dependent on moisture than the stiffness of the coarse graded material, which is consistent with expectation. In general the effect of water on the behavior of granular materials is greatly related with the amount and nature of the fines.

Density

The density of the grain skeleton is one of the most important factors influencing the stiffness and resistance to permanent deformation. Barksdale [26] studied the effect of density on the deformation behavior of granular assemblies using cyclic load triaxial tests. He observed an increase in stiffness and resistance to permanent deformation with an increase in degree of compaction expressed as percent of maximum Proctor dry density. Similar results were obtained by Marek [45].

Hicks and Monismith [38] reported that the effect of density on the resilient modulus was greater for a partially crushed material than for a crushed material. They also found that the effect of density decreased with increasing fines content. An increasing dry density increases the shear strength of a material [22, 46]. A material having high shear strength may be more difficult to compact, as they also resist the shear stresses induced by the compaction.

Van Niekerk [47] has also investigated the influence of the degree of compaction (DOC) on the resilient modulus and resistance for permanent deformation for recycled mix granulates in 3 different gradings. He concluded that the resilient modulus in general increases significantly with increasing DOC. The rate of the increase was also found to be related to the grading. A 50%, 80% and 30% increase in modulus was observed for the upper, average and lower specification limits respectively for an increase from 97% to 105% DOC (expressed in percent maximum standard Proctor dry density). Figure 2.6 shows the increase of the resilient modulus (Mr-θ relation) with DOC for the average limit (AL) gradation after 4 days curing.

Uthus [24] showed that a well-graded material is mostly influenced by the dry density up to a certain level of fines content. The dry density of a material with a relatively high amount of fines seems to be important under dry conditions, but as the fines content increases the moisture content seems to override the effect of the dry density of the samples. For equal dry densities both the resilient and permanent deformation seems to increase as the fines content increases. Hence a high amount of fines gives a lower resilient modulus as well as lower shear strength depending on the moisture content. The mechanical behavior of unbound granular layers in pavements is complex. A granular layer is a particulate, not a continuous medium. The response of an element of granular material in a pavement depends on its stress history and the current stress state in addition to the degree of saturation and density.

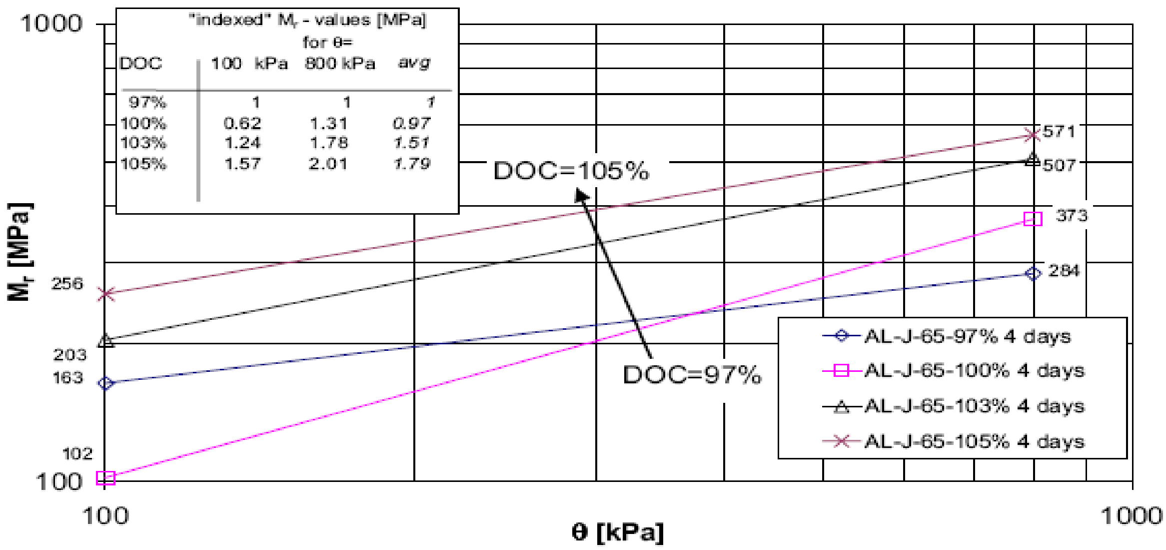


Figure 2. 5 Mr –θ relation as a function of DOC for AL after 4 days curing [47].

Stress level

The response of a material to cyclic loading is very dependent on the stress level. Hicks and Monismith [38] reported that the stress level affects the resilient modulus most significantly. They found that in all cases the resilient modulus increased considerably with increasing confining stress, and slightly with increasing axial stress. Allen and Thompson [39] also found that the testing variable that affected the resilient modulus the most was the applied state of stress.

Uthus [24] also found that for all his tests the general trend is that the resilient modulus and the resistance to permanent deformation increase with an increasing mean stress and increasing confining stress. The stiffness and strength of the material tested is more dependent on the confining stress than on the deviatoric stress. The confining stress seems to be 3 to 5 times as powerful as the deviatoric one. He found it reasonable to interpret the resilient modulus as a function of mean stress or bulk stress, as the confining stress is the dominant parameter when using bulk or mean stress.

These trends mentioned above are true for relatively low load levels. However, Van Niekerk [47] has classified the loading regime into 'mild' and 'severe' loading and observed that the resilient deformation behavior is clearly influenced by the severity of loading. For his recycled crush concrete and masonry mix granulates developed granulate bonds are much more damaged under severe loading than under mild loading. For these materials the Mr-θ relations obtained under mild loading lay higher than those under severe loading.

2.7. Characterization of Unbound Granular Materials

Characterization of pavement materials is a key requirement for the pavement design process. For the mechanistic-empirical pavement design process the characterization task involves obtaining material properties that identify the material response to external stimuli of pavement loading and environmental conditions.

2.7.1. Pavement loadings

The traffic loading that a pavement sustains can be divided into two key elements namely the stress applied and the number of repetitions of that stress. For design purposes these are often simplified into the number of passes of a standard axle load expressed in units of an equivalent standard axle [48]. However, from a fundamental point of view the actual pavement loadings are more complex, as the duration, frequency and magnitude of stress applied are not necessarily consistent throughout the pavement's life [20, 49]. The deeper within the pavement, the longer the stress pulse lasts for a given vehicle travelling at the same speed. In addition, the magnitude of stress varies depending on the magnitude of the traffic loads and the properties and thickness of the overlying pavement layers.

Considering the stress regimes typically induced by a moving wheel load, pavement elements experience various combinations of horizontal (σh), vertical (σv), and shear (τ) stresses as shown in Figure 2.7 [50].

Figure 2.7 shows the variations of stresses with time under a moving wheel load. The shear stress is reversed as the load passes and there is thus a rotation of the axes of principal stress. Chan [51] demonstrated that the rotation of principal stress doesn't have a significant influence on the stiffness modulus of granular pavement materials for a given applied stress. However, this phenomenon does have a major bearing on the permanent deformation of materials [49, 51].

The ability of test equipment to reproduce the fundamental stress state under a moving wheel is discussed further under section 2.5.2. However the reproduction of the rotation of principal stresses is not essential to being able to directly measure the material's stiffness modulus. The intention of this study is also not to develop or apply a fundamental laboratory testing technique but rather to simplify the advanced testing technique in order to approximate the fundamental properties in a more practical way.

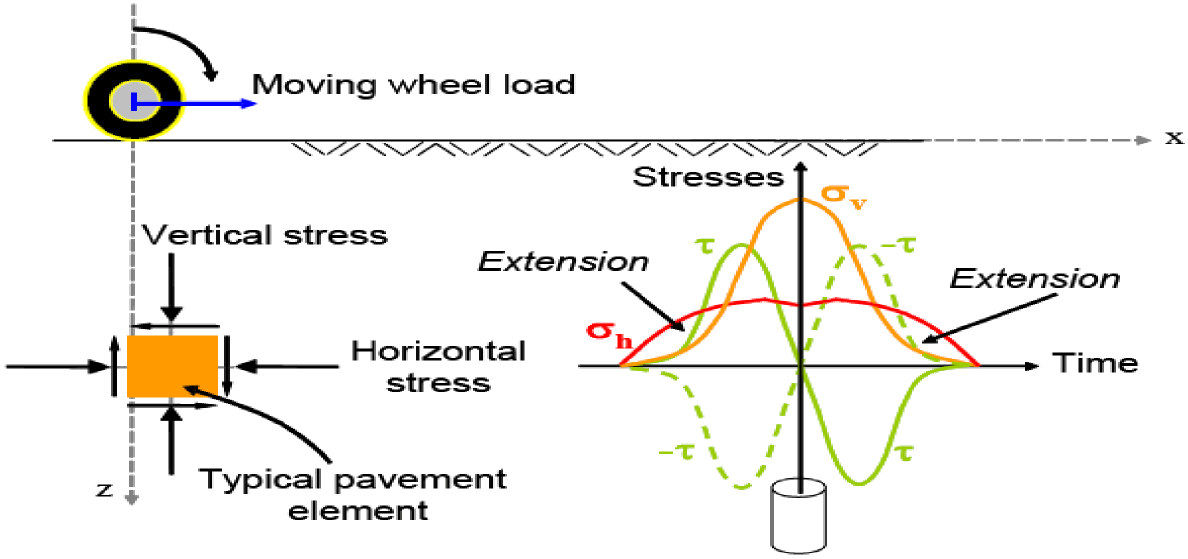


Figure 2. 5 Stress regimes experienced by a pavement element under a moving wheel load [50].

2.7.2. Laboratory characterization techniques

A large number of laboratory testing techniques are presently being used to investigate compaction, bearing capacity and degradation of UGMs. Most of these tests are index tests, developed to provide input-data for empirical pavement design procedures or to provide a means of qualitative comparison of different materials. However, the fundamental material properties of UGMs cannot be derived from classification or index tests of the materials [20]. Direct measurement of those properties in-situ or within the laboratory is preferred. The main focus of this research study is on direct measurement under laboratory conditions.

A review of performance related tests of aggregates for use in unbound layers undertaken by the NCHRP [52] details the range of testing equipment available (from a US perspective) for determination of various material properties. In their assessment some of the techniques that can be used for determination of the stiffness modulus such as Hollow Cylinder Triaxial (HCT), K-mould etc appear only for shear strength measurement. On the other hand, they conclude that the shear strength of an aggregate skeleton has a much greater influence on the performance of an unbound aggregate pavement layer than any other aggregate property. Its stiffness is directly related to shear strength they agree that stiffness has a similarly large effect on performance. In this section some of the main testing techniques that can be used to measure the stiffness modulus of UGMs will be reviewed. In addition these methods will be evaluated in terms of simplicity, affordability and availability of such techniques from developing countries perspective. Figure 2.8 shows the laboratory test methods and their stress states during testing.

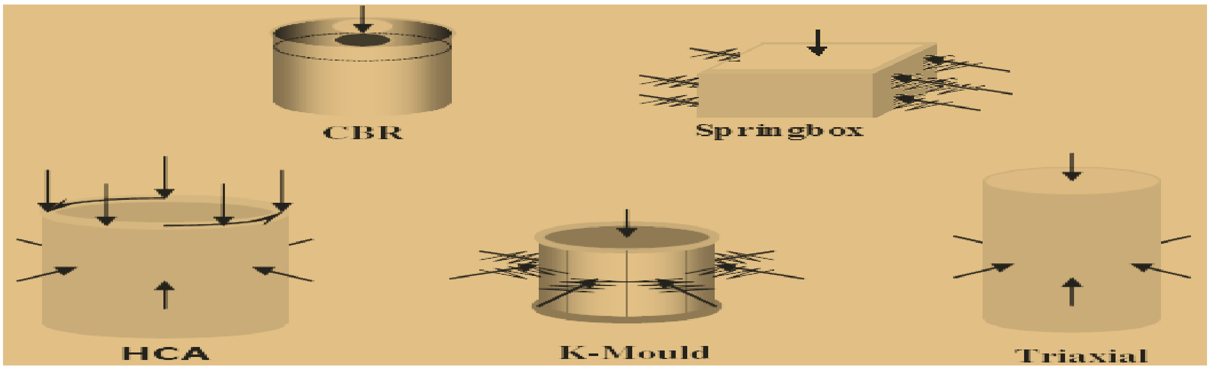


Figure 2. 6 Laboratory tests showing stress states during testing [19, 49]

2.7.2.1. Hollow cylinder triaxial and cyclic load triaxial tests

Hollow cylinder triaxial(HCT).

For an analytical design method, the most obvious way of measuring parameters from laboratory testing is to reproduce the loading conditions that will occur during the pavement service life. Hollow cylinder tests can replicate in a close match the complex pavement field loadings including the reversal of shear stresses. In a hollow cylinder triaxial (HCT) reversed shear stresses are simulated by applying torsion (cyclically) to a specimen shaped as a thin-walled hollow cylinder. Tests on hollow cylinder specimens of soil were perhaps first reported by Cooling and Smith in nineteen thirties [53]. They applied torque on an unconfined specimen of soil. Since then, a number of researchers have conducted tests on hollow cylinder specimens to investigate various aspects of the mechanical behavior of soils and rocks. Saada and Baah [54] used the hollow cylinder specimen to study anisotropy in the deformation and strength of clays. Lade [55] put efforts towards the influence of stress reorientation on the stress–strain behavior of sands. Hight et al. [56] investigated the effects of principal stress rotation in soils. Seaad [57] discussed the advantages and limitations of hollow cylinder tests.

The testing apparatus usually includes an axial-torsional loading system, a confining pressure system, and a hollow cylinder triaxial cell. Three independent external stresses are applied: radial confining stress both inside and outside the specimen, axial and torsional stresses. The desired stress path is implemented through the changes in the confining pressure, axial stress, and torsional stress, Figure 2.9.

The HCT is, however, a research tool with a number of practical limitations including complexity, availability and productivity. It is usually used for fine grained subgrade soils and sand particles; the maximum aggregate size is reported as being 12.5 mm [49]. HCT testing for fine grained materials for research application is already extremely complex, and such HCT testing for coarser grained base and subbase materials is even less feasible. The use of the relatively more accessible and simpler cyclic load triaxial test dominates most resilient modulus research.

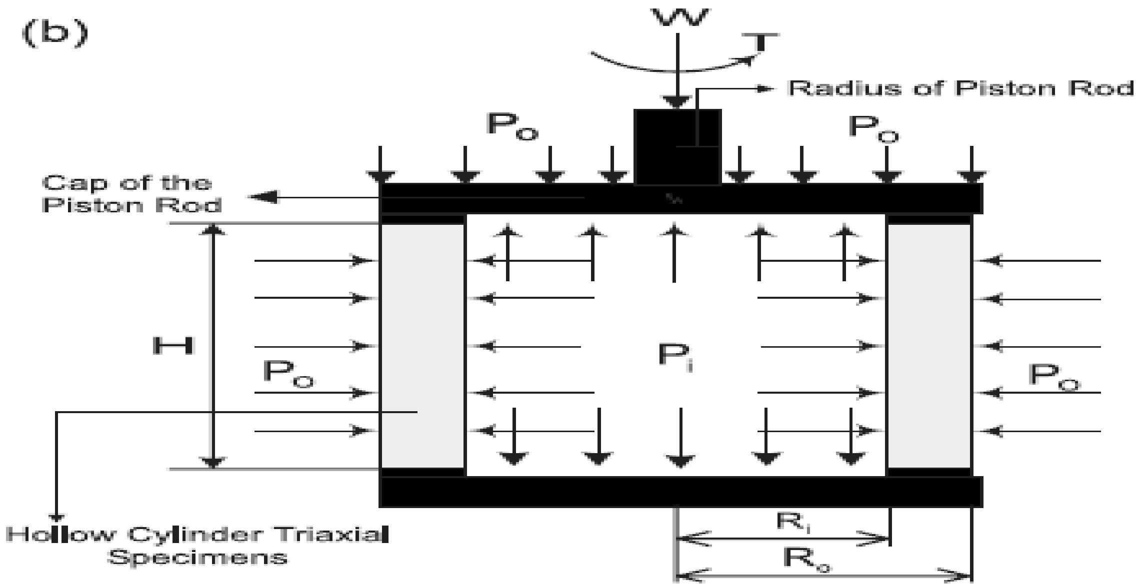


Figure 2. 7 Schematic diagram of HCT and loading conditions [53]

Cyclic load triaxial

Triaxial testing was first developed for the determination of failure properties, namely the angle of internal friction and cohesion, for use in geotechnical engineering. Seed et al. [58] recognized that the monotonic slow stress increase used in such test does not necessarily give a satisfactory indication of the performance of soils under repeated loading condition. Moreover the in-service loading condition associated with granular pavement layers is likely to be well below that of failure [20]. Therefore, standard triaxial test equipment requires significant adaptations to simulate the large number of repeated loadings applied to pavement structures.

