KYAMBOGO UNIVERSITY

DIRCTORATE OF RESEARCH AND GRADUATE TRAINING

INVESTIGATION OF CRACKS IN FLEXIBLE PAVEMENT ON A SECTION OF KAMPALA – MASAKA ROAD

BY

COSTA ODWAR MOSES

16/U/15108/GMET/PE

A DISSERTATION SUBMITTED TO KYAMBOGO UNIVERSITY DIRECTORATE OF RESEARCH AND GRADUATE TRAINING IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR AWARD OF A MASTER OF SCIENCE IN CONSTRUCTION TECHNOLOGY AND MANAGEMENT DEGREE OF KYAMBOGO UNIVERSITY

NOVEMBER 2022

DECLARATION

I, Costa Odwar Moses, hereby declare that this submission is my original work and that, to the best of my knowledge and belief, it does not contain any previously published or written works by other authors, nor does it contain any works that have been approved for the award of any other degree by the University or other institution of higher learning, with the exception of those works that have been properly acknowledged in the text and reference list.

Costa Odwar Moses

Sign.. Date 05/11/2022

CERTIFICATION

We, the undersigned, certify that we have read and hereby recommend for acceptance by Kyambogo University a thesis titled "Investigation of cracks in flexible pavement on a section of Kampala-Masaka Road", in fulfilment of the requirements for the award of degree of Master of Science in Construction Technology and Management of Kyambogo University.

Dr. Mugume Rodgers Bangi (Supervisor)

Sign: .... Date: 09/11/2022

Dr. Muhwezi Lawrence (Supervisor)

Sign: .... Date: 08/11/2022

DEDICATION

To my Mum Mrs. Sarah,

To my brothers: Denis, George and Innocent,

To my uncle. David,

To my wife Mrs. Sharon Nabuya, and

To my children: Anna, Abigail, Isabel, and Alvin.

For their love, support, and encouragement……

ACKNOWLEDGEMENTS

I am so deeply indebted to the family of Mr. Odwar Moses & Sarah Akongo and my beloved wife Mrs. Sharon Nabuya who by their sharp vigilance and keen awareness of problems have contributed substantially to my education. For their considerable contribution, these few lines can hardly do justice.

The most important acknowledgement, by far is to my Supervisors, Dr. Mugume Rodgers Bangi and Dr. Muhwezi Lawrence for their expertise, criticisms, technical advice and considerable experience is rendered to me when working on my project. I am especially indebted in this regard to my main supervisor, Dr. Mugume Rodgers Bangi for his assistance, advice and helpful suggestions throughout the preparation of this write-up.

Heartfelt gratitude goes to my family and friends who have been always an encouragement to me. They have made a significant contribution. God bless you all.

Above all, I thank the Almighty God for his providence and the gift of life, which he has given to us.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF ACRONYMS

AADT	Annual Average Daily Traffic
AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highways and Transport Officials
ADT	Average Daily Traffic
ASTM	American Society of Testing and Materials
CI	Condition Index
CV	Commercial Vehicles
CDV	Corrected Deduct Value
DBM	Dense Bituminous Mix
DV	Deduct Value
EF	Equivalence Factor
ESA	Equivalence Standard Axle
ESAL	Equivalent Single Axle Load
FHWA	Federal Highway Administration
FWD	Falling Weight Deflectometer
Gmm	Maximum Theoretical Density
GVW	Gross Vehicle Weight
HMA	Hot Mix Asphalt
Km	Kilo Meter
KN	Kilo Newton
MSA	Millions of equivalent Standard Axles.
MoWT	Ministry of Works and Transport
NDT	Non-Destructive Test
PCI	Pavement Condition Index
PCR	Pavement Condition Rating
TRL	Transport Research Laboratory
TRRL	Transport and Road Research Laboratory
UNRA	Uganda National Roads Authority
VDF	Vehicle Damage Factor
WIM	Weigh-in-motion

ABSTRACT

The study investigated cracks in flexible pavement on a section of Kampala-Masaka Road. The continuous occurrence of cracks which has affected the riding quality, increase in travel times as the vehicles be moving at a slow speed rather than the design speed and eventually the road in future will become not motorable. To realise the study objectives, primary data from the field comprising surface condition survey, traffic counts, FWD test and coring of the pavement for laboratory tests using, Marshall Test, indirect tensile stress test and secondary data for axle load data were acquired from UNRA. The PCI of the subject pavement section was 46.8% which falls between the limit of Poor (41-55) %. Therefore, the condition of the selected pavement section was qualitatively rated as poor. The estimated traffic loading of 54.0 MESA determined was not significantly different from the actual design value of 44.1 MESA indicating that the pavement structure was strong enough to carry the current traffic loading. FWD deflection bowl for the Subgrade layer was still in sound structural condition and for Base layer and Asphalt concrete layers were in warning structural condition. The materials characteristics were within the acceptable specification requirements except for the air voids which were at 5.7% being greater than 5.0% which is the maximum value provided for in the specification. Finally, the study concludes that for the section of the road being along the swamp and the road rehabilitation, which was completed in 2013, much attention was not catered for to strengthen the subgrade, therefore the underground movements in the swamp contributed to the cracks on the asphalt. It is recommended that the existing asphalt layer should be milled off and reconstructed with strict quality control regimes in place.

Keywords: Cracks; flexible pavement, PCI.

CHAPTER ONE

INTRODUCTION

1.1 Background to the Study

An important factor in a nation's economic development and growth is its transportation infrastructure. Goal 9 of the United Nations 2030 agenda deals with the relevance of infrastructure in the development (United Nations 2018). In line with the United Nations (UN) Sustainable Development Goals (SDGs), roads form an important infrastructure for strengthening positive economic and social links between cities and peri-urban and rural areas (United Nations 2018). The most traditional and possibly most extensively used form of transportation used by humans is by road (Mathakiga, 2016). Road travel is by far the most popular form of transportation in Uganda, and it is essential to the country's plans for social and economic advancement. To support its booming economy, the government of the Republic of Uganda has over the years made large financial investments in the construction and renovation of highways (Lubwama, 2008).

To achieve the United Nations SDGs, the Government of Uganda established the Uganda Vision 2040 (National Planning Authority, 2013) to address the issue of poor infrastructure (such as road) across the country. The Uganda Vision 2040 is supported by the National Development Plans (NDPs) especially the NDP III (2020-2025) aimed at implementing crucial developmental strategies (National Planning Authority,2020). A proper road network or infrastructure is important for achieving SDGs 11 (Promote sustainable industrialization), 14 (Build inclusive, safe, and sustainable cities and human settlements) and 15 (Strengthen positive economic and social links between cities and peri-urban and rural areas).

Due to the combined effects of climate, environment, and traffic loads, flexible pavements experience functional as well as structural deterioration simultaneously or independently. The decline in the riding quality, which can be tested using straightforward methods, is a sign of functional deterioration and is shown by changes in the pavement's surface condition. By adding a profile correction course and a resurfacing layer, the pavement's surface can also be restored to its original state. The stability of the current pavement structure and component layers, the size and operation of traffic wheel loads, the growth rate of traffic loads, the efficiency of the pavement drainage system, the severity of environmental and climatic factors, and other factors all affect how quickly flexible pavement deteriorates structurally (Khana, 2014).

The road's pavement begins to deteriorate as soon as it is opened to traffic, however it may even begin during construction. This process begins so slowly that it might not even be seen, but as time goes on, it quickens. The road must be planned, designed, built, and maintained according to best practices to reduce the danger of premature deterioration. This is accomplished by looking at pavements that have failed, with the goal of identifying the root causes so that future failures can be avoided. In-depth examinations may provide a clearer knowledge of pavement failures that could be helpful in lowering future expenditures related to pavement failures. In many circumstances, insufficient maintenance and weak evaluation programs can be directly blamed for the failure of the pavement structure. It's crucial to figure out a way to reduce maintenance expenditures on a tight budget (Madanat, 1994).

1.2 Problem Statement

Roads which are well maintained frequently the minimised the rate of accidents, in a optical developing country like Uganda a lot of money is spent on maintaining road due to, too many defects.

A section of Kampala — Masaka road is facing a distress predominantly cracking experienced on the road. The extensive cracked section of the road is mainly observed from Km 97+000 at Kamuwunga trading centre to Km 102+000 at Lukaya town.

The continuous widening of these cracks will lead to infiltration of rainwater into the voids which has developed into potholes hence affecting the functional and structural performance requirements of the road. Highway pavement deterioration is a very serious issue that leads to excessive traffic jams, distorted pavement aesthetics, vehicle damage, and most importantly, road traffic accidents that result in the loss of life and property.

> *"It is expected that the road will further become less motorable and resulting into high costs of reconstruction in future and increased risks of accidents. According to local officials, more than 200 people have died since January on the Kampala-Masaka highway, which skirts Lake Victoria and connects traffic from Kenya's coast all the way through Uganda to Rwanda, Burundi and the Democratic Republic of Congo beyond. On 2 July 2016 a single accident claimed 21 lives, including that of a six-year-old child, when a trailer truck slammed into two full minibuses and later that month, a district traffic police boss was himself among the victims when five died and 50 were injured in a pile-up involving six vehicles on the road. Its reputation as a death trap means some travelers now avoid using the Kampala-Masaka highway altogether. Despite each accident leading to new calls for something to be done with the road, no one can actually agree on what causes so many incidents to occur" (The Independent, 2016).*

Therefore, this study aims at developing an appropriate strategy to mitigate the cracking observed on the road to restore the functional and structural performance requirements of the road to sustain its design life. It is expected that if the strategy is implemented, it will lead to saving the high costs of reconstruction of the road in future and reduction in travel times as a good road reduces the risks of accidents.

1.3 Research Objectives

1.3.1 Main Objective

The main objective of this study was to investigate the cracking currently on a section of Kampala-Masaka Road and come up with appropriate measures to mitigate the observed distress.

1.3.2 Specific Objectives

This research specifically aimed at addressing the following:

i. To assess the extent of cracking currently existing on Kampala-Masaka Road in order to analyze its functional performance requirement;

ii. To evaluate the strength of the underlying layers of the pavement in order to establish the structural performance in relation to the current traffic loading on the road; and

iii. To develop an appropriate strategy to mitigate the cracking on Kampala-Masaka Road in order to restore the road to achieve its design life.

1.4 Research Questions

The study was guided by the following research questions:

i. What is the current extent of cracking being experienced on Kampala - Masaka road?
ii. What is the current strength and the residual remaining life of the existing pavement?
iii. What appropriate strategy can be developed to mitigate the cracking on Kampala-Masaka Road?

1.5 Research Justification

Highway pavement deterioration is a highly serious issue that leads to excessive traffic jams, distorted pavement aesthetics, vehicle damage, and most importantly, road traffic accidents that result in the loss of life and property. The degree of craftsmanship, materials utilized, quality of design, and oversight during road building all play a role in how quickly roads lose their functional quality.

The continuous occurrence of cracks will further develop into potholes which will affect the riding quality, increase in travel times as the vehicles will be moving at a slow speed rather than the design speed and eventually the road will become not motorable. Thus, the continuous infiltration of rainwater into the cracks will lead to deterioration of a section on Kampala-Masaka Road, from Km 97+000 at Kamuwunga trading centre to Km 102+000 at Lukaya town not to meet its design life.

1.6 Significance

This study developed an appropriate strategy to mitigate the distresses currently experienced on Kampala-Masaka Road. It is expected that if the strategy is implemented, it will save the high cost of reconstruction of the road in future if it were to become un-motorable. Ultimately, this study provided a basis for other academic and practical research studies concerning mitigation of distresses on flexible pavements.

1.7 Project Scope

1.7.1 Contents Scope

This study aimed at assessing distresses currently experienced on a section of the Kampala – Masaka highway by conducting a pavement surface condition survey by visual inspection to identify and classify the defects with the possible causes. By measuring the variation in pavement deflections both along and across the pavement, back calculating the moduli of the pavement layers, and determining the linearity of the pavement response to load, the FWD test structurally evaluates the pavement. To ascertain the axle weight distribution of the heavy trucks using the road, an axle load survey will be conducted. Finally, decision tree method will be adopted to identify an appropriate strategy for the highway engineers to mitigate the cracking currently observed on a section of Kampala - Masaka road from Km 97+000 at Kamuwunga trading centre to Km 102+000 at Lukaya town.

I.7.2 Time Scope

This research was conducted for a period of three years and three months, starting in July 2019 and completed in January 2022.

1.7.3 Geographical Scope

The Kampala-Masaka highway runs through central Uganda and links Kampala, the nation's capital, with Masaka, a town in the country's southwest. The route starts in the Makindye division's Kibuye neighborhood in southwest Kampala and travels through Kyengera, Nsangi, Mpigi, Buwama, Kayabwe, and Lukaya before ending at Masaka, a distance of roughly 127 kilometers. The investigation was done on the portion between the Kamuwunga trading center at Km 97+000 to the Lukaya town at Km 102+000.

1.8 Conceptual Framework

The scheme of concepts or variables were operationalized in the study in order to achieve the set of objectives as shown diagrammatically in Figure 1.1. This entails the ultimate goal (research focus), independent variables, dependent variables, analysis methods and ultimate outcome.

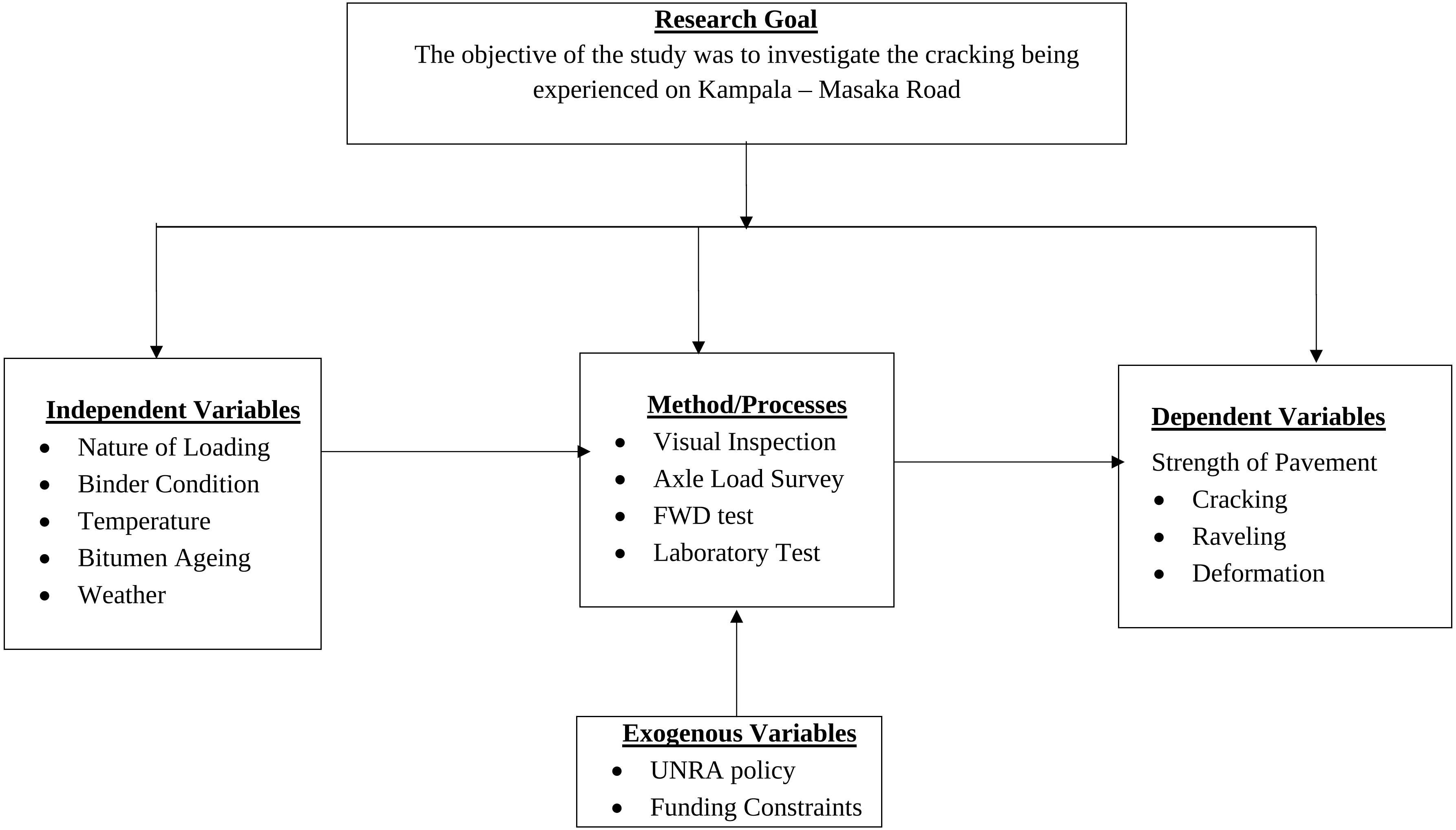


Figure 1-1: Conceptual framework of the study

1.9 Chapter Summary

This chapter has given the brief statement on road transport in the Uganda and how it is widely to be used. It has revealed the distresses in flexible pavements which undergo both functional and structural deteriorations simultaneously due to effects of climate, environment, and traffic loads. In this chapter the method of pavement condition survey by visual inspection and method of structural evaluation of pavement by FWD has been sought of to develop appropriate strategy to mitigate the distresses on Kampala - Masaka road. This research findings will be conducted on section from Kamuwunga trading centre from Km 97+000 to Lukaya town at km 102+000 along Kampala - Masaka road.

CHAPTER TWO

REVIEW OF LITERATURE

2.1 Introduction

One of the most popular ways of transportation is the road. Even in the prehistoric era, roads in the form of human walkways and trackways were used. Since then, numerous trials have been conducted to improve the safety and comfort of riding. As a result, building roads became integral to many civilizations and empires.

The most prevalent component of the transportation infrastructure is pavement, which is constructed to offer the general public a safe and enjoyable ride (Feng and Dar, 2009). The infrastructure, notably roadways, has needed ongoing renovation and expansion in recent years. The elements that have caused significant levels of wear and tear on the highway networks are the growing volume of traffic, load, and environmental conditions. To discover how these elements affect the performance of pavement, more research is required (Mathakiga, 2016).

2.2 Flexible pavement

The reason why flexible pavements get their name is because the entire pavement structure flexes or deflects when it is loaded. Usually made up of many layers of material, a flexible pavement construction accepts loads from the layer above it, disperses them, and then transfers them to the layer below. As a result, a layer's load (measured in terms of force per area) is lower the lower it is in the pavement structure. Other asphalt-surfaced pavements, such as those with bituminous surface treatments, are also categorized as flexible pavements (Mathakiga, 2016).

2.2.1 Flexible pavement structure

The surface course with the underlying base and subbase courses makes up a typical flexible pavement system, as shown in Figure 2.1. Each of these layers helps with drainage and structural stability. According to robust modulus, the surface course (usually an HMA layer) is the stiffest and makes up the majority of the pavement's strength. Although the bottom layers are less rigid, they are nevertheless crucial for drainage and frost protection as well as pavement strength. A typical structural design produces a sequence of layers whose material quality gradually degrades with depth (Hoffman, 2008).

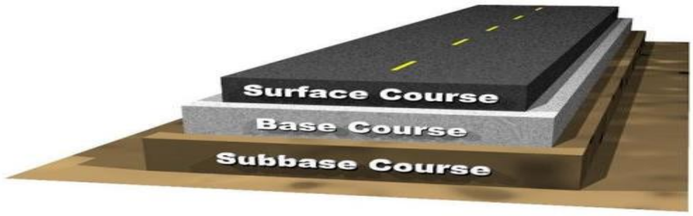


Figure 2-1: Basic flexible pavement structure

Source: (Hoffman, 2008)

a. Surface Course

The surface course is the layer in contact with traffic loads and normally contains the highest quality materials. It provides characteristics such as friction, smoothness, noise control, rut and shoving resistance and drainage. In addition, it serves to prevent the entrance of excessive quantities of surface water into the underlying base, subbase, and subgrade. This top structural layer of material is sometimes subdivided into two layers (Hoffman, 2008):

- **Wearing Course:** This is the layer in direct contact with traffic loads. It is meant to take the brunt of traffic wear and can be removed and replaced as it becomes worn. A properly designed (and funded) preservation programme should be able to identify pavement surface distress while it is still confined to the wearing course. This way, the wearing course can be rehabilitated before distress propagates into the underlying intermediate/binder course.

- **Intermediate/Binder Course**: This layer provides the bulk of the HMA structure. Its chief purpose is to distribute load.

b. Base Course

The base course is immediately beneath the surface course. It provides additional load distribution and contributes to drainage and frost resistance. Base courses are usually constructed out of the following:

- **Aggregate**: Base courses are most typically constructed from durable aggregates that will not be damaged by moisture or frost action. Aggregates can be either stabilized or un-stabilized.
- **HMA**: In certain situations where, high base stiffness is desired, base courses can be constructed using a variety of HMA mixes. In relation to surface course HMA mixes, base course mixes usually contain larger maximum aggregate sizes, are more open graded and are subject to more lenient specifications.

c. Subbase Course

The subbase course is between the base course and the subgrade. It functions primarily as structural support but it can also: (a) minimize the intrusion of fines from the subgrade into the pavement structure; (b) improve drainage; (c) minimize frost action damage; and (d) provide a working platform for construction.

The subbase generally consists of lower quality materials than the base course but better than the subgrade soils. A subbase course is not always needed or used. For example, a pavement constructed over a high quality, stiff subgrade may not need the additional features offered by a subbase course so it may be omitted from design. However, a pavement constructed over a low-quality soil such as expansive clay may require the additional load distribution characteristic that a subbase course can offer. In this scenario the subbase course may consist of high-quality fill used to replace poor quality subgrade (Hoffman, 2008).

d. Subgrade

The natural material along the pavement's horizontal alignment is called the subgrade (It serves as the foundation of the pavement structure). Depending on the type of pavement being built, it may be essential to treat the subgrade material to obtain the desired strength attributes (Ayat, 2013).

2.2.2 Failure in Flexible Pavements

The key to a successful evaluation is differentiating pavement distress categories and tying them to a root cause. When choosing a suitable maintenance or rehabilitation strategy, understanding the underlying causes of the existing issues is crucial.

Environmental factors and structural issues are what cause pavement distresses and degeneration. Finally, loading causes structural induced distresses while weathering and loading alone typically cause pavement deterioration. Environmental induced distresses are caused by weathering, moisture, and aging (Lavin, 2003).

Pavement deterioration is typically brought on by a number of variables, including traffic volume, environmental conditions, initial design, and construction quality. Therefore, traffic-induced anguish, environmentally related distress, and their interplay may all contribute to pavement deterioration. For instance, longitudinal and transverse cracking are seen as environmental or non-load-related distresses but rutting and alligator cracking are considered traffic-induced distresses (Walker, 2002).

