UNIVERSITY OF NAIROBI

DEPARTMENT OF CIVIL & CONSTRUCTION ENGINEERING

SCHOOL OF ENGINEERING

COMPARISON OF THREE DESIGN STANDARDS ON NAKURU – NYAHURURU ROAD OF FLEXIBLE PAVEMENTS IN KENYA

BY

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F56/81438/2015

A THESIS SUBMITTED IN PARTIAL FULFILMENT FOR THE AWARD OF THE DEGREE OF MASTER OF SCIENCE IN CIVIL ENGINEERING (TRANSPORTATION ENGINEERING)

UNIVERSITY OF NAIROBI

MAY, 2018

DECLARATION

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

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Signed: ……………………………………….…Date: ……………………….……………….

This thesis has been submitted for examination with our approval as university supervisors.

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ABSTRACT

The study road Nakuru – Nyahururu diverts from A109 Highway in Nakuru town passes through mountainous escarpments and small towns like Bahati, Subukia and the seat of Laikipia County (Nyahururu). It covers a total distance of 60 kilometers.

Alignment soil and the borrow pits were tested and analyzed, falling weight deflectometer (FWD) were analyzed at 100 meters interval for determination of uniform sections and to categorize the condition of the road, roughness measurement IRI (International Roughness Index) were analyzed at 100 meters interval and used to determine the condition of the road. Analyzing the traffic count was held in key places along the project road. Due to under design; roads are dilapidating before their design period is reached and will be forced to do the reconstruction or excessive maintenance making road construction uneconomical.

It is recommended that structurally sustainable and economical design methods among the usual Kenyan road design manual Part- III & IV(1987), Tanzanian design manual (1999), AASHTO 1993 and South African mechanistic-empirical pavement analysis design software (mePADS) for a rehabilitation of flexible pavement on Nakuru – Nyahururu road.

From the analysis, the cost among three methods in the beginning of the design options: - Tanzanian design manual 1999, AASHTO 1993 and South African software looks relatively more expensive than the Kenyan design manual. But the Kenyan road design manual needed a major rehabilitation. On the aspect of structurally sustainable pavement, Tanzanian design manual (1999) is safe when analyzed through the mechanistic-empirical Pavement Analysis Software (mePADS) whereas Kenyan design manual fails to reach half way the design period. AASHTO 1993 will reach its service life one or two years before the design period.

Therefore; it is recommended that the Kenyan design manual RDM, 1987 needs to be revised for the design and construction of economical and sustainable road pavement structures with in the design period.

DEDICATION

This work is dedicated to my family: my dad Mr. Abera Mosisa Wayu, my mom Kebedech Abdisa Adula, my brother Tsehay Abera Mosisa, Sister-in-law Dr. Lemane Kaba Ebba, my dear wife Muluwork Kaba Ebba, my daughter Lelo Michael Abera and my son Kenawak Michael Abera.

ACKNOWLEDGEMENTS

First of all; I want to thank my God "The creator of this universe" for giving me the opportunity and courage to undergo the course works and finish the thesis.

I would like express my great appreciation and gratitude to my supervisors, Prof. Sixtus K. Mwea and Eng. George Paul K. Matheri for unwavering support, guidance, encouragement, correction and advice you gave me to make this thesis a reality from the beginning up to the final level.

I am very grateful to my dear wife Muluwork Keba Ebba who encouraged me throughout the entire study.

My sincerely thanks goes to my teachers Prof. Oyuko O. Mbeche & Prof. F.J. Gichaga, my friend Eng. Ermias Abate who assisted me by providing the software and all my classmates for the correction and encouragement you gave me.

I also want to thank Eng. Daniel Wanyiri and his team from the ministry of transport and infrastructure for wonderful job done in the material sampling, testing and delivering results on time. Also my great appreciation goes to Ms. Ruth Wanyonyi a secretary to Runji & Partners Consulting Engineers who assisted in making copies of the proposal as well as the thesis.

May God Bless You All!!!

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ABBREVIATIONS

AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ACV	Aggregate Crushing Value
CBR	Californian Bearing Ratio
CMA	Cold Mix Asphalt
CUSUM	Cumulative Sum Method
DBM	Dense Bitumen Macadam
DCP	Dynamic Cone Penetrometer
E80	Equivalent Standard Axle (8,160 kg)
ESAL	Equivalent Standard Axle Load
FI	Flakiness Index
FWD	Falling Weight Deflectometer
GCS	Graded Crushed Stone
GPR	Ground Penetrating Radar
HMA	Hot Mix Asphalt
ICT	Inter-Continental Consultants and Technocrats
IRI	International Roughness Index
KRB	Kenya Road Board
LAA	Los Angeles Abrasion Test
LAPSSET	Lamu Port South Sudan Ethiopia Transport
LL	Liquid Limit
MC	Medium Curing
MDD	Maximum Dry Density
MEPADS	Mechanistic-Empirical Pavement Analysis Design Software
MEPDG	Mechanistic-Empirical Pavement Design Guide
MOR	Ministry of Roads
MR	Resilient Modulus
NDT	Non-Destructive Testing
PI	Plasticity Index

PL	Plastic Limit
PPP	Public-Private Partnership
PRD	Percentage Refusal Density
RC	Rapid Curing
RDM	Road Design Manual
SAPEM	South African Pavement Engineering Manual
SATCC	Southern African Transportation and Communications Commission
SD	Standard Deviation
SN	Structural Number
SNC	Modified Structural Number
SNP	Adjusted Structural Number
VCS	Visual Condition Survey
VIM	Voids in the Mix
VMA	Voids in the Mineral Aggregate
W18	Predicted number of 18-kip equivalent single load application
WMA	Warm Mix Asphalt
WMAAT	Weighted Monthly Average Annual Temperature

CHAPTER ONE

INTRODUCTION

1.1- Background

Flexible pavement is composed of a bituminous material surface course and underlying base and sub-base courses. The bituminous material is more often asphalt whose viscous nature allows significant plastic deformation. Most asphalt surfaces are built on a gravel base, although some 'full depth' asphalt surfaces are built directly on the subgrade. Depending on the temperature at which it is applied, asphalt is categorized as hot mix asphalt (HMA), warm mix asphalt, or cold mix asphalt. Flexible pavement is so named as the pavement surface reflects the total deflection of all subsequent layers due to the traffic load acting upon it. The flexible pavement design is based on the load distributing characteristics of a layered system (Russel, 2011).

According to Russel (2011) developing a rehabilitation design generally requires extensive investigation into the condition of the existing pavement structure, performance history, and laboratory testing of materials to establish suitability of existing and proposed materials for use in the rehabilitation design. The field investigation will require a deflection survey, drainage survey, and additional non-destructive testing (NDT) surveys such as dynamic cone penetrometer (DCP) and falling weight deflectometer (FWD). Once these preliminary surveys are conducted, locations for material sampling can be established. In addition, for projects where full-depth reclamation is being considered, samples of the structure should be taken at intervals not exceeding 0.5-km. These samples are evaluated in the laboratory to verify field survey conclusions and establish basic properties necessary to quantify moisture susceptibility, stabilizer compatibility and blending requirements. The preferred rehabilitation strategy should consider:

- Cost-effectiveness
- Repair of the specific problems of the existing pavement

- Prevention of future problems
- Meeting all existing constraints of the project

The study area Nakuru – Nyahururu diverts from A109 Highway in Nakuru town which passes through mountainous escarpments and small towns like Bahati, Subukia and the seat of Laikipia county (Nyahururu).The project traverses through three counties mainly: Nakuru, Laikipia, and Nyadarua. It covers a total distance of 60 km.

The climate is fairly cool to warm and in some places temperate. The area has a significant amount of rainfall during the year. The mean annual temperature varies between 14-20°C. In a year, the average annual rainfall varies between 800-1600 mm. The climate is classified as semi humid to sub humid. Precipitation is the lowest in January, with an average of 31 mm. With an average of 151 mm, the maximum precipitation is recorded in August. At an average temperature of 15.1 °C, March is the hottest month of the year while July has the lowest average temperature of the year at 13.0 °C.