The ratio of specimen diameter to maximum particle size is another aspect that is still a topic of discussion in testing UGMs. Since most of the cyclic load triaxial equipment presently available have a specimen diameter of 150 mm or less, it only allows for testing of materials with a maximum particle size of say 25 mm as there is a general suggestion for the specimen diameter being at least 5 to 6 times the largest particle size [59-61]. Therefore triaxial tests on coarse base and subbase materials having nominal grading of, for instance, 0/50 mm or 0/63 mm are usually carried out on a so called scaled- down gradings, which means that particles larger than say 25 mm are replaced by finer particles.

The effect of scaling down the grading on the resilient behavior of UGMs has been studied at the Nottingham University and the Delft University of Technology [18, 62]. It is reported that a significant decrease in stiffness was found on reduction of the maximum grain size. Similarly, Thom [63] investigated the maximum particle size of granular materials to have a significant effect on stiffness. These results indicate that granular materials should be tested at their full grading to obtain the stiffness parameters needed for pavement design.

For that purpose a large scale triaxial testing facility is established at the Delft University of Technology in the Road and Railway Engineering Laboratory. Several researches [18, 47, 25] have been carried out with this large scale triaxial test set-up in testing coarse base and subbase materials. In this research, see chapter 4, this large scale triaxial testing (300 mm specimen diameter and 600 mm specimen height) is used to characterize the failure and resilient behavior of tropical and European coarse grained base and subbase materials. The triaxial test is basically used for determination of a number of parameters. From the monotonic test e.g. one determines the internal angle of friction and cohesion and from the repeated load test the resilient strain and the permanent strain parameters. In the cyclic load triaxial test for establishment of the resilient and permanent deformation behavior, it is necessary to accurately measure the specimen deformations under the applied stress directions and magnitudes. This requires accurate displacement measuring devices such as linear variable displacement transducers (LVDTs) as shown in Figure 2.11.

As stated earlier, section 2.5.3, UGMs are highly stress dependent. For this reason resilient strain parameters like the resilient modulus Mr have to be determined at a number of stress levels for each material tested and these resilient characteristics of UGMs are not affected by loading history. Therefore a large number of tests for the determination of resilient parameters can be carried out on the same specimen, provided that the stresses applied are kept low enough to prevent substantial permanent volume-change of the specimen.

Permanent strains in UGMs are, on the contrary, affected significantly by the loading history [18, 47, 64]. Therefore, several triaxial specimens have to be tested to obtain the relationship between applied stress ratio and permanent deformation. Each test usually involves a large number of load applications (65 - 66) on each specimen, which renders the determination of permanent strain characteristics to be quite time-consuming.

Constant confining pressure (CCP) vs. Variable confining pressure (VCP)

As shown in section 2.5.1 the lateral pressure applied to an element of material beneath the pavement gradually increases as a vehicle approaches and then decreases as the vehicle moves away. In CCP tests, it is only possible to apply one constant stress path. The VCP type test enable s to apply a wide combination of stress paths by application of both a cyclic confining pressure and a cyclic vertical deviator stress. Such stress path loading tests better simulate actual field conditions, since in a pavement structure the confining stresses acting on the UGM are cyclic in nature.

1 specimen
2 membrane
3 specimen cap
4 specimen base
5 load cell
6 axial linear variable displacement transducers
7 radial linear variable displacementt transducers
8 triaxial cell wall
9 pressurere transducer
10 studs supporting the displacement transducers
11 drainage circuit

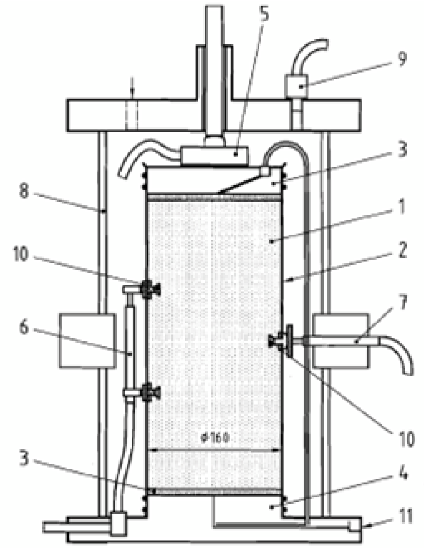


Figure 2. 8 Schematic diagram of standard triaxial test measurements [60]

In his investigation of aggregate skeletons of asphalt mixtures Muraya [67] has demonstrated the significant difference in permanent deformation resulting from the two confining methods. He has shown that for triaxial tests conducted at similar vertical to maximum (failure) stress ratios, tests conducted at cyclic confinement resulted in higher permanent deformation in comparison to the tests conducted at constant confinement. The VCP triaxial test is considered to be a closer simulation of reality as both the constant overburden stress and the variable traffic induced increase of the horizontal stress can be simulated by applying horizontal stress by a constant component and a variable component in phase with the variable vertical stress. The major drawback of both the CCP and VCP confining stress triaxial tests is that only normal principal stresses can be applied. The shear stresses developing under a moving wheel load cannot be applied, unlike the hollow cylinder triaxial test.

Advantages and limitations of cyclic triaxial testing UGMs

The advantage of using cyclic triaxial systems to measure dynamic properties are widely discussed, and are primarily related to the relative capability of simulating the traffic loading actions in pavements. Compared to other methods for testing UGMs (e.g. CBR), the cyclic triaxial test is well suited. Some of the main advantages are:

- flexible load applications (amplitude, frequency, number of pulses);
- controlled confining pressure (variable or constant);
- accurate axial and radial strain measurements (permanent and resilient);

- results can be used directly in advanced material models to predict performance of UGMs in a pavement structure.

The method has also some drawbacks. The most important are:

- unable to simulate continuous rotation of principle stress directions;
- confinement is imposed by controlling externally applied confining pressure unlike the field confinement which develops as a result of resistance to material deformation and reorientation;
- unable to test undisturbed samples from the field;
- real size aggregates often require unpractical large samples for coarse materials;
- expensive and time consuming (much work required per sample);
- Particularly for developing countries technically it is too complex, and too expensive to be used for routine road projects.

Many researches [49, 68, 69] have been carried out to look for an alternative characterization technique that can deliver a good estimation of the mechanical behaviors such as the resilient and shear properties of UGMs. Some of these techniques, the South African K-mould, the modified Hveem stabilometer and the Nottingham Springbox will be discussed in the next section as well as their pro's and con's.

2.7.2.2. The K-mould, modified Hveem stabilometer and springbox tests

K-mould

The K-mould was developed in South Africa by the Division of Roads and Transport Technology for rapid determination of elastic and shear properties of pavement construction materials [69]. The K-mould consists of an internal thick-walled cylinder (with an internal diameter of 152.4 mm) made up of eight equal case-hardened circular segments. Each segment is mounted on two horizontal shafts, which fit into two mounted linear ball bearings to allow each segment to move freely in a radial direction.

Semmelink [69] recognized that the main advantage of this test device is that the stiffness of the mould is infinitely variable and can therefore be adjusted to simulate the inherent lateral support of the material in its natural state. The K-mould spring plates can either be locked in place (preventing horizontal deformation of the specimen) or the specimen can be permitted to horizontally deform via the variable confinement provided by the spring plates.

The advantage of the K-mould compared to triaxial test is that it is more productive (in terms of ease of test set-up and instrumentation). The test only requires one specimen to determine a Mohr coulomb envelope and all the elastic and shear properties can be determined for each specimen.

Moreover, the confining stress developed in such mechanism is close to the reality in the field. That is the confining stress in K-mould test is a result of material deformation and reorientation.

Disadvantages of the K-mould as recognized by Van Niekerk [47] are its present limited specimen height to diameter ratio and the fact that the rigid steel wall segments and springs result in a uniform deformation and thus most likely a non-uniform horizontal stress over the height of the specimen. Edward [49] has also noticed the complex construction of the segmented mould as main disadvantage.

Modified Hveem stabilometer

Ter Huerne [70] has also modified the Hveem stabilometer (HSM) in order to simulate hot mix asphalt mixtures compaction. In Hveem stabilometer the radial confining pressure on the sample occurs due to radial deformation, which is more or less identical to the way the radial stress develops during compaction under field conditions. The fundamental concept behind the Hveem stabilometer was that the characterization of a granular based material could be achieved by measuring its ability to carry a reasonable axial load without too much radial deformation [71]. However, in its standard form it does not have adequate control of the sample volume. To achieve accuracy on volume control over the sample and make the confining stress adjustable Ter Huerne [70] has modified the stabilometer.

The basic principles of the modified Hveem stabilometer (MHSM) are the same as the Hveem stabilometer; a vertical loading on the sample generates a radial displacement and this radial deformation generates a radial confining stress on the sample. Due to the modification, the confinement stress strain relationship on the sample is now approximately linear and the radial expansion volume can be measured accurately. The MHSM test provides the axial loading, axial deformation, radial stress and radial displacement. From these the axial and radial stresses and strains can be determined and the Voids in Mineral Aggregates (VMA) of the mixture can be derived to characterize the material behavior at any stage of the test.

A small uncertainty during the test is the way the sample deforms radially. For calculating the radial deformation from the piston displacement the assumption of homogeneous radial deformation of the sample was made i.e. no barreling. Laboratory measurements, however, indicated that little barreling did occur i.e. the diameter in the middle is 2 to 3% larger than the diameter at the top and the bottom of the sample which will have a small error on the calculated radial strains.

Spring box

A new laboratory test equipment known as 'Springbox' [49] was developed at Scott Wilson Pavement Engineering Limited for the characterization of unbound and weak hydraulically bound mixtures under repeated loading. It is basically based around the principle of a variable confinement test (self-controlled), similar to the South African K-Mould. The Springbox specimen is a cube with dimensions of 170 mm. The mould consists of a pair of horizontal faces, which are spring-supported and thus permit a horizontal strain, and the other pair fixed. The form of test is therefore to apply a vertical pulsed load to the full upper surface of the specimen, and recording both the vertical displacement and the displacement in the movable horizontal direction. A schematic representation along the longitudinal section of the Springbox mould is shown in Figure 2.11. The fact that the horizontal stress is not controlled but develops as a result of the vertical deformation is considered to be as an advantage in terms of simplicity of equipment and execution of testing. An important aspect of such apparatuses is however that the confinement is dependent on the spring stiffness characteristics of the apparatus. Another limitation for application of such testing apparatus, particularly in developing countries, is its complexity and the fact that it has been designed for use within the Nottingham Asphalt Tester (NAT) loading frame which is not available in these countries.

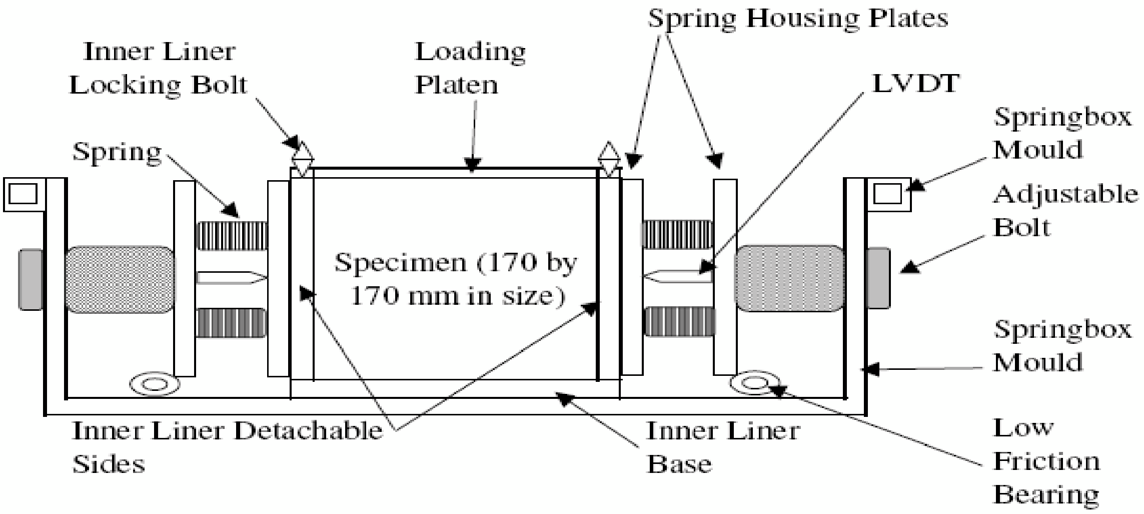


Figure 2. 9 Longitudinal section through the Springbox apparatus [49]

2.7.2.3. CBR tests

California Bearing Ratio

The California Bearing Ratio (CBR) test is a long established, very extensively applied test yielding an empirical measure of the quality of granular road materials. The CBR-test was developed initially for the evaluation of the laboratory and in-situ subgrade strength. Presently, the laboratory CBR-test is used throughout the world as a quick means of characterizing qualitatively the bearing capacity of soils and unbound base and subbase materials. The CBR-value still is an input value to many pavement design procedures, such as AASHTO [72] and TRRL [73] design methods.

The test is a penetration test in which a plunger with a cross sectional area of 1935 mm2 (49.63 mm dia.) is pushed with a constant 1.27 mm/min displacement rate into a sample contained in a steel cylinder with a diameter of 152.4 mm (6 inch). Although vast experience is built up with this specific test it is actually at best a strength test which gives some information on the shear resistance of the material in relation to its degree of compaction and moisture content. The CBR value is determined on the basis of the force Fa at 2.54 mm (0.1 inch) penetration, CBR1, and the force Fb at 5.08 mm (0.2 inch) penetration, CBR2, using equation 2-2.

$$CBR_1 = \frac{F_a}{1935 * 6.9} * 100\%$$

$$CBR_2 = \frac{F_b}{1935 * 10.3} * 100\% \qquad 2\text{-}2$$

Where Fa, Fb = force at 2.54 and 5.08 mm penetration respectively [N], 1935 = surface of the load area [mm2], 6.9 = contact stress on a standard sample of crushed rock at 2.54 mm penetration [MPa], 10.3 = contact stress on a standard sample of crushed rock at 5.08 mm penetration [MPa]

According to the European standard [74] the CBR value of the material is the higher percentage of the two, in most cases CBR1 is larger than CBR2. In testing unbound granular materials problems arise regarding the ratio of mould and plunger dimensions to maximum particle size of the material to be tested. If for instance, the CBR-test is performed on a 0/45 mm graded material, the diameter of the CBR-plunger and the largest particles would be almost equal. The rigid CBR-mould of 152.4 mm internal diameter gives an unknown and uncontrollable confining stress to the material specimen. To avoid such problems, test specifications often prescribe removal of coarse particles from the test material. In case of coarse graded materials, this removal leads to testing of a material having a grading which differs substantially from the original material which influences the parameters to be measured.

Because of its longtime worldwide use, the CBR-test is also being used to obtain material stiffness parameters for input to analytical design procedures. Since these procedures require fundamental material properties like elastic modulus, E, for input, several empirical correlations between E and CBR have been developed. Sweere [18] noted, however, that deformation occurring in the CBR-specimen is a combination of elastic and plastic deformation. Since these two types of deformation cannot be distinguished in the test and since the ratio of elastic to plastic deformation may differ from one material to another, the standard CBR-test is unsuited for determination of a purely elastic parameter like an elastic modulus.

CHAPTER-THREE
RESEARCH METHODOLOGY

3.1. Study Area

The research was conducted on the already existing trunk road that are completed and substantially constructed during the different road sector development stages having high deterioration problems at different locations from Jimma – Sekoru. Jimma to Sekoru road segment is located at Oromia National Regional State, part of the south west Ethiopia, at a distance 250 Km from Addis Ababa. The study area is characterized by rugged topography having a maximum elevation of around 1936m and a minimum elevation value of around 1718.4m. Its astronomical location is (3 6º50'00'', 7º40'01'') to (37º25'59'', 7º55'37'') of longitude and latitude of the two towns respectively.