It is possible to think of the issue of thermal cracking of flexible pavements in hot climates as a novel type of pavement distress that has only lately been noticed in those places of the world (Abdulwahhab, 1998).

The choice of asphalt grading for use in pavements and the stability of the pavement are both greatly impacted by temperature fluctuations. Pavement engineers will benefit immensely from being able to perform accurate back calculations of pavement modulus values based on the asphalt pavement temperature at various depths and horizontal locations. It will also assist engineers in choosing the asphalt grade to be used in different pavement lifts (Bashir, 2006).

Thermal condition, if not addressed, can lead to significant problems, including the following (OECO, 2008):

a) Cracking caused by large temperature differentials between the interior of concrete and the external environment;
b) Strength loss caused by the freezing of concrete before it has reached sufficient strength; and
c) Strength loss caused by high internal temperatures within the asphalt concrete mass.

2.3 Deterioration in Flexible Pavement

By visually seeing and documenting the many kinds of flaws on the pavement's surface, the state of the pavement can be evaluated. Pavement condition surveys identify surface distresses including cracks, ruts, and other surface flaws. In some circumstances, they can also include an assessment of pavement roughness (Miller and Bellinger, 2003).

The elements of visual assessment of the situation are as follows:

a) Distress Type – categorizing each type of distress as cracking, patching and potholes, surface deformation, and surface defects;
b) Distress Severity – identifying distress severity as high, medium, and low severity; and
c) Distress Amount – identifying the magnitude of each distress type characterized by severity level.

Pavement severity is defined as a qualitative measurement of the rate of deterioration over the pavement surface, with severity levels ranging from low to severe (Bianchini, 2007). Follow the safety procedures before conducting any site inspections to guarantee their safety and the success of the inspection process (Adarkwa, 2013).

The general descriptions of the major types of distress that may be encountered in both flexible (asphalt concrete) and rigid pavements is a typical description of three distress severity levels associated with each distress (AASHTO, 1993). A pavement moisture accelerated distress identification system for these descriptions are provided as a guide to user agencies only and should not be viewed as a standard method for distress type severity identification. This information, along with an estimate of the amount of each distress severity combination, represents an example of the minimum information needs required for a thorough condition (distress) survey (AASHTO, 1993).

2.4 Pavement Distress

Distresses in the pavement are flaws that can be seen on the surface. They are signs of some issue or pavement deterioration occurrence, such as cracks, spots, and ruts. A pavement's kind and degree of distress can offer important clues about what its future maintenance and/or rehabilitation needs will be. The level, extent, and type of the distress are typically reported.

However, the methods for measuring and identifying distress may slightly differ from one organization to another (Luo, 2005). On the basis of appearance, defects in asphalt pavement can be categorized into classes as shown in the following section (David, 2006).

2.4.1 Pavement Cracks

One of the main reasons pavements deteriorates is due to cracks. Numerous research conducted over the last few decades revealed that the most common type of pavement cracking is alligator cracking (Ullidtz, 1987). Pavement engineers must take into account cracking as one of the major in-service forms of deterioration for asphalt concrete pavements when designing new pavements (Smith and Romine, 2001). It is significantly influenced by the bituminous mix's quality, which is directly tied to the right choice of components, including bitumen and aggregates. The temperature and length of time that the asphalt mixes are created, laid, and compacted, for example, are additional elements that affect the qualities of bituminous mixes (Mugume, 2020). Alligator cracks, block cracks, transverse cracks, longitudinal cracks, and edge cracks are among the several types of cracks (Miller and Bellinger, 2003). This section provides a description of pavement cracks:

a) Alligator cracking

Fatigue cracks and crocodile cracks in the surface layer are two more names for the phenomenon known as alligator cracks (Miller and Bellinger, 2003). When it first forms on the pavement surface, it resembles longitudinal cracks in the wheel pathways, but as it develops and grows, it interconnects and takes on the appearance of an alligator crack before finally showing potholes as seen in Figure 2.2. (Hadjidemetriou, 2019). There are numerous variables that contribute to this fracture, including excessive traffic, the base course, a thin surface, weak subgrade strength, and poor drainage, which allows moisture to penetrate the base course and subgrade and damage the pavement (Bianchini, 2010).

b) Block cracking

Block cracks are a network of cracks that divide the pavement surface into square and rectangular sections. The cracks were between $0.1m^2$ and $10m^2$ in size (Miller and Bellinger, 2003). The blocks' sizes could range from roughly (3 by 0.03 m) to (3 by 3m). Block cracking typically signifies that the asphalt has severely hardened. As shown in Figure 2.2, block cracking typically affects a sizable section of the pavement area. However, it can occasionally solely affect non-traffic regions. Alligator cracking is different from this kind of discomfort in that it breaks into smaller, more complex pieces with acute angles. Additionally, unlike block cracks, alligator cracks only appear in traffic zones because they are brought on by repetitive traffic loadings (Zaltuom, 2011).

c) Transverse cracking

Thermal cracks, often referred to as transverse cracks, typically run perpendicular to the pavement centerline (Miller and Bellinger, 2003). These cracks can form on the surface, as seen in Figure 2.2, and typically occur perpendicular to the centerlines. Some factors that contribute to these fractures include improper asphalt mixture setup, low temperatures, and sub-base of pavement layers (Yang and Deng, 2019).

d) Longitudinal cracking

Longitudinal cracks are primarily parallel to the centerline of the pavement and are brought on by reflection cracks in the underlying pavement, worn asphalt on the pavement surface, and poor bonding during construction. Wheel path longitudinal cracks are an indication of fatigue failure from severe truck loads (Miller and Bellinger, 2003). Longitudinal cracks are a type of fraction that primarily appears in the centerlines of pavements (Lan, 2019). according to Figure 2.2. The asphalt layer shrinks due to climate, which promotes the expansion of these fissures. Another source of these fissures is when two pieces of the pavement are improperly joined together. Other potential causes include temperature fluctuations or unfair paver operations (Karimi, 2021).

e) Edge cracking

Pavements without paved shoulders will exhibit edge cracking as continuous or crescent-shaped cracks. Edge cracks are seen within one to 0.6m of the outside pavement edge, near to the pavement shoulder (Miller and Bellinger, 2003). Edge cracks that start from the edge and spread are a sign of distress in narrow pavements; they resemble fatigue cracks in appearance. As seen in Figure 2.2, they occur as a result of weak material or excessive moisture that does not adequately support the pavement's shoulder. Different causes depend on the state of the pavement. As an illustration, it might be caused by inadequate drainage where water is close to the edge, soil movement beneath the pavement, heavy traffic close to the edge, infiltration of water to the base, or a lack of strength in the surface and base (Nega, 2015).

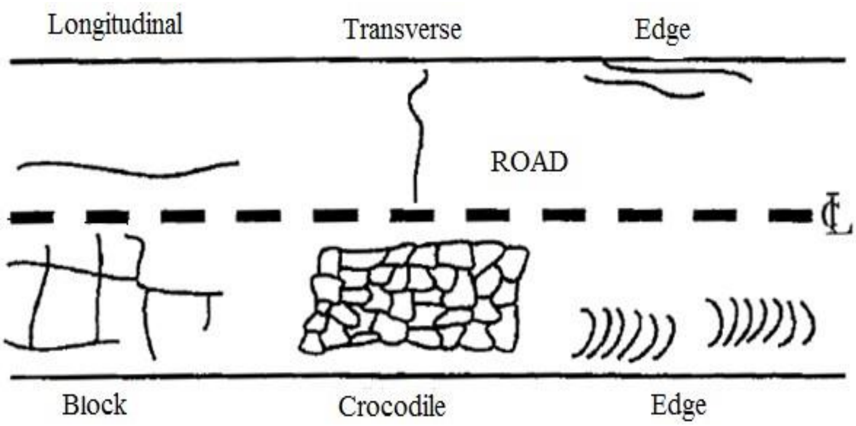


Figure 2-2: Cracks categories

Source: (Hye, 1992)

2.4.2 Patching and Potholes Deterioration

a) Patch deterioration

Patch deterioration is described as the removal and replacement of more than 0.1 m2 of pavement surface or the installation of new material to the pavement following its initial construction (Miller and Bellinger, 2003). Small asphalt pavement fragments that are larger than 0.1 square meters are present on the surface, and they are either replaced or their materials are added in a process known as patch deterioration, as depicted in Figure 2.3. Incorrect compaction during patching, improper material mixing, and rut settlement at the perimeter or within the inner side of the patch are just a few of the problems that lead to patching. For example, when the initial distress expands (Thant and War, 2019).

b) Potholes deterioration

The deteriorating potholes are tiny, bowl-shaped holes of varied diameters in the pavement (Miller and Bellinger, 2003). A cumulative failure known as a pothole causes holes that resemble depressions in a bowl. As illustrated in Figure 2.3, the lowest layers of the pavement experience distress once a little portion of the first layer is damaged. Lack of pavement thickness to manage traffic during freeze or thaw periods is one of the causes. Meanwhile, there are potholes due to inadequate drainage and raveling of fissures. They can be fixed by excavating or rebuilding, but the region may redevelop huge potholes (Yadav, 2019).

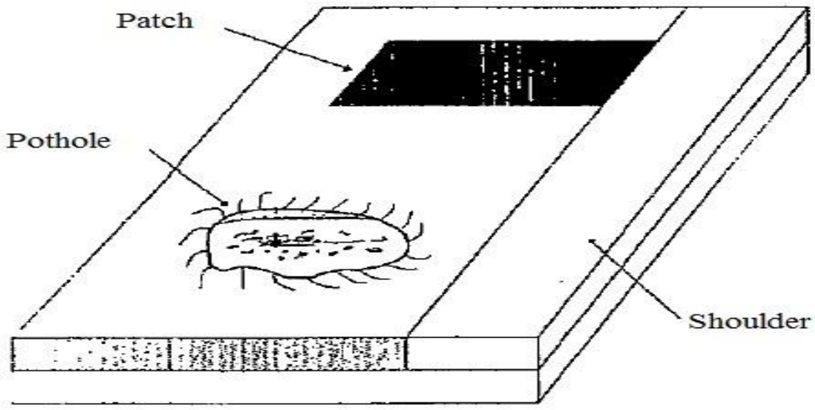


Figure 2-3: Patching and potholes

Source: (Hye, 1992)

2.4.3 Surface Deformations

a) Rutting

Rutting is a depression of the longitudinal surface in the route of the wheels, which damages the asphalt and results in additional cracks. Actually, the cause of the water in the wheel path is a surface failure (Vaitkus, 2014). Rutting, also known as persistent pavement deformation, results in longitudinal depressions that form channels in wheel pathways. This is impacted by material consolidation or lateral displacement brought on by traffic loads, inadequate construction site compaction, an unstable mixture, and pavement breakdown in the lower layers (Miller and Bellinger, 2003). Figure 2.4 illustrates a subgrade failure with a wide, narrow crack. It is conceivable to attribute causes such as insufficient air spaces, excessive dust, rounded aggregate, high asphalt content, and an excessive amount of natural sand (Zhang, 2019).

b) Shoving

Shoving, often referred to as rippling, is a type of plastic movement characterized by a bulge in the road's surface perpendicular to the direction of traffic that is brought on by automobiles pressing up against the pavement (braking or accelerating vehicles). It typically happens at the beginning and end of traffic and acceleration lanes (Miller and Bellinger, 2003).

Pavement deformation occurs when the road surface departs from the profile it had when it was first built, possibly as a result of traffic, environmental factors, insufficient quality control during construction, or all three. It will degrade riding performance and could cause cracking issues. Inadequate pavement thickness, incorrect compaction, low mix stability, layer settlement, a lack of bonding between layers, and pausing at intersection stop signals or roundabouts are some potential reasons of pavement deformation (Shafie, 2007). Figure 2.4 depicts the flexible pavement rutting and shoving.

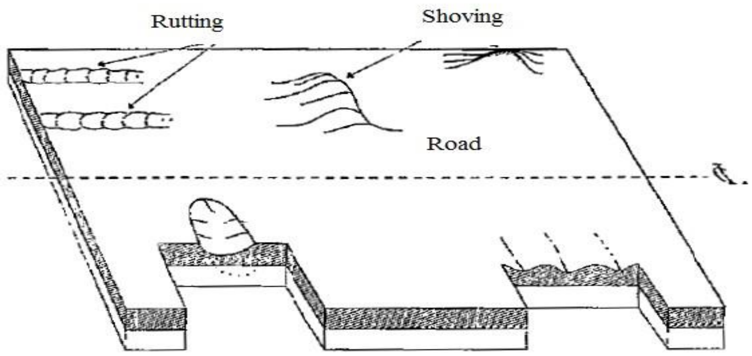


Figure 2-4: Pavement deformation

Source: (Hye, 1992)

2.4.4 Surface Defects

Ravelling, polishing, and bleeding are examples of surface flaws. These flaws significantly impact serviceability, ride quality, and safety concerns (Miller and Bellinger, 2003).

a) Bleeding

The effects of excess bitumen binder on the pavement surface, which are typically found in the wheel paths, can range from a surface that is discolored in comparison to the rest of the pavement to one that is losing surface texture as a result of asphalt overuse to one in which the aggregate may be obscured by excess asphalt and have a shiny, glass-like, reflective surface that may be tacky to the touch (Miller and Bellinger, 2003).

b) Pavement Polishing

The flexible and rigid varieties of pavements both experience pavement polishing. The low proportion of angular-shaped aggregate in the mix is the primary contributor to polishing, which manifests itself in pavement with little to no angular aggregate. Repetition of traffic loads reduces surface friction.

c) Raveling

Asphalt hardening, insufficient asphalt content, loss of asphalt binder and aggregate particles, and insufficient compaction are the causes of raveling. Surface deterioration is caused when aggregate is forced out of the mixture. Figure 2-5 shows the polishing, bleeding, and raveling events.

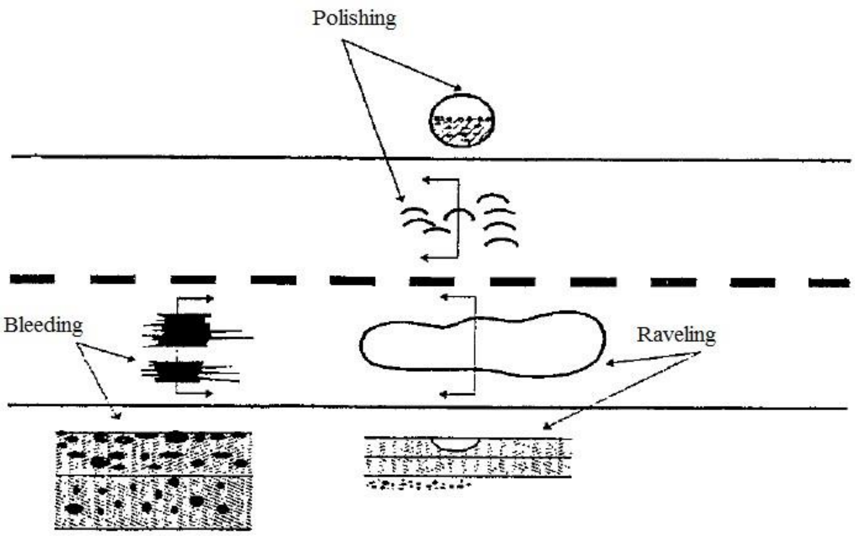


Figure 2-5: Surface defects

Source: (Hye, 1992)

2.4.5 Miscellaneous Distresses

Other flexible pavement issues include reflection cracking at joints, lane-to-shoulder drop-off, and water bleeding and pumping where asphalt pavement has been laid on top of concrete pavement.

2.5 None-wheel path cracking - thin bituminous seals

Thin bituminous sealants on roads make them more resistant to failures that aren't caused by traffic since they have a higher strain tolerance. Particularly surface treatments are less likely to break alongside road markings or at construction joints. Additionally, they are less prone to heat or shrinkage cracking.

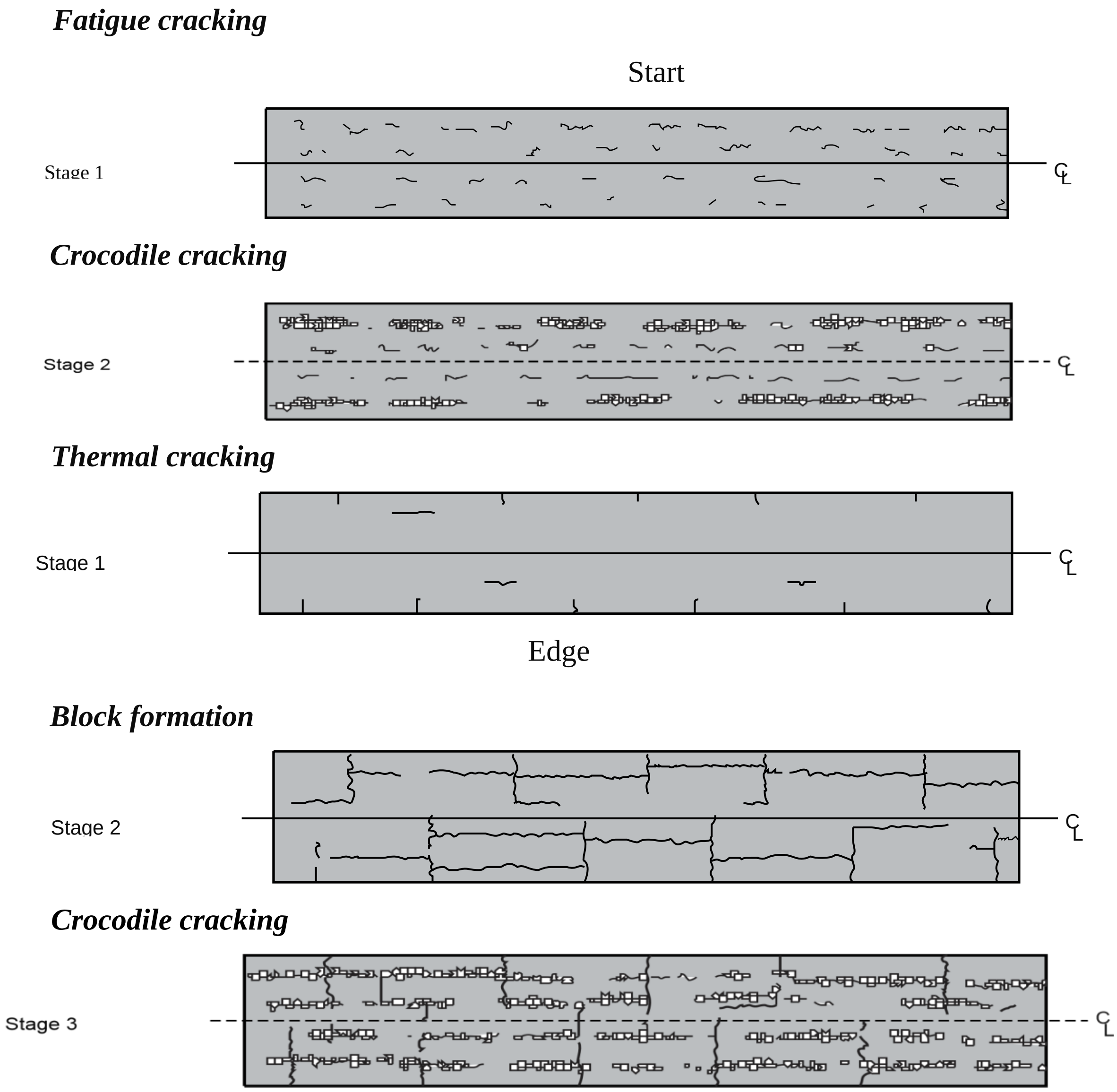


Figure 2-6: Crack development patterns in bituminous surfaces

Source: (Dickinson, 1984)

Where strains are large, however, as in the case of reflection cracking from a stabilized road base or from subgrade movement, the surfacing failure will be similar to that described for asphalt surfacing as seen in Figure 2.6 (Overseas Road Note 18, 1999).

2.6 Assessment of surface condition

Depending on the road's current state, it could be required to divide it again after cutting it into pieces of presumably similar design. You can accomplish this by conducting a windshield survey. In order to accurately record the status of the road pavement using a selection of the road pavement deterioration criteria listed below in surfacing flaws, it is advisable for the survey vehicle to halt at intervals of 500 meters or one kilometer. Keep in mind that if the vehicle is not stopped and survey crew is not given the chance to closely check the road, critical features of road deterioration may be missed. These measurements are necessary for the economic appraisal and are useful in defining sections of road in similar condition. The road can then be subdivided into shorter uniform sections based upon the following: a) time since construction; b) traffic loading; c) type of road deterioration; and d) topography (Overseas Road Note 18, 1999).

Detailed condition surveys of the sections are then carried out. When the uniform sections are relatively short, the detailed condition survey is best carried out over the entire length of the section. However, where resources are limited then a number of representative one-kilometre lengths of road can be used to identify the cause of pavement distress. The length of road investigated by this method should represent no less than 10 per cent of each section (Overseas Road Note 18, 1999).

Before the detailed surface condition is carried out, the section or representative one-kilometre length is permanently marked into ‘blocks’ of equal length. For inter-urban roads the maximum block length should be either 50 or 100 meters, however, the length may be reduced to as short as 10 metres if the road is severely distressed.

During the detailed surface condition survey, the nature, extent, severity, and position of the following defects is recorded:

i Surfacing defects such as bleeding, fretting, stripping

ii cracking

iii deformation (excluding rutting)

iv patching and potholes

v edge failures

vi Rutting is recorded once at the beginning of each of the blocks. It is important that rutting is measured at a discrete point as its severity may need to be compared with other non-destructive tests carried out at the same location.

2.6.1 Detailed condition survey

Four technicians/laborers, a support vehicle, and a driver are part of the team doing the thorough surface condition study, which is a walking survey. At all times, a secure workplace should be maintained. There are many on-site protocols that must be followed by many organizations. After the road has been permanently demarcated, the crew will need the following tools: a) traffic control signs or flags; b) a 2-meter straightedge and wedge; c) a crack width gauge; d) a distance measurer; and e) surface condition forms and a clipboard. (International Road Note 18/1999).

2.6.2 Condition Rating Systems

The rating is based on metrics from the data collecting procedure, including roughness, skid resistance, deflection, and others. The performance of two road sections is compared using condition ratings as the basis. Most importantly, it assists organizations with prioritizing treatment processes, assessing the severity and scope of pavement faults, and estimating the cost of repair and rehabilitation. It serves as a foundation for budget planning as well. The political pressure that makes up the majority of the decision-making process has been lessened as a result of condition rating indices (Adarkwa, 2013).