With the upgrading of the Isiolo - Moyale Road (A2) to bitumen standard, the traffic levels on project road are expected to increase significantly. The implementation of the LAPPSET corridor in future is likely to further enhance traffic on the project road. The project road therefore needs reconfiguration, rehabilitation and strengthening based on the expected level of traffic. The project road rehabilitation or up-gradation is expected to provide faster access to country side farming and thus enhance the economy. The improvement of the roads in the neighborhood of the study road will result in the increased traffic.

Therefore; in order to meet the demand of rehabilitating of the road in the stretch there was a need to find the most cost effective as well as structurally sustainable method of designing a road which will stay to its design period. Several studies have been done on flexible pavement but they have not addressed this shortcoming and this research tried to fill this gap by providing a cost effective and sustainable pavement design methods by considering design manuals from South Africa, Tanzania and America. This was meant to design a cost effective pavement structure. The location map of the study road is indicated in Figure 1.1 below.

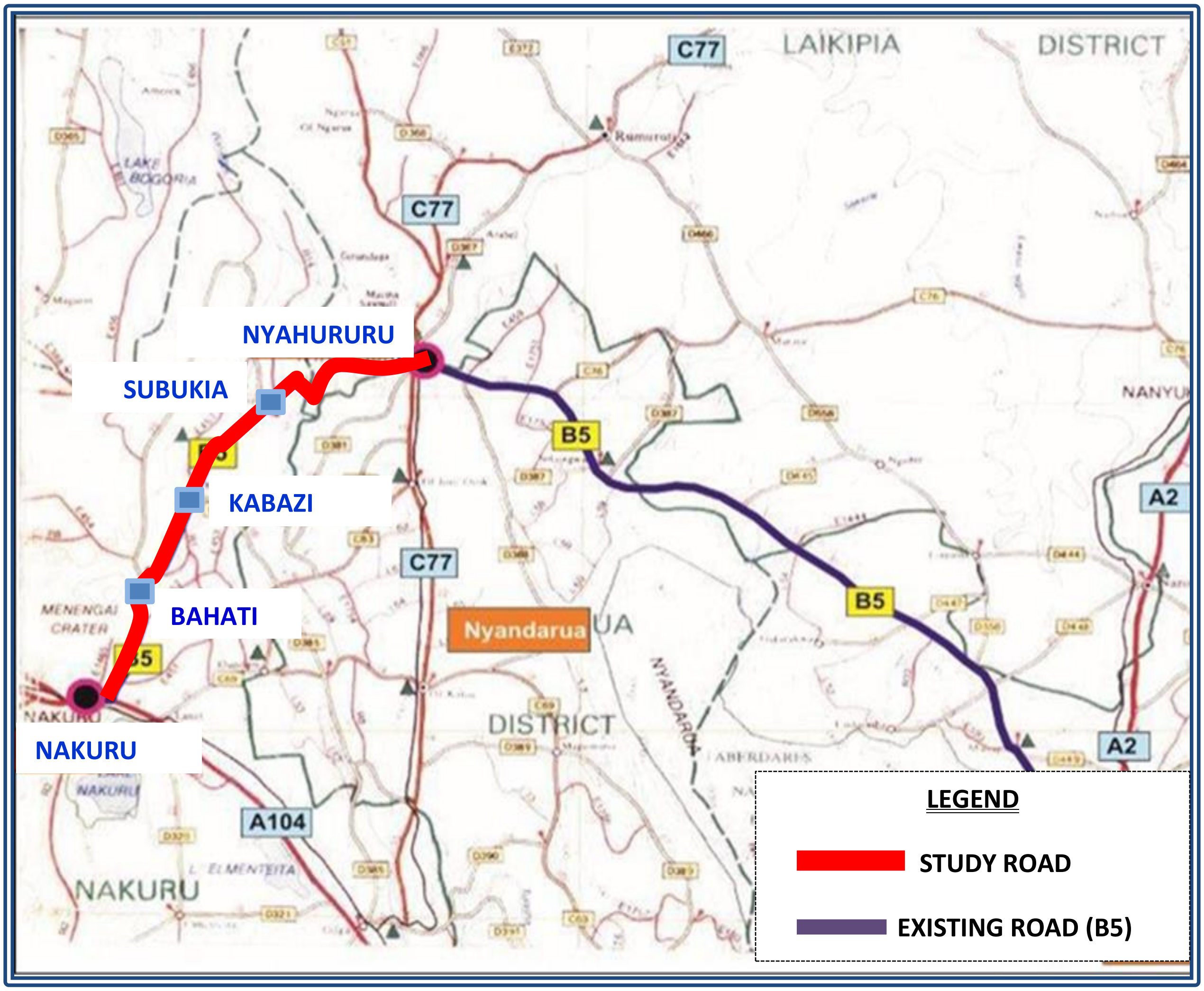


Figure 1.1: Location Map of the Project

Source: Inter- Continental Technocrats & Runji, 2016

1.2- Problem Statement

Due to bad condition of the existing road, the road was found to be full of potholes, rutting, raveling, shoving and cracks were observed on the road which led to discomfort to the road users as will be seen on Figure 1.2 a-d below. The dilapidated road condition for so many reasons became a problem to the drivers as well as to the commuters. Those are:-

i. - Problem of Road Safety (Increased Road Accidents)

The study road was dilapidated with some road sections were full of potholes, and in a poor state which led to high numbers of road accidents.

ii. - Lack of proper drainage

Due to encroachment of the road reserve and development, the existing drainage system is blocked with debris and in some places an erection of permanent structures was observed.

iii. - Increased Travel Time

Using such a road in a bad condition takes a lot of time to reach a destination.

iv. - Extra Cost of Fuel

Since the road is full of potholes and difficult for a smooth ride the drivers were forced to make a slow drive or many stop and rides which result in a high use of fuel. The other alternative is to use a long distance route which forces to use extra fuel.

v. - Extra Cost on Vehicles Repair

When using a road with a lot of potholes and in a bad condition the probability of damage to the vehicle is very high.

vi. - Lack of Comfort

While driving or commuting in such a bad road it makes the ride uncomfortable.

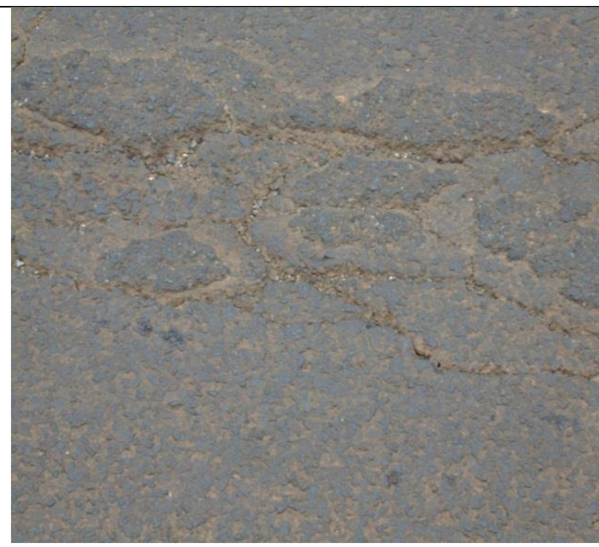
	
a- Dilapidated road (Maili Tisa)	b- Raveling and edge break (Bahati Town)
	
c- Edge break potholes and raveling (Kaburi- junction outskirt of Nyahururu)	c- Raveling and cracks (Past Bahati town)

Figure 1.2:- Existing Pavement Condition of the Road (Nakuru –Nyahururu)

Source: - Author, 2016

Murunga (1983) studied on the performance of flexible pavements in Kenya and recommended that there was need for further research on the compaction characteristics of subgrade soils. Kipyator (2013), dealt with cost comparison of concrete versus flexible pavement design along A104 road (Nakuru - Eldoret) at Timboroa and concluded that in the long-run concrete road is less expensive than flexible pavement. Temu (2012), compared axle load data for the year 2011 and 1971 and come out with a conclusion that within 40 year the axle load data has increased by almost 4 times.

This research was intended to study which methods of designing a flexible pavement is economical, structurally sustainable to accommodate traffic of 15 years and more.