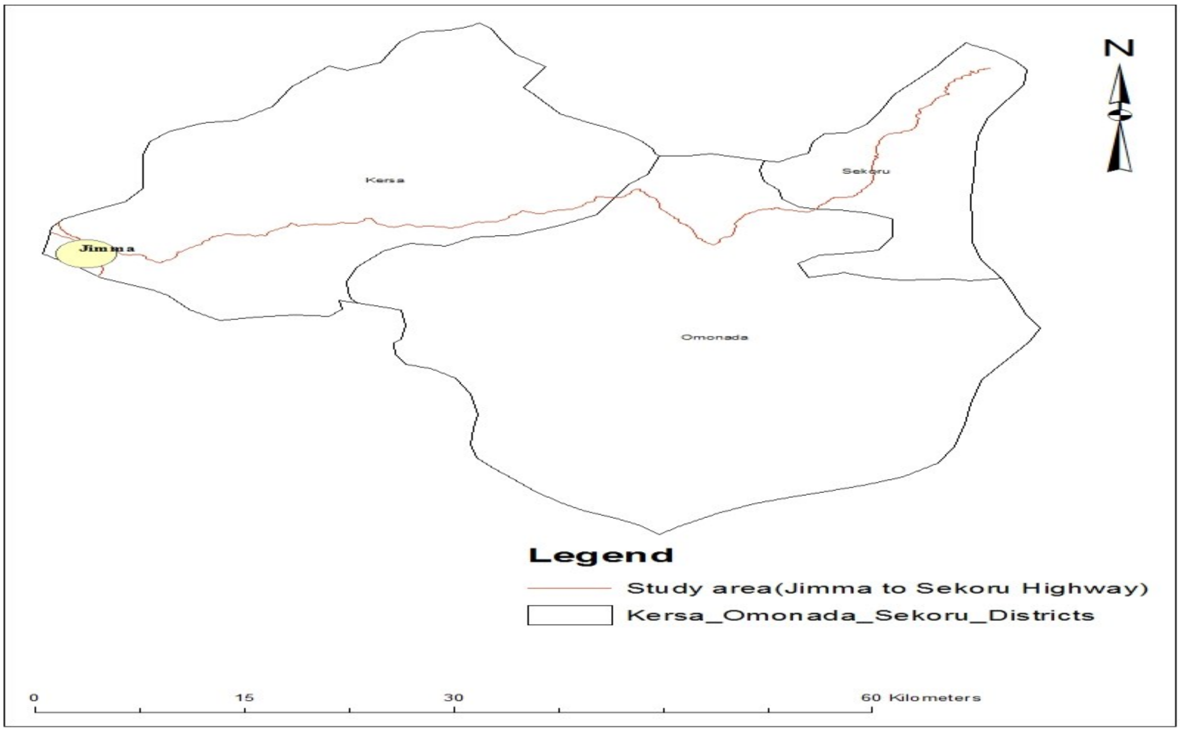


Figure 3. 1 Location of the study area (Source: Google Map, 2017)

3.1.1 Identification of Soil in the Study Area

Site visiting is the first and foremost for investigation of pavement distress. Site visit was made along the roadway from Jimma town to Sekoru and identify the roadway which are damaged. Furthermore, Consulting with the town municipality administrative body and other concerned people were also arranged to collect information about the geology of subgrade soil along the roadway and quarry site of sub base material geological natures, soil texture and other related problems along the roadway. After observation of the distressed pavement surface from Jimma to

Sekoru road, sample areas were selected from different stations of pavement structure based on the geological nature of the soils having different subgrade soils and sub base materials. Test pits were excavated up to a layers of each pavement structure and, three representative samples from the sampling area were taken from subgrade soil and one representative sample from sub-base materials.

3.2 Study Design

The procedure utilized throughout the conduct of this research study are as follows: Continuous Reviewed related literatures on relevant areas of Effects of Subgrade and sub base soil material on pavement structure, includes articles, reference books, research papers, standards specifications like ERA,AASHTO and ASTM. Necessary data collection, organization, comparison and analysis were obtained, and then subsequently compared the results with preexisting literature and standard specifications, as shown in Figure 3.2.

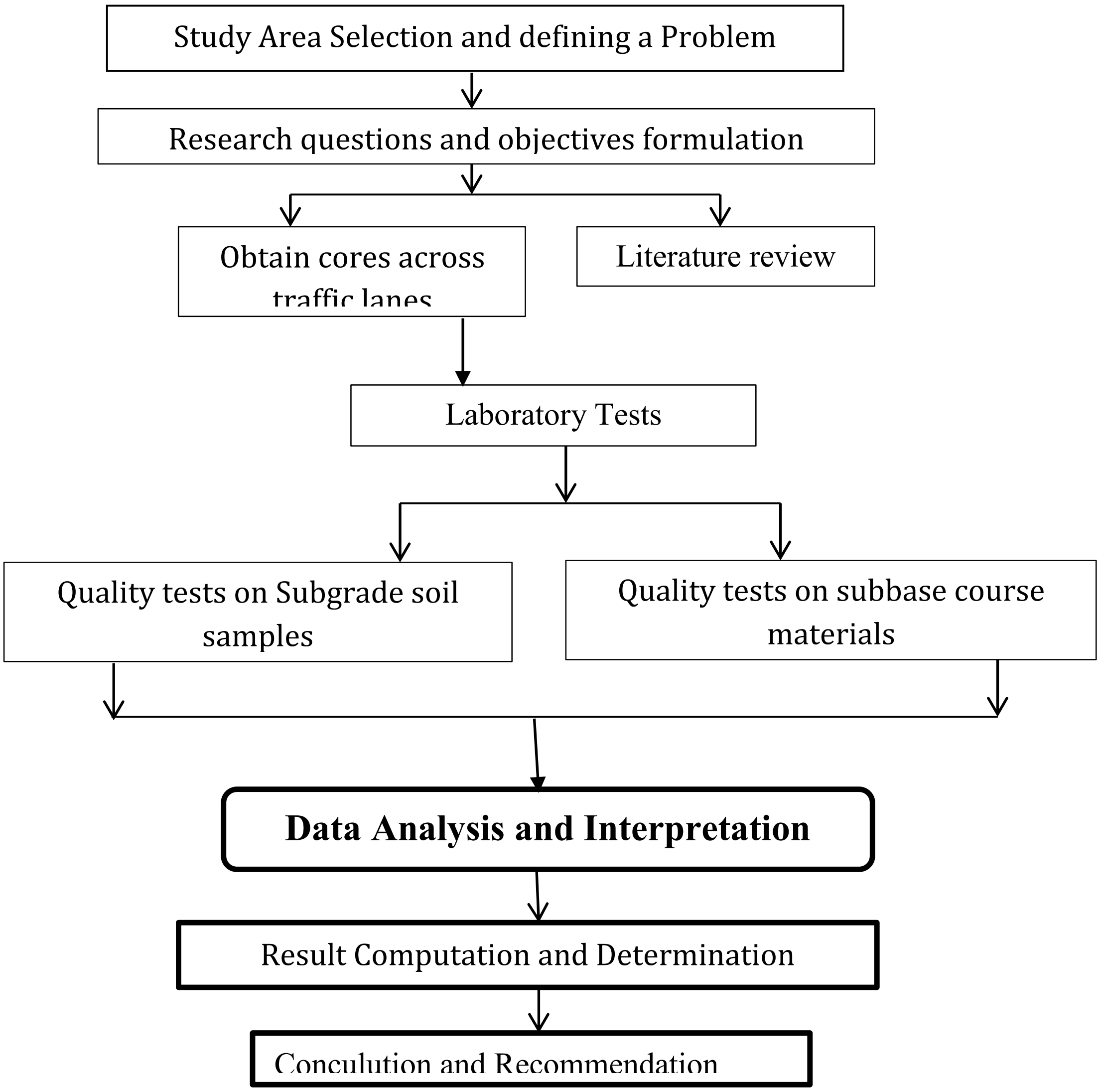


Figure 3. 2 Research design Chart

The study involves investigating the quality of subgrade and sub base materials which highly affects the corridor pavement performance. After choosing an area and a period of investigation, the analysis method will be designed or carried out in following phases: Evaluating and collecting traffic and construction data of the subject trunk-road selected for the case-study, Obtaining cores of subgrade and sub base across traffic lanes of the selected sections, Conducting laboratory tests on the core samples, and conducting statistical correlation analysis between the present and the recommended material strength.

3.3 Population

3.3.1 Source Population

The existing road segment is highly affected by deterioration. Surface cracks, alligator cracks, block cracking, shoving, corrugation and others are pavement problems which happened in the road corridor. So these problems are the source of population.

3.3.2 Study Population

The study population of this research is the material quality of subgrade and sub base course of the pavement from Jimma to Sekoru segment.

3.4 Sampling Technique and Sample Size

3.4.1 Sampling Technique

Since Non-probability sampling represents a group of sampling techniques (like; purposive sampling) and has free distribution that help researchers to select a unit sample from a population, this sampling method is adopted.

3.4.2 Method of Sample Size

For evaluation of the effects of subgrade and sub base material quality on deterioration performance of flexible pavement, simple and multiple regression techniques will be used. A minimum of traffic data, axle load data, subgrade CBR, sub base quality and pavement section details of sampling sections, details of sampling and field measurement locations, and others were collected from both ERA (project completion report) and the one who consults the road segment. The following information will be recorded, traffic data, axle load data, project completion report, subgrade CBR, sub base material quality, pavement section details etc.

3.5. Study Variables

There are two types of variables that will be taken into consideration;

3.5.1. Independent Variables

- Performance of the flexible pavement

3.5.2. Dependent Variable

- Aggregate Properties of the sub base course
- Subgrade soil properties of the corridor

3.6 Data Collection Process:

The research was conducted first by identifying the effects of subgrade and sub base material quality on deterioration performance and traffic safety through literature review, desk study at the selected locations, report documents, and scientific researches. The laboratories tests followed then from the findings, statistical correlation methods and regression analysis was developed and interpreted.

3.6.1 Data Collection Methods:

The primary research data was collected through site visits, direct core sampling on the traffic lanes whereas the secondary data are collected through the existing relevant documents, project completed report, literatures, traffic data and axle load data for further analysis.

3.6.2 Data Types and Sources:

Quantitative as well as qualitative data types will employ. Of the total data the major component of the research data was collected from field observations/investigation and laboratory test results, the rest is collected from secondary data sources from ERA and the consultant firm.

3.7 Data Processing and Analysis:

For evaluation of the effects of subgrade and sub base material quality on deterioration performance flexible pavement beside laboratory tests, simple and multiple regression techniques were used. For the results to be considered significant, final regression equations will have a regression coefficient of R^2 that is significant for simple and multiple regression analyses. Thus, from the findings correlations will be developed and interpreted using a purpose built statistical modeling programs to solve deterioration as well as traffic safety problems. Final conclusions were drawn after the interpretation of each explanatory variable.

3.8 Ethical Consideration:

Prior to data collection an official letter was written by JIT to obtain the relevant data from the respective agencies and other administrative offices. Before the collection of the data, the purpose of the data collection was clearly described to the organizations by the data collectors and the principal investigator. The data were collected based on the willingness of the organizations to give information. It was kept confidential and was used only for the research purpose.

3.9 Data Quality Assurance:

In order to increase the quality of the data I were prepare a field work manual to check every day progress for reliability and accuracy. Pre –test of the available road, axle load data and traffic data would have to be done before the main data collection period begin and the data was collected after gaining an awareness on how to collect relevant data would be done by the principal investigator.

3.10 Software and instruments

The following instruments and software were used for this study: Meter tape, plastic bags, manual hand auger equipment, laboratory equipment's, and field test instrument, Digital Camera for documentation, MS word and Excel to analysis laboratory data were used in this study.

3.11 Field work

After reviewing the literature and before conducting laboratory test, the following activities were undertaken. This section includes the identification of geological nature of subgrade and sub base soils depending on their grain sizes, and the condition of the existing structures around the study area was conducted.

Field observations, were carried out and representative samples were taken to laboratory tests. During the field observation, it was necessary to begin by conducting visual inspection and site inventory of the whole soil classification zones from Jimma town to Sekoru road segment.

After finishing the initial visual inspection and categorizing the soil conditions of the road segments from Jimma to Sekoru and, the next step was then to select the representative locations for sampling based on the availability of sub base and sub grade soil structures.

3.12 Laboratory tests

3.12.1 Subgrade and sub base Soil Classification

Subgrade and sub base is the trimmed or prepared portion of the formation on which the pavement is constructed and the material used to construct the layer needs different experimental works. Soil classification is the arrangement of soils into different group in order that the soils in a particular group would have similar behavior. A classification scheme provides a method of identifying soils in a particular group that would likely exhibit similar characteristics. There are different classification devises such as USCS and AASHTO classification systems, which are used to specify a certain soil type that is best suitable for a specific application. These classification systems divide the soil into two groups: cohesive or fine-grained soils and cohesion-less or coarse-grained soils. The method of classification used in this study was the AASHTO System. The AASHTO Classification system is useful for classifying soils for highways. The particle size analysis and the plasticity characteristics are required to classify a soil. The soils with the lowest number, A-1, is the most suitable as a highway material. The experimental work comprises for subgrade and sub base materials are the following. Thus are particle size distribution, determination of plasticity property: LL, PL and PI, estimation of MDD and OMC by modified proctor test, and, determination of CBR strength.

3.12.2 Grain-size Distribution of Soil

The Standard grain size analysis test determines the relative proportions of different grain sizes as they are distributed among certain size ranges. One of the main descriptors of soil used for engineering purposes is the distribution of grain sizes in the soil mass.

In any soil mass, the sizes of the grains vary greatly. To classify a soil properly, you must know its grain size distribution. The grain-size distribution of coarse-grained soil is generally determined by means of sieve analysis. For a fine-grained soil, the grain-size distribution can be obtained by means of hydrometer analysis, but it is enough to use the sieve analysis tests in this research study in order to classify the subgrade soils in to Fine-grained and Coarse-grained soils.

3.12.3 Sieve Analysis

A sieve analysis is conducted by taking a measured amount of dry, well-pulverized soil and passing it through a stack of progressively finer sieves with a pan at the bottom. The amount of soil retained on each sieve is measured, and the cumulative percentage of soil passing through each is determined. This percentage is generally referred to as percent finer. The distribution of different grain sizes affects the engineering properties of soil. Grain size analysis provides the grain size distribution, and it is required in classifying the soil. ASTM D 422 - Standard Test Method for Particle-Size Analysis of Soils is used in the test analysis.

3.12.4 Atterberg limits

When clay minerals are present in fine-grained soil, the soil can be remolded in the presence of some moisture without crumbling. This cohesive nature is caused by the adsorbed water surrounding the clay particles. At very low moisture content, soil behaves more like a solid. When the moisture content is very high, the soil and water may flow like a liquid. Hence, on an arbitrary basis, depending on the moisture content, the behavior of soil can be divided into four basic states; solid, semisolid, plastic, and liquid. The moisture content, in percent, at which the transition from solid to semisolid state takes place, is defined as the shrinkage limit. The moisture content at the point of transition from semisolid to plastic state is the plastic limit, and from plastic to liquid state is the liquid limit. These parameters are also known as Atterberg limits. The plasticity is measured by the "plasticity index". The plasticity index is defined as the numerical difference between its Liquid limit and Plastic limit.

Liquid Limit: The liquid limit (LL) is the water content, expressed in percent, at which the soil changes from a plastic to liquid and principally it is defined as the water content at which the soil pat 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. A soil containing high water content is in the liquid state and it offers no shearing resistance.

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. The conventional plastic limit test is carried out as per the procedure of AASHTO T 90 or ASTM D 4318. The soil in the plastic state can be remolded into different shapes. When the water content is reduced the plasticity of the soil decreases changing into semisolid state and it cracks when remolded.

3.12.5 Test Procedures

Atterberg Limits were determined for air-dried samples. It was done based on the Standard Reference: ASTM D 4318-Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils. The air- dried samples were prepared by spreading the specimen in the air until it dried. About 250gm sample of soil passing sieve No 40(.425mm) is used to determine the Atterberg Limits.

The moisture content, in percent, required to close a distance of 12.7 mm (0.5 in.) along the bottom of the groove after 25 blows is defined as the liquid limit. It is difficult to adjust the moisture content in the soil to meet the required 12.7 mm (0.5 in.) closure of the groove in the soil pat at 25 blows. Hence, at least three tests for the same soil are conducted at varying moisture contents, with the number of blows, N, required to achieve closure varying between 15 and 35. About 15-20 gm of soil passing through sieve No. 40 (ASTM) mixed thoroughly with water. The soil is rolled on a glass plate with the hand, until it is about 3 mm in diameter. This procedure of mixing and rolling is repeated till the soil shows signs of crumbling when the diameter is 3 mm. The water content of the crumbled portion of the thread is determined. This is called as plastic limit.

3.12.6 Moisture - Density Relationship

Compaction of a soil improves the engineering properties, i.e. it increases the shear strength of the soil and hence, the bearing capacity. It increases the stiffness and thus, reduces future settlement, void ratio and permeability. At lower water content than the optimum the soil is rather stiff and has a lot of void spaces and hence, the dry density is low. On the other hand, at water content more than the optimum the additional water reduces the dry density as it occupies the space that might have been occupied by solid particles.