Because most pavement designers and maintenance staff must take pavement condition into consideration in their actions, (Haas and Hudson, 1978) said that pavement condition and performance are topics of fundamental importance in pavement management.

2.6.3 Pavement Condition Index (PCI)

The US Army Corps of Engineers created the PCI measured condition rating system, which was subsequently embraced by the American Public Works Association and the American Society for Testing and Materials (ASTM) (ASTM D6433-11, 2011). It is based on a 0-100 scale, as illustrated in Figure 2.7. (Illinois Center for Transportation, research Report). Based on the kind, intensity, and extent of each distress found on the pavement, a value is assigned to it. The rating of the pavement condition is then determined by adding all the points and subtracting them from a total of 100. Condition of the entire section as determined by the weighted average of the PCIs for multiple sub-layers of pavements and 19 distress kinds for flexible pavements. There are 39 distresses with high, medium, and low severity levels. There are 20 problems with asphalt paving (Adarkwa, 2013).

Figure 2-7 Pavement condition index, rating scale and suggested colors
Source: (ASTM D 6433-07)

2.7 Traffic in Flexible Pavement

Traffic volume and the surrounding environment are the two main causes of pavement deterioration, claims Huang (2004). In a road test conducted by AASHO in the late 1950s, the relationship between traffic load and other variables on pavement performance was developed (Schwartz and Carvalho, 2007). The number of load cycles, pavement structural capacity, and performance as assessed by serviceability are all correlated according to the AASHTO (1993) design equation, which is principally based on the AASHO road test (Schwartz and Carvalho, 2007).

2.7.1 Axle load surveys

To ascertain the axle weight distribution of the heavy trucks using the road, an axle load survey is conducted. The mean number of comparable standard axles for a typical vehicle in each vehicle class is then determined using this survey results. The entire expected traffic loading for the road over its design life, expressed in millions of MSA, is then calculated by combining these numbers with traffic flows and predictions (Kumar, 2014).

An axle load survey's primary goal is typically to calculate the average equivalence factor (EF) for each survey vehicle type. It is currently standard procedure to determine the EF for each axle of each vehicle, add these values, and then get the EF for each and every vehicle. The number of standard (80 KN) axles that would inflict the same amount of damage is the equivalence factor, which measures the typical harmful effect of a vehicle on the pavement.

It is true that a single maximum load could result in a failure, however this load would need to be extremely high. Fatigue failure, or the application of axle weight repeatedly, is the main cause of pavement failure. The equivalent single axle load (ESAL), which is the single traffic parameter for design purposes, is obtained by adding together the equivalent effect of all axle loads during the design life of pavement (Huang, 2004). The following equation predicts the total number of ESAL during the pavement design phase.

$$ESAL = AADT * T * T_f * G * D * L * 365 * Y \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(Equation 2.1)}$$

Where:

ESAL = Equivalent Standard Axle Load

AADT = Annual Average Daily Traffic

T = Percentage of trucks

T_f = Truck factor

G = Growth factor

D = Directional factor

Y = Design period in years

Details of all variables of Equation 2.1 are discussed in Huang (2004). The truck factor (T_f) in equation 2.1 is expressed as:

$$T_f = \sum (pi * LEFi) * A \ldots\ldots\ldots\ldots\ldots\ldots \text{(Equation 2.2)}$$

Where:

Pi = Percentage of repetitions for i^{th} load group

LEFi = Load Equivalence Factor for the i^{th} load group

A = average number of axles per truck

In terms of how to take vehicular load and traffic effects into account while designing a pavement, there are three basic approaches: fixed traffic, fixed vehicles, and variable traffic & vehicles. According to Huang (2004), the wide range of axle loads and traffic volumes, as well as their unavoidable effects on the performance of pavements, necessitate that the majority of design techniques used today be based on the fixed-vehicle concept.

The number of repetitions of a standard vehicle or axle load determines the pavement thickness in the fixed vehicle technique. The destructive force of a vehicle axle in relation to a standard axle load, typically 8,200 kg, is taken into account using the load equivalence factor (Huang, 2004). The 18-kip single axle weight was chosen because, at the time of the AASHO Road Test, it was the highest legal load permissible in many states in the United States of America (USA) (Schwartz and Carvalho, 2007).

2.8 Flexible Pavement Evaluation using the Falling Weight Deflectometer

When evaluating a pavement's structural reaction, surface deflections under forward-moving traffic must be measured, and the appropriate combination of layer attributes must be optimized to meet the deflection basin. The pavement's current state is described by these qualities. The projected characteristics are used to estimate how long the pavement will last (Uzan, 1994).

2.8.1 The Falling Weight Deflectometer (FWD)

The FWD has been proven to be the most efficient testing tool for assessing a pavement's structural integrity (Brown and Tam, 1987). An impact load is applied to the pavement surface by a trailer-mounted NDT device using a falling weight on a circular plate with a 300 mm diameter. By adjusting the weight's mass or the height of the drop, the load magnitude can be changed. The applied force is measured using a load cell, and the impact load has an overall duration that normally ranges between 25 and 30ms and a peak force of up to 125 KN. Peak deflections are measured using velocity transducers (geophones) in contact with the pavement surface, as illustrated in Figure 2.8, at the load plate's center and six different places out from the plate. The seven-deflection data can indicate the deflection basin under the FWD load, assuming symmetry around the load, because the geophones are typically configured to be 300 mm apart.

The FWD has the following advantages over laboratory testing (Ali and Khosla, 1987):

i. Low operating cost a laboratory test programme for measuring layer moduli will be 60-80 times more costly than a corresponding field test program using the

FWD (Houston, 1992). This includes factors such as traffic control cost and the monetary value of project delay;

ii. Short test duration and rapid data collection;

iii. Simple testing procedures;

iv. No physical damage to the pavement structure;

v. No disturbance effect to the sample;

vi. It can simulate the effect of moving traffic loads; and

vii. Full scale model test where the test measures the insitu pavement behaviour under traffic load.

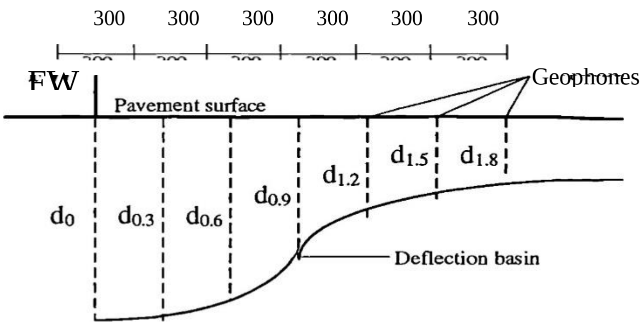


Figure 2-8: Configuration of the FWD

Source: Ali and Khosla, (1987)

2.8.2 Background to original deflection bowl parameter benchmark methodology

It is usual practice to use a Falling Weight Deflectometer to measure the deflection response of a road surface structure (FWD). When a 40kN weight is dropped from a standard height onto a load plate with rubber cushions using an automated FWD, the deflection bowl is measured. The diameter of the FWD loading plate is 300mm (Horak, 1989). The measuring configuration of the FWD's geophones is shown in Figure 2.10. The standard FWD geophones in South Africa are configured to measure deflection

(D) at zero (D0), 200mm (D200), 300mm (D300), 450mm (D450), 600mm (D600), 900mm (D900), 1200mm (D1200), 1500mm (D1500), and 1800mm (D1800). The 40kN dumped weight is equal to 50kN of a truck's typical 80kN axle load. These distinct measurement locations on the deflection bowl enable straightforward spreadsheet computations of deflection bowl characteristics representing different zones or sections of the entire deflection bowl.

Figure 2.8 also shows the deflection bowl's three distinct zones: the positive curvature zone near the point of loading, the inflection curvature zone between 300 and 600 millimeters from the loading point centroid, and the outer edges, which are typically referred to as the negative curvature zone from roughly 600 millimeters from the load centroid up to a distance of 2 meters. It has been discovered that the zones shown in Figure 2.9 correlate extremely well with the structural reaction of particular structural layers or combinations of structural layers in the pavement structure (Horak, 2006).

The most popular deflection bowl or basin parameters used in pavement structural evaluation are included in Table 2.1. (Horak, 2006). Additionally, it shows which combinations of pavement structural layers have shown to best correspond with the various deflection bowl metrics. The slope parameters, designated as BLI, MLI, and LLI (parameters 2, 3, and 4) in Table 2.1, served as the foundation for the first developed benchmark technique (Horak, 1989). Additionally, it was later demonstrated that the radius of curvature (RoC) parameter can be used to assess the structural reaction of asphalt surfacing and the top of the base layer.

Table 2-1: Deflection Bowl Parameters (Horak, 2006)

Parameter	Formula	Structural indicator and association with pavement zone
1. Maximum deflection	D_0 as measured under the Centre of the load	Gives an indication of all structural layers with about 70% contribution by the subgrade
2. Base layer index (BLI) Previously referred to as surface curvature index (SCI)	$BLI = D_0 - D_{300}$	Gives an indication of primarily the base layer structural condition
3. Middle layer index (MLI) Previously referred to as base curvature index (BCI)	$MLI = D_{300} - D_{600}$	Gives an indication of the subbase and probably selected layer structural condition
4. Lower layer index (LLI) Previously referred to as base damage index (BDI)	$LLI = D_{600} - D_{900}$	Gives an indication pf the lower structural layers like the selected and the subgrade layers
5. Radius of curvature (RoC)	$RoC = \left[\frac{(L)^2}{2D_0(1 - D_{200}/D_0)}\right]$	Gives an indication of the structural condition of the surfacing and top of the base condition

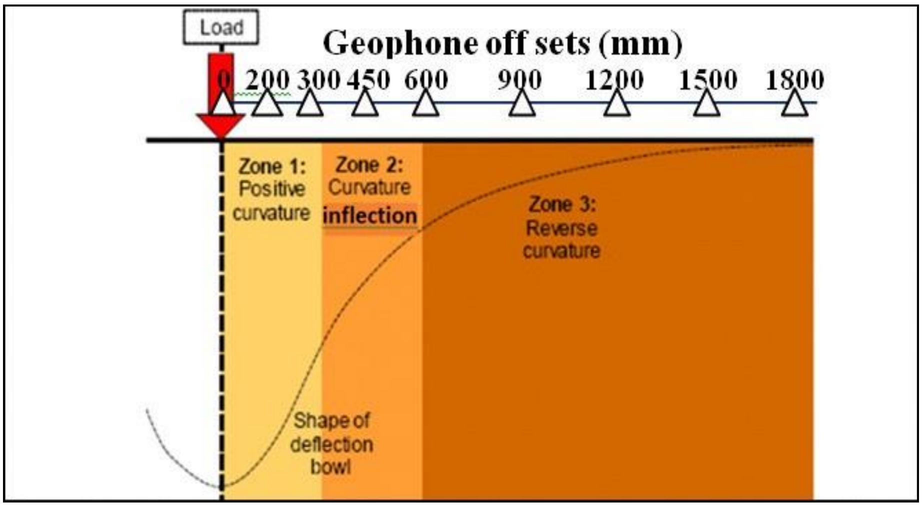


Figure 2-9: FWD deflection bowl illustration with measuring geophone set-up
Source: (Horak, 2006)

Maximum deflection, which has a long history in empirical structural response relationships with older equipment like the Benkelman Beam, where rebound deflections with plastic deformation elements included in the elastic rebound response were used, reflects the elastic response to loading of the entire pavement structure. Due to the dropping load imitating a moving wheel traveling at about 60 kph, the FWD can monitor largely elastic response. The subgrade elastic response may account for up to 60% to 70% of the maximum deflection (D0), as measured with the FWD (Horak, 2006)

Initial investigations using an accelerated pavement testing apparatus (Heavy Vehicle Simulator - HVS) and its modified Benkelman Beam provided the foundation for the deflection bowl specifications (Road Surface Deflectometer - RSD). Later, as previously mentioned, these parameters were modified to fit FWD deflection bowls and standardized metric offsets (Horak, 2006).

2.9 Internal Factors

Material characteristics and pavement structure are internal elements that could influence flexible pavement cracking. The sections that follow go through these elements.

2.9.1 Asphalt Binder

Asphalt binder comes in a variety of forms, including natural asphalt and asphalt made from the refining of petroleum. Asphalt is a cementation material that can be found in nature or created by the refining of petroleum, according to ASTM D 8-02 from 2003.

Asphalt binder grade has a big impact on pavement cracking since the key component affecting pavement cracking is the binder viscosity (Ali, 2006). At a given temperature, the viscosity of asphalt varies from grade to grade. The likelihood of flexible pavement cracking is reduced by flexible pavement with a tougher and less temperature vulnerable binder (Ali, 2006).

2.9.2 Air Voids in Total Mix

The most influential characteristics of asphalt mixes that may affect pavement cracking are the number of air voids in the total mix (VTM) and an excessive amount of asphalt binder in the total mix (AC) (Brown and Cross, 1989). AASHTO (1997) calculated the percentage of voids in the compacted mixture to represent the voids in the whole mixture. One of the significant factors that significantly affects pavement performance under traffic loads is the VTM content. When there is enough stability and air spaces in a mixture, it performs well (Wagner, 1984).

2.9.3 Layers Thickness

One of the most crucial components of a pavement's flexible pavement mechanism is the degree of stress; in addition, the amount of stress is influenced by the thickness of the pavement layers and the traffic loads (Gillespie et al., 1993). According to Isa et al. (2005), flexible pavement with thicker layers will disperse less loads to the subgrade and hence experience less vertical critical strain. Ali (2006) shown that the thickness of the surface layer influences pavement cracking; consequently, a thin surface layer with uneven traffic loads causes pavement cracking because of high stresses in the layer, which causes cracking instability.

2.9.4 Voids in the Mineral Aggregate

The percentage of voids in the compacted asphalt mixture is known as voids in the mineral aggregate (VMA). (VMA) is the inter-granular void space that exists between the aggregate particles in a compacted asphalt mixture and is occupied by asphalt and air, according to Roberts et al. (1996). The volume of absorbed asphalt binder is not included in VMA since it does not take into account air voids or the effective asphalt in the overall mix. Roberts and others (1996). Because the asphalt binder won't coat each individual particle, small void spaces between the particles will result in low VMA, whereas a combination with excessive VMA will have low mixture stability. To obtain an appropriate VMA and subsequently an acceptable mixture, the asphalt binder should coat each individual aggregate particle in the mixture (Rahman, 2006)

The pavement mixture may be significantly impacted by the aggregate gradation. Therefore, altering the gradation will have an impact on VMA and VTM, as well as durability, usability, mixture stability, and surface skid resistance. As a result, when designing the mix, the aggregate gradation should be chosen to adhere to the design requirements (Chadbourn et al., 1999).

Due to minimal voids and plastic flow, mixtures with elongated and flat particles have a propensity to densify under traffic (Roberts, 1996). On the other hand, a mixture with a lot of crushed aggregates and crushed aggregates with more angles tends to have a higher VMA and crack more easily (Chadbourn et al., 1999).

2.9.5 Marshall Stiffness

The Marshall Mix Design Method was created by Mississippi Highway Department employee Bruce Marshall in the late 1930s (White, 1985). After that, the U.S. Army refined it, and around 38 states employed it to some level (White, 1985). Marshall Stability and Marshall Flow are two crucial measurable metrics in this procedure. To achieve appropriate stability and flow, engineers might choose the amount of asphalt binder component at a particular density in the mix (Kandhal and Koehler, 1985, Usmen, 1977).

Marshall Stability is used to gauge the aggregate asphalt cement mixture's mass viscosity. In order to help choose the ideal asphalt content, this property is used to assess the performance of asphalt under loads and the change in mix stability with increasing asphalt content. The stability of the mix is influenced by the angle of aggregate friction and the viscosity of the binder (Abukhettala, 2006). A stable mixture is one that can support traffic loads and resists deterioration of the pavement for the duration of the mixture's design life (Asphalt Institute, 2001).

A mixture with a high Marshall Stability is therefore stable and will not cause pavement cracking.

The specimen's vertical deformation is known as Marshall Flow. Up until the moment where Marshall Stability starts to decline under loading, it is assessed concurrently with Marshall Stability. Marshal Flow should be approximately 16 in an acceptable mix design and construction, whereas mixtures with Marshall Flow above 16 are more likely to display rutting (Abukhettala, 2006).

Marshall Stiffness (MS), calculated as Marshall Stability divided by Marshall Flow, determines the material's resistance to rutting in pavement and estimates the load deformation properties of the combination (Asphalt Institute, 2001). Low Marshall Stiffness mixtures are stiffer and less likely to cause pavement cracking (Abukhettala, 2006).

2.9.6 Subgrade Material Stiffness

The most frequent subgrade material characteristics are material stiffness, which is the capacity of subgrade material to carry repetitive of traffic loads, material strength, and bearing capacity. The subgrade material's stiffness needs to be sufficient to support and distribute the applied traffic loads; as a result, the higher the subgrade material stiffness, the less rutting there will be in the pavement, and the lower the subgrade material stiffness, the more cracking there will be in the pavement. The most popular ways to characterize subgrade stiffness are the California Bearing Ratio (CBR), resistance value (R- Value), and resilient modulus (MR) (WAPA, 2002).

2.9.7 Pavement Structural Strength

The roadbed layers' capacity to support constant traffic loads and evenly distribute vertical deformation to the lowest layer defines the structural strength of the pavement. The structural number (SN), a metric for pavement structural strength used in the AASHTO method of pavement design, depends on the thickness and type of the surface, base, and subbase layers.

2.10 Appropriate Strategy to mitigate the cracks on the study road

According to the definition of maintenance strategies, these are several tasks that are chosen for each highway system after study in order to raise the pavement rating over a predetermined minimum standard (Lu, 1976). There are three categories for pavement repair techniques: Pavement maintenance strategies include preventative maintenance, corrective maintenance, and emergency maintenance. It entails surface operations and treatments aimed at slowing the progression of failures and lowering the demand for regular maintenance and repair work. When a flaw in the pavement manifests itself, such as a lack of friction, moderate to severe rutting, or extensive cracking, corrective maintenance is carried out. Another name for it is "reactive" maintenance. When there is an urgent need for repair, such as a severe pothole or blowout, emergency maintenance is carried out. It also comprises short-term fixes intended to hold the surface together until a long-term fix can be made (Shafie, 2007).

Various indicators, whether they are standalone indices or a mix of them, have been created to assess pavement performance (Zhang, 1993). The Present Serviceability Index (PSI) and the IRI are functional performance indices used to describe a pavement's ride quality, whereas the Structural Number (SN) is a structural performance index used to assess structural capacity (Horak, 2008).

2.11 Other Studies

The biggest issue with cracks is that they let moisture into the pavement, causing it to deteriorate more quickly. Cracks can appear in many different patterns. They may be caused by a variety of factors, but typically arise from the aging and brittleness of the pavement's surface, environmental factors, structural or fatigue breakdown of the pavement, or any other factors (Jain and Kumar, 1998).

Numerous issues arise from the development of cracks in the pavement surface, including irritation for users and a decline in safety. In addition to the aforementioned, a significant issue with pavements is water intrusion, which weakens lower layers and decreases the carrying capacity of subgrade soil by pumping soil particles through fissures (Ahmed, 2008).

CHAPTER THREE

RESEARCH METHODOLOGY

3.1 Introduction

This chapter presents the methods and procedures used in data collection and analysis which consisted of collection of data from the field. The fieldwork involved pavement condition survey through conducting visual inspection and field sampling through which asphalt core samples where extracted for laboratory tests; structural evaluation of pavement was by conducting FWD test on the existing pavement; axle load surveys were conducted by static method of weigh bridge in Lukaya town to determine the axle load distribution of the heavy vehicles and finally developed an appropriate strategy to mitigate the cracking on a section of Kampala-Masaka Road in order to restore the road to achieve its design life.

3.2 Research design

The study was a case study of experimental type for investigation of cracks in flexible pavement on a section of a road. Common feature included collecting and analyzing data, cases were studied in their real-life context, cases would naturally occur in the sense that they were not manipulated as in an experiment. The use of multiple sources of data included observations, laboratory testing, archival documents, and even physical artefacts to allow triangulation of findings. Case studies were most commonly associated with qualitative research and qualitative data, but this need not be so and quantitative data can readily be incorporated into a case study where appropriate.

3.3 Research approach

The flow of the research was a qualitative approach which included selection of study road, survey types which for both pavement condition and axle load surveys followed by field sampling for extraction of cores for laboratory tests followed by structural evaluation of pavement by FWD test and finally data analysis and reporting. The choice of the study road was considered after a reconnaissance survey to establish a road with cracks and be able to conduct field tests such as axle load and FWD test and pavement condition surveys provided later in this chapter. This study involved both destructive and nondestructive tests with four pavement distress types, namely: depression, pothole, rutting and cracks.

3.4 Description of research area

Kampala-Masaka Road is located in central region of Uganda, connecting the Capital City of the republic of Uganda known as Kampala to the south-western town of Masaka in Masaka district. It is entirely paved and comprised of a single carriageway. Kampala-Masaka Road is one of the busiest arterials from Kampala to the south-western towns of Masaka, Mbarara to Katuna, and the Rwanda boarder with very high vehicular traffic. This could be attributed to the many commercial and social facilities abutting the road such as trading centres, which attracts considerable traffic. The study was conducted on an approximately 5 km stretch of the road which resulted from the reconnaissance survey conducted on a section of Kampala-Masaka Road observed from Km 97+000 at Kamuwunga trading centre to Km 102+000 at Lukaya town, section of the road registered deterioration informs of cracking.

3.5 Data collection

The research started by collecting secondary data from the website, books, journal, newspaper, reports and related documents to the subject matter. It was then followed by primary data from the field which comprised pavement condition survey at section from Km 97+000 to Km 102+000 to determine the pavement condition index and its rating, axle load survey from a static weighbridge at Km 102+000 in Lukaya to determine the load distribution of the heavy vehicles on the road which followed with the FWD test to evaluate the structural strength of the pavement and finally coring of the pavement to extract cores which were using for laboratory tests for the material properties of the asphalt pavement.