1.3- Research Objectives

1- To come up with a plan so that the existing road can be rehabilitated to accommodate a traffic load of 15 years and more.
2- To find other design methods other than Kenyan standard design method so that the existing road can be designed economically in order to accommodate future traffic for 15 years
3- To find other design techniques which are structurally sustainable for accommodating a traffic load of 15 years

1.4- Justification of the Study

Historically the study road has been carrying medium level of traffic and no substantive upgrading has been undertaken on the study Road. The study road therefore needs reconfiguration, rehabilitation and strengthening based on the expected level of traffic.
In order to design and build a more sustainable and economically viable road, there was a need to look into other countries design manual other than Kenyan design manual. The other manuals selected for comparison were from Tanzanian design manual 1999, North America AASHTO 1993 and software for pavement design analysis from South Africa (mePADS) has been utilized. Therefore; finding most economical and structurally sustainable design method for the proposed upgrading or rehabilitation of the Study Road is based on this justification.

1.5- Scope and Limitation

The study is limited to identifying ways of rehabilitating the road from Nakuru – Nyahururu. The study was conducted in the following way:-

Scope of the Study:-

1- 68 samples from the alignment soil and the borrow pit tested and analyzed in the laboratory of Ministry of Transport and Infrastructure.
2- Falling weight deflectometer (FWD) were analyzed at 100 meters interval for determination of uniform sections and to categorize the condition of the road.
3- Roughness measurement IRI (International Roughness Index) were analyzed at 100 meters interval and used to determine the condition of the road.
4- Analyzing the traffic count that was held in key places along the project road. The traffic counts were done in - Nakuru, Bahati junction, Nyahururu and Subukia to determine the cumulative axle load within the design period.
5- Analyzing the selected pavement options from Road Design Manual of Kenya, Tanzania and AASHTO 1993 with software from South Africa mePADS (mechanistic empirical pavement analysis design software).
6- Comparing the most cost effective of the design methods among the Kenyan, Tanzanian, AASHTO 1993 and mePADS.
7- Cost comparison of all the design methods I used the current unit price from the average of contractor's rate for analysis.

LIMITATIONS

Checking all the design methods by using mePADS was limited to:-

1- Keeping all the allowable data that are embodied in the software such as allowable stress, strain and deflection.
2- The moduli (modulus of elasticity) were used from Kenyan road design manual (RDM-III & IV) for subgrade, sub-base, base and surfacing.
3- According to the South African standard, the altitude above 1,000 (> 1,000m) is considered as “wet”.

4- Recommendation of rutting depth for South Africa is 10mm but Kenya, Tanzania, Uganda and Ethiopia recommends using 20mm depth, for this thesis a 20mm rutting depth is adopted.
5- The standard design load for South Africa is a 40kN dual wheel load. (Based on the legal axle load of 80 KN allowed on public roads) at 350mm spacing between centers and a uniform contact pressure of 520kPa was adopted.

CHAPTER TWO

LITERATURE REVIEW

2.1 Introduction

This chapter explains what flexible pavement is and all the available methods of rehabilitation that are practiced around the world and also try to illustrate the available formulae to calculate pavement structural number.

Road pavement is referred to as "flexible" because the bituminous materials are capable of flexing slightly under traffic loading. For thinly surfaced pavements, the road base is often unbound granular material. The base course immediately beneath the surface course can be composed of crushed stone, crushed slag or other untreated or stabilized materials. The sub-base course is the layer beneath the base course. The reason that two different granular materials are used is for economy. Instead of using the more expensive base course material for the entire layer, local and cheaper materials can be used as a sub-base course on top of subgrade (Huang, 2004), (Yoder, 1975).

2.1.1 Causes of Flexible Pavement Deterioration

According to (Awang, 2009), there are different factors for causes of pavement deterioration. These are:-

a) Traffic

Traffic is the most important factor influencing pavement performance. The performance of pavements is mostly influenced by the loading magnitude, configuration and the number of load repetitions by heavy vehicles. The damage caused per pass to a pavement by an axle is defined relative to the damage per pass of a standard axle load, which is defined as 80 kN single axle load (E80). Thus a pavement is designed to withstand a number of standard axle load repetitions (E80’s) that will result in a certain terminal condition of deterioration.

b) Moisture (Water)

Moisture can significantly weaken the support strength of natural gravel materials, especially the subgrade. Moisture can enter the pavement structure through cracks and holes in the surface, laterally through the subgrade, and from the underlying water table through capillary action. The result of moisture ingress is the lubrication of particles, loss of particle interlock and subsequent particle displacement resulting in pavement failure.

c) Subgrade

The subgrade is the underlying soil that supports the applied wheel loads. If the sub-grade is too weak to support the wheel loads, the pavement will flex excessively which ultimately causes the pavement to fail. If natural variations in the composition of the sub-grade are not adequately addressed by the pavement design, significant differences in pavement performance will be experienced.

d) Construction Quality

Failure to obtain proper compaction, improper moisture conditions during construction, quality of materials, and accurate layer thickness (after compaction) all directly affect the performance of a pavement. These conditions stress the need for skilled staff and the importance of good inspection and quality control procedures during construction.

e) Maintenance

Pavement performance depends on what, when, and how maintenance is performed. No matter how well the pavement is built, it will deteriorate over time based upon the mentioned factors. The timing of maintenance is very important, for a pavement is to deteriorate to a very poor condition or not. Thus, postponing maintenance because of budget constraints will result in a significant financial penalty within a few years.

2.1.2 Pavement Rehabilitation

Pavement rehabilitation is defined as a structural or functional enhancement of a pavement which produces a substantial extension in service life, by substantially improving pavement condition and ride quality. Pavement maintenance activities, on the other hand, are those treatments that preserve pavement condition, safety, and ride quality, and therefore aid a pavement in achieving its design life (Kathleen, 2001).

Individual rehabilitation treatments are often categorized as belonging to one of the “4-

R's"–restoration, resurfacing, recycling, or reconstruction. There are some problems with trying to fit each rehabilitation treatment into one of these four major categories. For example, some treatments may be done as part of a restoration effort or as part of a resurfacing effort. Each of the four types of rehabilitation is defined below.

i. Restoration is a set of one or more activities that repair existing distress and significantly increase the serviceability (and therefore, the remaining service life) of the pavement, without substantially increasing the structural capacity of the pavement.

ii. Resurfacing may be either of the following:

(a) A **Structural overlay**, which significantly extends the remaining service life by increasing the structural capacity and serviceability of the pavement, usually in combination with pre-overlay repair and/or recycling. A structural overlay also corrects any functional deficiencies present.

(b) A **Functional overlay**, which significantly extends the service life by correcting functional deficiencies, but which does not significantly increase the structural capacity of the pavement.

iii. Recycling is the process of removing pavement materials for reuse in resurfacing or reconstructing a pavement (or constructing some other pavement). For asphalt pavements, this process may range from in-place recycling of the surface layer, to recycling material from all pavement layers through a hot mix plant. Recycling of asphalt-overlaid concrete pavement may be either surface recycling or removal and recycling of both asphalt and concrete. In this case, the asphalt and concrete layers are removed and recycled separately.

iv. Reconstruction is the removal and replacement of all asphalt, and often the base and sub base layers, in combination with improvement of the subgrade and drainage, and possible geometric changes. Other circumstances, such as obsolete geometrics, capacity improvement needs, and/or alignment changes, are often involved in the decision to reconstruct a pavement (Kathleen, 2001).

2.2 Traffic

Deterioration in paved roads caused by traffic is a function of the magnitude of the individual wheel loads and the frequency with which they are applied (Kadiyali, 1989).

For pavement design purposes, therefore, it is necessary to know not only the total number of vehicles using the road but also the axle loads. Traffic loading is normally expressed in terms of ‘equivalent standard axles’, ‘ESA’, a concept developed following the AASHO Road Test carried out in the USA in the late 1950s. An axle carrying 8.16 tons was arbitrarily defined as a ‘’standard axle’’, to which axles of different weights were correlated to derive equivalence factors, thereby obtaining an expression of the damaging effect with a formula shown in eq. 2.1:

$$\mathrm{ESA} = (\mathrm{L}/80)^{4.5} \ldots\ldots\ldots\ldots.\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..\ldots\ldots \mathrm{Eq.}\ 2.1$$

Where: - ESA is the equivalent standard axle,

L is the axle load in kN divided by the standard 80kN axle and

4.5 is the exponent representing the relative damage.