The laboratory standard proctor and modified proctor tests are performed as per (AASHTO T 99 or ASTM D 698) and (AASHTO T 180 or ASTM D 1557) respectively. This laboratory test is performed to determine the relationship between the moisture content and the dry density of a soil for a specified compactive effort. The compactive effort is the amount of mechanical energy that is applied to the soil mass. Several different methods are used to compact soil in the field, and some examples include tamping, kneading, vibration, and static load compaction. This laboratory will employ the tamping or impact compaction method using the type of equipment and methodology developed by R. R. Proctor in 1933, therefore, the test is also known as the Proctor test.

Two types of compaction tests are routinely performed: (1) The Standard Proctor Test, and (2) The Modified Proctor Test. In the Standard Proctor Test, the soil is compacted by a 5.5 lb/2.5kg hammer falling a distance of one foot/300mm into a soil filled mold. The mold is filled with three equal layers of soil, and each layer is subjected to 25 drops of the hammer. Soil compaction consists of closing, packing the soil particles together the soil particles, so that increases the dry unit weight. Soil compaction only reduces the air void in the soil. The Modified Proctor Test is identical to the Standard Proctor Test except it employs, a 10 lb/5kg hammer falling a distance of 18 inches/450mm, and uses five equal layers of soil instead of three. There are two types of compaction molds used for testing. The smaller type is 4 inches in diameter and has a volume of about 1/30 ft3 (944 cm3), and the larger type is 6 inches in diameter and has a volume of about 1/13.333 ft3 (2123 cm3). If the larger mold is used each soil layer must receive 56 blows instead of 25. Mechanical compaction is one of the most common and cost effective means of stabilizing soils. An extremely important task of geotechnical engineers is the performance and analysis of field control tests to assure that compacted fills are meeting the prescribed design specifications. Design specifications usually state the required density (as a percentage of the "maximum" density measured in a standard laboratory test), and the water content. In general, most engineering properties, such as the strength, stiffness, resistance to shrinkage, and imperviousness of the soil, will improve by increasing the soil density. The optimum water content is the water content that results in the greatest density for a specified compactive effort. Compacting at water contents higher than (wet of) the optimum water content results in a relatively dispersed soil structure (parallel particle orientations) that is weaker, more ductile, less pervious, softer, more susceptible to shrinking, and less susceptible to swelling than soil compacted dry of optimum to the same density. The soil compacted lower than (dry of) the optimum water content typically results in a flocculated soil structure (random particle orientations) that has the opposite characteristics of the soil compacted wet of the optimum water content to the same density. The overall objective of this test is to obtain the moisture content –dry density relationship for a different soils type and hence to determine the optimum moisture content and maximum dry density. **Moisture Content (OMC Optimum):** moisture content of a soil at which a specified amount of compaction will produce the maximum dry density under specified test conditions. Dry density is determined based on the moisture content and the unit weight of compacted soil. The corresponding water content at which the maximum dry density occurs is termed as the optimum moisture content. Grading and Atterberg limits alone are not sufficient to qualify the performance of construction materials since variation of moisture content and density play a considerable role.

3.12.7 California Bearing Ratio (CBR)

California Bearing Ratio is a measure of shearing resistance of the material under controlled density and moisture conditions/ it is the **ratio of force per unit area** required to penetrate a soil mass with standard circular piston. The California Bearing Ratio Test (CBR Test) is a penetration test developed by ***California State Highway Department (U.S.A.)*** for evaluating the bearing capacity of subgrade soil for design of flexible pavement. 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 as per AASHTO T-193. Tests are carried out on natural soils in water soaked conditions and the results so obtained are compared with the curves of standard test to have an idea of the soil strength of the subgrade soil by adding lime and cement with soils having different grain sizes. **One point CBR Test** was made for all tests. California bearing ratio test results (CBR test) for four days soaked samples at their maximum dry density and OMC were compared with the standard specifications. The standard load values were obtained from the average of a large number of tests on different crushed stones. The greatest value calculated for penetrations at 2.54mm and 5.08mm was have been recorded as the CBR "for the 56 blows". However, if the greater recorded value was available first for penetration at 5.08mm the laboratory test, was repeated again and result were taken as it is for the next penetration result. The equation to be computing the CBR value is as follows.

CBR (%) = 100 *(x/y),………………………………………………………………Eq.3.1

Where, ‘X’ = material resistance or the unit load on the piston (pressure) for 2.54mm or 5.08 mm of penetration, y = standard unit load (pressure) for well graded crushed stone. For 2.54mm Penetration = 6.9mpa and for 5.08mm penetration = 10.3mpa. The **CBR test** is one of the most commonly used methods to evaluate the strength of a sub grade soil, sub base, and base course material for design of thickness for highways and airfield pavement. The California bearing ratio test is penetration test meant for the evaluation of subgrade strength of roads and pavements. The results obtained by these tests are used with the empirical curves to determine the thickness of pavement and its component layers. This is the most widely used method for the design of flexible pavement. This instruction sheet covers the laboratory method for the determination of **CBR** of undisturbed and remolded /compacted soil specimens, both in soaked as well as unsoaked state. The summary of the test result is tabulated below and the laboratory test analysis and plots are given in Appendix D.

CHAPTER FOUR

RESULTS AND DISCUSSIONS

4.1 Field Test Results

4.1.1 Pavement Condition Survey results

The pavement condition surveys investigation along the study area showed that the road segment has highly exposed to many types of deterioration like raveling, put hole, grade depression, corrugation, block cracking and alligator crack as shown in figure 4.1 and Appendix E. Those deteriorations happened on the study road segment repeatedly. Thus the sampling areas identified depending on the deterioration sites along the segment.

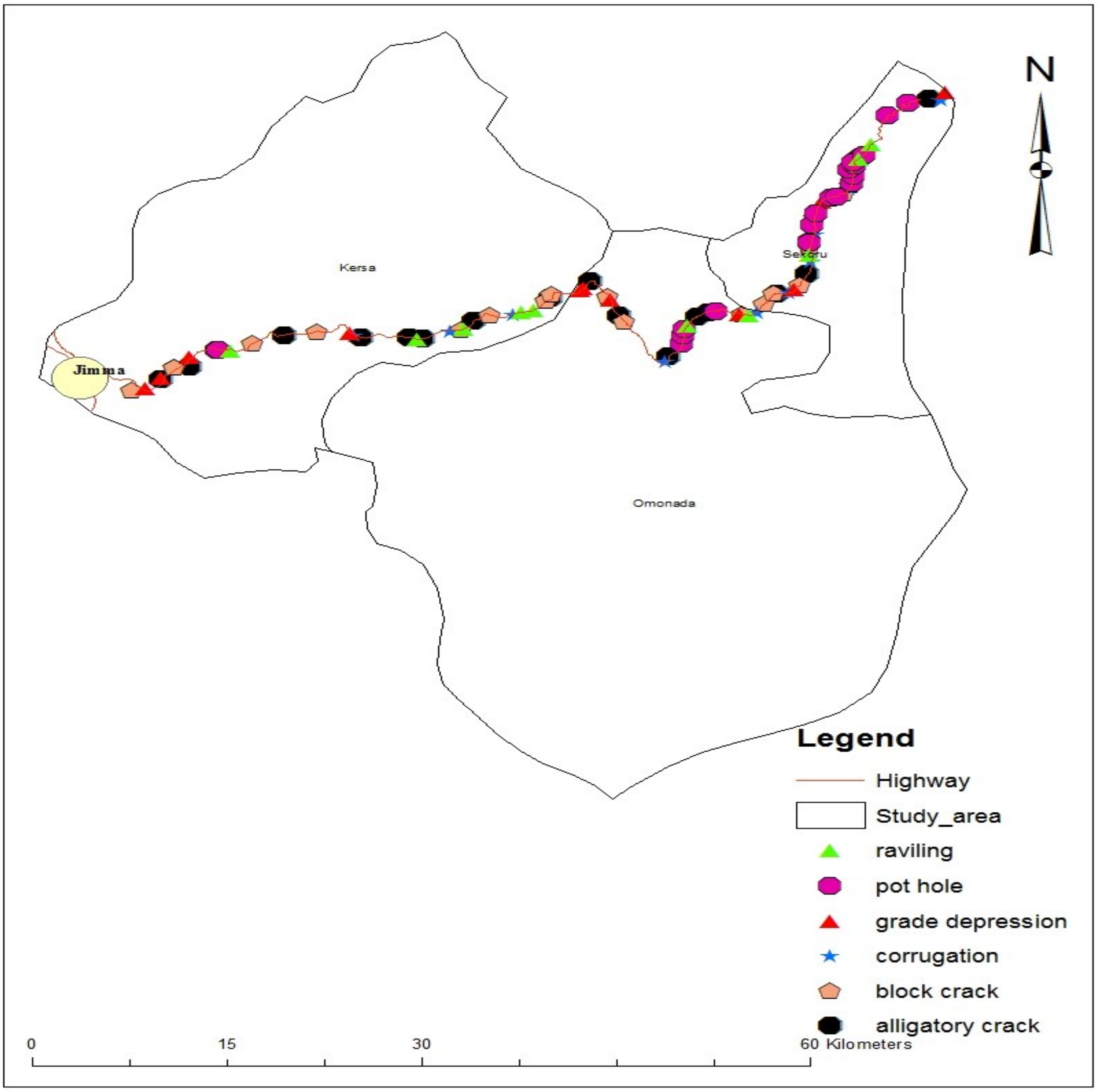


Figure 4. 1 Pavement condition along the study segment

The subgrade soil types were observed as normal granular soils, black cotton soils and white clay soils at those sites. Hence, the three soil samples were taken for further laboratory work for checking the quality of the subgrade strength and identifying the soil classes by AASHTO soil classification system. Atterberg limits, moisture content, dry density and California bearing ratio test were employed to check the quality of the subgrade layer of the study segment.

The subbase course materials pits were also taken at the distress sites for further investigation of the layer quality. Eighty nine (89) representative samples took at the distress sites for both subgrade and subbase layer each as presented in Appendix E. The subgrade layer soils were, generally, grouped to three soil types as stated above. The subbase materials had possessed the aggregate and soil properties as checked by laboratory tests. Thus, the study took the same representative for as shown below.

4.2 Laboratory Test Results and Discussions

4.2.1 Grain Size Analysis of Subgrade Soil Samples

The mechanical analysis consists of the determination of the amount and proportion of coarse material by the use of sieves analysis. The grain size analysis results are plotted below and the data is given in appendix A. The normal granular soil was designated as the normal soils which had almost similar laboratory results relatively. The white clay and the black cotton soils were also possessed a relatively similar mechanical properties at all test pits thus they were grouped in two types of soil as presented in the following sections.

Table 4. 1 Sieve Analysis test results of subgrade soil samples

Sieve Size (mm)	% Passing		
	Normal Granular Soil	White Clay Soil	Black Cotton Soil
2.36	64.885	62.29	62.845
1.18	51.1	47.14	48.445
0.6	41.035	35.335	35.845
0.3	19.985	18.965	20.22
0.15	7.395	7.035	10.805
0.075	2.98	2.405	3.17

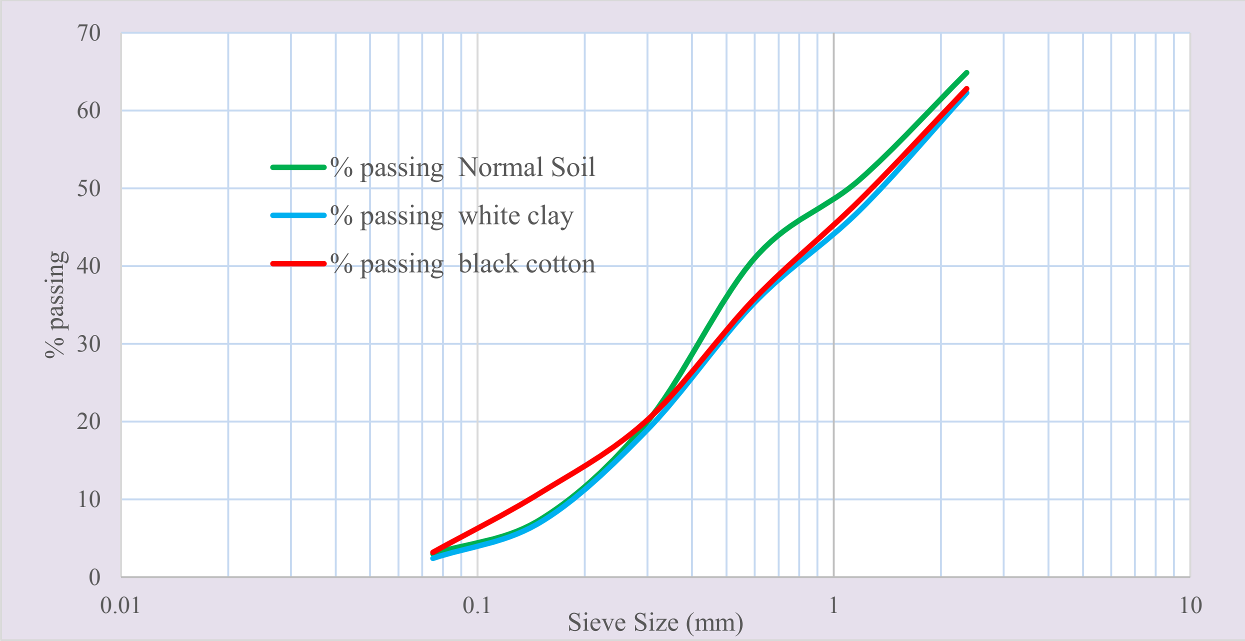


Figure 4. 2 Grain-size distribution curve of the three subgrade soil samples

i. For normal subgrade soil

- D10=0.19, D30=0.41, D60=1.9, Where,
  - ✓ D10=particle size such that 10% of the soil is finer than this size,
  - ✓ D30= particle size such that 30% of the soil is finer than this size and
  - ✓ D60= particle size such that 60% of the soil is finer than this size.
- Determine the Uniformity coefficient CU and coefficient of curvature (Coefficient of gradation) CC.
  - ✓ Cu=D60/D10=1.9/0.19=10>6 &
  - ✓ $Cc=[(D30)^2/D60]*D10=[(0.41)^2/1.9]*0.19=0.02<1$

The larger the numerical value of Cu was the more the range of the particle. Fine soils with a value of Cu of 6 or more are well-graded.

ii. For white clay soil

- Determine the Uniformity coefficient CU and coefficient of curvature (Coefficient of gradation) CC.
- D10=0.18, D30=0.49, D60=2.1,
  - ✓ Cu=D60/D10=2.1/0.18=11.67>6 &
  - ✓ $Cc=[(D30)^2/D60]*D10=[(0.49)^2/2.1]*0.18=0.02<1$

The larger the numerical value of Cu was the more the range of the particle. Sands with a value of Cu of 6 or more are well-graded.

iii. For black cotton soil

- Determine the Uniformity coefficient CU and coefficient of curvature (Coefficient of gradation) CC.
- D10=0.15, D30=0.47, D60=2.1,
    - ✓ Cu=D60/D10=2.1/0.15=14>6 &
    - ✓ $Cc=[(D30)^2/D60]*D10=[(0.47)^2/2.1]*0.15=0.016<1$

The larger the numerical value of Cu was the more the range of the particle. Sands with a value of Cu of 6 or more are well-graded.

The general shape of the particle size distribution curve is described by another coefficient known as the coefficient of curvature (Cc).it may be noted that the gap grading of the soil cannot be detected by Cu only. The value of Cc is also required to detect it. For well graded soil, the value of the coefficient of curvature (Cc) lies between 1 and 3, therefore the calculated value doesn't fulfill these criteria in all the three subgrade soil samples and generally the given subgrade soils are poorly-graded soil. The sieve analysis results and the gran size distribution curves for the three soils are attached at Appendix A.

4.2.2 Grain Size Analysis of Sub-base Course Sample

The sub-base layer is an important load spreading layer in the competed pavement structures. It also acts as a separation layer between the subgrade layer and the base course layer. The Grain-size distribution curve of the materials taken from the sub-base course of Jimma to Sekoru road is as follows.