3.5.1 Pavement Condition Survey

a. Introduction

The objective of the road and pavement condition surveys was to identify defects and sections with similar characteristics. All defects were systematically referenced, recorded and quantified for the purpose of determining the optimum design/maintenance alternative. The pavement condition surveys were carried out using visual observations, supplemented by actual measurements and in accordance with the widely accepted methodology as per the guidelines (ASTM D6433-20, 2011). The measurement of rut depth was conducted using standard straight edges. The shoulder and embankment conditions were evaluated by visual means and the existence of distress modes (cuts, erosion marks, failure, drops) and extent (none, moderate, frequent and very frequent) of such distress manifestations are recorded. Various distresses were measured and recorded in the developed visual condition survey format that bifurcated 18 different types of distresses as per the guidelines suggested by (ASTM) (ASTM D6433-11, 2011). The road section was divided into various subsections of 500m interval each. Subsequently, the distresses were recorded manually for each road section and photographs of each pavement section are taken. However, sample photographs of distress identified at each road section was provided in Appendix A. The detailed quantification of each type of distress were carried out as per the guidelines suggested by (ASTM D6433-11, 2011)

b. Estimation of PCI (ASTM D6433-11, 2011)

PCI value for each sub-section was estimated based on the percentage contribution of each distress from the total area of each sub-section as per the guidelines suggested by the ASTM D 6433-11. The severity level of each distress was designated in three categories (low, medium, and high) based on the unit length and area. Figure 2.7 shows recommended typical PCI rating scale of 0 to 100 each distress has been assigned by a deduct value according to the severity and intensity levels as shown in table 3.1. The generic procedure adopted for the calculation of PCI value from the calculated percentage contribution of each distress for each sub-section of the selected pavement section is discussed below. Therefore, the mean PCI value was estimated for each pavement section from the estimated PCI value for each sub-section of the selected pavement section. The detailed calculation sheets of distress intensity and PCI values for each sub-section of each pavement section is provided in Appendix B.

The calculation procedure is summarized in following steps:

i. Determination of pavement distresses and their severity, which can be low, medium, or high.

ii. Determination of deduct values from the deduct value curves for each distress. Figure 3.1 shows typical deduct value curve for longitudinal cracking.

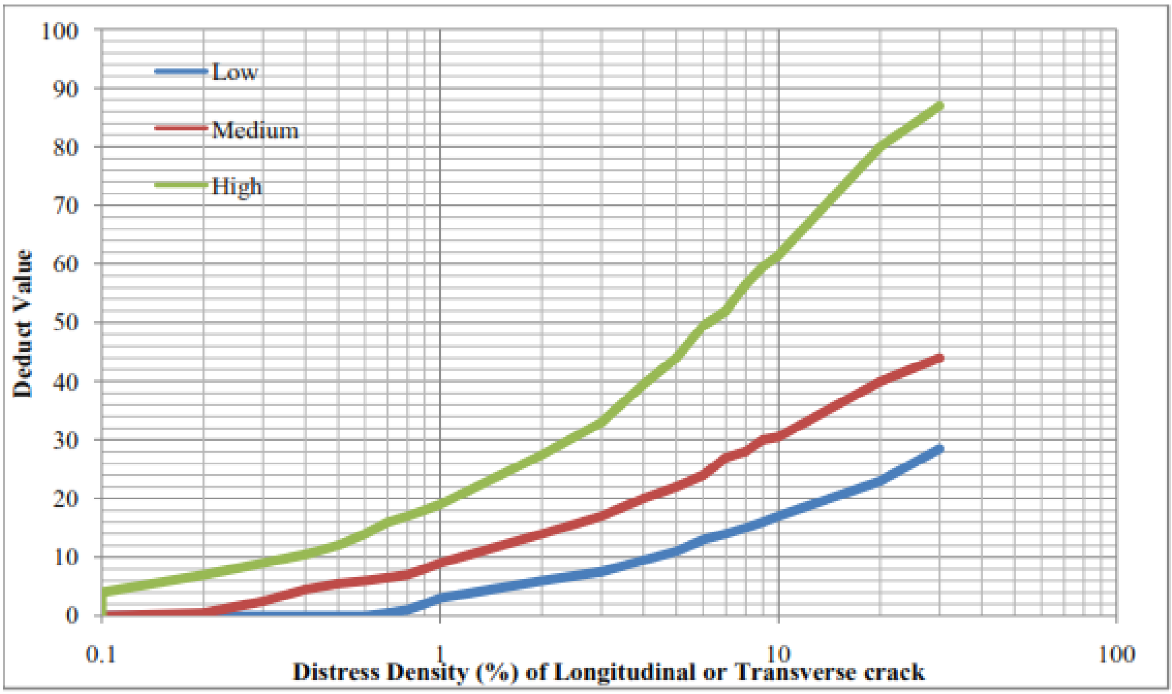


Figure 3-1: Typical deduct value curves for Longitudinal or Transverse crack

Source: (ASTM D-99, 1999)

iii. Calculation of maximum number of deduct values from the maximum allowable deduct number, by using Equation 3.1.

$$Mi = 1 + (9/98)\,(100 - HDV) \text{ …………………. (Equation 3.1)}$$

Where, Mi = maximum allowable number of deduct values and HDV= greatest individual deduct value.

iv. Determination of q, for the number of deducts values greater than 2.

v. Determination of the total deduct value (TDV), which is the summation of all deduct values.

vi. Determination of the corrected deduct value (CDV) based on the correction curves using q and the TDV

vii. Reductions of the smallest deduct value greater than 2 to exactly 2.

viii. Repetition of steps 4 through 7 until q is equal to 1.

ix. Determination of the maximum CDV (CDVmax) and computation of the PCI using Equation 3.2.

$$PCI = 100 - CDV_{max} \quad \text{(Equation 3.2)}$$

Table 3-1: Pavement Condition Rating (Shahin, 1990)

Condition Rating	PCI	Remarks
Failed	0-10	Totally Unserviceable
Very Poor	11-25	
Poor	26-40	
Fair	41-55	
Good	56-70	Critical Range
Very Good	71-85	
Excellent	86-100	Newly Constructed

c. Steps for calculating PCI of flexible pavement:

Data collected during the visual inspection were used to calculate the PCI. This paragraph explains how to calculate the PCI for the first section. An important item in the calculation of the PCI is the "deduct value." A deduct value is a number from 0 to 100, with 0 indicating the distress has no impact on pavement condition and 100 indicating an extremely serious distress which causes the pavement to fail.

i. Determination of distress types and severity levels and measurement of density: Each sample unit was inspected and distress data (type and severity levels) recorded on data sheet form.

ii. Determination of deduct values:

Added up total quantity of each distress type at each severity level and recorded them in the "Total Severity" section. The total quantity of each distress type was divided at each severity level by the total area of the sample unit and multiplied by 100 to obtain the percent density.

iii. Determine the deduct value (DV) for each distress type and severity level combination from the distress deduct value.

iv. Determine the corrected deduct value (CDV):

If none or only one individual deduct is greater than two, the total value is used in place of the maximum CDV in determining the PCI; otherwise, maximum CDV must be determined.

Determination of the allowable number of deducts **m**, using the following formula:

$$m = 1 + (9/98)\,(100 - HDV) \leq 10 \qquad \text{(Equation 3.3)}$$

Where:

m = allowable number of deducts including fractions (must be less than or equal to ten).

HDV = highest individual deduct value.

For example:

$$m = 1 + (9/98)\,(100-48) = 5.77 \qquad \text{(Equation 3.4)}$$

Table 3-2: Pavement condition survey data sheet for road section from Km 97+000 to Km 102+000

Asphalt surfaced roads
Condition survey data sheet for Kampala – Masaka Road Section from Km 97+000 to Km 102+000

Road Section: Km 97+000 – Km 97+500
No. of section: 01
Surveyed by: Costa Odwar
Date: October,2019

7.5 m

Direction of survey → 500m ← N

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

Distress severity	**Quantity**									**Total**	**Density**	**Deduct value**
1L	2.5*4.7	0.5*1.5	0.4*8	1.3*6.5	1*1.8	1*4.5					**30.45**	**3.80**	**23**
1 M	2.5*5	4.5*3.3	1.8*2.3	1.3*1	1.3*1.2	1.1*1.2	1.4*4.5	1.6*6.5	3.5*6.6	4*1.1			
	0.6*0.5										**80.17**	**10.02**	**48**
3 L	0.8*6.1										**4.88**	**0.61**	**0**
3 H	1.4*4.5										**6.3**	**0.78**	**5**
10 L	2.4	2.1	3.6	1.8	2.1	2.1	2.9	1.05	3.35		**21.4**	**2.7**	**6**
10 M	7.5	3.8	3.4	3.3	4.8	4.9	1.45	3.36	4.44	3.37			
	1.4	2.5	3.37	3.56	4.25	3.45	1.4	4.2	1.48	3.42	**69.35**	**8.67**	**18**

The number of individual deduct values was reduced to the m largest deduct values, which included the fractional part. For the example in Table 3.3, the values were 48, 23, 18, 6 and 3.85. (The 3.85 was obtained by multiplying 5.0 by (5.77- 5.0) = 3.85).

d. Determination of total deduct value by summing individual deduct values.

e. Determination of q as the number of deducts with a value greater than 2.0.

f. Copied DVs on current line to the next line and changed the smallest DV greater than two to two and finally repeated 4,5,6 until q =1

g. Determination of the CDV from total deduct value and q and looked up the appropriate correction curve for AC pavements in Figure 3.3.

h. Calculated PCI by subtracting the maximum CDV from 100

PCI = 100 - max CDV …………… (ASTM D 6433-99, 1999)

$m = 1 + (9/98)(100 - \text{Max DV}) \leq 10$

$m = 1 + (9/98)(100-48) = 5.77$

$0.77*5 = 3.85$

Table 3-3: Calculation of corrected PCI value (ASTM D 6433-99, 1999)

#	**Deduct value**									**Total**	**q**	**CDV**
1	48	23	18	6	3.85					**98.85**	**5**	**52**
2	48	23	18	6	2					**97**	**4**	**56**
3	48	23	18	2	2					**93**	**3**	**60**
4	48	23	2	2	2					**77**	**2**	**56**
5	48	2	2	2	2					**56**	**1**	**56**

Max CDV = 60

PCI = 100- Max CDV = 40

Rating = Poor

3.5.2 Axle load survey to determine the axle load distribution of the heavy vehicles

Axle loads and gross vehicle weights were measured by a static method. In a static method, vehicles were stopped to measure their axle loads at the weighbridge which was located at Km 102+000 at Lukaya town.

a) Data Analysis of Axle Load data

A standard computer spreadsheet program was used very effectively to analyse axle load data to make sure errors were eliminated, especially with data inputs and calculation formulae. The general steps used in the analysis were as follows.

b) Calculating the EF/vehicle (for each vehicle class)

i. The method of analysis was based on the use of a simple spreadsheet program using automatic calculations. For manual calculations, the same general method was used although, when converting from axle loads to EF per axle.

$$EF = \left[\frac{\text{axle load (in tonnes)}}{8.16}\right]^{4.5} \quad \text{………………… Equation (3.3)}$$

ii. It was important to note that the relationship between axle load and damage was a power relationship. This means that doubling the axle load would not simply double the damaging effect but would increase it by over 22 times. For example, whilst a standard axle load of 8.16 tonnes would have an EF = 1, an axle load of 16.32 tonnes would have an EF = 22.6.

iii. Calculating the EF for each vehicle

A column was needed to calculate the EF per vehicle by summing the EFs for all the axles of each vehicle. It should be noted that each axle of a multiple axle set should be treated as a single separated axle.

iv. Calculated the average EF per vehicle for each vehicle type

Keeping the directions separate, the average EF per vehicle for each vehicle type was calculated. This must include all vehicles in the category, whether loaded or empty.

The definition of vehicle classes may vary between countries. For example, in many countries there are several classes for vehicles with five axles or more, usually depending on the precise configuration of axles, whilst in other countries these categories may be combined.

v. When inputting the axle load data, it can sometimes be useful to input some of the other information from the survey such as load type, even though it may not be required for the calculation of EFs. This information can be useful during additional analysis, for example, to determine the types of goods carried by vehicles that are overloaded.

vi. Determination of the cumulative equivalent standard axles over the design life of the road, the following procedures were followed:

- *Average daily traffic flow for each vehicle class:*

  From the results of a classified traffic count (as well as any other recent traffic count information that is available), the average daily traffic flow for each class of vehicle in each direction is calculated.

- *Average EF per vehicle (for each vehicle class):*

  From the axle load survey, the average equivalence factor (EF) per vehicle for each class of vehicle in each direction was calculated.

- *Average ESA per day (for each vehicle type):*

Again, keeping the two directions separated, for each vehicle category, the average ESA/day was calculated by multiplying the average EF per vehicle by the average traffic flow for that category.

- *Daily traffic loading (one way):*

  The sum of the ESA/day for all vehicle categories gives the total daily traffic loading (in ESA/day) for each direction.

- *Annual traffic loading (one way) if required:*

  Some designs require annual traffic loading. By multiplying the total daily traffic loading (ESA/ day) values by 365 the annual traffic loading for each direction is obtained. This figure is normally presented as millions of equivalent standard axles per year (MESA/year) for each direction. The larger of the two directional values should be used for pavement design purposes.

- *Design life and traffic growth:*

  ✓ The required design life of a pavement or rehabilitation treatment in terms of years is usually clearly specified in project documentation. Most design manuals cater for varying traffic levels in terms of millions of ESA over the design period.

  ✓ The design life will usually start at the anticipated opening year for the pavement, which might not be for several years due to the processes of gaining approval, funding and actually building the pavement. For example, if a road is not expected to open for a further three years, the design loading will be the sum of traffic loading from year 4 to year 18 inclusive. If this is not done, and the design life is calculated from current traffic levels, the error in ESA over the design life is likely to be sufficiently large to affect the design.

- ✓ An estimate must be made of future traffic growth over the chosen design life, which should consider normal traffic, diverted traffic and generated traffic. Future traffic growth is usually expressed as a fixed percentage rate (for example 5% per year) (MoWT, 2005).

- *Total traffic loading over design life:*

  Using data from the lane with the highest traffic loading the total traffic loading over the design life is calculated. The following equation 3.4 can assist in the calculations:

Total cumulative ESA (one way) = $\left(\frac{a \text{ X } 365 \text{ x } 100}{b}\right) \text{x}\left(\left(1+\frac{b}{100}\right)^{c+d} - \left(1+\frac{b}{100}\right)^{d}\right)\ldots$

(Equation 3.5)

Where: a = current average annual daily traffic loading in ESA per day (one way), b = annual growth rate (%), c = design life (years) and d = number of years to start of design life

The results are usually expressed in units of millions of equivalent standard axles, one way.

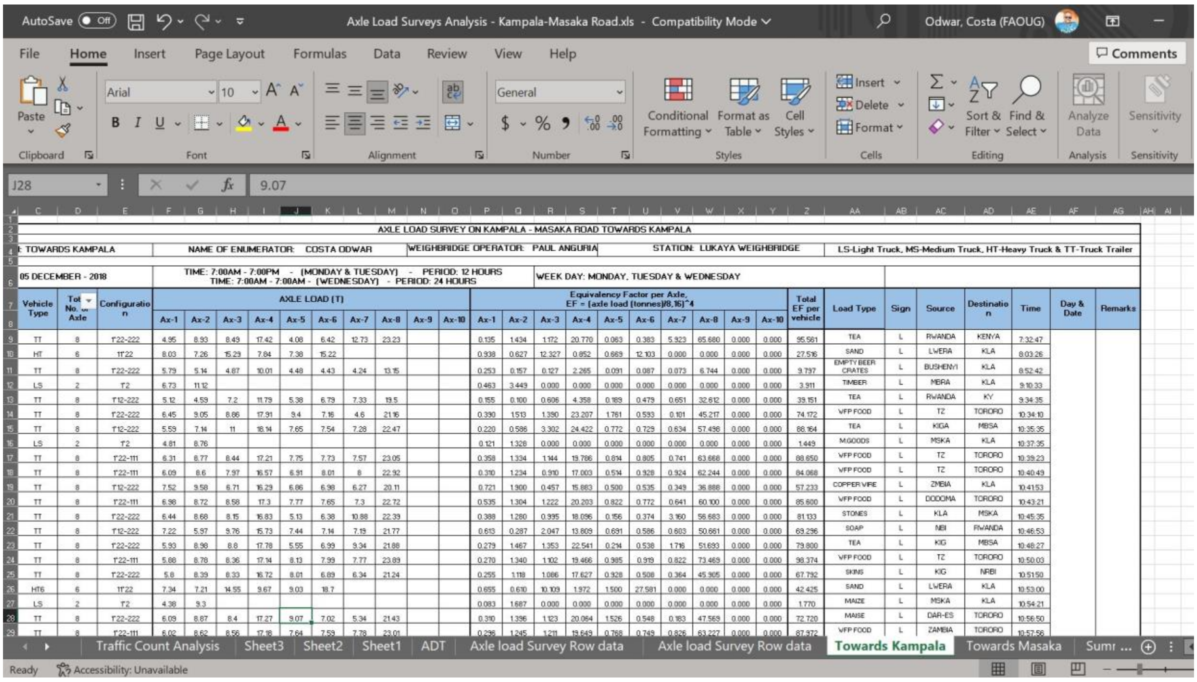


Figure 3-2: Data analysis example - Completed analysis table from the research study

Table 3-4: Traffic Classes (MoWT, Pavement design manual, 2005)

Traffic Class Designation								
	T1	**T2**	**T3**	**T4**	**T5**	**T6**	**T7**	**T8**
Traffic ranges) (Million ESAs	< 0.3	0.3-07	0.7-1.5	1.5-3	3-6	6-10	10-17	17-30

If calculated design values are very close to the boundaries of a traffic class, the values used in the forecasts should be reviewed and sensitivity analyses carried out to determine which category is most appropriate.

The lowest traffic class T1, for design traffic of less than 0.3 million ESAs, is regarded as a practical minimum since realistic layer thicknesses as well as materials specifications tend to preclude lighter structures for lesser traffic. The current level of knowledge on pavement behaviour, in any case, limits the scope for rational design of such lighter structures.

However, in the unlikely case that design traffic is estimated at less than 0.1 million ESAs (that is, traffic significantly less than the lowest class T1), since this guide is aimed primarily at the Regional Trunk Road Network, the Engineer is recommended to also consider alternative designs proven locally for this very light trafficking (MoWT, Pavement Design Manual, 2005).

3.5.3 Evaluation of the structural condition with deflection bowl parameters

Pavement structural strength evaluation is undertaken to determine the adequacy of the existing pavement to support traffic without developing appreciable structural distress. The intention of the structural evaluation is to determine both the current adequacy of the existing pavement and to predict its future service life with respect to the forecast traffic loading. On the basis of the evaluation results, appropriate remedial measures have been designed to provide the intended level of service.

a) Deflection Bowl Parameters

This measurement of the deflection bowl by means of the FWD led to the definition of various deflection bowl parameters that describe various aspects of the measured deflection bowl. In Table 3.5, a selected number of deflection bowl parameters and their formulae are summarised as linked to the deflection bowl zones and their formulae based on the measured deflection bowls. The radius of curvature (RoC) and the base layer index (BLI) have been found to correlate well with zone 1 (mostly surface and base layers), the middle layer index (MLI) correlates with zone 2 (mostly sub-base layer), and the lower layer index (LLI) correlates with zone 3 (mostly selected and subgrade layers) as shown in Table 3.5 and 3.6, Owing to the closeness (200mm) of the geophone to the edge of the loading plate and the associated surface disturbances observed, the RoC is used with less confidence and BLI is used with more confidence to describe zone 1.

Table 3-5: Behaviour states for granular base pavements (SAPEM, TRH-10, 2014)

Behaviour State	**Traffic Range (MESA)**	**Maximum Deflection (mm)**	**BLI (mm)**	**MLI (mm)**	**LLI (mm)**
Very stiff	12 – 50	< 0.3	< 0.08	< 0.05	< 0.04
Stiff	3 – 8	0.3 – 0.5	0.08 – 0.25	0.05 – 0.15	0.04 – 0.08
Flexible	0.8 – 3	0.5 – 0.75	0.25 – 0.5	0.15 – 0.2	0.08 – 0.1
Very flexible	< 0.8	> 0.75	> 0.5	> 0.2	> 0.1

Table 3-6: Deflection bowl parameter structural condition rating criteria (SAPEM. TRH-10, 2014)

Pavement Base Type	**Structural Condition Rating**	**Deflection Bowl Parameters**				
		Y_{max}	**RoC**	**BLI (mm)**	**MLI (mm)**	**LLI (mm)**
Granular base	Sound	< 500	> 100	< 200	< 100	< 50
	Warning	500 – 750	50 – 100	200 – 400	100 – 200	50 – 100
	Severe	> 750	< 50	> 400	> 200	> 100
Cementitious base	Sound	< 200	> 150	< 100	< 50	< 40
	Warning	200 – 400	80 – 150	100 – 300	50 – 100	40 – 80
	Severe	> 400	< 80	> 300	> 100	> 80
Bituminous base	Sound	< 400	> 250	< 200	< 100	< 50
	Warning	400 – 600	100 – 250	200 – 400	100 – 150	50 – 80
	Severe	> 600	< 100	> 400	> 150	> 80

These variabilities have also been observed in other methods of analysis that tended to rely on the deflection value at 200 mm, such as the Australian method where a curvature ratio is calculated based on that value (Horak, 2008).

3.5.4 Coring and laboratory test on the extracted cores

Coring was done to ascertain base layer thickness, depth of cracks and to obtain samples for laboratory testing at the various locations on the road section. Cores extracted from the road of which seven cores were tested for material testing in the laboratory as obtained from the field.

a) Laboratory test on core samples are as follows

i. Extraction of Cores

Seven cores of 150 mm diameters were extracted from locations pre-determined through visual survey of the road, within two 150 m sections with one representing a typical “distressed area” and the other representing a relatively “good area”. The extraction of cores was done in accordance with ASTM D 3549 of thickness and density of pavement cores.

ii. Marshall (Flow and Stability) tests

Marshall Stability and flow of asphalt mixtures, along with field mixture density, VA, VMA and/or VFA, were used for bituminous mix design in laboratory and in-situ control. In addition, the Marshall parameters are very useful for controlling the plant production process of asphalt mixtures (ASTM D6927-15, 2015). The Marshall Test was performed on cylindrical samples of 4 in (102 mm) in diameter and thickness of 2.5 ± 0.10 in (63.5 ± 2.5 mm). The specimens were exposed to a water bath with a temperature of 60 ± 1°C. The load was applied by means of a gage-equipped hydraulic jack at a rate of 2.00 ± 0.15 in/min (5.08 ± 0.38 cm/min) until break point is reached or the load begins to decrease. Commonly, the maximum load reached is defined as stability, while the vertical deformation at maximum load was recorded as flow.