This equation was derived by Liddle (1962) for the test conditions at the time. Although Liddle’s formula is safe only up to axle weights of 130kN (13 tons), nevertheless, in the absence of anything better, current practice is still to use this equation for greater axle weights. A more secure practice would be to determine the proportion of axle weights greater than 130kN and then to adjust the traffic category accordingly.

A tandem axle may inflict slightly more or slightly less damage than two separate axles depending on various factors. It is recommended that they are treated separately in the calculation of ESA. The ultimate objective in design is thus to determine the cumulative number of ESA in the design period. This is achieved in a number of operations:

- The axle load distribution of the traffic is evaluated
- The axle loads converted into ESA
- The initial daily number of ESA calculated, and
- An annual growth rate over the design period selected.

The Maximum Gross weight of a vehicle is defined in Table 2.1:

Table 2.1: Maximum Permissible Gross Vehicle Weights

Vehicle Type	Legal Limit (kg)
Vehicle with two axles	18,000
Vehicle with three axles	24,000
Vehicle & semi-trailer with total of three axles	28,000
Vehicle & semi-trailer with total of four axles	34,000
Vehicle & drawbar trailer with total of four axles	36,000
Vehicle & semi-trailer with total of five axles	42,000
Vehicle & drawbar trailer with total of five axles	42,000
Vehicle & semi-trailer with total of six axles	48,000
Vehicle & drawbar trailer with total of six axles	48,000

Source: Kenyan RDM, 1987

2.3 Evaluation of Traffic for Design Purposes and Traffic Counts

The loads imposed by private cars and light goods vehicles with axle weights < 1.5 tons do not contribute significantly to the structural damage of a paved road and thus, for design purposes, can be ignored according to road design manual, 1987. However, for economic and congestion forecasting, the total traffic is determined and routine traffic counts are carried out annually by the Ministry of Transport & Communications at a number of census points. They distinguish between cars, light goods, buses, medium goods and heavy goods vehicles. Where such results are available, the initial daily traffic can be estimated by extrapolation.

2.3.1 Evaluation of Axle Loads

According to the Kenya roads design manual (1987), the axle loads is obtained by multiplying the average daily number of commercial vehicles by the appropriate equivalence factor and then summing the ESA for all the vehicle types. To estimate the total number of ESA for the pavement design, it is necessary to forecast the annual traffic growth rate and decide the length of the design period.

Projection for future traffic is important and it is done over the design period of the pavement. The total traffic considered for projection are those expected to be attracted to the improved road. This is done by consideration of normal, diverted and generated traffic.

Guidance can be obtained from the following factors: historical growth, economic trends, geometric capacity of the road, increases in vehicle numbers and loading and social realities. Typical growth rates range from 2 to 15% per annum, averaging about 4% per annum (RDM, 1987).

According to Kenyan roads design manual (1987), the cumulative number of ESA, T, for the chosen design period, N (in years), is then obtained from equation 2.2:

$$T = 365t_1 \frac{(1+i)^N - 1}{i} \quad \text{……………………………………………………..(Eq. 2.2)}$$

Where:

t_1 - the average daily number of standard axles in the first year after

Opening

i - The annual growth rate expressed as a decimal fraction

N- Design period (years)

From the results of Eq. 2.2 above, the cumulative number of standard axle class is summarized in table 2.2 below.

Table 2.2: Traffic classes according to RDM, 1987

Traffic Class	Cumulative Equivalent of Standard Axle (Millions)
T_1	25 – 60
T_2	10 – 25
T_3	3 – 10
T_4	1 – 3
T_5	0.25 - 1

Source: Kenyan RDM, 1987

2.4 Pavement Materials

According to Charles (1997), pavement materials which include the alignment soils and borrow materials for the subgrade and other materials for the overlying layers should be assessed both for quality and quantity. Materials that do not meet the required specifications are improved using materials such as cement or lime that reduces plasticity and/or increases strength if they are to be used in the pavement layers.

Gichaga and Parker (1988) state that the main objective of road construction material selection is to confirm overall economy of the road project by selecting materials that will require minimum haul and that will rather require no treatment to improve their strengths. Thus ideally, the most economical road material would be that which lies along the proposed road alignment or adjacent to it.

2.5 Asphalt Concrete / Flexible Pavement

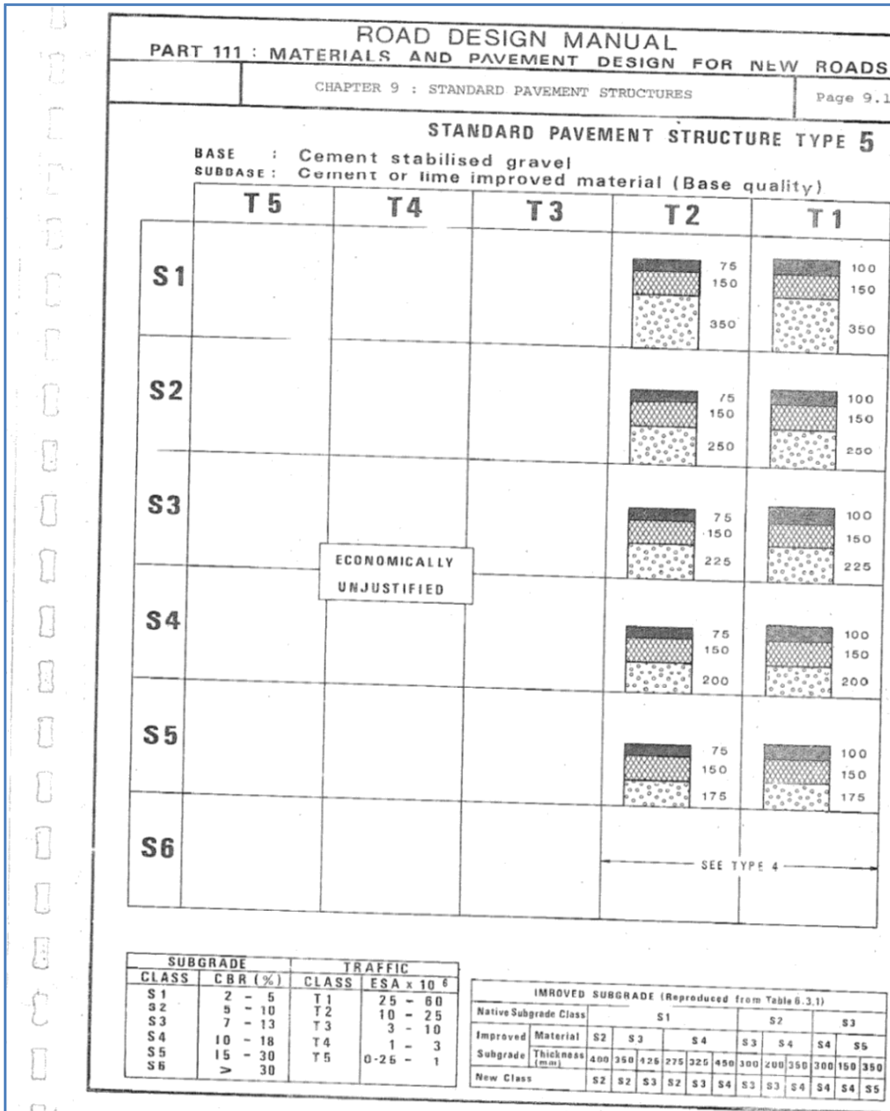
2.5.1 General

Asphalt has been widely used since 1920 among the developing and developed nations (Anon, 1991). The viscous nature of the bitumen binder allows asphalt to sustain significant plastic deformation, although fatigue from repeated loading over time is the most common failure mechanism. Most asphalt surfaces are built on bases and sub-bases of various materials; although some 'full depth' asphalt surfaces are built directly on the native subgrade. In areas with very soft or expansive subgrades such as clay or peat, thick gravel bases or stabilization of the subgrade with Portland cement or lime may be required. Anon (1991) & Gransberg (2005), states that depending on the temperature at which it is applied, asphalt is categorized as hot mix asphalt (HMA), warm mix asphalt, or cold mix asphalt.