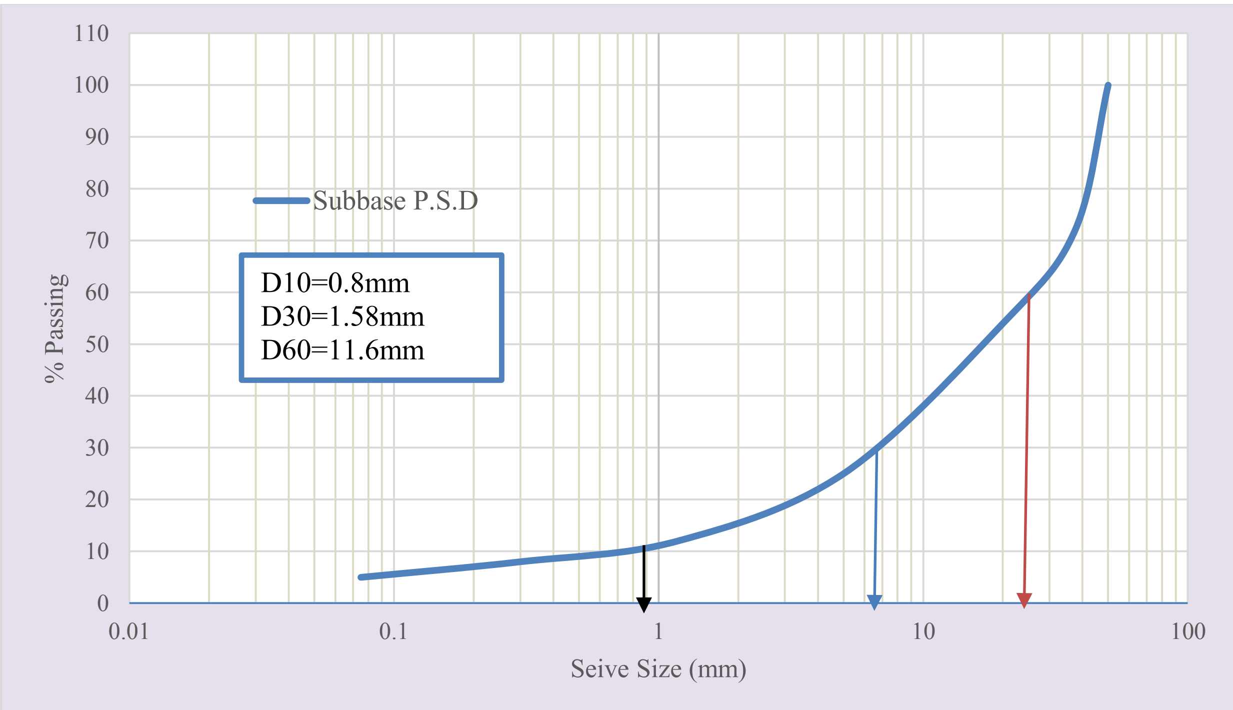


Figure 4. 3 Grain-size distribution curve of the sub-base course sample

- Determine the Uniformity coefficient CU and coefficient of curvature (Coefficient of gradation) CC.
- D10=0.8, D30=1.58, D60=11.6,
  - ✓ Cu=D60/D10=11.6/0.8=14.5>6 &
  - ✓ $Cc=[(D30)^2/D60]*D10=[(1.58)^2/11.6]*0.8=0.172<1$

The larger the numerical value of Cu was the more the range of the particle. Sands with a value of Cu of 6 or more are well-graded. The calculated value of coefficient of curvature doesn't fulfill the criteria in all sub-base course samples and generally the given sub-base material is poorly-graded soil.

The materials of the sub-base course have to fulfill the requirements of ERA specification. The grain size found from the Jimma –Sekoru road segment is out of the requirement of ERA pavement design standard specification as we can from table 4.2 and figure 4.3. Thus, aggregate size of the sub-base course layer materials are one of the effect for the deterioration of the pavement layer.

The gradation curve of the sub-base material is even below the lower limit of the ERA specification. The graph have shown that there is a lack of course aggregate in the subgrade materials. Coarse aggregates are the key for the load distribution of the pavement layer from the top surface. This means the sub-base course can't transfer load but it is deformed due the load coming from the base course. This in turn results the failure of the surface course deterioration, crack, and deformation.

Table 4. 2 Sieve analysis test results and ERA specification limit for subbase course materials

Sieve Size (mm)	Sub-base P.S.D	ERA Specification	
		Lower Limit	**Upper Limit**
50	100	100	100
37.5	72	80	100
20	54	60	100
5	25	30	100
1.18	12	17	75
0.3	8	9	50
0.075	5	5	25

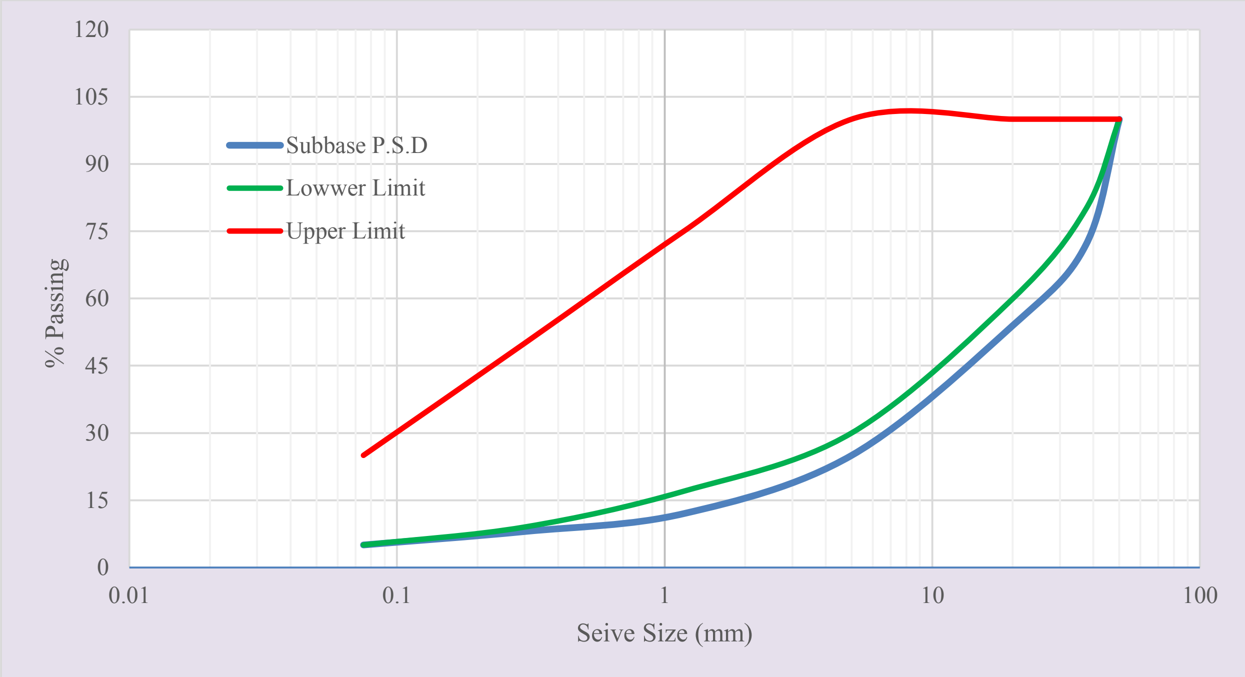


Figure 4. 4 Grain-size distribution curve of the sub-base course sample with ERA upper and Lower limit specification

4.2 Atterberg Limits

The test procedure adapted for the determination of Liquid limit, Plastic Limit and plasticity index are in accordance with AASHTO T89 or ASTM D 4318. The liquid limit of subgrade and sub base soil is the boundary between the water content at which soil behaves practically like a liquid, but has small shear strength. Its flow closes the groove in just 25 blows in Casagrande's liquid limit device. Hand mixing in a porcelain pan is the method of mixing. A sample of air dried soil passing # 40/0.425m and weighting about 250gm was taken from the mixture prepared for liquid limit test. A minimum of three trial tests were conducted for each samples. Additional soil wasn't needed to make the soil dry, exposed the mix to a fan or dried it by continuously mixing it with the spatula and obtained a minimum of three trials with values of N ~ 15 to 40 was enough to determine the LL of the soil. The plasticity of the soil was measured by the "plasticity index". The plasticity index is defined as the numerical difference between its Liquid limit and Plastic limit.

4.2.1 Atterberg Limits for subgrade soils

Based on the procedures of AASTO T89 or ASTM D4318, the laboratory test were delivered to determine **Atterberg Limits** of subgrade soils types which are normal soil, black cotton soils and white clay soils. Figure 4.4 show relationship between number of blow and water content for determinations of liquid limit of subgrade soils and Summaries of the test results are tabulated in table 4.3 below and the detail laboratory data's are attached as an Appendix-B.

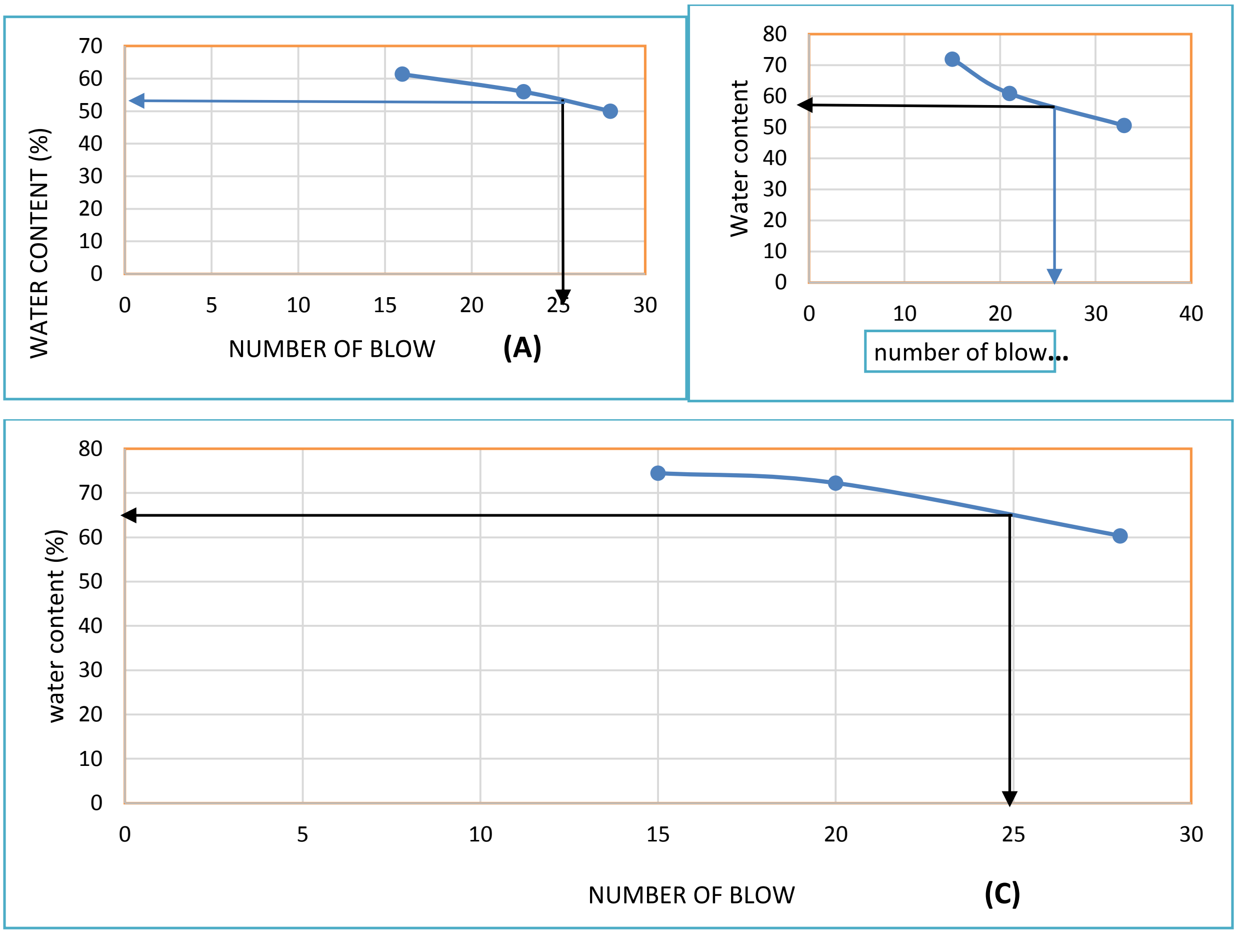


Figure 4. 5 liquid limit determinations for sub grade soil of: (A) normal soil, (B) black cotton soil and (C) white clay soil.

Table 4. 3 Summary of laboratory results of subgrade soils for Atterberg limits.

Sn	Subgrade soil types	Liquid limits (%)	Plastic limits (%)	Plastic index (%)	AASHTO classification (M145-91)	General rating as sub-grade
1	Normal soils	54.42	41.48	12.95	A-7-6	Fair to poor
2	Black cotton soils	57.47	43.63	13.83	A-7-6	Fair to poor
3	White clay soils	64.82	31.72	33.10	A-7-6	Fair to poor

Atterberg limits analysis:- Depending on the calculated values of **LL**, **PL**, and **PI** and its sieve analysis It was possible to identify the soil types according to AASHTO soil classification system based on the **Criteria's** of Passing No.0.075mm, liquid limit and plasticity index value, the soil Group was became A-7-6 which Consistent of silty-clay soil material and general rating as subgrade material ranges fair to poor.

4.2.2 Atterberg Limits for sub base soil

Based on the procedures of AASTO T89 or ASTM D4318, the laboratory test were deliver to determine **Atterberg Limits** of sub-base soils. Figure 4.5 show relationship between number of blow and water content for determinations of liquid limit of sub-base soils and Summaries of the test results are tabulated in table 4.4 below and the detail laboratory data's are attached as an Appendix-B.

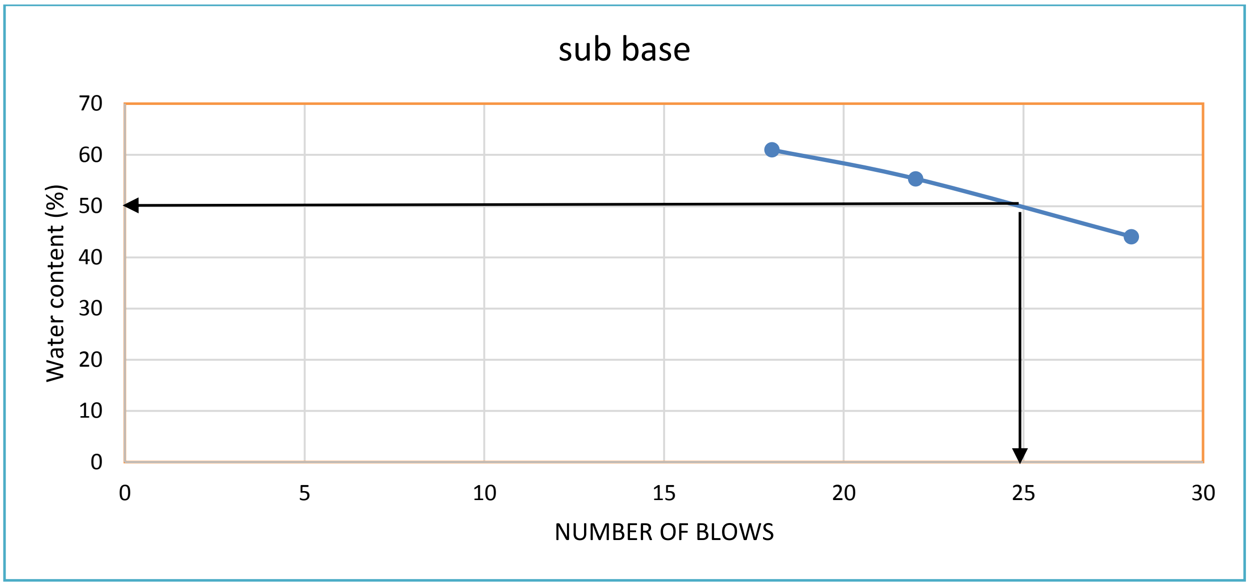


Figure 4. 6 liquid limit determinations of sub base soil material

Table 4. 4 Summary of Atterberg limits laboratory results of sub-base soils.