Stability of the mixture reflects the internal friction and cohesion whereby cohesion is a measure of the bitumen binding strength, and internal friction a benchmark of the interlocking and friction resistance of aggregates. On the other hand, flow is a measure of the sample deformation. High flow values generally indicate a plastic mixture that will undergo permanent deformation under traffic, whereas low values may indicate a mixture with larger than normal voids and insufficient asphalt to ensure durability and premature cracking could be experienced due to the fragility of the mixture (Veropalumbo, 2019).

iii. Aggregate gradation

The ignition method was used to obtain the aggregates from the AC cores, this procedure removes the surface areas (containing cut aggregate) from each core. The material is then combined, split, asphalt cement is removed in the ignition oven, and finally gradation is determined. The aggregate grading curves, constructed according to specifications (MoWT, 2005) are depicted in Figure 3.3. The sieves' opening range related with sieve percentage passing is indicated, as well as the working formula and mix design grading, considering a 1/2 in (12.5 mm) maximum aggregate size. The working formula is obtained from the design curve plus/minus the tolerance values that depend on sieve size.

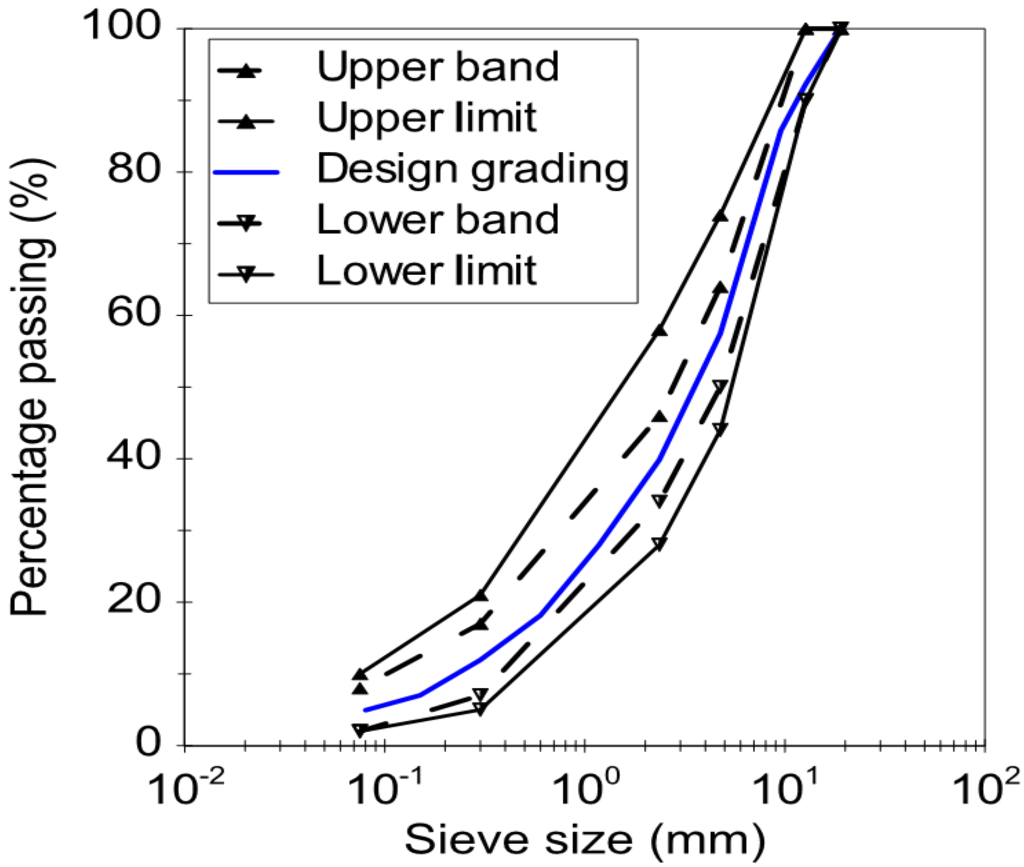


Figure 3-3: Aggregate grading curves and mix design band

iv. Indirect Tensile Stress Test

The effect of traffic load in a long period could affect the strength of an asphalt mixture showing fatigue cracks and/or rutting. To determine this damage is commonly used the Indirect Tensile Strength (ITS) (ASTM D6931-12, 2012). To evaluate the ITS, the cylindrical specimen must be placed in the compression testing machine between the loading strips and be loaded diametrically along the direction of the cylinder axis with a constant speed of displacement until it breaks (ASTM D6931-12, 2012). The indirect tensile strength is the maximum tensile stress calculated from the peak load applied at break and the dimensions of the specimen according to the following equation (Veropalumbo, 2019):

$$ITS = \left[\frac{2P}{\pi DH}\right] \text{ ……………………… Equation (3.5)}$$

where *ITS* is the indirect tensile strength, expressed in Giga-Pascal's (GPa); *P* is the peak load, expressed in Kilo Newton's (kN); *D* is the diameter of the specimen, expressed in millimeters (mm); and *H* is the height of the specimen, expressed in millimeters (mm). Besides the strength of mixture is also compromised by water effect thus to evaluate its behaviour and durability the ITS value was determined before and after a soaked in a water bath for 72h at 40°C. The relation of the strength values before and after water storage is called Indirect Tensile Strength Ratio (AASHTO T283, 2004-2005). Therefore, the following equation has been adopted (Veropalumbo, 2019):

$$ITSR = \frac{ITS\ (wet)}{ITS\ (dry)} \cdot 100 \text{ ……………………… Equation (3.6)}$$

Where $ITSR$ is the indirect tensile strength ratio (%); ITS is the average indirect tensile strength of the wet group (KPa); and ITS is the average indirect tensile strength of the dry group (KPa) as per table 3.6 in the series 4000 of the general specifications for the MoWT.

v. Bulky Density

Density is one of the most important parameters in construction of asphalt mixtures. A mixture that is properly designed and compacted will contain the optimum amount of air voids (Safwat, 2016).

The amount of voids in the asphalt mixture is probably the factor that most affects performance throughout the life of an asphalt pavement. Voids are primarily controlled by asphalt content, compactive effort during construction, and additional compaction under traffic.

High voids lead to permeability of water and air, resulting in water damage, oxidation, raveling, and cracking. Low voids lead to rutting and shoving of the asphalt mixture.

The primary method that has been used include measurement of bulk density of cores taken from the in-place pavement.

vi. Binder Content Determination

Selection of the appropriate binder is a very important aspect in the asphalt mix design process. Usually, binder grades are affected by temperature and traffic loading expected in service, with stiffer binder grades selected when heavily loaded or slow-moving vehicles are expected. However, utilization of stiffer binders often results in stiff mixes that have workability related issues since they are not easy to place and compact to the desired density (MoWT, 2005).

Table 3-7: Design requirements for asphalt concrete surfacing (MoWT, 2005)

Property of mixture and laboratory test method			Asphalt Concrete, continuousl graded, (AC20, AC14, AC10)
Marshall flow (mm)			2 – 4
Marshall stability (Newton), all severely loaded areas*)			Minimum 9000
Marshall stability (Newton)	Traffic Loads	> 10 x 10^6 esa's	8000 – 18000
		1 - 10 x 10^6 esa's	7000 – 15000
		< 1 x 10^6 esa's	6000 – 10000
Air voids (%)			3 – 5
Voids in mineral Aggregate (%)	Asphalt Mix	AC 20	min. 14
		AC 14	min. 15
		AC 10	min. 16
Voids Filled with Bitumen (%)	Traffic Loads	> 10 x 10^6 esa's	65 - 75
		1 - 10 x 10^6 esa's	65 - 78
		< 1 x 10^6 esa's	70 -80
Requirement after refusal laboratory compaction BS 594 - Part 598 (severely loaded areas only) *)			Air voids shall be minimum 3%
Indirect tensile strength (KPa) AASHTO T 283			Minimum 800 Tested at 25 ^oC
Indirect wet tensile strength (KPa) AASHTO T 283			80 % of dry strength
*) The appropriate Traffic Load Class, and whether requirements for severely loaded areas apply to any location, shall be as given in the drawings or Special Specifications. Where such information is not given, the decision of the Engineer shall apply.			

3.6 An appropriate strategy to mitigate the cracking on a section Road

Suggested methods of maintenance for the different types of pavement deterioration for roads having thin bituminous seals and asphalt surfacing are given in Table 3.8.

Table 3-8: Existing Road Surface-Asphalt surfacing (Overseas Road Note 18, 1999)

Primary failure	Remedial treatment	New surfacing	Comments
Surface defects Fretting or stripping	Local patching	Surface dressing or slurry seal	
Fatting-up		Surface dressing	Where texture depth has decreased to an unacceptable level.
Bleeding	Remove surfacing	Asphalt surfacing	Asphalt surfacing that are bleeding will rapidly deform and may need to be removed.
	Apply heated fine aggregate		Where failures are localized.
Loss of texture	Surface dressing		
Polished aggregate		Surface dressing or slurry seal	Use aggregate having suitable Polished Stone Value for the expected traffic (See Note 2).
Rutting without shoving Secondary compaction		Thin overlay	
Excessive traffic loading or inadequate pavement thickness	Reflection crack treatment if necessary	Regulating layer followed by strengthening overlay	
Rutting with shoving Inappropriate surfacing material	Remove surfacing that has failed	Replace with new asphalt surfacing material	(See Note 1)

Primary failure	Remedial treatment	New surfacing	Comments
Surfacing out of specification	Remove surfacing that has failed	Replace with new asphalt surfacing material	
Inadequate road base Too thin	Remove surfacing and increase thickness of road base with a granular overlay.	Asphalt surfacing	
Too weak	Remove surfacing. Replace or modify existing road base.	Asphalt surfacing	Existing road base may be suitable for mechanical stabilization or modification with lime or cement.
Wheel path cracking Isolated slippage	Remove affected surfacing and patch		
Extensive slippage	Remove surfacing and replace	Asphalt surfacing	
Cracks confined to the top of the surfacing		Double surface dressing	
Poor bond	Remove affected surfacing and patch	Where the failures are extensive the surfacing will need to be removed and the road resurfaced with asphalt	
Poor surfacing material	Remove areas of cracking of intensity 3 or greater and patch. Chase out cracks more than 3mm wide and seal with proprietary crack sealant.	Double surface dressing or asphalt surfacing	Where the failures are Extensive the surfacing will need to be removed and the road resurfaced with asphalt.

Primary failure	Remedial treatment	New surfacing	Comments
Fatigue cracking	Remove areas of cracking of intensity 3 or greater and patch. Chase out cracks more than 3mm wide and seal with proprietary crack sealant.	Double surface dressing or asphalt surfacing (See Note 2)	Where the failures are Extensive check whether the road needs strengthening.
Reflection cracking	Remove areas of cracking of intensity 3 or greater and patch. Chase out cracks more than 3mm wide and seal with proprietary crack sealant.	Double surface dressing or asphalt surfacing (See Note 2)	If a crack relief interlayer is to be used under an asphalt surfacing then areas of crack intensity 4 or greater should be removed and patched
Non-wheel path cracking **Longitudinal cracks** i) At construction joints and road markings	Chase out cracks and seal with proprietary crack sealant.		
ii) Subgrade movement	Immediately chase out and seal all cracks with proprietary crack sealant to prevent the ingress 0f water.	Pavement should be sealed with a double surface dressing after the crack development has stabilized (See Note 3).	
iii)Reflection cracks	Chase out cracks more than 3mm wide and seal with proprietary crack sealant.	Double surface dressing if reflection cracking has an extent greater than 1 (See Note 2).	

Primary failure	Remedial treatment	New surfacing	Comments
Transverse cracks i) At construction joints and structures	Chase out cracks and seal with proprietary crack sealant.		
ii) Thermal or shrinkage cracks	Chase out cracks more than 3mm wide and seal with proprietary crack sealant.	Double surface dressing (See Note 2)	
iii)Reflection cracks	Chase out cracks more than 3mm wide and seal with proprietary crack sealant.	Double surface dressing if reflection cracking has an extent greater than 1 (See Note 2).	
Block cracking i) Thermal or shrinkage cracks	Chase out cracks more than 3mm wide and seal with proprietary crack sealant.	Double surface dressing (See Note 2)	If block cracking is severe then the surfacing will need to be removed and replaced.
ii) Reflection cracks	Chase out cracks more than 3mm wide and seal with proprietary crack sealant.	Double surface dressing if reflection cracking has an extent greater than 1 (See Note 2)	If block cracking is severe then the surfacing will need to be removed and replaced.
Crocodile cracking	Remove surfacing	Asphalt surfacing	

i Road Note 31 includes a mix design procedure for bituminous surfacing suitable for severe loading conditions. Many authorities also use bitumen modifiers for asphalt surfacing subject to severe loading.

ii Some organizations have shown that the inclusion of fabrics improves the performance of surface dressings. However, it is recommended that initially these techniques be introduced on a pilot scale basis to ensure contractors are trained in the techniques.

CHAPTER FOUR

DATA PRESENTATION, ANALYSIS AND DISCUSSION OF RESULTS

4.1 Introduction

The results obtained from field and laboratory investigations conducted, their discussion and analysis in line with acceptable standards, design reports, investigation reports, journals and publications have all been presented and discussed in this chapter:

4.2 Assessment of the extent of cracking currently being experienced on a section of Kampala-Masaka Road in order to analyse its functional performance requirement

Assessing the extent of cracking being experienced on the study road in order to analyze the functional performance involved: pavement condition survey and analysis, and traffic analysis (Traffic counts and Axle load measurements). The results obtained are discussed in subsequent sections.

4.2.1 Pavement Condition Survey and Analysis

a) **Visual Assessment and Quantification**

The visual assessment conducted identified various distress types including alligator and longitudinal cracking, deformation, surface defects, potholes and patching already provided as well as their severity. All the above distresses were presented in percentages of total area and the results are as shown in Figure 4.1.

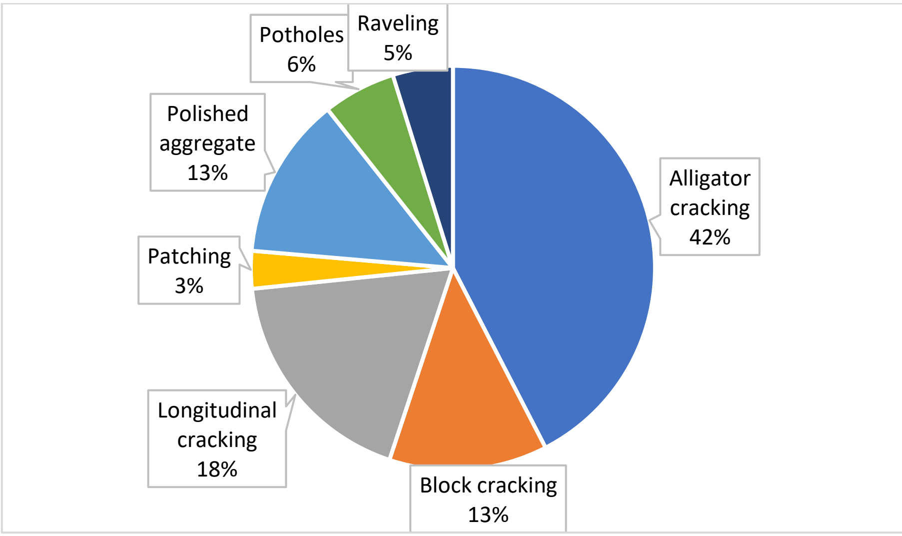


Figure 4-1: Distress percentage for the section of Kampala-Masaka Road from Km 97+000 to Km 102+000.

From Figure 4.1, it is observed that alligator cracking and patching were the most and least dominant defects at 42.0% and 3.0% respectively. The severe crocodile cracking and spalling were observed on both lanes starting from Km 97+700 and ending at Km 100+500. The severe crocodile cracking mostly on the wheel paths were from Km 99+000 to Km 99+500. Some patches had cracks mostly at Km 99+200 up to Km 100+100; and finally, a section with no cracks was from Km 101+000 to Km 102+000. This is probably because the study section of the road was in a swampy area. Swampy areas have high groundwater levels which significantly impact on the engineering properties of subgrade soil, as a result of repeated fluctuations of groundwater with the variation of atmospheric environment. In that situation, groundwater that migrates into the subgrade under capillary action not only causes long-term strength attenuation of subgrade soil but also produces large plastic deformation in early stage of the subgrade (Cary and Zapata, 2011). This plastic deformation leads to fatigue cracking.

b) Road Condition Rating

Rating was done to establish the details and extent of each distress identified on the road. The results obtained are presented in Table 4.1.

Table 4-1: Condition rating of the detailed visual condition survey on the section of Kampala – Masaka road (ASTM D6433-11, 2011)

Section of the Road (Km)	Deformation or Rutting		Visible Cracking		Potholes and Potholes Patching		Overall Rating
	Depth (mm)	Condition Rating	Intensity (m/m^2)	Condition Rating	Area (m^2 as %)	Condition Rating	
97+000 – 97+500	<10	Good	<1	Fair	<1	Good	Fair
97+500 – 98+000	<10	Good	1-2	Poor	<1	Good	Poor
98+500 – 99+000	<10	Good	1-2	Poor	<1	Good	Poor
99+000 – 99+500	<10	Good	1-2	Poor	1-15	Fair	Poor
99+500 – 100+000	<10	Good	<2	Poor	<1	Good	Poor
100+000 – 100+500	<10	Good	1-2	Poor	<1	Good	Poor
100+500 – 101+000	<10	Good	<1	Fair	<1	Good	Fair
101+000 – 101+500	<10	Good	<1	Fair	<1	Good	Fair
101+500 – 102+000	<10	Good	<1	Fair	<1	Good	Fair

From Table 4.1, depth of all the ruts measured were less than 25mm indicating that the section of the study road was in a good condition with respect to rutting. Pavement rutting is classified into three categories based on severity (magnitude of depression), that is, Low (13 to 25 mm), Medium (25 to 50 mm) High (>50mm) (Kannemeyer, 2003).

It was also observed as shown in Table 4.1, that the section from Km 97+500 to Km 100+500 was in poor state with respect to cracking since the intensity of the cracks was more than 1 - 2m/m^2 with an area of moderately or severely spalled cracks forming the characteristic alligator pattern (Zafar, 2019).

Table 4.1 shows that the area covered by potholes was very low with the depth and surface area being less than 25mm and less than 0.1m^2 respectively with potholes observed from Km 99+000 to Km 99+500 as the road being still very motorable and considered to be in a good state with respect to potholes.

c) Pavement Condition Index (PCI)

Pavement Condition Index (PCI) of the pavement section was calculated from the data collected during the Pavement Condition Survey conducted. PCI of the individual sample units was calculated on Corrected Deduct Value (CDV) calculation sheets as shown in Table 4.2 and detailed in Appendix B. The combined PCI for the entire selected section was calculated by determining the average of the random sample unit PCIs and the effect of additional sample unit PCIs was taken into account by the way of weighted average. Table 4.2 shows that the qualitative Rating achieved was 46.8% of the subject pavement section which falls between the limit of Poor (41-55) (ASTM D6433-11, 2011). Therefore, the condition of the selected pavement section may be qualitatively rated as poor probably as the result of the observed distress on the section of the road.

Table 4-2: PCI results for different sections along the study road

Section of the Road (Km)	PCI (%)
Km 97+000 – Km 97+500	40
Km 97+500 – Km 98+000	42
Km 98+000 – Km 98+500	48
Km 98+500 – Km 99+000	50
Km 99+000 – Km 99+500	50
Km 99+500 – Km 100+000	56
Km 100+000 – Km 100+500	40
Km 100+500 – Km 101+000	50
Km 101+000 – Km 101+500	42
Km 101+500 – Km 102+000	50
Average PCI	**46.8%**

4.2.2 Pavement Design

a) Traffic assessment

To assess whether the visible cracks were as a result of change in the traffic loading from the original designed traffic loading and comparing traffic data obtained for the current counts. Therefore, those carried out at design stage for light single truck, medium single truck and heavy or semi-trailers and trailers corresponding to traffic loading in terms of ESA was established.

From the study and analysis of traffic data for the section of Kampala – Masaka road, the total traffic (AADT) on the road was 6226 representing an increase from the design AADT, as the traffic was dominated by light goods vehicles and small bus categories. Therefore, the significant quantity of trucks ranging from light goods to track trailers and semi-trailers total to 1620 as shown in appendices C, D, E and F composition of traffic was 398 heavy vehicles, 903 Medium vehicles and 2319 light vehicles giving average percentages as: heavy vehicle 24.57%, medium vehicle 55.74 %and light vehicle 19.69%.

b) Axle load Surveys

Axle loading for each vehicle category was established through axle load measurements. Weights of each axle were taken (See Appendix H), the equivalent axle load factors were determined and ultimately the Vehicle Damage Factors (VDF) computed. Table 4.3 shows the VDF computed for the heavy goods category vehicles is 54.1. It should be noted that VEF values used at the design stage could not be found hence no comparison was done between the design and measured values.

Table 4-3: Calculated Vehicle Damage Factor

Vehicle type	Average EF per vehicle
Light Single Unit Truck/Large Bus/Medium Single Unit Truck	3.145
Medium- Large Single Unit Trucks – 3 and 4 axles	22.447
Heavy Trucks & Trailer or Heavy Truck & Semi Trailer (More than 5 axles)	54.10

In order to compute traffic loading, Vehicle Equivalency Factors (VEF) were calculated from the axle loads using an equivalent single axle load of 80 KN as shown in appendix H. As stated earlier, VEF values used at the design stage could not be found hence no comparison was done between the design and measured values. Table 4.4 shows the estimated traffic loading of approximately 51.0 MESA was determined detailed on appendix H. This was not significantly different from the actual design value of 44.1 MESA (Mugume, 2020). Therefore, indicating that the designed pavement structure should be strong enough to carry the current traffic loading.

Table 4-4: Calculated Cumulative ESA over design life

Vehicle type	Average EF per vehicle
Light Single Unit Truck/Large Bus/Medium Single Unit Truck	3.0 MESA
Medium- Large Single Unit Trucks – 3 and 4 axles	17.1 MESA
Heavy Trucks & Trailer or Heavy Truck & Semi Trailer (More than 5 axles)	33.9 MESA
Total Cumulative ESA	54.0 MESA

4.3. Evaluation of strength of the underlying layers of the pavement.

4.3.1 Pavement Deflection Measurements

Pavement deflection measurements were carried out using the FWD to determine the flexural rigidity of pavement layers (Asphalt Concrete, Base and Subbase) and sub-grades so as to determine the residual strength of the existing pavement. It was also intended to evaluate the load carrying capacity which is an indicator that quantitatively expresses the soundness of pavements. The load bearing capacity indicates the capacity to support traffic loads. If the load bearing capacity declines, pavements become unable to support the traffic load and deflect under the load. The results of the FWD deflection bowl are shown in Figures 4.2, 4.3 and 4.4 for the Subgrade layer, the Base layer and the Asphalt Concrete layer respectively. The Lower Layer Index (LLI) benchmarking results shown in Figure 4.2 show that the green colour code represents the subgrade which is still in sound structural condition indicating that the deflection bowl parameters for those sections are greater than 50mm. The amber colours at Km 97+00, Km 98+000, Km 99+000 and Km 100+000 shows that the subgrade is in warning structural condition indicating that the deflection bowl parameters are ranging from 50mm to 80mm.