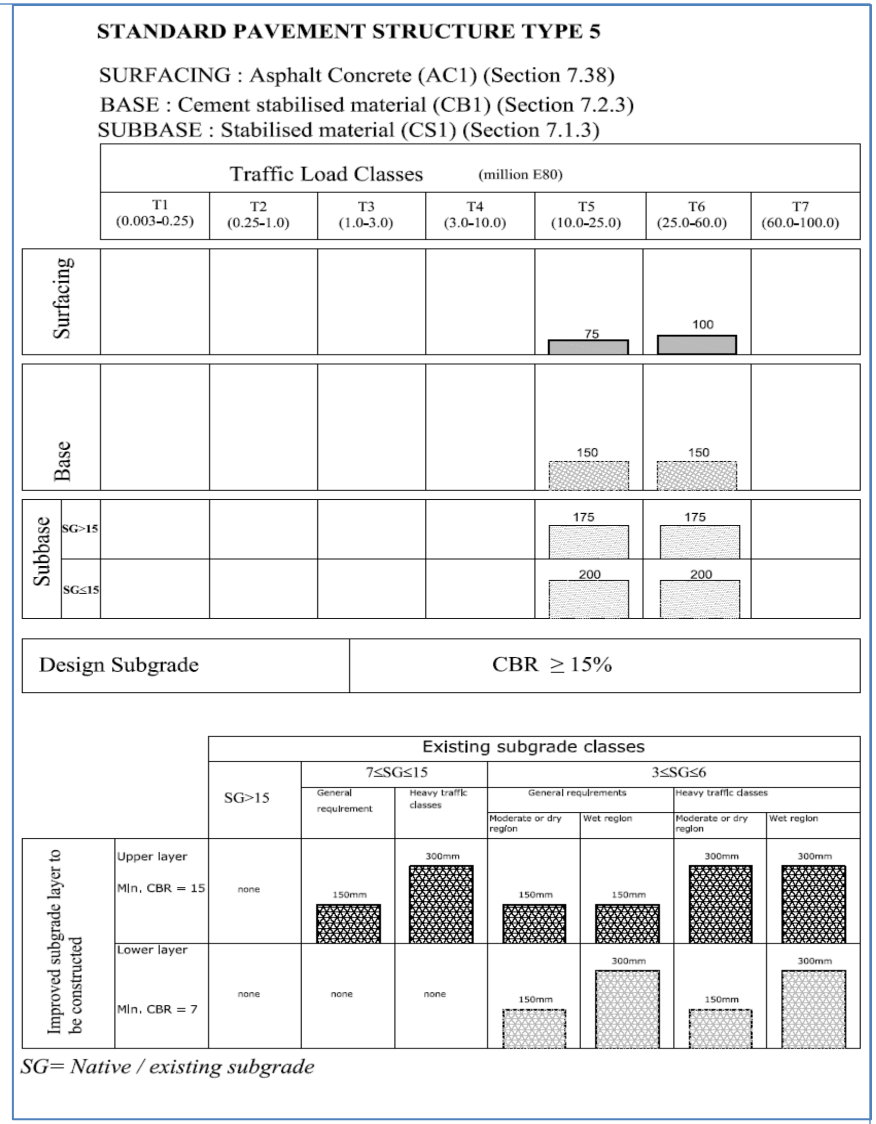
Anon (1991) states that concrete road will generally be constructed for high volume primary highways having an average annual daily traffic load higher than 1200 vehicles per day. Advantages of asphaltic roadways include relatively low noise, relatively low initial cost compared with other paving methods, and perceived ease of repair.

Kenya Roads Design Manual (RDM, 1987) is the most commonly used design method of roads in Kenya. After consideration of the traffic loading and the alignment material strengths, the most appropriate pavement type and materials is selected from a set of charts. The method is specific to prevailing conditions in Kenya and therefore best suited for the design. Samples of RDM, 1987 and the draft design manual charts are as shown in Figure 2.1 for comparison.

Tanzanian Manual (1999) is the most popular manual in East Africa where it was revised after thorough evaluation of the existing road design manual of 1989 with the assistance of Norwegian agency for international development (NORAD). The manual has considered temperature or climate as a great factor in the design of a pavement.



RDM Part-III, 1987 (Old design manual)



Design manual for roads and bridges (Draft – 2009)

Figure 2.1:- Comparison of Kenyan Old and draft New Design Manuals

Source: - Kenyan Road Design manual (RDM 1987 & draft 2009)

2.6 Flexible Pavement Design (Ministry of Roads Design Method)

2.6.1 General

Kenyan design manual part III deals with both the pavement and respective materials. The manual has developed different cross-sections and defined various components of a paved road. Figures 2.2 and 2.3 show the terms used in describing the principal pavement and cross section components.

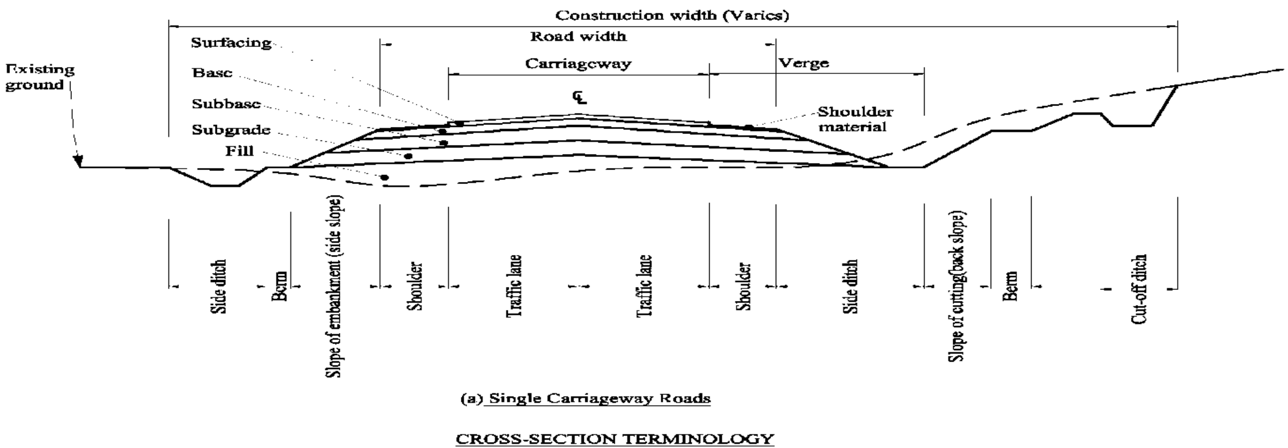


Figure 2.2: Single carriageway cross-section

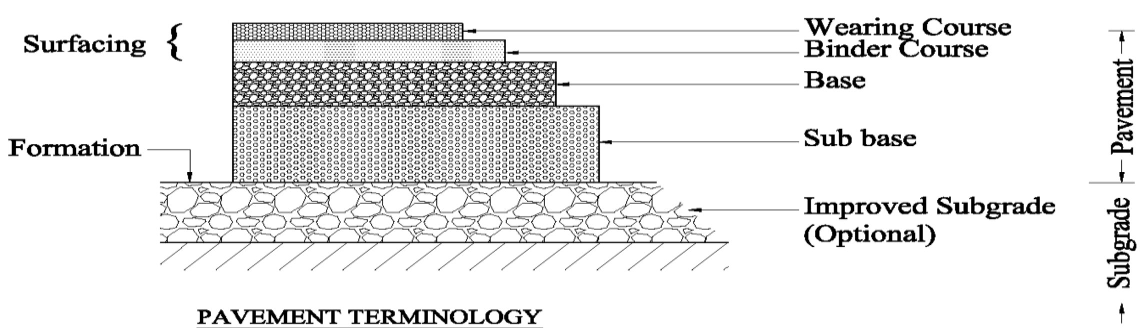


Figure 2.3: Road pavement terminology

Source: RDM, draft design of Kenya manual (2009)

2.6.2 Sub-Grade

i. - Subgrade determination and classification

Kadiyali (1989) gives specifications for processing subgrade while RDM, Part III (1987), gives the detailed procedures and material specifications for the subgrade. According to the Kenyan design Manual 1987; the pavement should be designed to cope with the lowest measured CBR. Any sub-grade with a CBR of 5% or less will require a capping layer, also known as a sub-grade improvement layer. The sub grade is classified into 6 groups S1, S2, S3, S4, S5 and S6 in relation to CBR value as indicated in table 2.3 below.