Sub base soil Atterberg properties	Values (%)	ERA recommended plasticity value for seasonal wet trop.	AASHTO classification (M145-91)	General rating as sub-base
Liquid limit	49.72	<45	A-7-6	Fair to poor
Plastic limit	37.28			
Plastic index	12.44	<12		

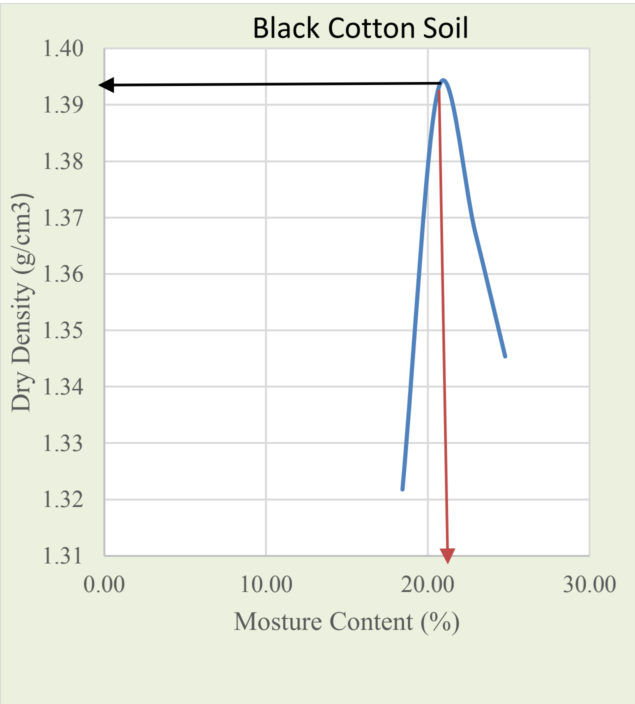
Atterberg limits analysis of sub base materials:- Depending on the calculated values of **LL**, **PL**, **PI** and its sieve analysis. It was possible to identify the soil types according to AASHTO M145-91 soil classification system based on the **Criteria's** of Passing No.0.075mm, liquid limit and plasticity index value, the soil Group was became A-7-6 which Consistent of silty-clay soil material and general rating as sub-base material ranges fair to poor. The study area of climatic conditions are seasonally wet tropical and the annual rainfall ranges greater than 500mm. According to ERA recommendation of plasticity value of sub base material for seasonally wet tropical area, the value of liquid limit and plasticity index less than 45% and 12% respectively. As shown in table 4.4 the value of sub base soil liquid limit and plasticity index values are greater than the value of ERA recommended.

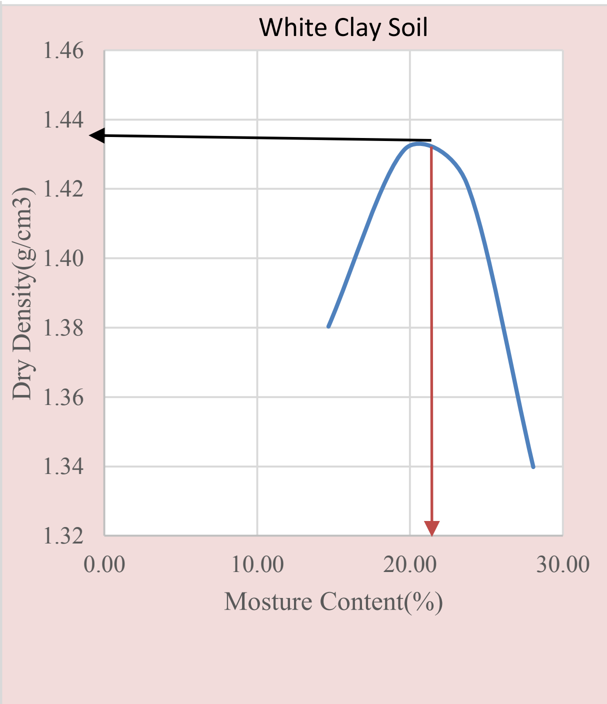
4.3 Moisture density relationships

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 representative sample of approximately 6kg passing 4.75 (No.4) and tests were conducted in accordance with AASHTO T99-97. The Standard Proctor compaction Test method was used or the soil was compacted by a 10lb/4.5kg hammer falling a distance of 18in/450mm into a soil filled mold. The mold was filled with three equal layers of soil, and each layer was subjected to 56 drops of the hammer. Soil compaction only reduces the air void in the soil. The overall objective of this test was to obtain the moisture content –dry density relationship for a different soils type and hence to determine the optimum moisture content and maximum dry density. Tests were conducted with the white clay, Black cotton soil, Normal soils all considered as subgrade soils and sub-base materials. Summarized results are tabulated in Table 4.5 below. The details of the test results are attached in Appendix A.

Table 4. 5 Summary of MDD and OMC laboratory results for Sub-base and Subgrade soils

Soil Types	Black cotton	Normal Soil	White Clay	Subbase soil
OMC (%)	20.67	19.66	19.70	21.90
MDD(g/cm^3)	1.39	1.46	1.43	1.94





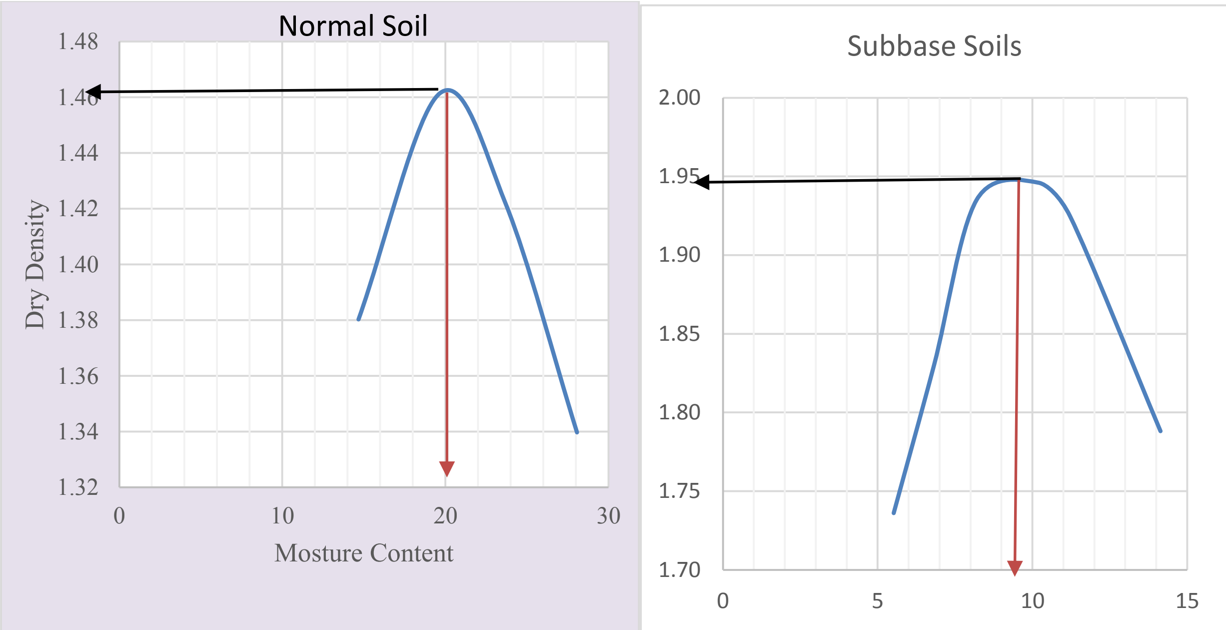


Figure 4. 7 Moisture-Density relations of Sub-base and Subgrade soil types

Compaction tests analysis:-

- ✓ The different subgrade soil samples and sub-base soil materials of Moisture-density relationship laboratory results were obtained.
- ✓ MDD increased of subgrade soils was increased from 1.39g/cm^3 (Black Cotton soil) to 1.46g/cm^3 (Normal Soil) whereas MDD value of the subbase course layer was became 1.95g/cm^3.
- ✓ This maximum value represents the most suitable subgrade and subbase soil material ready to filling up the voids spaces and densely packing the soil particles together.

4.4 California Bearing Ratio (CBR)

4.4.1 Subgrade Soils CBR

The CBR Values are determined based on AASHTO T-193. Tests were conducted with white clay, Black cotton and Normal subgrade soils as shown in table 4.6 and figure 4.7. Specimens are molded at respective optimum moisture content as determined in moisture density relationships. Moisture content and density before soaking were determined. Results are tabulated and illustrated in Appendix D. As we can from the table, the CBR values of normal subgrade soil, white clay soils and black cotton soils are 7.90mm, 8.81mm and 5.65 respectively. These values are grouped under poor, fair and very poor CBR classes according to ERA pavement design manual specifications. This means that the subgrade materials have mainly developed the deterioration effect of the surface course.

Table 4. 6 CBR results for Subgrade soil samples

SN	**Soil Type**	**Compacted Soil Type**	**CBR Value (%) 56 Blows**		**Remarks**	**ERA Criteria**
			2.54mm	**5.08mm**		
1	Subgrade Materials	Normal Soil	7.90	7.47		poor
		White Clay Soil	6.81	8.81	Repeated	fair
		Black Cotton Soil	2.82	3.65	Repeated	Very Poor

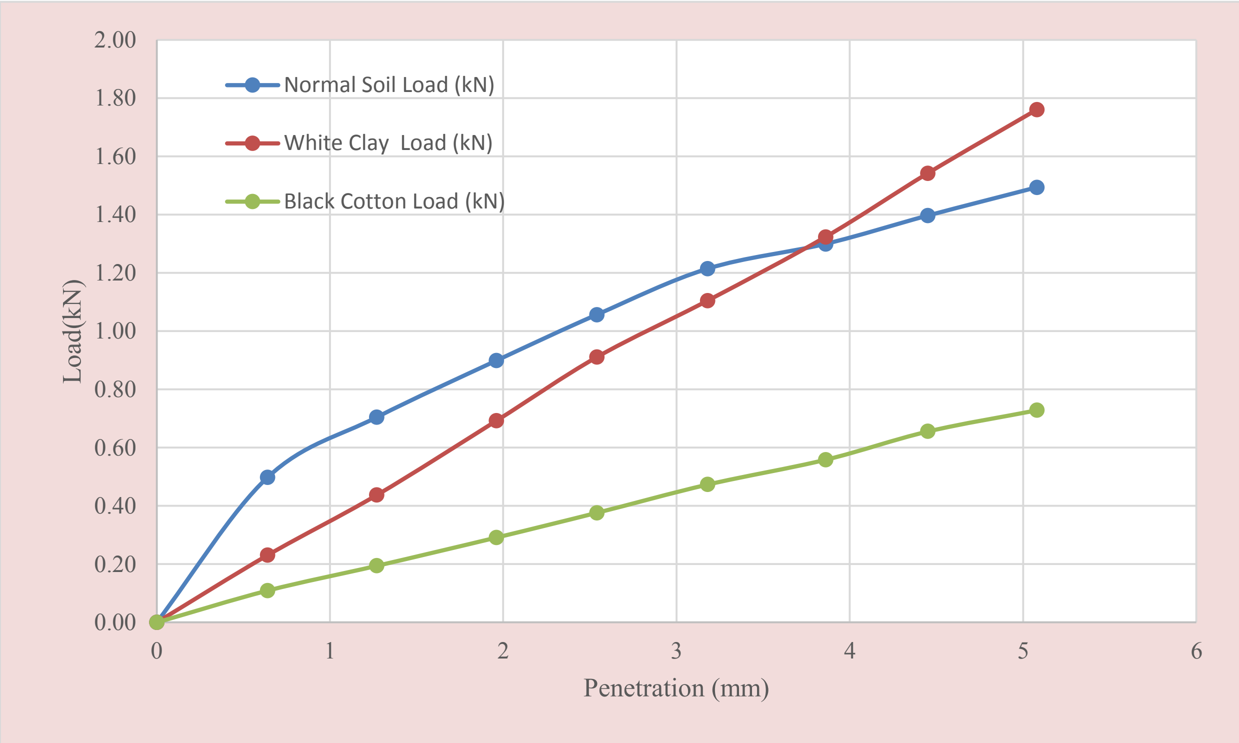


Figure 4. 8 Penetration vs load graph for CBR of subgrade samples

4.4.2 Sub-base Layer Materials CBR

The CBR Values are determined based on AASHTO T-193. CBR tests were employed to investigate the strength sub-base course materials. Specimens are molded at respective optimum moisture content as determined in moisture density relationships. Moisture content and density before soaking were determined. Results are tabulated and illustrated in Table 4.7 and figure 4.8 below. The details of the laboratory results are attached in Appendix D. As we can see from figure 4.2 the maximum CBR value is 15%. But a minimum CBR value for the highest anticipated moisture areas is 30% as ERA pavement design manual. Thus, as the CBR value showed the sub-base materials of Jimma-Sekoru road has failed. This means the deterioration of surface course is developed due to a low quality strength materials of sub-base course materials.

Table 4. 7 CBR results for Subgrade soil samples

No. Of Blows	10 blows	30 blows	65 blows
CBR for 2.54 penetration	4.36	12.44	15.26
CBR for 5.08 penetration	4.68	12.82	20.90
Max. CBR	4.68	12.82	20.90
Dry Density,g/cc	1.50	2.11	1.93

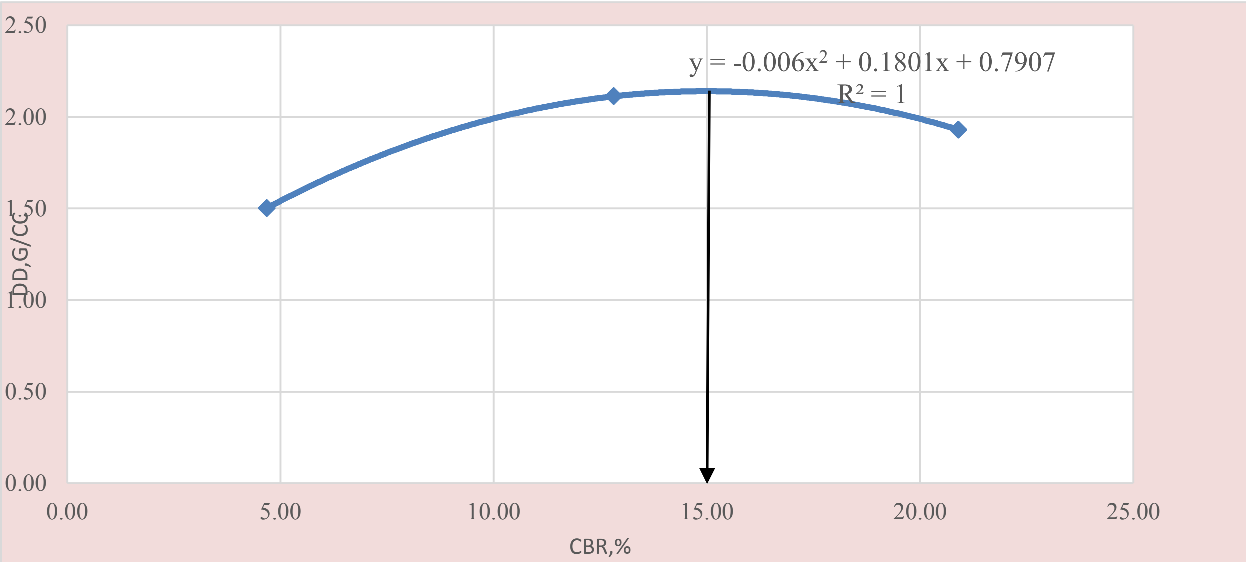


Figure 4. 9 penetration vs load graph for CBR of subgrade samples

Generally as we have observed from the gradation analysis, Atterberg limits, moisture content, and CBR results of both subgrade and sub-base materials, their result is below the quality and the specification of ERA pavement design specification. The low quality subgrade soils and sub-base materials also results the deterioration of pavement top layer. The defects, fatigue cracks, deformations, potholes, alligator cracks and other road defects developed because of the low quality materials used in Jimma-Sekoru road segment beside other problems.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATIONS

5.1 CONCLUSION

Based on the pavement condition survey and laboratory tests result the following conclusions are drawn:

- The pavement condition survey along the study area affected by different failures types such as raveling, put hole, grade depression, corrugation, block cracking and alligator crack problems related to road failures was identified during field investigation; this indicates that lack of routine and periodic maintenance along on a road section.
- The results of the subgrade soils investigation along Jimma-sekoru road showed that the road pavement structures are underlined by A-7-5, and A-7-6 category of soils which showed that the soil is very poor subgrade materials according to AASHTO and USCS shows that the soil categorize in to white clay, black cotton, normal subgrade and subbase soils. The liquid limit varies from 49.72 - 64.82% and Plasticity index from 12.44 - 33.10%. The soaked CBR values of subgrade soil materials are between 3.65 - 15% compared with 15% minimum specified, therefore, the failures frequently observed on the road surface are significantly influenced by subgrade soil.
- From the CBR value of subgrade soil, the three representative soils have fair, poor and very poor for white clay, normal subgrade and black cotton soils as per ERA pavement design specification. The minimum CBR value for subbase layer for moisture susceptible areas is 30% as per ERA but Jimma –Sekoru road segment subbase materials CBR is 15%. Therefore the two layers are the major causes of road failures along study area.
- Neither the sub-base layers materials satisfies minimum standard requirements nor the subgrade soil materials along Jimma-Sekoru Road section did satisfied the standard requirements set by ERA; therefore the failures observed on road surface significantly affected due to subgrade soils and subbase materials.