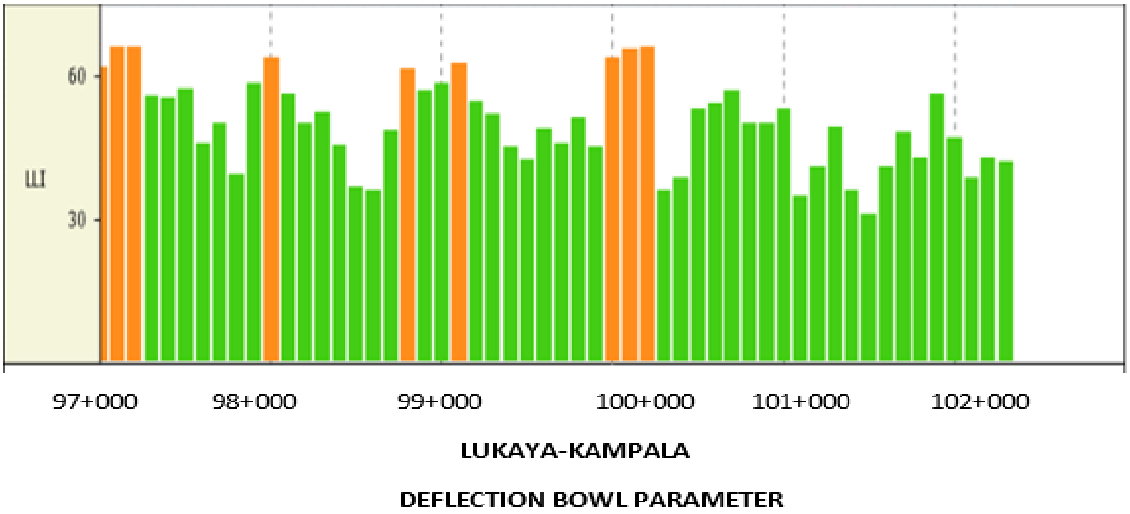


Figure 4-2: Lower Layer (Subgrade) Index benchmarking for Km 97+000 to Km 102+000

The Middle Layer Index (MLI) values shown in Figure 4.3 indicates that the green colour code at Km 101+500 shows a sound structural condition of the base layer representing that the deflection bowl parameter is greater than 100mm. The remaining section of the base layer showing amber colour code represents the structural condition and is warning indicating that the deflection bowl parameters are between 100mm to 150mm.

In locations where the subgrade structural condition represented as LLI shows some warning and the MLI shows lack of structural support from the subgrade.

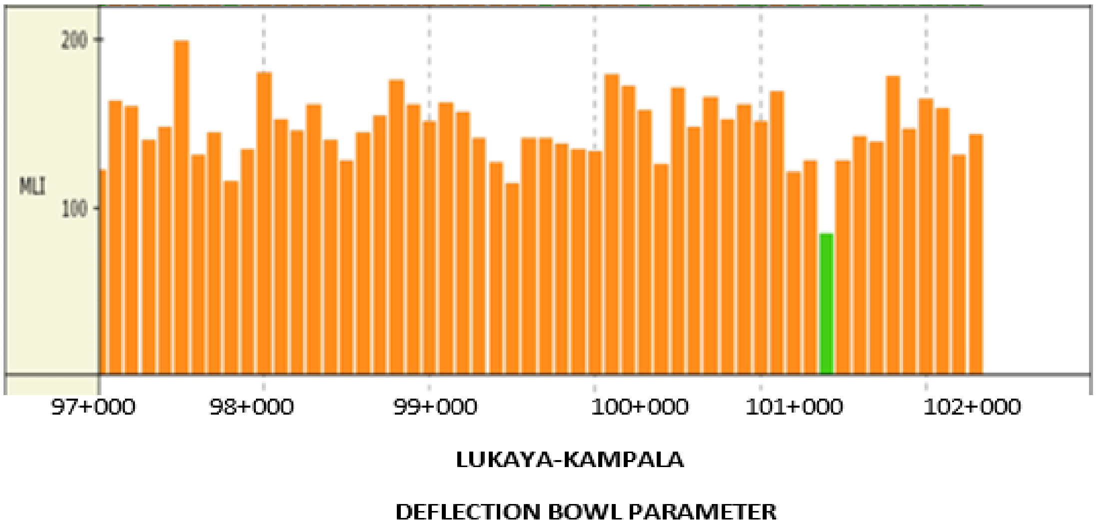


Figure 4-3: Middle Layer (Base Layer) Index benchmarking for Km 97+000 to Km 102+000

The Base/Top Layer Index (BLI) benchmarking shown in Figure 4.4 shows that Km 101+000 to Km 102+000 represented by green colour indicates that the structural condition is sound with the deflection bowl parameters being greater than 200mm. The remaining section of the study road is represented by amber colour indicating that the structural condition is warning with the deflection bowl parameters ranging from 200mm to 400mm.

These indicate that the base layer is increasingly drifting into the warning condition over large sections of pass. The surveyed visual condition rating over this section was mostly warning owing to cracks permitting the ingress of large quantities of water which soak the base thereby compromising its structural integrity. The distress thus observed therefore possibly originate largely from the asphalt and the base layer and, to a lesser extent, from the sub-base/subgrade.

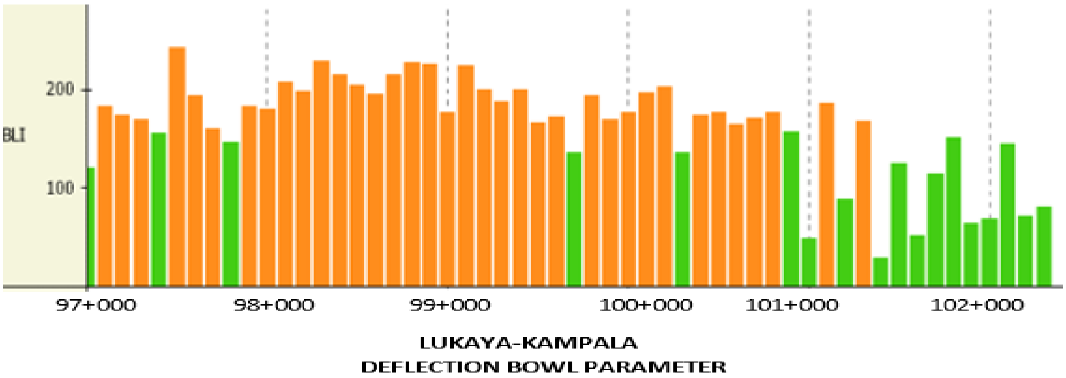


Figure 4-4: Base Layer Index benchmarking for Km 97+000 to Km 102+000

4.3.2 Investigation of Material Properties

Material tests were done to establish whether the materials used in the pavement construction met the design and specification requirements so as to rule out the possibility of cracking being related to material properties non compliances. The results obtained are presented in the subsequent sections.

a) Layer thicknesses and Depth of Cracking

Coring was done at the various locations on the road section to ascertain the thickness of the Asphalt Concrete and base layer, to establish depth of cracks and obtain samples for laboratory testing to check conformity with the general specifications for roads and bridges (MoWT, 2005). Table 4.5 provides the summary of results for the cores thicknesses taken from the road and detailed in Appendix K.

Table 4-5: Established Layer thicknesses and crack depths (mm)

Core location (Km)	97+300 (LHS)	97+700 (RHS)	98+210 (LHS)	98+790 (RHS)	99+330 (LHS)	100+190 (RHS)	101+440 (LHS)	Average
Crack depth (mm)	19	14	26	25	31	17	29	23

From the table above, the established average thickness for the AC and base were 50 mm and 300 mm respectively. The average crack depth was 23 mm which is less than the AC layer thickness. This implies that the cracks observed on the surface are top – bottom cracks and are within the AC layer and therefore do not reach the base layer.

b) Laboratory Test Results

Material samples collected from the Asphaltic surfacing layer and pavement layers were tested and analysed for the different fundamental parameters of this study as the results obtained for each test are presented and discussed in the following sections.

i. Coarse Aggregate Gradation

One of the most relevant factors in performance-based design of asphalt mixtures is the gradation of the aggregates. The role of aggregates in the performance and durability of an asphalt mixture is critical (Lambebo, 2020). Knowledge of particle size distribution is necessary to determine the grading of materials proposed for use in road pavement. Therefore, its result gives the percentages of different aggregate fractions present in the aggregate sample. Accordingly, the particle size distribution of the aggregate sample as compared with national specifications (MoWT, 2005) are shown in Figure 4.5.

As it can be seen from Figure 4.5, the coarse aggregate meets the requirements for the particle size distribution for use as wearing course material in roads. The shape of aggregates were observed to be angular, therefore the cracks observed on the surface are not as a result of the particle size distribution, hence aggregates were not part of the cause of cracking.

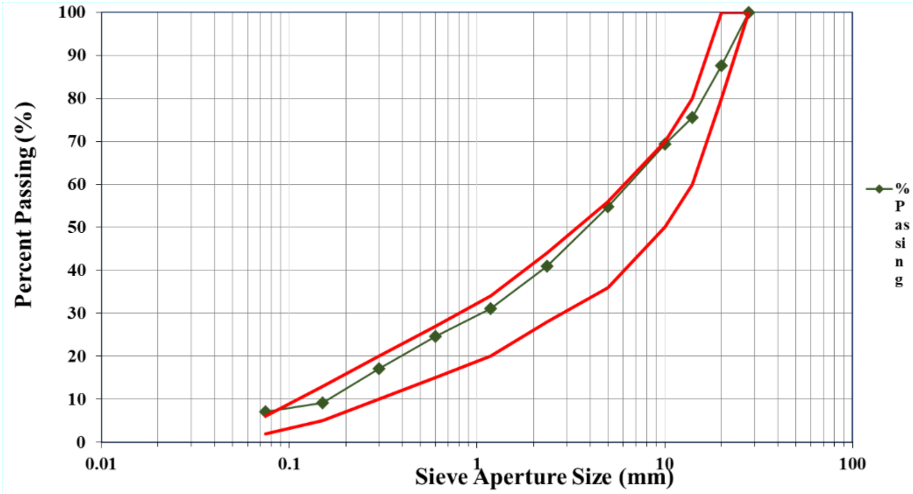


Figure 4-5: Gradation curve for the aggregate from the extracted cores

ii. Binder Content

The results for binder content extracted from the cores are summarized in Table 4.6 and detailed in appendix L. From the table, the highest binder content was 5.05% while the lowest binder content was 4.91 %.

Table 4-6: Binder content results

Sample	1	2	3	Average
Binder Content (%)	4.91%	5.05%	5.04%	5.0%

On average, the extracted bitumen content was 5.0% which falls within the tolerable limit of ± 0.3% on the optimum binder content (OBC) as per specification (MoWT, 2005). However, the research notes that the average bitumen content of 5.0% was on the higher side of the AC 20 wearing course. The amount of binder to be added to a bituminous mixture cannot be too excessive or too little. The principle of designing the optimum amount of binder content is to include sufficient amount of binder so that the aggregates are fully coated with bitumen and the voids within the bituminous material are sealed up. As such, the durability of the bituminous pavement can be enhanced by the impermeability achieved. However, the binder content cannot be too high because it would result in the instability of the bituminous pavement. In essence, the resistance to deformation of bituminous pavement under traffic load is reduced by the inclusion of excessive binder content.

iii. Indirect Tensile Strength (ITS)

The IST results obtained are as summarized in the Table 4.7 and detailed in appendix M. From the results, the minimum ITS (dry) obtained is 1809.3 KN/m3 while the minimum ITS (wet) obtained is 1242.4 kN/m3. The minimum values obtained are higher than 800 KPa for ITS (dry) tested at 25 ºC, whereas the minimum values obtained is higher than 650 KPa for ITS (wet) as per the sections (MoWT, 2005). Similarly, the ratio of the wet to dry for sample 1 was less than 80%, whereas sample 2 and 3 tested cores were more than 80% as shown in Table 4.8. This implies that the AC for samples 2 and 3 has a high resilience to moisture damage. The average ITS $_{(dry)}$ of 1547.1 kN/m^3 is higher than the minimum required 800KPa, whereas the average ITS $_{(wet)}$ value obtained was 1242.4 kN/m^3 which was within the 80% of dry strength as required. Therefore, the value of the ITSR is 80.3% which is greater than 80% and this guarantees that the AC has better performance in terms of stiffness and strength resistance under traffic loading.

Table 4-7: Indirect Tensile Strength results

Sample	**ITS $_{(Dry)}$ (KN/m^2)**	**ITS $_{(Wet)}$ (KN/m^2)**	**TSR**
1	1809.3	1292.6	71.4%
2	1284.9	1992.18*	155%
3	1062.6	855.8	80.5%
Average	1547.1 KN/m^2	1242.4 KN/m^2	80.3%

*This was regarded as outlier and was not computing the averages.

iv. Marshall flow and stability

The results obtained for Marshall Stability and flow of asphalt mixtures are as summarized in Table 4.8 and detailed in appendix N. From the table, stability values ranged from 14.3KN to 23,3KN, all values were higher than the 9KN which is the minimum value provided for in the specification (MoWT, 2005). The increase of stability could be attributed to additional compaction after laying, and to asphalt stiffening that increases the overall stability. Therefore, the stability of the mixture reflects the internal friction (a benchmark of the interlocking and friction resistance of aggregates) and cohesion (a measure of the bitumen binding strength) with extremely high stability but it can be concluded that the high stability asphalt concrete had high rutting resistance (Japan Road Association, 2010).

The flow parameter obtained show 2.4 mm as the lowest and 4.0 mm as the highest as shown in Table 4.8 and detailed in appendix E-4. These values are within the range from 2mm to 4mm provided for in the specification (MoWT, 2005).

It is noted that there were higher values of flow to the upper limit which implies possibility of a plastic mixtures susceptible to permanent deformations under traffic. Very low values less than 2 would indicate that the mixture had larger than normal voids and insufficient asphalt to ensure durability; premature cracking could be experienced due to the fragility of the mixture.

Table 4-8: Marshall Flow and stability results

Sample	**A**	**B**	**C**	**D**	**E**	**F**
Stability (KN)	14.3	23.5	14.3	21.6	32.7	23.3
Flow (mm)	3.6	2.9	2.4	4.0	3.2	3.9

v. Maximum theoretical Density of the asphalt mixes

The results for maximum theoretical density (G_{mm}) are presented in Table 4.9 and detailed in appendix O. From the results, the highest G_{mm} was 2.538 g/cm^3 while the lowest G_{mm} was 2.429 g/cm^3.

The theoretical maximum specific gravities and densities of bituminous paving mixtures are fundamental properties whose values are influenced by the composition of the mixture in terms of types and amounts of aggregates and bituminous materials. Maximum specific gravity is used in the calculation of air voids in the compacted bituminous paving mixture, in calculating the amount of bitumen absorbed by the aggregate, and to provide target values for the compaction of paving mixtures. The purpose of maximum theoretical density G_{mm} was to enable the determination of the air voids. Density is one of the most important parameters in design and construction of asphalt mixtures. A mixture that is properly designed and compacted will contain enough air voids to prevent rutting due to plastic flow but low enough air voids to prevent permeability of air and water. Obtaining adequate density is a major requirement in the construction of hot mix asphalt (HMA) pavements as density is very much related to the air voids.

Table 4-9: Maximum theoretical density results

Sample	**A**	**B**	**C**
G_{mm} (g/cm^3)	2.429	2.538	2.448

vi. Air Voids

Air voids are small airspaces or pockets of air that occur between the coated aggregate particles in the final compacted mix. A certain percentage of air voids is necessary in all dense-graded highway mixes to allow for some additional pavement compaction under traffic and to provide spaces into which small amounts of asphalt can flow during this subsequent compaction. The durability of an asphalt pavement is a function of the air-void content. This is because the lower the air-voids of less than the recommended lower limit of 3%, the less permeable the mixture becomes. Too high an air-void content higher than the recommended upper limit above 5% provides passageways through the mix for the entrance of damaging air and water. A low air-void content, on the other hand, can lead to flushing, a condition in which excess asphalt squeezes out of the mix to the surface. Density and void content are directly related. The higher the density, the lower the percentage of voids in the mix, whereas the lower the density, the higher the percentage of voids in the mix.

The results obtained for air voids of asphalt mixtures are as summarized in Table 4.10 and detailed appendix P. From the results, the air voids for the three cores ranged from 4.7% to 5.7% from the lowest to the highest respectively, with the highest air voids being greater than 5.0% which is the maximum value provided for in the specification (MoWT, 2005).

For most mixes used for surfacing, a range of 3.0 –5.0% of air voids in laboratory samples is considered adequate. Too high air voids lead to a permeable mix which is susceptible to the damaging effect of air and water, that’s because the more air voids a pavement has, the more that pavement is compromised in terms of pavement strength, fatigue life and susceptibility to moisture damage, therefore the probable causes of the cracks observed could be as a results of high air voids slightly being higher than the recommended range because the study section was in the swampy area.

Table 4-10: Void Content (Air Voids) results

Sample	A	B	C
Air void content (%)	4.7	5.7	4.7

4.3 An appropriate strategy to mitigate the cracking on a section of Kampala-Masaka Road in order to restore the road to achieve its design life

Defects on pavement is often because of a combination of factors, rather than just one root cause, a series of interconnected cracks caused by fatigue failure of the surface under repeated traffic loading. Therefore, the PCI result classifies the case study pavement section as being considered in “poor” condition and it falls between the values of 41 and 55, which mean that the pavement in the selected road section needs major rehabilitation operations whereas the current traffic loading estimated is 54 MESA above the projected design traffic loading 44.1 MESA implying that increase in traffic loading (that is, the pavement is being loaded more heavily than anticipated in design). Hence attributing to probable causes of the observed cracks on the section of the road and weakening the subbase.

Finally, the evidence of the deflection measurement which determined the flexural rigidity of the subbase to be in a warning state owing to lack of stable support.

If it is established that the road does not require strengthening, the method of maintenance should be based upon the type of the existing surfacing and the cause of failure. Pavement maintenance will generally result in two operations. Firstly, those areas where failure has already occurred should be repaired by some form of remedial treatment and, secondly, the road should generally be resurfaced to prevent other lengths failing in a similar manner (Overseas Road Note 18, 1999).

The appropriate strategy to mitigate cracking on the section of the study road shall be to mill off the existing top layer and consider applying an asphalt overly on top of the existing asphalt concrete.

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

An investigation was conducted to develop a better understanding of the defects observed on a section of Kampala – Masaka road from Km 97+000 to Km 102+000. Cracking was the major defect observed on the road in form of interconnected and extensive crocodile cracks that spread across the full width of the carriageway, as well as longitudinal cracks along the wheel paths. Cracking was confined to the wearing course layer of the surfacing with the other underlying layers performing well and their materials characteristics were within the acceptable specification requirements. Based on the analysis carried out, the following conclusions on the failures observed on the road are drawn.

1. PCI for the entire selected section calculated by determining the average of the random sample unit PCIs was 46.8% of the subject pavement section which falls between the limit of Poor (41-55). Therefore, the condition of the selected pavement section may be qualitatively rated as POOR based on the results of the observed distress on the section of the road.
2. The computed traffic loading showed the estimated traffic loading of approximately 54.0 MESA was determined and when compared with the actual design value of 44.1 MESA this implies there was an increase in traffic loading higher than the design traffic on the road.

3. The deflection bowl parameters for the Lower Layer Index shows that the subgrade is still in sound structural condition indicating that the deflection bowl parameters for those sections are greater than 50mm.
4. Generally, the subgrade was good in sound state, except few localised areas at Km 97 +000, Km 98+000, and Km 100+000 which were warning.
5. Laboratory test results for the cored samples obtained from different sections of the road indicated that they didn't have a problem, and all were within the specifications except for the air void for the bitumen which were at 5.7% slightly higher than the recommended design air void of 5%. According to MS-2, the void ration after construction stage should be between 4 to 8% to allow for additional compaction during the design life of the road. Therefore, the structural integrity of the roadbase and subbase was still in sound condition to receive asphalt surfacing and hence was deemed not to be the main cause for defects observed on the asphalt surface.
6. Finally, the study concludes that the road being a rehabilitation project, much attention was not catered for to strengthen the original subgrade, therefore proper investigations should be conducted before using existing layers as part of a new construction.

5.2 RECOMMENDATIONS

Based on the findings of this investigation, the following recommendations are provided:

1. The existing asphalt layer should be milled off and reconstructed with strict quality control regimes in place.

2. Good documentation and record keeping with respect to completed projects should be practiced to aid and facilitate research.
3. In case rehabilitation of an existing road is to be carried out, always a comprehensive investigation must be conducted rather than considering re-use of the existing subbase, widen the carriageway and construct a new base and finally an asphalt pavement which this research has proved that the road will not achieve its design life, yet the cost of maintenance is too high.
4. Emphasis should be laid on adequate pavement support. Further investigations on chemical analysis of improved natural material forming pavement and visual conditions surveys are recommended to be undertaken.
5. It is recommended that amounts of stabilizer for improving strength of pavement layers to be varied in accordance with the properties of the natural materials being used and be based on frequency set out in the standard specifications.
6. It is recommended to mill off the existing top layer and consider applying an asphalt overly on top of the existing asphalt concrete.

REFERENCES

AASHTO T283, 2004-2005. *The American Association of State Highway and Transportation Officials.* Wasington, D.C., USA: AASHTO designation.

AASHTO, 1993. *American Association of State Highway and Transport Officials.* Washington , D.C.: guide for design of pavement structures.

Abdulwahhab H. I., 1998. *Temperature Impact on Pavement Structures in Hot Arid Environment,* Saudi Arabia: Civil Engineering Department, Research Institute King Fahd Univeristy of Petroleum and Minerals.

Adarkwa, A.-O. a. O., 2013. *Pavement Conditions Surveys - Overview of Current Practices,* Delaware: Delaware Center for Transportation University.

Ahmed, 2008. *Pavement Distresses Study: Identification and Maintenance,* Khartoum: University of Sudan.

Ali, N.A and Khosla, N.P., 1987. *Datermining Layer Moduli Using a Falling Weight Deflectometer.* Wasington D.C: Transportation Research Board.

ASTM D 6433-99, 1999. *American Society for Testing and Materilas,* West Conshohocken, PA, USA: Guide for design of pavement structures.

ASTM D6433-11, 2011. *Practice for Roads and Parking Lots Pavement Condtion Index Surveys.* s.l.:s.n.

ASTM D6927-15, 2015. *American Society for Testing and Measurement,* Wasington DC: s.n.