Table 2.3 Sub-grade classification Kenyan RDM

Subgrade Class	CBR Range	Median
S1	2 – 5	3.50
S2	5 – 10	7.50
S3	7 – 13	10
S4	10 – 18	14
S5	15 – 30	22.50
S6	>30	

Source: Kenyan RDM- Part III, 1987

ii. - Improved Subgrade

Placing an improved subgrade not only increases the bearing strength of the pavement support but also:

- Protects the upper layers of earthworks against adverse weather conditions (protection against soaking and shrinkage),
- Facilitates the movement of construction traffic,
- Permits more effective compaction of the pavement layers,
- Reduces the variation in the subgrade bearing strength, and
- Prevents pollution of open-textured sub-bases by plastic fines from the natural subgrade.

iii. - Tanzanian Sub grade determination

Different countries have their own way of determining design sub grade, Tanzanian design manual 1999 has a means how to determine the sub grade CBR (CBR_{design}) by use of:-

a. Statistical analysis

The CBR_{design} for a section is the 90%-ile value of the CBR test results for a section with homogenous strength. The method is illustrated in Figure 2.4 and is used an example for determination of CBR_{design} of each homogenous section. CBR values are plotted in ascending order.

CBR data	Example
6	
8	
9	
9	
9	
11	
12	90% -ile 7.6
12	
14	CBR design
	8%

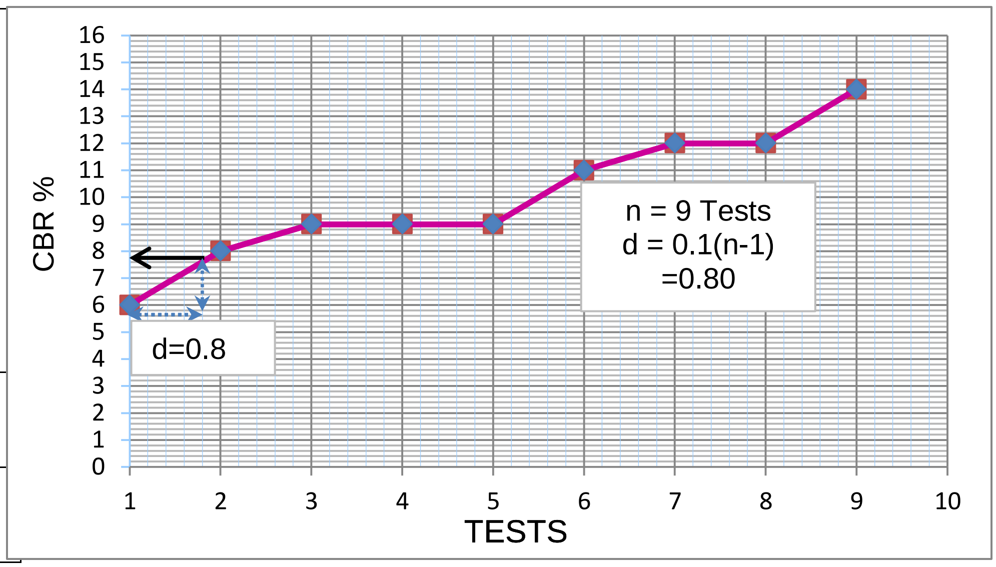


Figure 2.4:- CBR_{design} as the 90%- ile value

Source: Tanzanian design manual, 1999

2.6.3 Sub-Base

The functions of the sub-base are to act as a construction platform for the upper pavement layers and as a separation layer between the subgrade and the road base. In certain circumstances it may also act as a drainage layer, especially in concrete roads. The selection of a suitable sub-base material will, therefore, depend on the design function of the layer and the anticipated moisture conditions, both at construction and in service.

2.6.4 Bases

Gichaga and Parker (1988) gave the tests that should be carried out on the base materials. The key tests that are carried out on base material include, CBR, Los Angeles abrasion tests, Aggregate Crushing Value, Atterberg limits. Kenyan road design manual (1987) states that provision of a road-base is dependent upon the cumulative number of standard axles anticipated over the design life of the pavement.

The main function of the base is to act as the load-spreading layer of the road pavement. Therefore, only strong materials will be suitable. Bases fall into two categories: unbound and bound.

- Unbound bases, such as natural gravels and crushed stone, rely on their intrinsic internal friction to develop the necessary bearing capacity.
- Bound bases have a binder, either bitumen or cement or lime, which is used to strengthen them and enhance their ability to reduce the traffic stresses on the layers below.

Base can be of the following materials:-

- Natural gravel
- Graded Crushed Stone
- Stabilized materials
- Lean Concrete
- Sand Bitumen Mixes
- Dense Bitumen Macadam
- Dense Emulsion Macadam

2.7 Surfacing and its types

Gichaga and Parker (1988), state that the most commonly used surfacing material in construction of a flexible pavement are bituminous materials which are normally applied in thin layers ranging from 25mm to 100mm. Such materials include asphaltic concrete, gap-graded asphalt, sand asphalt, emulsion slurry seal and surface dressing. It further gives the key tests that should be done on surfacing materials which include Los Angeles Abrasion tests, aggregate crushing value, flakiness index, Sulphate content, grading amongst other tests. The road user mainly requires an asphalt concrete premix surfacing to offer an adequate riding quality and convey an appropriate skid resistance under all weather conditions. There are two generic types of asphalt premix surfacing according to the Kenyan design manual (1987).

- Interlocked aggregate mixes which derive stability from the aggregate interlock, obtained by careful adjustment of the mix grading, and from the cohesion provided by the bitumen, and
- Mortar type mixes, such as gap-graded asphalt or sand asphalt, which derive stability from the cohesion of the fines-filler-bitumen mortar.

Type I asphaltic concrete is a fairly stiff type of mix designed to resist rutting and high stresses. Type II asphalt concrete is a more flexible mix, designed to resist comparatively high flexural deformation. It must be placed in a thin layer, maximum 50mm.

2.8 Overlaying and rehabilitation of existing asphalt layers

Roads in Kenya vary widely in their geometric standard and the traffic they carry. They have been constructed and maintained over a period of years; indeed, many have ‘evolved’ rather than been designed and constructed by a formalized process. The range in topography and traffic loading results in roads having a wide range of construction thickness and strength. However, the common ground is that they have a granular road base and either a relatively thin asphalt concrete or surface dressing surfacing. The road network, both in flat and hilly terrain, is also criss-crossed with patches and utility trenches. These often contribute to road deterioration through poor reinstatement.

The present overlay practice is either to mill the existing surface and overlay with 40-50mm (periodic maintenance) of asphalt, or to apply a new surface dressing. This tries to rationalise decisions reached from the gathering of data regarding the road condition. A number of procedures are recommended, namely visual surveys, roughness surveys, FWD, DCP, test pits from which the engineer can choose to investigate the road condition, but the procedures do not all have to be done to decide what maintenance measures need to be carried out.

The proposed empirical overlay design method, described in the Kenyan design manual, is also based upon the AASHTO recommendations (1993) and uses the concept of structural number (SN) to establish the thickness of the overlay. The procedure uses a relationship to convert FWD deflection measurements to the adjusted structural number (SNP) of the existing pavement, allowing designs to be completed quickly and at relatively low cost. The design process envisages the following two levels of survey as in the Figure 2.5.

- Network level surveys, consisting of roughness and visual condition, carried out to demarcate road sections of equivalent condition, followed if necessary by:
- Project level surveys, more detailed in scope, consisting of visual condition, FWD and Test Pit investigations, carried out to determine the level of maintenance required.