5.2 RECOMMENDATION

- Scarification and reconstruction of the distress road section specially affected by raveling.The sections with various sizes of potholes should be patched with good quality asphalt and distress sections of pavement with poor material due to material quality problems shall be removed and replaced to required quality and strength. Seal coats shall be applied to prevent infiltration of water through cracked surfaces.
- The designer and contractor shall follow the minimum requirement set by ERA regarding the properties and structural thickness of embankment of a road, so as to prevent the pavement from defects.
- The designer and contractor shall allow improving or treating the available subgrade soil type using some stabilizing agents such as:-Lime, Cement, Fly ash, and Try to add an additional thickness for subbase course layers.
- Poor drainage courses, level of groundwater table, variation of geologic materials along the road route and poor construction materials shall be thoroughly addressed before beginning rehabilitation the road section in the future.
- Timely pavement maintenance practices should be employed to reduce pavement failure.
- Detail investigation shall be carried out on project areas; also the properties of material and method of construction should be according to the design specification of project in order to serve the design period of a project in order to avoid the failure.
- For future research, it is recommended that detailed in-depth investigation should be carried out on related project; compliance with quality of materials and construction methods in accordance with ERA Standard Specifications in order to avoid future failure.

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APPENDIX A

Sieve Analysis Test Results

1. Normal Subgrade soil Sieve analysis test results

Table A1 Normal Subgrade soil Sieve analysis test results

SIEVE SIZE	Wt. RETAINED	% retained	Commu. Percent Retained	% passing
2.36	702.3	35.115	35.115	64.885
1.18	275.7	13.785	48.9	51.1
0.6	201.3	10.065	58.965	41.035
0.3	421	21.05	80.015	19.985
0.15	251.8	12.59	92.605	7.395
0.075	88.3	4.415	97.02	2.98
Pan	59.6	2.98	100	0
Total	2000			

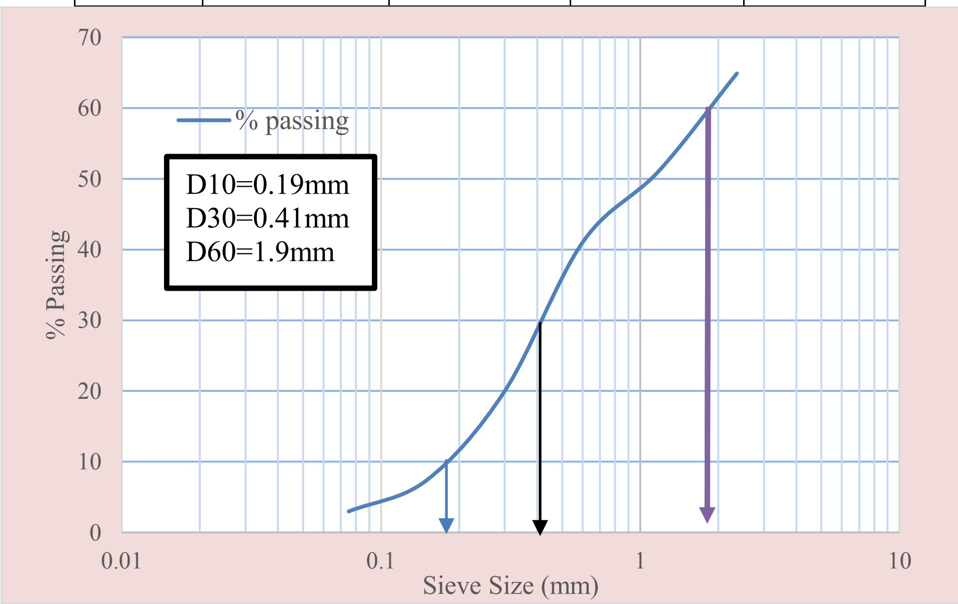

Figure 1A- Grain-size distribution curve of the normal subgrade soil sample

2. White clay subgrade soil Sieve analysis test results

Table A2 White clay soil sieve analysis test results

SIEVE SIZE	Wt. RETAINED	% retained	Commu. Percent Retained	% passing
2.36	754.2	37.71	37.71	62.29
1.18	303	15.15	52.86	47.14
0.6	236.1	11.805	64.665	35.335
0.3	327.4	16.37	81.035	18.965
0.15	238.6	11.93	92.965	7.035
0.075	92.6	4.63	97.595	2.405
Pan	48.1	2.405	100	0
Total	2000			

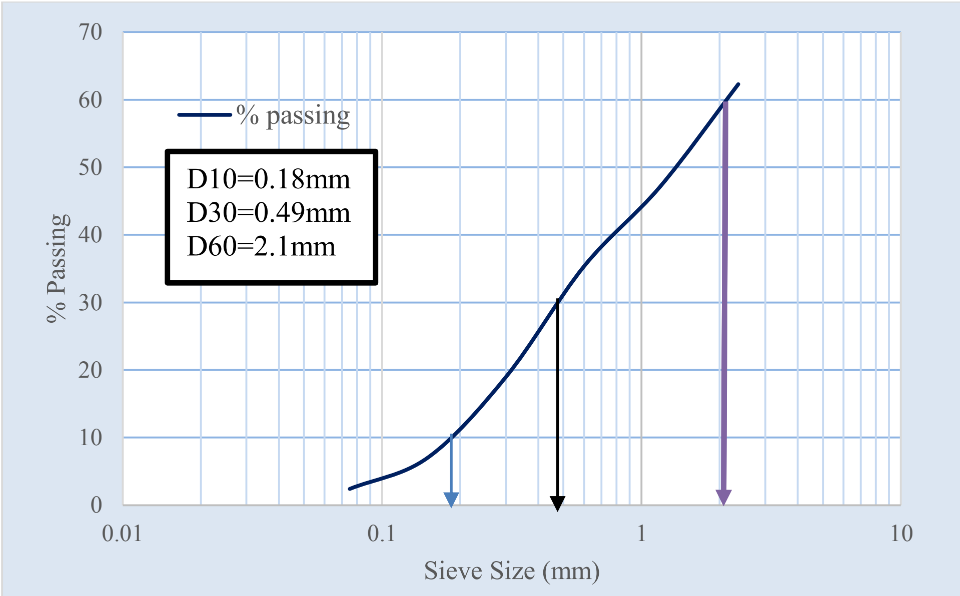

Figure 2A- Grain-size distribution curve of the white clay subgrade soil sample

3. Black cotton subgrade soil Sieve analysis test results

Table A3 Black cotton soil Sieve analysis test results

SIEVE SIZE	Wt. RETAINED	% retained	Commu. Percent Retained	% passing
2.36	743.1	37.155	37.155	62.845
1.18	288	14.4	51.555	48.445
0.6	252	12.6	64.155	35.845
0.3	312.5	15.625	79.78	20.22
0.15	188.3	9.415	89.195	10.805
0.075	152.7	7.635	96.83	3.17
Pan	63.4	3.17	100	0
Total	2000			

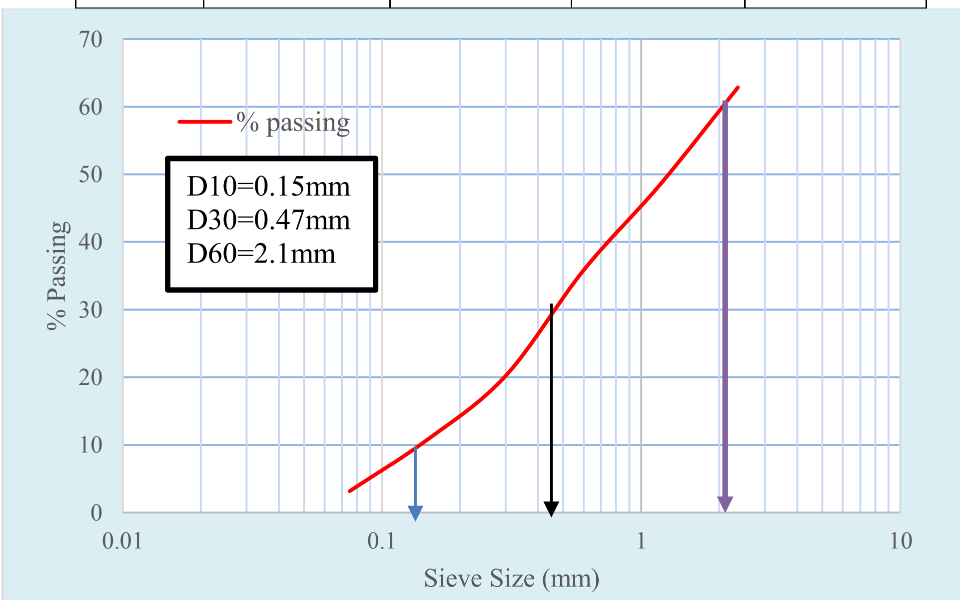

Figure 3A- Grain-size distribution curve of the black cotton subgrade soil sample

4. Sub-base Course Sieve analysis test results

Table A4 Sub-base course materials sieve analysis test results

SIEVE SIZE	**Wt. RETAINED**	**% retained**	**Commu. Percent Retained**	**% passing**
50	0	0	0	100
37.5	561.4	28	28	72
20	360.9	18	46	54
5	581.45	29	75	25
1.18	260.65	13	88	12
0.3	80.2	4	92	8
0.075	60.15	3	95	5
Pan	100.25	5	100	0
Toatal	2005			

APPENDIX B

Atterberg Limits for subgrade soils

1. Normal subgrade soils

DETERMINING LIQUID LIMIT、PLASTIC LIMIT、PLASTIC INDEX AND CLASSIFICATION OF SOIL TEST METHOD：AASHTO T89, T90 AND M145

Liquid Limit WL			
Run Number	1	2	3
Tare Number	A	O	E
A. Weight of Wet Soil + Tare	51.8	59.8	56.5
B. Weight of Dry Soil + Tare	41.1	46.2	42.2
C. Weight of Water (A – B)	10.7	13.6	14.3
D. Weight of Tare	19.7	21.9	18.9
E. Weight of Dry Soil (B -D)	21.4	24.3	23.3
Water Content % (C / E x 100)	50	55.96708	61.37339
Number of Blows	28	23	16
Liquid Limit %	**54.42241751**		

Plastic Limit WP			
Run Number	1	2	3
Tare Number	O7	O8	m1
F. Weight of Wet soil + Tare	17.6	15.6	16.6
G. Weight of Dry soil + Tare	15.8	14.1	15.2
H. Weight of water (F – G)	1.8	1.5	1.4
I. Weight of Tare	11	10.8	10.8
J. Weight of Dry soil (G – I)	4.8	3.3	4.4
Water Content % (H / J x 100)	37.5	45.45455	31.81818
Plastic Limit % (Average)	**41.47727273**		
Plasticity Index (PI = WL - WP) :	**12.94514479**		

Grading				AASHTO Classification (M145-91)
ASTM SIEVE No.	**10**	**40**	**200**	
Diameter f mm	**2.00**	**0.425**	**0.075**	***A-7-6***
% Passing	***99***	***94***	***74.2***	

2. Black cotton subgrade soils

DETERMINING LIQUID LIMIT、PLASTIC LIMIT、PLASTIC INDEX AND CLASSIFICATION OF SOIL TEST METHOD：AASHTO T89, T90 AND M145

Liquid Limit WL			
Run Number	1	2	3
Tare Number	A29	CD	BF
A. Weight of Wet Soil + Tare	53.4	63.2	57.2
B. Weight of Dry Soil + Tare	39	47	44.7
C. Weight of Water (A – B)	14.4	16.2	12.5
D. Weight of Tare	19	20.4	20
E. Weight of Dry Soil (B -D)	20	26.6	24.7
Water Content % (C / E x 100)	72	60.90226	50.60729
Number of Blows	15	21	33

Liquid Limit %	**57.47059958**			
Plastic Limit WP				
Run Number	1	2	3	
Tare Number	O2	O1	k1	
F. Weight of Wet soil + Tare	20.3	19.1	20.4	
G. Weight of Dry soil + Tare	16.9	16.5	17	
H. Weight of water (F – G)	3.4	2.6	3.4	
I. Weight of Tare	9.4	10.3	9.4	
J. Weight of Dry soil (G – I)	7.5	6.2	7.6	
Water Content % (H / J x 100)	45.33333	41.93548	44.73684	
Plastic Limit % (Average)	**43.6344086**			
Plasticity Index (PI = WL - WP) :	**13.83619097**			
Grading				AASHTO Classification (M145-91)
ASTM SIEVE No.	**10**	**40**	**200**	
Diameter f mm	**2.00**	**0.425**	**0.075**	***A-7-6***
% Passing	***99***	***95***	***73.5***	

3. White clay subgrade soils

Liquid Limit WL				
Run Number	1	2	3	
Tare Number	Z60	XY	O	
A. Weight of Wet Soil + Tare	40.4	37.8	39.4	
B. Weight of Dry Soil + Tare	32.5	30.5	32.4	
C. Weight of Water (A – B)	7.9	7.3	7	
D. Weight of Tare	21.9	20.4	20.8	
E. Weight of Dry Soil (B -D)	10.6	10.1	11.6	
Water Content % (C / E x 100)	74.5283	72.27723	60.34483	
Number of Blows	15	20	28	
Liquid Limit %	**64.81947764**			
Plastic Limit WP				
Run Number		1	2	3
Tare Number		ZM	NS	LO
F. Weight of Wet soil + Tare		17.9	17.7	17.4
G. Weight of Dry soil + Tare		16.2	15.8	15.7
H. Weight of water (F – G)		1.7	1.9	1.7
I. Weight of Tare		10.8	10.1	10.1
J. Weight of Dry soil (G – I)		5.4	5.7	5.6
Water Content % (H / J x 100)		31.48148	33.33333	30.35714
Plastic Limit % (Average)	**31.72398589**			
Plasticity Index (PI = WL - WP) :	**33.09549175**			
Grading				AASHTO Classification (M145-91)
ASTM SIEVE No.	**10**	**40**	**200**	
Diameter f mm	**2.00**	**0.425**	**0.075**	***A-7-6***
% Passing	***99***	***94***	***73.0***	

4. Atterberg limits for sub-base soil materials

Liquid Limit WL			
Run Number	1	2	3
Tare Number	A29	a4	BY1
A. Weight of Wet Soil + Tare	44.5	46.5	47.5
B. Weight of Dry Soil + Tare	36.7	36.7	36.7
C. Weight of Water (A – B)	7.8	9.8	10.8
D. Weight of Tare	19	19	19
E. Weight of Dry Soil (B -D)	17.7	17.7	17.7
Water Content % (C / E x 100)	44.0678	55.36723	61.01695
Number of Blows	28	22	18
Liquid Limit %	**49.71751412**		

Plastic Limit WP			
Run Number	1	2	3
Tare Number	O7	10	5P
F. Weight of Wet soil + Tare	16.5	17.1	18.1
G. Weight of Dry soil + Tare	14.5	15.4	15.9
H. Weight of water (F – G)	2	1.7	2.2
I. Weight of Tare	9.4	10.3	10.3
J. Weight of Dry soil (G – I)	5.1	5.1	5.6
Water Content % (H / J x 100)	39.21569	33.33333	39.28571
Plastic Limit % (Average)	**37.27824463**		
Plasticity Index (PI = WL - WP) :	**12.43926949**		

Grading				AASHTO Classification (M145-91)
ASTM SIEVE No.	**10**	**40**	**200**	
Diameter f mm	**2.00**	**0.425**	**0.075**	***A-7-6***
% Passing	***99***	***94***	***73.0***	

APPENDIX C

Moisture content and Dry density test Results

1. Normal subgrade soil Moisture-Density Relation test results

Trial No.		1	2	3	
Mold +Weight of wet soil(g)	W1	9434	9681	9628	
Weight of Mold(g)	W2	6033.4	6033.4	6033.4	
Weight of Wet Soil(g)	W3=W1-W2	3400.6	3647.6	3594.6	
Volume of Mold (cc)	V	2123	2123	2123	
Wet Density, (g/cm^3)	ϸ=W3/V	1.60178992	1.718134715	1.693170042	
Moisture Content Determination					**NMC**
Can-No.		15A	140	25	AE
Wight of Wet soil+can wt.(g)	a	163.1	160.4	168.7	487.6
Weight of Dry soil+can wt.(g)	b	143	135.8	137.6	449
Can wt.(g)	c	32.8	34	32.7	78
Weght of water (moisture)(g),	d=a-b	20.1	24.6	31.1	38.6
Weight of Dry soil(g)	e=b-c	110.2	101.8	104.9	371
Moisture content (%)	w=d/ex100	14.67	19.66	23.7	28.08
Dry Density(g/cm^3)		1.38	1.46	1.42	1.34