ASTM D6931-12,, 2012. *Standard Test Methods for Detrmining Indirect Tensile Strength of Compacted Bituminous Mixtures.* West Conshocken, PA, USA: ASTM International.

Bashir, M., 2006. *Effect of thermal cracking and environmental condition on asphalt Pavement,* Khoms: Master Dissertation, Al-meraheb University.

Bianchini, A. B. P, 2010. Interrater reliability of manual pavement distress evaluations. *Journal of Transportation Engineering,,* 154-172(https://ascelibrary.org/doi/pdf/10.1061), p. 136.

Bianchini, A., 2007. *Prediction of pavement performance through neuro-fuzzy reasoning,,* New Mexico: Ph.D., New Mexico State University, Unitec States.

Brown S. F and Tam, B., 1987. *Structural Evaluation and Overlay Design: Analysis ans Implementation.* s.l., Proc. of the 6th Int. Conf. on the Struct Design of Asphalt Pavements.

Cary, C. E. and Zapata. C, 2011. Resilirnt modulus for unsaturated unbound materials.. In: V. 12(3), ed. *Road Materials and Pavement Design.* s.l.:s.n., pp. 615-638.

D6433-11, A., 2011. *Practice for Roads and Parking Lots Pavement Condtion Index Surveys.* s.l.:s.n.

Dickinson, E. J., 1984. *Bituminous roads in Australia,* Sydney: Australian Road Resaeach Board.

Feng, D. H., 2009. *Effects of surface preparation, thickness and material on asphalt pavement overlay transvers crack propagation.* Eng., s.n., p. 1411.

GTC, 1998. *Geotechnical Testing Center.* Riyadh-KSA: guide of pavement defects and proposals for maintenance.

Haas, R. H. W. and Hudson, W., H., 1978. *Pavement Management Systems,* New York: Mc Graw-Hill.

Hadjidemetriou, G. M. E, 2019. Vision and Entro-Based Detection of Distressed Areas for Integrated Pavement Condtion Assessment. *Journal of Computing in Civil Engineering,* p. 33.

Hoffman, 2008. Basic Structural Elements, Pavement Interactive.

Horak, E. S., 2008. *Falling Weight Deflectometer Bowl Parameters as analysis tool for pavement structural evaluation.* Brisbane, Australia, 22nd Austrialian Road rearch Board (ARRB) International Conference.

Houston, W, M. M., 1992. *Laboratory Versus Nondestructive Testing for Pavement Design.* Vol. 118, Journal of Transportation Engineering, ASCE.

Hye, K. T., 1992. *A Guide to Visual Assessment of Flexible Pavement Surface Conditions.,* Kuala Lumpur, Malaysia: s.n.

Kannemeyer, L., 2003. Modelling rutting in flexible pavments in HDM-4. In: s.l.:s.n.

Karimi, R. H, 2021. Study on Asphalt Pavement Distress: A case study on Turkish Republic on Northern Cyprus. *British Journal of Earth Sciences Research,* Volume Vol.8 (1) pp. 59-70.

Khana, C. J, 2014. *In Highway Engineering.* India, Nem Chand & Bros Roorkee.

https://www.independent.co.uk/news/world/africa/uganda-s-masakakampala-highway-is-this-the-world-s-most-dangerous-road-a7331391.html

Lambebo, B. A., 2020. Mix Design for Wearing Course of Flexible Road Pavement by Marshall Method. *International Research Journal of Engineering and Technology*.

Lan, X., 2019. *Study on the Management Mode of Asphalt Pavement Cracks in Different Climatic Zones.* s.l., Cham. https://doi.org/10.1007/978-3-030-152352_176.

Lavin, P. G., 2003. *Asphalt Pavement.* London and New York: Taylor & Francis e-Library.

Lu, D., 1976. *Strategic Planning for Pavement Rehabilitation and Maintenance Management Systems..* s.l., s.n.

Lubwama, I., 2008. *Final Year Project Report,* Kampala: Makerere University.

Luo, Z., 2005. *Flexible Pavement Condition Model Using Clusterwise Regression and Mechanistic-Empirical Procedure for Fatigue Cracking Model,* s.l.: Ph.D Dissertation, The University of Toledo.

Madanat, 1994. *Optimal Inspection and repair policies for infrastructure facilities*: Transportation Science.

Mathakiga, A., 2016. *Strengthening and widening of flexibel pavement,* s.l.: s.n.

Miller,. J., and Bellinger, W, 2003. *Distress identification manual for long term pavement performance program (Fouth Revised Edition,* s.l.: Rep. FHWA-RD-03-031.

MoWT, P. D. M., 2005. *Pavement Design Guide.* Kampala: MoWT.

Mugume, R. B., 2020. Effect of Unstable Mix under Severe Traffic Loading on Performance of Asphalt Pavements in Tropical Climate. Volume 2020(https://doi.org/10.1155/2020/8871094), p. 4.

Mugume, R., 2020. Effect of Inappropiate Binder Grade Selection on Initiation of Asphalt Pavement Cracking. *MDPI,* p. 1.

Muhammad, S. Z and Naveed, S. R., 2019. Condition Survey for Evaluation of Pvement Condition Index of Highway. *Researchgate,* p. 1369.

Naiel, Asmaie Odan A., 2010. *Flexible Pavement Rut Depth Modeling for Different Climate Zones,* Detroit, Michigan.

National Planning Authority (2013) The Uganda Vision 2040; Government of Uganda: Kampala, Uganda, 2013, Retrieved March 10, 2021, http://www.npa.go.ug/uganda-vision-2040

National Planning Authority (2020) Third National Development Plan (NDP III) 2020/21 – 2024/25. Government of Uganda, 265pp

Nega, A. N. H. h. S. &. G. B., 2015. Distress identification, cost analysis and pavement temperature prediction for the long-term pavement perfromance for Western Austrailia. *Internal Journal of Engineering and Technology (IJET),* Volume http://Doi 10.7763/IJET.2015.V7.803, p. 7.

OECO, 2008. *Omran Engineering Consultants Office*, Tripoli, Libya: Book of general technical specifications for the establishment and paving roads. Roads ans land Transpotation Authority.

Overseas Road Note 18, 1999. *Overseas Road, Aguide to the pavement evaluation and maintenance of Bitumen-surfaced roads in tropical and sub-tropical countries.* Crowthorne, Bertshire: Transport Research Laboratory.

The Independent, 2016. Uganda's Masaka-Kampala highway: Is this the world's most dangerous road? Retrieved online via https://www.independent.co.uk/news/world/africa/uganda-s-masakakampala-highway-is-this-the-world-s-most-dangerous-road-a7331391.html (accessed: 21st November 2022).

United Nations (2018) Sustainable Development Goals. Retrieved online via the link https://sustainabledevelopment.un.org/sdgs (accessed: 16th January 2020)

Veropalumbo, R., 2019. *Indirect Tensile Strength Method for Defining a proper practice of Asphalt Mixtures Design.* s.l., University of Naples Federico II.

Jain, S. and Kumar, P., 1998. *Report on causes of cracks Occuraence in Ramghat - Aligarh Road in U.P.,* Aligarh: Report submitted to PWD.

Safwat, F. S., 2016. Evaluation of rutting of asphlat concrete pavement under fiedl-like conditions. *E&E Congress* , pp. 1-2.

Shafie, A. R. M., 2007. *Flexible Pavement Maintenance along the north South Interurban Toll Expressway from Pagoh to Machap in Section S3, Southern region,* Johor, Malaysia: PhD, University Technology Malaysia.

Shahin, M. Y., 1990. Pavement maintenance management for roads and streets using the PAVERS system. In: *Construction Engineering Research Lab (Army).* s.l.:s.n.

Smith, K. & Romine, A., 2001. *Matrials and Procedures for Sealing and Falling Cracks in Asphalt-Surfaced Pavements-Manuals of Practice,* McLean, VA, USA: s.n.

Thant, N. and War S, 2019. Study on Distress Patterns, Causes and Maintenance of Flexible Pavement for Selected Portions.. Issue https://doi.org/10.31142/ijtsrd25233.

Ullidtz, P., 1987. *Pavement Analysis,* Elsevier, Amsterdam: s.n.

Uzan, J., 1994. *Advanced Backcalculation Techniques, Nondestructive Testing of Pavement and Backcaluclation of Moduli.* (Second Volume) ASTM STP 119 ed. Philadelphia: s.n.

Vaitkus, A. C. D., 2014. Research of asphalt pavement rutting in Vilnius city streets.. *Environmental Engineering. Proceedings of the International Conference on Environemental Engineering.ICEE,*Vol.9(Deaprtment of Construction Economics & Property. http:// enviro.vgtu.lt978-609-457-640-9), p. 1.

Walker, G., 2002. *Flexible Surface Evaluation and Rating.* Madison, University of Wisconsin.

Yadav, C. S., 2019. Assessment of Potholes using QGIS & AutoCAD Software. Issue https:59827687 IRJETV61457520190622-113266, p. 6(04).

Yang, Q., & Deng, Y., 2019. Evaluation of cracking in asphalt pavement with stabilized base course based on statistical pattern recognition. *International Journal of Pavement Egnineering.*

Zaltuom, A. M., 2011. *Evaluation Pavement Distresses using Pavement Condtion Index,* s.l.: s.n.

Zhang, W. S. S., 2019. Effects of In-Place Volumetric Properties on Field Rutting and Cracking Performance of Asphalt Pavement. *Journal of Materials in Civil Engineering.* , Issue https://doi.org/10\.61/(ASCE)MT. 1943-5533.0002767, pp 8 - 31.

Zhang, Z. S., 1993. *Comprenhensive Ranking Index for Flexible Pavement Uisng Fuzzy Sets mODEL,* s.l.: s.n.

APPENDICES

Appendix. A: Photographs showing visual Condition Survey

	Bleeding of bitumen from Km 97+200 to Km 97+590 –
	Alligator cracks with wider width >3mm from Km 98+180 to Km 98+270 –
	Longitudinal cracks developing from series of alligator cracks from Km 98+150 to Km 98+280 –

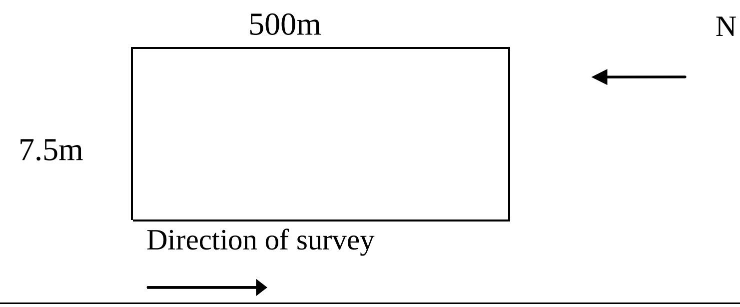
	Measuring the length of longitudinal cracks from Km 98+640 to Km 98+890 –
	Aggregate loss from bitumen from Km 99+710 to Km 99+870 –
	Measuring the width and depth of Alligator cracks at Km 101+330 and Km 101+490

APPENDIX B: Calculation of PCI for Sections

B.1 Pavement condition survey data sheet for road for section on Km 97+000 -Km 102+000

Asphalt surfaced roads
Condition survey data sheet for Kampala – Masaka Road Section from Km 97+000 to Km 102+000

Road Section: Km 97+500 – Km 98+000
No. of section: 02
Surveyed by: Costa Odwar
Date: October,2019



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

Distress severity	Quantity										Total	Density	Deduct Value
1 L	2.3*3.5	0.2*0.6	0.9*2.7	1.1*5.8							16.98	2.12	17.5
1 M	1.8*19.2	1.1*4.1	2.0*30	3.8*0.80	0.9*2.2	1.0*4.3	1.2*3.0				114.63	14.32	50
10 L	2.4	2.9	2.4	2.6	2.4	2.4	2.4	2.8	1.3	2.1			
	2.7	1.6									29.4	3.7	2.5
10 M	2.5	2.0	7.5	2.2	2.5	3.65					20.35	2.6	15
12	8*30	8*5									280	35	9
13 L	4										4	0.5	12

B.2 Calculation of corrected PCI value

m = 1+ (9/98) (100-50) = 5.59

Use highest 5 deducts and 0.59 of six deduct

0.59 *2.5=1.5

#	Deduct value										Total	q	CDV
1	50	17.5	15	12	9	1.5					105	5	56
2	50	17.5	15	12	2	1.5					89	4	56
3	50	17.5	15	2	2	1.5					99	3	56
4	50	17.5	2	2	2	1.5					55	0	55
5	50	2	2	2	2	1.5					58.5	1	58

Max CDV = 58

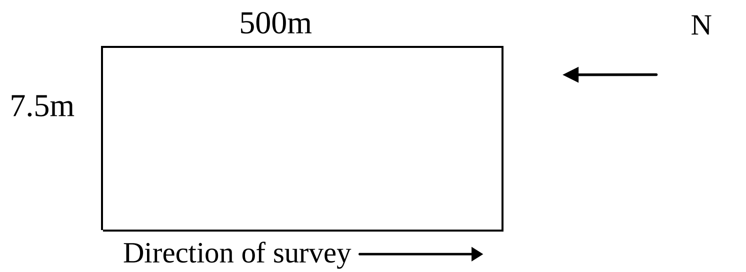
PCI = 100- Max CDV = 42

Rating = Fair

B.3 Pavement condition survey data sheet for road for section on Km 97+000 -Km 102+000

Asphalt surfaced roads
Condition survey data sheet for Kampala – Masaka Road Section from Km 97+000 to Km 102+000

Road Section: Km 98+000 – Km 98+500
No. of section: 03
Surveyed by: Costa Odwar
Date: October,2019



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

Distress severity	Quantity									Total	Density	Deduct Value
1 L	2*3.5	3.5*3.0	5.4*2.7	2.35*5.1	2.45*3	3.5*3				61.91	7.74	30
1 M	1.5*3	2.25*3	1.7*3.7	1.9*2						21.34	2.66	31
3 M	1*3	1.5*0.6								3.9	0.48	0
10 M	6.80	2.70	4.00	2.90	2.40	2.20	2.50	2.16	2.80	3.80			
	3.50	3.90	4.05										
										43.45	5.4	12
11 L	12									12	1.5	5
13M	1									3	0.125	7

B.4 Calculation of corrected PCI value

m = 1+ (9/98) (100-58) = 4.90

Use highest 4 deducts and 0.90 of six deduct

0.90 *5=4.50

#	Deduct value										Total	q	CDV
1	31	30	12	7	4.50						84.5	5	44
2	31	30	12	7	2						82	4	48
3	31	30	12	2	2						77	3	52
4	31	30	2	2	2						67	2	50
5	31	2	2	2	2						39	1	40

Max CDV = 52

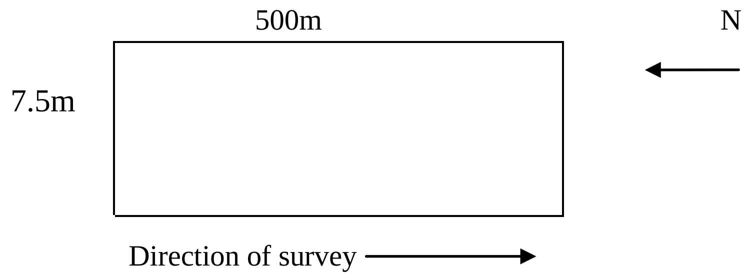
PCI = 100- Max CDV = 48

Rating = Fair

B.5 Pavement condition survey data sheet for road for section on Km 97+000 -Km 102+000

Asphalt surfaced roads
Condition survey data sheet for Kampala – Masaka Road Section from Km 97+000 to Km 102+000

Road Section: Km 98+500 – Km 99+000
No. of section: 04
Surveyed by: Costa Odwar
Date: October,2019



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

Distress severity	Quantity										Total	Density	Deduct Value
1 L	5.07	26.25	2.325	5.04	14.04	19.25					71.97	8.99	32.5
1 H	2.25	2.34	2.1								12.27	0.8	28
3 H	15.3										15.3	1.91	7.5
11 L	22										22	2.75	5
13 M	2										2	0.25	12.5

B.6 Calculation of corrected PCI value

m = 1+ (9/98) (100-32.5) = 7.19

Use highest 6 deducts and 0.96 of six deduct

0.19 *5=0.95

#	Deduct value										Total	q	CDV
1	32.5	28	12.5	7.5	0.95						81.45	4	48
2	32.5	28	12.5	2	0.95						75.95	3	50
3	32.5	28	2	2	0.95						65.45	0	50
4	32.5	2	2	2	0.95						39.45	1	40

Max CDV = 50

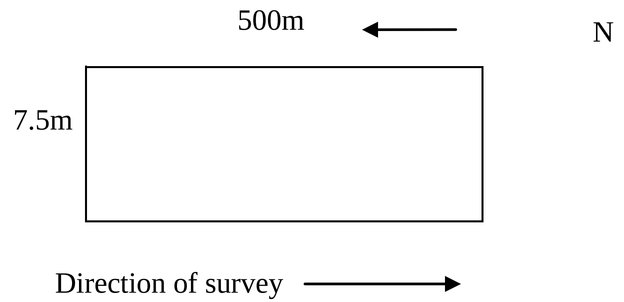
PCI = 100- Max CDV = 50

Rating = Fair

B.7 Pavement condition survey data sheet for road for section on Km 97+000 -Km 102+000

Asphalt surfaced roads
Condition survey data sheet for Kampala – Masaka Road Section from Km 97+000 to Km 102+000

Road Section: Km 99+000 – Km 99+500
No. of section: 05
Surveyed by: Costa Odwar
Date: October,2019



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

Distress severity	Quantity									Total	Density	Deduct Value
1 M	3.3*0.6	3.4*1.2	0.5*1.3	5.3*4.4	1.7*8.5					44.48	5.56	40
10 L	2.20	2.00	0.70	2.50	2.00	1.20	2.80	1.80	2.50	1.50		
	2.10	2.40	2.10							25.80	3.30	2
10 M	3.30	1.80	3.20	2.30	2.40	3.00	2.20	3.30	3.90		25.40	3.20	7
11 L	4										4	0.50	2.5
13 L	1										1	0.20	5
19 M	8*10										80	10	18

B.8 Calculation of corrected PCI value

m = 1+ (9/98) (100-32.5) = 7.19

Use highest 6 deducts and 0.96 of six deduct

0.19 *5=0.95

#	Deduct value										**Total**	**q**	**CDV**
1	40	18	7	5	2.5	0.28					**72.78**	**5**	**38**
2	40	18	7	5	2	0.28					**72.28**	**4**	**42**
3	40	18	7	2	2	0.28					**69.28**	**3**	**46**
4	40	18	2	2	2	0.28					**64.28**	**0**	**48**
5	40	2	2	2	2	0.28					**48.28**	**1**	**50**

Max CDV = 50

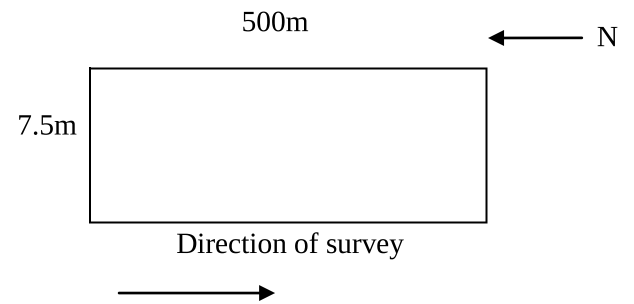
PCI = 100- Max CDV = 50

Rating = Fair

B.9 Pavement condition survey data sheet for road for section on Km 97+000 -Km 102+000

Asphalt surfaced roads
Condition survey data sheet for Kampala – Masaka Road Section from Km 97+000 to Km 102+000

Road Section: Km 99+500 – Km 100+000
No. of section: 06
Surveyed by: Costa Odwar
Date: October,2019



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

Distress severity	Quantity										Total	Density	Deduct Value
1 L	1.0*23	1.70*6.2	1.40*5.7	1.30*1.70	0.90*6.2						49.31	6.16	29
1 M	2.0*6.6	2.2*5.30	2.00*4.0	3.7*3.0							43.96	5.5	40
10 L	2.15	2.10	3.15	1.50	1.80	1.40	1.10	2.20	1.50	3.70			
	1.60	1.40									23.60	2.95	2
10 M	3.70	2.70	1.20								7.60	0.95	3
11 L	4										4	0.50	2.50
13 L	3										3	0.375	7

B.10 Calculation of corrected PCI value

m = 1+ (9/98) (100-40) = 6.51

Use highest 6 deducts and 0.51 of six deduct

0.51 *2.0=1.02

#	**Deduct value**										**Total**	**q**	**CDV**
1	40	29	7	3	2.5	1.02					**82.52**	**5**	**44**
2	40	29	7	3	2	1.02					**82.02**	**4**	**48**
3	40	29	7	2	2	1.02					**81.02**	**3**	**54**
4	40	29	2	2	2	1.02					**76.02**	**0**	**56**
5	40	2	2	2	2	1.02					**49.02**	**1**	**50**

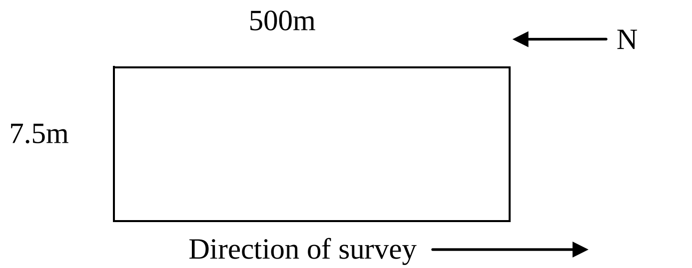
Max CDV = 56

PCI = 100- Max CDV = 44

Rating = Fair

B.11 Pavement condition survey data sheet for road for section on Km 97+000 -Km 102+000

Asphalt surfaced roads
Condition survey data sheet for Kampala – Masaka Road Section from Km 97+000 to Km 102+000



Road Section: Km 100+000 – Km 100+500
No. of section: 07
Surveyed by: Costa Odwar
Date: October,2019

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

Distress severity	Quantity										Total	Density	Deduct Value
1 L	1.1*6.30	1.0*6.4	7.0*5.0	2.5*3.6							57.33	7.1	30
1 M	0.95*4.2	1.5*0.50	0.55*5.5	1.0*16.0							23.76	2.97	32
3 L	5.0*0.45	2.3*3.2	0.9*8.1	6.80*3.5							40.70	5.1	5
3 M	2.2*0.90										1.98	0.25	-
10 L	1.00	1.80	1.90	0.70	2.70	3.10	5.30				16.5	2.10	-
10 M	1.60	2.60	3.00	0.70	1.30	1.50	2.65	3.40	1.00		17.75	2.22	6
11 M	8										8	1	10
12	8*20										160	20	7