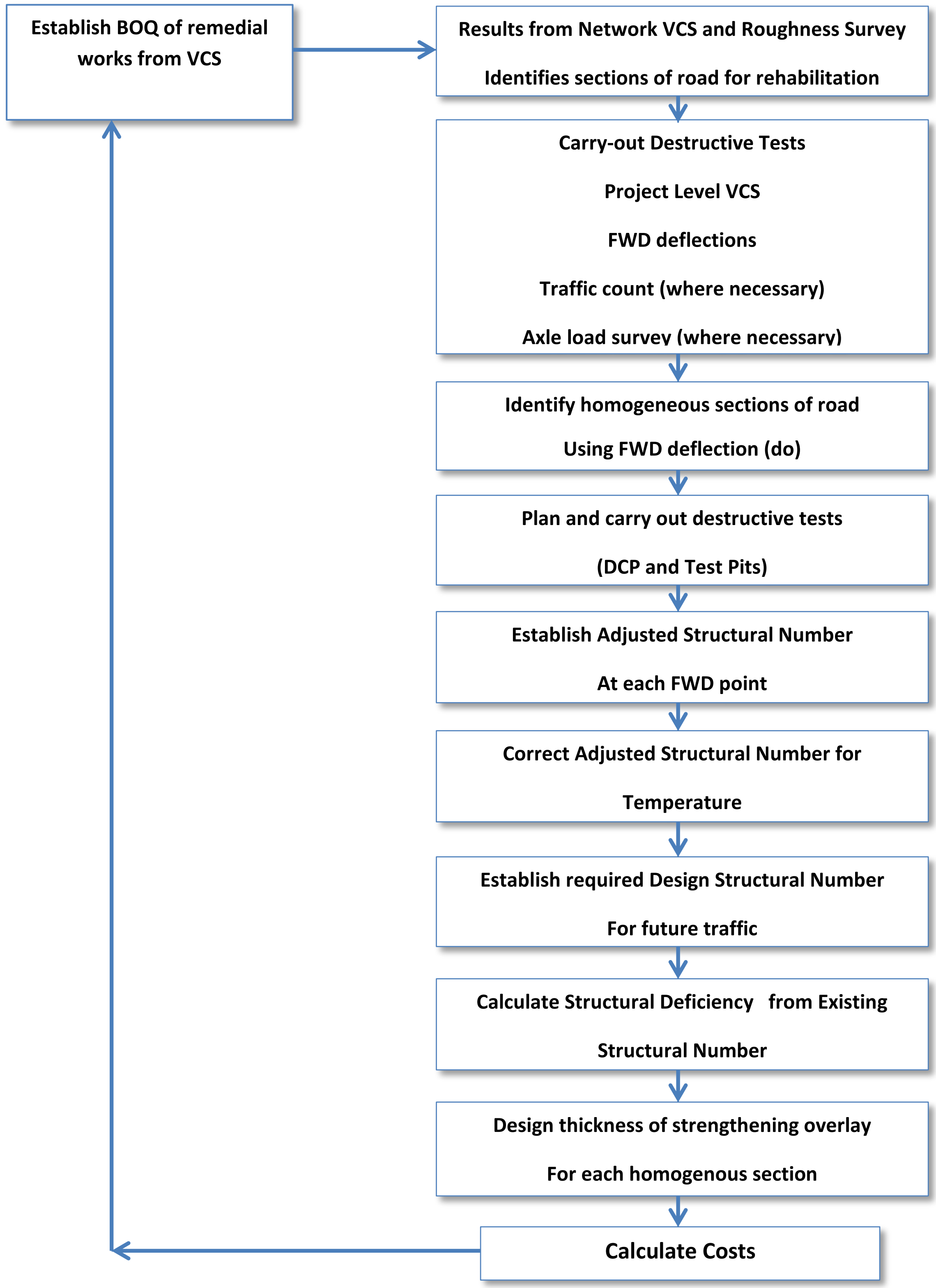


Figure2.5:- Determination of condition pavement and design procedures

Source: - Kenyan design manual for roads and bridges, draft - 2009

2.8.1 Network Level Evaluation

The network level evaluation comprises visual inspection and road roughness investigations in order to sub-divide road sections into the following:

- those where only minimal routine or periodic maintenance is needed,
- those where major treatment, such as reconstruction is needed, and
- those of intermediate condition

Normally, it is only those of intermediate condition where further project-level investigation is needed to decide what maintenance measures to take.

2.8.2 Visual Inspection

Visual surveys are the most basic yet the very useful surveys. The objective of visual inspection is to classify the type and severity of the distress in a measurable manner in order to evaluate maintenance involvements and also to enable the function of performance modelling devices needed.

2.8.3 Roughness Condition Data

Roughness is generally measured using a bump integrator and results are expressed through the international roughness index (IRI), in m/km, or mm/m The bump integrator used must be calibrated with a standard device; an example is the 'Merlin', "Roughometer II" and "Rasor Profilometer" (draft RDM, 2009). Typical values of the IRI with reference to the type and condition of the road are indicated in Table 2.4.

Table 2.4: Roughness criteria

IRI Ranges	Road Condition
0 – 3.0	Excellent (Very Good)
3.00 – 5.00	Good
5.00 – 8.00	Fair
8.00 – 15.00	Poor
>15.00	Very Poor

Source: Kenyan road board, 2015

2.8.4 Falling weight deflectometer (FWD) survey

The FWD is probably the most effective of a series of devices which consist of a standard weight dropped on the road from a known height, causing a reaction from the road, called deflection. Measurement of deflection is a non-destructive means of assessing the performance of the pavement under load. The stiffness of the response provides a means to evaluate the potential of the pavement to carry further multiple load applications.

The falling weight deflectometer (FWD) gives a very precise measurement of absolute deflection, to an accuracy of 2µm (Micro-meter), and has several other good features, as follows:

- The weight applied can be matched to a typical wheel load: normally in surveys the load is standardised at 50kN
- The loading time of the weight is similar to that of a moving vehicle
- The measurements are accurate and absolute, and
- Deflection measurements are taken at the loading point and at selected distances from it: this enables a 'deflection bowl' to be drawn, making it possible to evaluate the particular arrangement of layer stiffness that generated the deflection: this procedure is known as 'back-analysis', or 'back-calculation', and there are various software applications available that can interpret the data. In this document the interest is confined to the central deflection where you get the maximum deflection at the loading point, or D_0.

2.8.5 Deflection Data Analysis and Curvature Indices

i. - Characteristic Deflection, D_{90}

Due to a large number of factors affecting deflection, variation in deflection from point to point should be expected. From experience, distribution of deflections in homogeneous section is expressed by Normal or Gaussian distribution Eq. 2.3:

$D_{90}= d +1.3 \times \sigma$ ……………………………………………...……………Eq. 2.3

Where: - D_{90} - is the characteristic Deflection,

d- is the average deflection for the homogeneous section

σ- is standard deviation for the deflections D_0

The condition of a pavement with respect to the deflection D_{90} to determine the pavement quality index is presented in Table 2.5.

Table 2.5: Pavement quality determination from condition survey & structural deflection

Structural Deflection / **Conditions: Visual Conditions**	**D_{90}< 300µm**	**300µm<D_{90}<600µm**	**D_{90}> 600µm**
Rating	**Sound**	**Warning**	**Severe**
Sound: 1	Q1	Q2	Q3
Warning:2-3	Q2	Q3	Q4
Severe: 4-7	Q3	Q4	Q5

Source: - SAPEM, 2014

Where; Q_i: Pavement Quality Index

ii. - Criteria for deflections D_{90}

The allowable deflection limits and pavement condition rating are as follows in table 2.6 according to South African pavement design method.

Table 2.6 Deflection limits for pavement condition rating

Deflection	**Lower Limit: Low deflection < 300µm**	**Middle Limit : Medium deflection b/n 300-600µm**	**Upper Limit: High Deflection > 600µm**
Bearing capacity	High	Medium	Poor
Pavement condition rating	Sound	Warning	Severe

Source: SAPEM, 2014

iii. - Characteristic radius of curvature (RoC)

The magnitude of deformation is a function of the radius of curvature of the deflection bowl. In effect, the greater the flexural rigidity of pavement the larger the radius of curvature of the deflection shape for a given load and the smaller the strain.

The radius of curvature rather than deflection is the controlling factor and indicates whether a pavement will crack or not. Equation 2.4 shows estimation of RoC from field data.

$$RoC = \frac{L^2}{2Do\left(1-\frac{D200}{Do}\right)} \qquad \text{Eq. 2.4}$$

Where:

L - is 200mm representing the position from the second geophone

D_0 – is deflection at central geophone

D_{200}- is deflection at 200mm away from central geophone

Criteria for radius of curvature:

Sound: $R_OC > 200\text{m}$

Warning: $100\text{m} \le R_OC \le 200\text{m}$

Severe: $R_OC < 100\text{m}$

The manual for rehabilitation of flexible pavement in tropical Africa, suggests traffic class with respect to pavement quality is mentioned in the Table 2.7 below.