2. White clay Subgrade soil Moisture-Density Relation test results

Trial No.		1	2	3	4	
Mold +Weight of wet soil(g)	W1	9393.5	9669.7	9767.1	9676.3	
Weight of Mold(g)	W2	6033.3	6033.3	6033.3	6033.3	
Weight of Wet Soil(g)	W3=W1-W2	3360.2	3636.4	3733.8	3643	
Volume of Mold (cc)	V	2123	2123	2123	2123	
Wet Density, (g/cm^3)	ϸ=W3/V	1.582760245	1.712859162	1.758737635	1.715967	
Moisture Content Determination						**NMC**
Can-No.		50	11	8	17	CK
Wight of Wet soil+can wt.(g)	a	160	167.5	168.4	162.4	445
Weight of Dry soil+can wt.(g)	b	144	145.6	142.8	134.1	425.3
Can wt.(g)	c	34.9	34.2	34.8	33.3	68.8
Weght of water (moisture)(g),	d=a-b	16	21.9	25.6	28.3	19.7
Weight of Dry soil(g)	e=b-c	109.1	111.4	108	100.8	356.5
Moisture content (%)	w=d/ex100	14.66544455	19.65888689	23.7037037	28.0753	5.525946
Dry Density(g/cm^3)		1.380328879	1.431451692	1.421734017	1.33981	0

3. Black cotton Subgrade soil Moisture-Density Relation test results

Trial No.		1	2	3	4
Mold +Weight of wet soil(g)	W1	9356	9693.6	9779.6	9596.8
Weight of Mold(g)	W2	6032.5	6032.5	6032.5	6032.5
Weight of Wet Soil(g)	W3=W1-W2	3323.5	3661.1	3747.1	3564.3
Volume of Mold (cc)	V	2123	2123	2123	2123

Wet Density, (g/cm^3)	p=W3/V	1.565473387	1.724493641	1.765002355	1.67889 7786	
Moisture Content Determination						**NMC**
Can-No.		127	17	4	121	AL
Wight of Wet soil+can wt.(g)	a	150.4	156	157.7	155.9	424.3
Weight of Dry soil+can wt.(g)	b	132	135	134.4	132.2	393.6
Can wt.(g)	c	32.2	33.4	32.8	36.6	67
Weght of water (moisture)(g),	d=a-b	18.4	21	23.3	23.7	30.7
Weight of Dry soil(g)	e=b-c	99.8	101.6	101.6	95.6	326.6
Moisture content (%)	w=d/ex100	18.43687375	20.66929134	22.93307087	24.7907	9.39987753
Dry Density(g/cm^3)		1.321778714	1.39	1.37	1.35	0

4. Sub-base course materials Moisture-Density Relation test results

Trial No.		1	2	3	4	
Mold +Weight of wet soil(g)	W1	10477.1	10585.2	10576.6	10441.2	
Weight of Mold(g)	W2	6032.6	6032.6	6032.6	6032.6	
Weight of Wet Soil(g)	W3=W1-W2	4444.5	4552.6	4544	4408.6	
Volume of Mold (cc)	V	2123	2123	2123	2123	
Wet Density, (g/cm^3)	p=W3/V	2.093499764	2.144418276	2.140367405	2.076589732	
Moisture Content Determination						**NMC**
Can-No.		25	14	21	50	B.G
Wight of Wet soil+can wt.(g)	a	208.4	186.9	160.3	140.7	450
Weight of Dry soil+can wt.(g)	b	195.1	172.7	147.6	126	428.2
Can wt.(g)	c	32.8	33.5	34.4	34.9	76.7
Weght of water (moisture)(g),	d=a-b	13.3	14.2	12.7	14.7	21.8
Weight of Dry soil(g)	e=b-c	162.3	139.2	113.2	91.1	351.5
Moisture content (%)	w=d/ex100	8.194701171	10.20114943	11.21908127	16.13611416	6.20199
Dry Density(g/cm^3)		1.934937425	1.945912803	1.924460605	1.788065449	0

APPENDIX D

CBR test results

1. Normal subgrade soil

Table D1 CBR test data

PENETRATION TEST DATA				
	56 Blows			
Penetration (mm)	**DIAL RDG**	**LOAD (kN)**	**COR. LOAD (kN)**	**CBR (%)**
0	0	0	0	
0.64	41	0.49774		
1.27	58	0.70412		
1.96	74	0.89836		
2.54	87	1.05618	0.544	7.90163542
3.18	100	1.214		
3.86	107	1.29898		
4.45	115	1.3961		
5.08	123	1.49322	0.77	7.472825543

SWELL	
No. OF BLOWS	10
RDG (BEFORE SOAKING)	0.2
RDG (AFTER SOAKING)	0.31
PERCENT SWELL	0.006

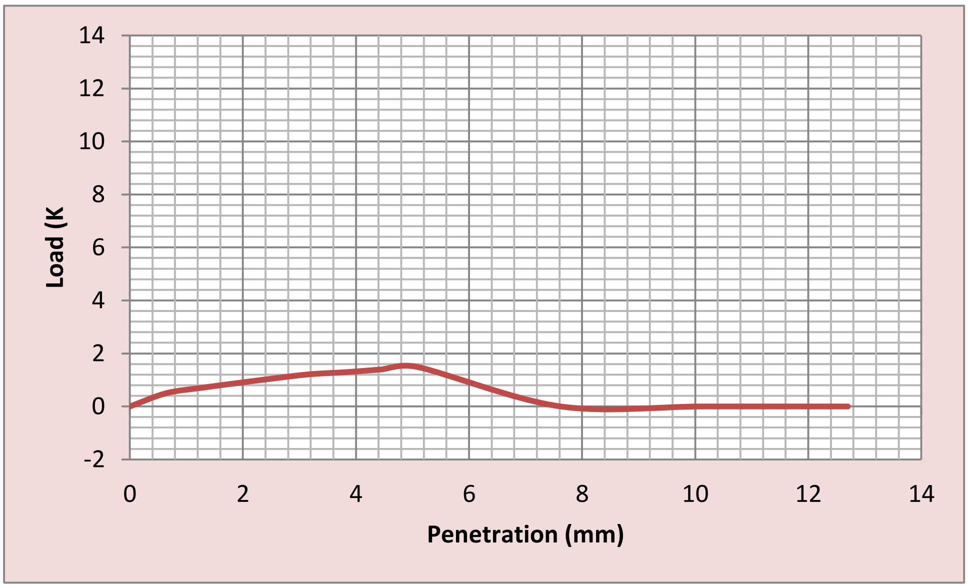

Figure 1D CBR graph for normal subgrade soil

2. White clay soil

Table D2 CBR test data

PENETRATION TEST DATA				
	56 Blows			
Penetration (mm)	**DIAL RDG**	**LOAD (kN)**	**COR. LOAD (kN)**	**CBR (%)**
0	0	0		

0.64	19	0.23066		
1.27	36	0.43704		
1.96	57	0.69198		
2.54	75	0.9105	0.469329897	6.811754672
3.18	91	1.10474		
3.86	109	1.32326		
4.45	127	1.54178		
5.08	145	1.7603	0.907371134	8.809428486

SWELL	
No. OF BLOWS	10
RDG (BEFORE SOAKING)	0.3
RDG (AFTER SOAKING)	0.4
PERCENT SWELL	0.005166624
AVERAGE PENETRATION SWELL	0.005166624

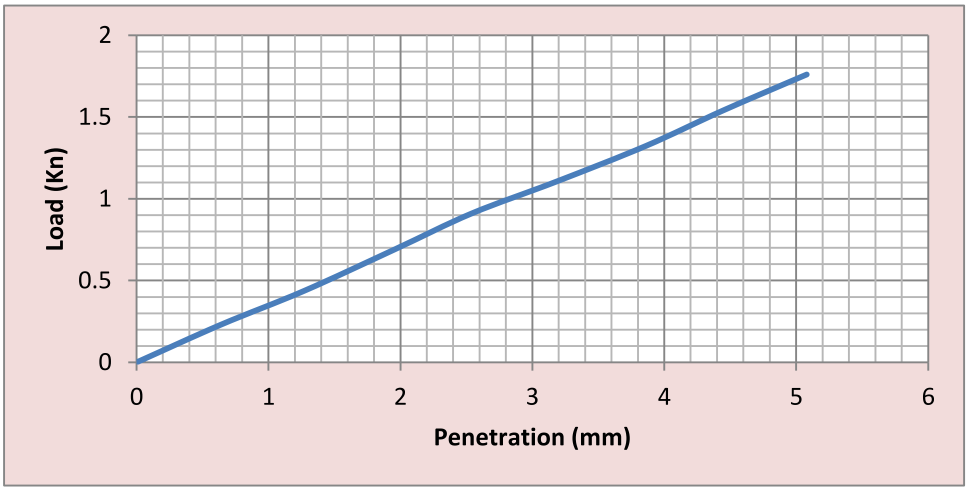

Figure 2D CBR graph for white clay subgrade soil

3. Black cotton soil

Table D3 CBR test data

PENETRATION TEST DATA				
	56 Blows			
Penetration (mm)	**DIAL RDG**	**LOAD (kN)**	**COR. LOAD (kN)**	**CBR (%)**
0	0	0		
0.64	9	0.10926		
1.27	16	0.19424		
1.96	24	0.29136		
2.54	31	0.37634	0.193989691	2.815525264
3.18	39	0.47346		
3.86	46	0.55844		
4.45	54	0.65556		
5.08	60	0.7284	0.375463918	3.645280753

No. OF BLOWS	56
RDG (BEFORE SOAKING)	0.25
RDG (AFTER SOAKING)	0.3
PERCENT SWELL	0.002583312

AVERAGE PENETRATION SWELL	0.002583312

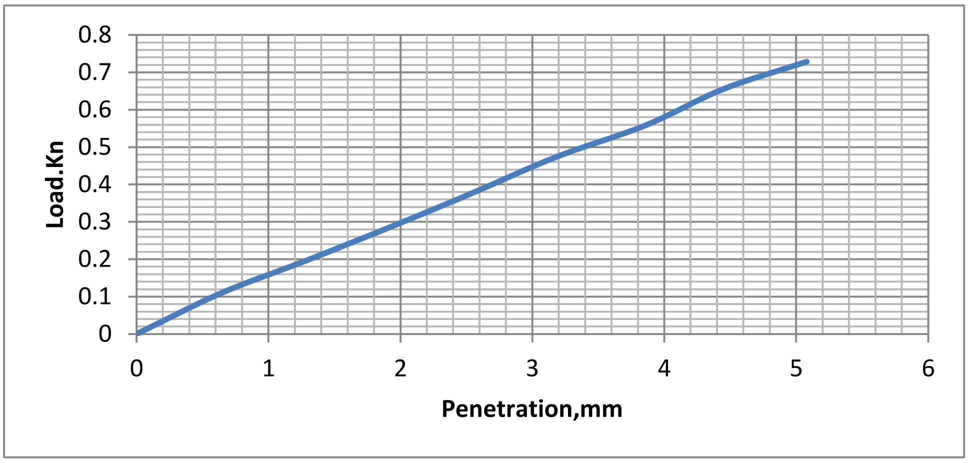

Figure 3D CBR graph for white clay subgrade soil

4. Sub-base course materials

Table D3 CBR test data

PENETRATION TEST DATA												
	10 Blows				30 Blows				65 Blows			
Penetration (mm)	DIAL RDG	LOAD (kN)	COR. LOAD (kN)	CBR (%)	DIAL RDG	LOAD (kN)	COR. LOAD (kN)	CBR (%)	DIAL RDG	LOAD (kN)	COR. LOAD (kN)	CBR (%)
0	0	0.00			0	0			0	0		
0.64	21	0.25			35	0.42			40	0.49		
1.27	32	0.39			72	0.87			69	0.84		
1.96	41	0.50			110	1.34			124	1.51		
2.54	48	0.58	0.30	4.36	137	1.66	0.86	12.44	168	2.04	1.05	15.26
3.18	57	0.69			161	1.95			213	2.59		
3.86	63	0.76			184	2.23			257	3.12		
4.45	70	0.85			198	2.40			298	3.62		
5.08	77	0.93	0.48	4.68	211	2.56	1.32	12.82	344	4.18	2.15	20.90

SWELL			
No. OF BLOWS	10	30	65
RDG (BEFORE SOAKING)	0.15	0.2	0.2
RDG (AFTER SOAKING)	0.21	0.29	0.34
PERCENT SWELL	0.0031	0.0046	0.0072
AVERAGE PENETRATION SWELL	0.0050		

APPENDIX E
Field Investigation Data's

Number	Easting (°)	Northing (°)	Distance (km)	Description
1	36.87	7.66	3.1	Block Crack
2	36.88	7.661	4.1	Grade Depression
3	36.89	7.67	6	Alligator Crack
4	36.89	7.671	6.5	Grade Depression
5	36.9	7.68	6.7	Block Crack
6	36.9	7.68	6.9	Block Crack
7	36.91	7.681	7.2	Alligator Crack
8	36.91	7.69	7.5	Grade Depression
9	36.929	7.697	10.4	Alligator Crack
10	36.93	7.697	10.1	Pot Hole
11	36.932	7.696	10.6	Corrugation
12	36.939	7.696	11.5	Raveling
13	36.954	7.702	13.6	Block Crack
14	36.976	7.71	16.6	Alligator Crack
15	36.999	7.713	19.3	Block Crack
16	37.022	7.712	22.6	Grade Depression
17	37.025	7.71	23	Corrugation
18	37.029	7.708	23.5	Alligator Crack
19	37.063	7.708	27.4	Alligator Crack
20	37.068	7.706	27.9	Raveling
21	37.072	7.707	28.3	Alligator Crack
22	37.091	7.714	30.7	Corrugation
23	37.099	7.715	31.5	Block Crack
24	37.101	7.716	31.8	Raveling
25	37.107	7.723	32.8	Alligator Crack
26	37.119	7.728	34.4	Block Crack
27	37.135	7.729	36.4	Corrugation
28	37.141	7.731	37	Raveling
29	37.149	7.733	38	Raveling
30	37.158	7.741	39.4	Block Crack
31	37.161	7.744	39.9	Alligator Crack
32	37.162	7.747	40.2	Block Crack
33	37.182	7.751	42.6	Grade Depression
34	37.184	7.753	43	Grade Depression
35	37.188	7.759	43.8	Alligator Crack
36	37.201	7.745	46	Block Crack
37	37.203	7.742	46.6	Grade Depression
38	37.208	7.728	48.3	Alligator Crack

39	37.212	7.722	49.1	Block Crack
40	37.241	7.686	55	Corrugation
41	37.243	7.691	55.6	Alligator Crack
42	37.252	7.702	57.3	Pot Hole
43	37.253	7.71	58.2	Pot Hole
44	37.254	7.716	58.8	Pot Hole
45	37.256	7.719	59.2	Raveling
46	37.263	7.727	60.5	Alligator Crack
47	37.27	7.731	61.4	Alligator Crack
48	37.276	7.732	62.1	Pot Hole
49	37.292	7.729	64	Grade Depression
50	37.294	7.729	64.3	Block Crack
51	37.299	7.728	64.9	Raveling
52	37.304	7.731	65.5	Corrugation
53	37.309	7.739	66.6	Block Crack
54	37.316	7.748	67.8	Block Crack
55	37.318	7.748	68	Alligator Crack
56	37.326	7.749	68.9	Corrugation
57	37.33	7.752	69.5	Grade Depression
58	37.334	7.756	70.2	Block Crack
59	37.339	7.766	71.4	Alligator Crack
60	37.341	7.791	74.3	Block Crack
61	37.341	7.798	75	Block Crack
62	37.341	7.795	74.7	Pot Hole
63	37.341	7.783	73.4	Raveling
64	37.342	7.778	72.8	Corrugation
65	37.343	7.811	76.7	Pot Hole
66	37.344	7.818	77.5	Block Crack
67	37.344	7.802	75.5	Corrugation
68	37.345	7.821	77.9	Pot Hole
69	37.346	7.827	78.6	Corrugation
70	37.349	7.832	79.2	Grade Depression
71	37.356	7.836	80.1	Pot Hole
72	37.36	7.837	80.7	Pot Hole
73	37.364	7.839	81.2	Block Crack
74	37.368	7.844	81.9	Corrugation
75	37.368	7.861	84	Pot Hole
76	37.369	7.847	82.3	Block Crack
77	37.37	7.849	82.6	Pot Hole
78	37.371	7.856	83.3	Pot Hole
79	37.371	7.868	85	Pot Hole

80	37.372	7.865	84.5	Pot Hole
81	37.375	7.871	85.5	Raveling
82	37.376	7.873	85.8	Alligator Crack
83	37.379	7.875	86.2	Pot Hole
84	37.383	7.884	87.3	Raveling
85	37.395	7.911	91.4	Pot Hole
86	37.409	7.922	94.4	Pot Hole
87	37.423	7.926	96	Alligator Crack
88	37.432	7.925	97.5	Corrugation
89	37.435	7.932	100.5	Grade Depression