B.12 Calculation of corrected PCI value

m = 1+ (9/98) (100-32) = 7.24

Use highest 7 deducts and 0.24 of six deduct

0.24 *5=4.35

#	**Deduct value**										**Total**	**q**	**CDV**
1	32	30	10	7	6	1.2					**86.2**	**5**	**46**
2	32	30	10	7	2	1.2					**82.2**	**4**	**46**
3	32	30	10	2	2	1.2					**77.2**	**3**	**50**
4	32	30	2	2	2	1.2					**69.2**	**0**	**50**
5	32	2	2	2	2	1.2					**41.2v**	**1**	**44**

Max CDV = 50

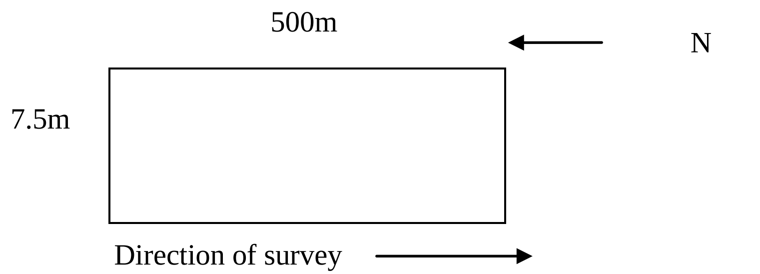
PCI = 100- Max CDV = 40

Rating = Fair

B.13 Pavement condition survey data sheet for road for section on Km 97+000 -Km 102+000

Asphalt surfaced roads
Condition survey data sheet for Kampala – Masaka Road Section from Km 97+000 to Km 102+000

Road Section: Km 100+500 – Km 101+000
No. of section: 08
Surveyed by: Costa Odwar
Date: October,2019



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

Distress severity	**Quantity**										**Total**	**Density**	**Deduct Value**
1 L	2.4*0.8	2.0*3.70		1.0*1.40	1.0*1.0	2.30*2.30	0.60*2.3				**18.39**	**2.29**	**18**
1 M	1.1*5.6	3.0*6.70	0.80*2.80	3.40*4							**42.10**	**5.26**	**40**
3 L	15*2.4	10.5*3.9									**76.95**	**9.60**	**8**
3 M	3.4*3.1	2.6*3.2									**18.86**	**2.35**	**7**
10 M	1.60	2.20	1.00	2.40	3.00	2.30	3.20	2.30			**18**	**2.25**	**6**
13 L	4										**4**	**0.50**	**12**

B.14 Calculation of corrected PCI value

m = 1+ (9/98) (100-40) = 6.50

Use highest 6 deducts and 0.51 of six deduct

0.50 *6=3

#	Deduct value										Total	q	CDV
1	40	18	12	8	7	3					88	6	42
2	40	18	12	8	7	2					87	5	46
3	40	18	12	8	2	2					82	4	48
4	40	18	12	2	2	2					76	3	50
5	40	18	2	2	2	2					66	2	50
6	40	2	2	2	2	2					50	1	50

Max CDV = 50

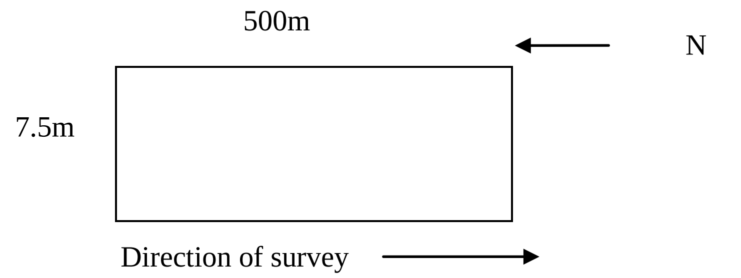
PCI = 100- Max CDV = 50

Rating = Fair

B.15 Pavement condition survey data sheet for road for section on Km 97+000 -Km 102+000

Asphalt surfaced roads
Condition survey data sheet for Kampala – Masaka Road Section from Km 97+000 to Km 102+000

Road Section: Km 101+000 – Km 101+500
No. of section: 09
Surveyed by: Costa Odwar
Date: October,2019



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

Distress severity	**Quantity**										**Total**	**Density**	**Deduct Value**
1 L	2.1*4.3	5.2*1.70	2*1.90	1.9*1.3	1.0*3	1.8*3.8	1.7*1.3	1.0*6.3			**42.49**	**5.31**	**28**
1 M	0.9*1.6	0.8*2.0	1.0*4.7								**51.25**	**6.4**	**40**
3 L	0.3*3.3	0.7*4.8	0.8*1.0	0.9*3.2							**6.03**	**0.75**	**-**
3 M	0.5*2.7	1.1*0.4									**2.03**	**0.25**	**-**
10 L	3.5	2.00	1.50	1.30	1.00	3.80	4.30	0.90	2.40	1.50			
	2.00	2.50									**26.70**	**3.3**	**3**
10 M	1.90	2.60	2.00	2.45	3.20	2.10	2.90	3.80	2.60		**23.55**	**2.9**	**8**
19 M	8*10										**80**	**10**	**18**

B.16 Calculation of corrected PCI value

$m = 1 + (9/98)(100-40) = 6.50 \leq 10$

Use highest 6 deducts and 0.50 of six deduct

0.50 *3.04 = 1.52

#	Deduct value										Total	q	CDV
1	40	28	18	8	1.50						95.5	4	56
2	40	28	18	2	1.50						89.5	3	58
3	40	28	2	2	1.50						73.5	2	54
4	40	2	2	2	1.50						47.5	1	48

Max CDV = 58

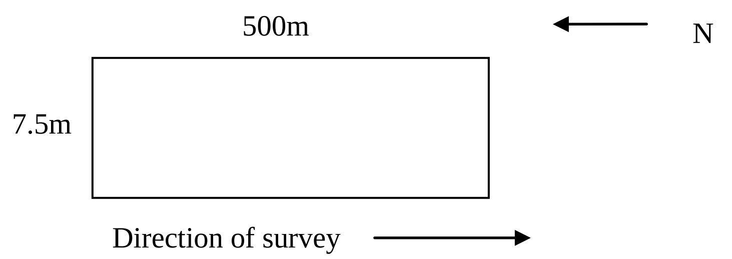
PCI = 100- Max CDV = 42

Rating = Fair

B.17 Pavement condition survey data sheet for road for section on Km 97+000 -Km 102+000

Asphalt surfaced roads
Condition survey data sheet for Kampala – Masaka Road Section from Km 97+000 to Km 102+000

Road Section: Km 101+500 – Km 102+000
No. of section: 10
Surveyed by: Costa Odwar
Date: October,2019



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

Distress severity	Quantity										Total	Density	Deduct Value
1 L	1.2*1.6	0.7*1.70	0.3*0.9	1,80*0.5	0.3*1.7	0.6*2.1					6.11	0.76	8
1 M	0.4*3	1.7*3	2.3*1.7								10.21	1.27	23
1 H	0.8*3	1*0.7	1.1*0.9	2*4.40							12.89	1.61	36
3 L	0.2*0.5	0.6*1.7	3.4*3.8	2,5*4.3							24.79	3.1	4
3 M	1.8*2.5	3.5*4.2									19.2	2.4	7
3 H	2.4*2.0										4.8	0.6	5
10 M	2.60	2.90	4.30	3.90	2.40						25.55	3.2	9

B.18 Calculation of corrected PCI value

m = 1+ (9/98) (100-36) = 6.87

Use highest 6 deducts and 0.87 of six deduct

0.87 *4 = 2.40

#	Deduct value										Total	q	CDV
1	36	22	9	8	7	5	3.48				90.48	7	46
2	36	22	9	8	7	5	2				89	6	46
3	36	22	9	8	7	2	2				86	5	46
4	36	22	9	8	2	2	2				81	4	46
5	36	22	9	2	2	2	2				75	3	48
6	36	22	2	2	2	2	2				68	2	50
7	36	2	2	2	2	2	2				48	1	48

Max CDV = 50

PCI = 100- Max CDV = 50

Rating = Fait

Appendix C: Traffic Analysis - Direction of Traffic from Kampala to Masaka

DIRECTION A		**MON**	**TUE**	**WED**	**THU**	**FRI**	**SAT**	**SUN**	**NTWKDAY**	**NTWKEND**
Motorized										
Motorcycles		408	503	401	503	422	342	236	73	65
Cars Special Hire Taxis		369	288	288	303	650	202	207	200	202
Pickups / Vans / 4WD		583	558	438	570	482	289	391	252	242
Minibuses		482	446	446	446	482	338	299	176	165
Medium Buses / Coasters		60	51	47	51	107	43	25	21	21
Buses		77	69	70	69	77	60	39	77	68
Single Unit Truck (Dynas / Tractors)		98	81	99	84	151	93	49	52	50
Single Unit Truck (Fuso's / Lorries)		151	150	241	150	151	232	103	264	283
Truck Trailers and Semi-Trailers		107	112	102	104	107	102	98	78	78
Total		**2,335**	**2,258**	**2,132**	**2,280**	**2,629**	**1,701**	**1,447**	**1,193**	**1,174**
Non-motorized										
Bicycles		101	116	109	116	104	104	90	12	21
	M	0	0	0	0	0	0	0	0	0
Pedestrian	F	0	0	0	0	0	0	0	0	0
	C	0	0	0	0	0	0	0	0	0
Total		101	116	109	116	104	104	90	12	21

Appendix D: Traffic Analysis - Direction of Traffic from Masaka to Kampala

DIRECTION B		MON	TUE	WED	THU	FRI	SAT	SUN	NTWKDAY	NTWKEND
Motorized										
Motorcycles		394	487	477	508	410	377	342	101	93
Cars Special Hire Taxis		307	262	179	223	307	223	202	98	93
Pickups / Vans / 4WD		468	384	332	391	468	391	294	136	133
Minibuses		442	356	308	406	442	299	338	126	119
Medium Buses / Coasters		34	39	28	38	34	38	43	25	25
Buses		69	51	62	53	70	53	60	67	62
Single Unit Truck (Dynas / Tractors)		69	70	73	66	69	66	93	102	100
Single Unit Truck (Faso's / Lorries)		102	108	139	108	102	108	232	337	337
Truck Trailers and Semi-Trailers		95	105	116	106	95	106	232	93	93
Total		**1,980**	**1,862**	**1,714**	**1,899**	**1,997**	**1,661**	**1,836**	**1,085**	**1,055**
Non-motorized										
Bicycles		105	128	60	115	111	104	104	15	31
	M	0	0	0	0	0	0	0	0	0
Pedestrian	F	0	0	0	0	0	0	0	0	0
	C	0	0	0	0	0	0	0	0	0
Total		105	128	60	115	111	104	104	15	31

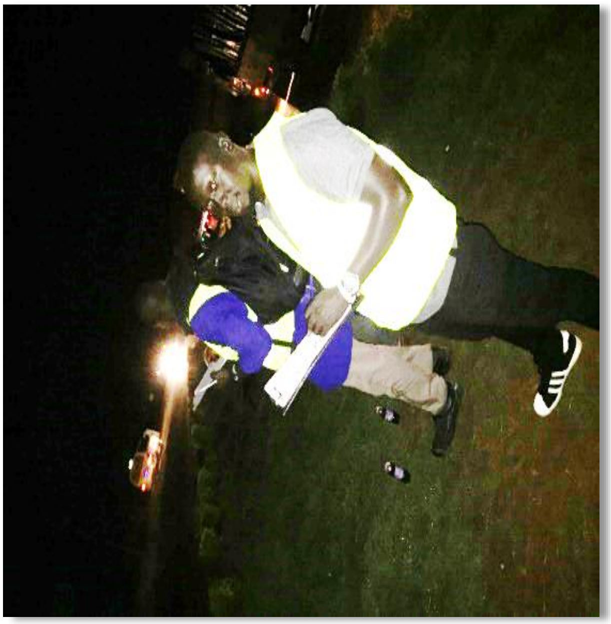
Appendix E: Traffic Analysis for both directions (A+B)

BOTH DIRECTIONS (A+B) **Type**	**MON**	**TUE**	**WED**	**THU**	**FRI**	**SAT**	**SUN**	**NTWKDAY**	**NTWKEND**
Motorized excl. M-bikes	3,513	3,130	2,968	3,168	3,794	2,643	2,705	2,104	2,071
Motorized incl. M-bikes	4,315	4,120	3,846	4,179	4,626	3,362	3,283	2,278	2,229
Non-motorized	206	244	169	231	215	208	194	27	52

Appendix F: Calculated Average Daily Traffic

Item	Vehicle Type	Overall ADT	Targeted ADT
1	Motorcycles	1,000	
2	Saloon cars and taxis	870	
3	Light goods (vans, pick-ups and 4WD)	1,247	
4	Small Bus; Minibuses and matatu	1,087	
5	Medium Bus; Coasters	137	
6	Buses	265	
7	Light Single Unit Truck; - Dynas and Tractor	319	319
8	Medium-Large Single Unit Trucks; - Lorries	903	903
9	Truck trailers and semi-trailers	398	398
	Total (**Motorized**)	**6,226**	**1620**
	Bicycles and Carts	214	

Appendix G: Photographs showing Traffic counts and Axle Loads Survey

	Traffic counting for 12 hours count at Lukaya
	Traffic counting for 24 hours count at Lukaya
	Axle Load survey at weigh bridge in Lukaya

Appendix H: Axle Load Analysis – Kampala – Masaka – (Section Km 97+500 to K 102+000)

Direction:	**Directional Summaries**								Station: Lukaya	Design Life = 15yesrs Annual growth rate = 5 years			Years to start of design life = 4years
Configuration	Traffic Class	Sign	Direction of Movement	Loaded/ Empty	No. of Vehicles	Equivalency Factors			Average EF per Traffic Class for L/E	Average Overall EF per Traffic Class	Average Traffic flow for each traffic class	Average ESA/day for each traffic class = (EF*ADT)	Cumulative ESA over design life = {(a*365*100)/b} *{(1+(b/100)^c+ d}-{{1+(b/100)}d}
						Max	Min	Average					
1*2	Light Single Unit Truck/Large Bus/Medium Single Unit Truck	LS	To Kampala	L	109	6.642	0.385	3.194	3.338	1.763	722	1272.58	-2,002,017
			To Masaka		214	16.662	0.385	3.411					
			To Kampala	E	5	0.749	0.099	0.262	0.187				
			To Masaka		16	0.819	0.027	0.164					
1*12	Heavy Trucks 3Axles	HT 3	To Kampala	L		-	-	-	-	12.061	903.00	10891.45	-17,134,381
			To Masaka			-	-	-					
			To Masaka		0	-	-	-					

1*22 1*21	Heavy Trucks 4Axles	HT 4	To Kampala	L	28	42.713	18.418	29.223	23.831				
			To Masaka		20	35.458	2.004	16.283					
			To Kampala	E	1	0.422	0.422	0.422	0.291				
			To Masaka		2	0.279	0.172	0.226					
1*222 1*1-22	Truck Trailers, Semi-Trailer 5Axles	TT5	To Kampala	L	3	116.956	24.896	72.340	73.446	54.100	398.00	21531.94	-33,873,954
			To Masaka		2	76.449	73.759	75.104					
			To Kampala	E	0	-	-	-	-				
			To Masaka		0	-	-	-					
11*22 1*2-222 1*1-222	Truck Trailers, Semi-Trailer 6Axles	TT6	To Kampala	L	74	188.490	1.983	49.902	47.313				
			To Masaka		26	95.577	1.226	39.943					
			To Kampala	E	0	-	-	-	-				
			To Masaka		0	-	-	-					

11*222 1*22-22	Truck Trailers, Semi-Trailer 7Axles	TT7	To Kampala	L	5	192.477	19.841	95.939	87.132				
			To Masaka		2	79.850	51.698	65.114					
			To Kampala	E	0	-	-	-	-				
			To Masaka		0	-	-	-					
1*12-222 1*22-222 1*12-111 1*22-221 1*22-111 1*22+2*22 1*2+22*22	Truck Trailers, Semi-Trailer 8Axles	TT8	To Kampala	L	170	161.935	4.606	77.945	74.606				
			To Masaka		196	178.211	1.219	71.710					
			To Kampala	E					-				
			To Masaka										
			To Masaka										
1*22+22*22 1*21+22*22 1*22+21-21 11*22-221	Truck Trailers, Semi-Trailer 10Axles	TT10	To Kampala	L	21	75.477	6.019	40.626	42.106				
			To Masaka		17	62.425	9.971	43.934					
			To Kampala	E					-				
			To Masaka										
Directional Traffic Volume Weighed			Towards Kampala		416						**Total cumulative ESA (one way)**		**- 53,010,353**
			Towards Masaka		495								

Appendix I: Photographs showing FWD data collection

	FWD testing at Km 97+700
	FWD testing at Km 98+210
	FWD testing at Km 99+330

Appendix J: Falling Weight Deflectometer Analysis

Appendix K: Established Layer thicknesses and crack depths (mm)

Core Location	**AC thickness**	**DBM thickness**	**Base thickness**	**Crack Depth**
Km 97+300 (LHS)	50	40	300	19
Km 97+700 (RHS)	50	40	300	14
Km 98+210 (LHS)	50	41	300	26
Km 98+790 (RHS)	50	40	300	25
Km 99+330 (LHS)	50	42	300	31
Km 100+190 (RHS)	50	41	300	17
Km 101+440 (LHS)	50	41	300	29
Average	**50**	**40.7**	**300**	**23**

Appendix L: Binder content results

No.	**Description**	**Unit**	**1**	**2**	**3**	**Average**
A	Mass of Sample Before Extraction	Gm	1239.9	1411.6	1260.5	
B	Mass of Sample After Extraction (> 0.075mm Sieve)	Gm	1146.2	1307.0	1130.1	
C	Mass of Dry Cap	Gm	215.6	280.6	219.6	
D	Mass of Dry Cap + Sample	Gm	248.4	313.9	286.5	
E	Mass of Sample in Cap (< 0.075mm Sieve) **D-C**	Gm	32.8	33.3	66.9	
F	Total Mass of Dry Sample **B+E**	Gm	1179.00	1340.30	1197.00	
G	Mass of Binder alone **A-F**	Gm	60.9	71.3	63.5	
	Binder Content **(G/A) *100**	%	4.91	5.05	5.04	**5.00**

Appendix M: Indirect Tensile Strength Results

Indirect Tensile Strength Test, ASTM D6931-07											
Our Sample Ref:	**Client's Sample Ref:**	**Labels**	**Condition**	**Sample Description**	**Avg Height of Specimen (mm)**	**Avg Diameter of Specimen mMm)**	**Divisions**	**Max. Load**	**ITS**	**Avg Indirect Tensile Strength**	**Specifications**
							No.	**kN**	**kN/m²**	**kN/m³**	
G/AC/20/	NA	M	Dry	AC	67.84	99.34	1658.0	19.2	1809.3		
G/AC/20/	NA	P	Dry	AC	80.62	99.18	1397.0	16.1	1284.9	**1547.1**	Minimum 800 kN/m³ tested at 25 °C
G/AC/20/0	NA	N	Dry	AC	84.72	99.34	1216.0	14.0	1062.6		
G/AC/20/	NA	B	Wet	AC	81.78	99.20	1425.9	16.5	1292.6		
G/AC/20/	NA	C	Wet	AC	76.64	100.08	1243.3	14.4	1192.1	**1242.4**	80% of dry strength
G/AC/20/	NA	G	Wet	AC	91.64	99.20	1057.9	12.2	855.8		

Appendix N: Marshall Flow and Stability Results

Our Sample Ref. No	Labels	Force Dial reading.	Flow Dial Reading.	Stability. (kN)	Flow. (mm)	Stability correction Factor	Corrected Stability (k N).
G/AC/20/	A	1239	360	14.3	3.6	1.0	13.3
G/AC/20/	E	2038	290	23.5	2.9	1.5	21.9
G/AC/20/	H	1237	240	14.3	2.4	0.9	13.3
G/AC/20/	Q	1869	398	21.6	4.0	1.0	20.1
G/AC/20/	D	2827	320	32.7	3.2	1.1	30.4
G/AC/20/	F	2021	392	23.3	3.9	1.1	21.7

Appendix O: Maximum theoretical Density results

Aggregate's condition		***Fully coated***	√	***Porous, not fully coated***	
Input Data					
Sample Identification			*AC Core 01*	*AC Core 02*	*AC Core 03*
$P_{bd\ aggregate}$	Bulk density of the total aggregate blend	*g/cm³*	-	-	-
P_{bit}	Density of Binder	*g/cm³*	-	-	-
p	Binder content in the asphalt mix	*%*	4.91	5.05	5.04
Results of weighing					
m_1	Mass of container in air	*g*	890.0	890.0	890.0
C	Mass of container in water	*g*	779.5	779.5	779.5
m_2	Mass of container and dry sample in air	*g*	4067.4	3311.2	3036.8
B	Mass of container and saturated sample in water	*g*	2647.1	2246.0	2048.0
A_s	Mass of surface dry sample in air	*g*	3179.4	2423.5	2148.0
A_d	Mass of dry sample in air (m_2-m_1)	*g*	3177.4	2421.2	2146.8
Calculations					
Gmm	Maximum theoretical density= (A_d/A_s-(B-C))/0.997	*g*	**2.429**	**2.538**	**2.448**

Appendix P: Void Content (Air Voids) Results

Sample ID	**AC Core 01**	**AC Core 02**	**AC Core 03**
Sample Ref:			
Gmm, Maximum Theoretical density of mix, ρd_{max} (g/cm^3)	2.429	2.538	2.448
Bulk Density of total aggregate blend, $\rho_{bd\ aggregate}$ (g/cm^3)			
Binder content, p (%)	4.91	5.05	5.04
Computations (based on 100cm^3)			
Average Height of Specimen, H (mm)	87.34	81.78	76.64
Average Diameter of Specimen, D (mm)	98.98	99.20	100.08
Weight in Air, $W1$ (g)	3177.4	2421.2	2146.8
Weight in Water, $W2$ (g)	1807.1	1396	1228
Saturated Weight in air, $W3$ (g)	3179.4	2407.5	2148.0
Volume, V (cm^3)	1372.30	1011.49	920.04
Bulk Density of core, $\rho_{bd\ mix}$ (g/cm^3)	2.315	2.394	2.333
Void Content, % $V_0 = (\rho_{d\ max} - \rho_{bd\ mix} / \rho_{d\ max}) * 100$	**4.7**	**5.7**	**4.7**

Appendix Q: Photographs showing field coring and laboratory tests

	Coring of pavement at Km 97+300
	Cored samples ready to be transported to the lab.
	Lab test for Bulky Density on Cores

	Extraction of cores to separate bitumen from aggregates
	Lab test on cores for ITS test