Table 2.7: Flexible pavement condition determination

Traffic Class / Pavement Quality	T_5 0.25-1 Million ESAL	T_4 1.5-3 Million ESAL	T_3 3-10 Million ESAL	T_2 10-25 Million ESAL	T_1 25-60 Million ESAL	T_0 >60 Million ESAL
Q1	Routine Maintenance					
Q2	Periodic Maintenance			Pavement Reconstruction		
Q3	Strengthening					
Q4	Overlay / Strengthening			Pavement Reconstruction		
Q5						

Source: SAPEM, 2014

2.8.6 Homogeneous Sections Determination from FWD data

Rehabilitation measures cannot be tailored to each and every variation in road characteristic. To produce cost-effective designs the road should be divided into lengths where the strength properties are similar, known as homogeneous sections. Each homogeneous section is then treated as a separate overlay design exercise. This will result in reduced costs as the overlay thickness changes, reflecting the existing strength of each homogeneous section. This procedure is best carried out by using the cumulative sum method (CUSUM) on FWD central deflection measurements (D_o). The method involves plotting the cumulative sum of the differences of the FWD deflection from the mean FWD value calculated from all the results. The calculations are based on equation 2.5. (RDM, draft 2009)

$$S_i = FWD_i - \mathrm{FWD}_{mean} + \mathrm{S}_{i-1} \quad \text{.. (Eq.2.5)}$$

Where:

FWD $_{mean}$ = Mean FWD deflection of the road

FWD_i = FWD deflection at chainage i

S_i = Cumulative sum of the deviations from the mean deflection

2.8.7 Test Pits (Trial pits)

The test pit data are used to determine the reasons for the weaknesses identified from the FWD investigation, which could include:

- Whether the existing granular base and sub-base meet normally acceptable material standards for partial or full reconstruction.
- Whether the existing granular base and sub-base meet normally acceptable standards for thickness for the appropriate road class.
- Confirmation of the pavement layers identified during DCP analysis.
- To enable mechanistic analysis of FWD measurements
- Test pits will be dug at points in the road where the detailed visual condition survey and FWD deflection profile show the road to abnormally weak.
- The results of these tests should be compared to standard material specifications, listed in the design for new bituminous, gravel and concrete roads. Where the road base and sub-base material do not meet these specifications the length of road affected should be deep patched.

2.8.9 Structural number

The structural number approach is probably the most reliable method of evaluating the 'strength' of pavements of similar type in terms of their likely traffic carrying capacity. According to AASHTO 1993, structural number is calculated from equation 2.6 as follows:-

$$SN = 0.0394 \sum_i a_i h_i \quad \text{(Eq. 2.6)}$$

Where: a_i = Layer coefficient of layer i

h_i= Thickness of layer i (mm)

The calculation of layer coefficients for existing pavement layers is based on the stiffness of bituminous materials and the CBR of granular materials. Layer coefficients for pavement layers in various conditions are presented in Table 2.8.

Table 2.8: Layer coefficients for Surfacing, Base and Sub-base materials

TYPE OF MATERIAL AND CONDITION OF THE LAYER	Material coef. (a_i)
SURFACING:	
Asphalt Concrete (AC), generally un-cracked and with little deformation in the wheel paths	0.4
Portland cement concrete layers, generally un-cracked	
Asphalt Concrete (AC) that exhibits some cracking but with little deformation in the wheel paths.	0.3
Portland cement concrete layers, generally stable but has some cracks, however Containing no pieces smaller than 1 m^2.	
Asphalt Concrete (AC) that exhibit appreciable cracking, with some deformation in the wheel paths, but is essentially stable	0.16
Portland cement concrete layer, deliberately broken into pieces less than 0.5 m across	
BASE COURSE:	
Bituminous layers other than AC, generally un-cracked and with little deformation	0.3
Penetration macadam without infiltration of fines into the layer	0.2
Cement stabilized base course, generally without reflected cracking to the surface	0.18
Cement stabilized base course, with extensive pattern cracking reflected to the surface	0.16
Bituminous layers other than AC, appreciably cracked and with some deformation	0.14
Granular layer of crushed or natural material, PI max 8, CBR min 80	
LOW GRADE BASE COURSE, SUBBASE OR EARTHWORKS LAYERS:	
Fully cracked cemented sub-base or granular layers of natural gravel or with small proportions of crushed particles, CBR min 60	0.12
Natural gravel of nominally sub-base quality, CBR min 25	0.10
Natural gravel in improved subgrade layers, CBR min 10	0.08

Source: Kenya road design manual, 1988

2.8.10 Use of structural number for overlay design

According to Kenyan (draft RDM, 2009), the overlay thickness is derived from equation 2.7:

$$Overlay thickness, mm = \left[\left(SNP_{design} - SNP_{existing}\right)/ a_1 * 25.4\right] \qquad \text{(Eq. 2.7)}$$

Where:

SNP $_{Design}$ = Structural Number for future traffic

SNP $_{Existing}$ = Structural Number of existing road

a_1 = Layer coefficient of asphalt overlay

25.4 = conversion mm to inches

SNP $_{Design}$ values are determined by the AASHTO (1993) design equation

SNP $_{Existing}$ values are based on FWD deflection measurements

Therefore to calculate the thickness of required overlay, the structural number of the existing road (SN $_{Existing}$) has to be measured. There are a number of ways of doing this, all of which have various procedures, requirements and restrictions as enumerated in Table 2.9.

Table 2.9: Methods, procedures and restrictions of determining SN $_{Existing}$

Method	**Procedure to calculate SN $_{Existing}$**	**Requirements**	**Operational restrictions**
Test pits	Direct calculation from thickness and strength (laboratory) of the different pavement layers	Field and laboratory testing	Poor coverage and slow rate of work but very accurate
DCP tests	Direct calculation from estimated thickness and in situ strength of the different pavement layers	Test pits needed to gain information on actual pavement layer thickness and material	Fair coverage, but care must be taken with the use of the equipment
FWD	Back calculation	DCP or test pits needed to establish pavement layer thickness	Good coverage but results need to be calibrated
	Estimate of SNC from FWD deflection bowl (SNP)	-	Good coverage

Source: Kenyan pavement design manual, draft- 2009

The structural number and modified structural number concept, whilst simple in principle, gives rise to a number of practical difficulties, especially on roads that have been in existence for many years. When DCP tests and test pits are carried out, the boundaries between the different materials are sometimes indistinct and differentiating bases from sub-bases, and sub-bases from the subgrade can be difficult. Changes of strength are expected to occur when passing from one layer to another but significant changes of strength also occur within reasonably well-defined layers. When the same pavement is tested with a DCP a more complex, many-layered structure is often revealed.

This can cause a problem in defining the layers in test pits for calculating the modified structural number. The same difficulty also applies when trying to define the appropriate layer thickness for back-analysis of FWD data and often makes this form of analysis somewhat unreliable.

A procedure is therefore required which takes account of the contribution to structural number of a pavement from all the pavement layers and the contribution of the subgrade, which is independent of where the subgrade boundary is defined. This value is called the adjusted structural number (SNP) (Rolt and Parkman, 2000).

2.8.11 Use of FWD to estimate SNP $_{\text{Existing}}$

The most suitable tool to measure the adjusted structural number of an existing road (SNP $_{\text{Existing}}$) is the DCP; its use to design overlays in Kenya is, however, often not ideal. This is because:

- It may not be practicable to take sufficient DCP measurements along each road to cope with the possible high variability found in Kenya, and
- The coarse granular road base in the Kenya roads may prevent the instrument's penetration or make the results unreliable.

The most effective form of the correlation between FWD measurements and SNP takes the form below in equation 2.8:

$$SNP = 1.394 + 4.548 * (d_0 * 0.8)^{-0.5} - 1.760 * \left(\frac{d_{900} - d_{1200}}{d_{900}} \right)^{-0.5}$$

………….. (Eq.2.8)

Where:

d_0 = Central deflection (mm)

d_{900} = Deflection at 900mm from the load (mm)

d_{1200} = Deflection at 1200mm from the load (mm)

(FWD deflection is measured in mm at a load of 50kN)

2.8.12 Overlay design procedure using the FWD

The required overlay thickness is calculated based on a comparison of the strength of the road required for the future traffic and the existing strength of the road, as assessed by FWD measurements.

SNP for future traffic (SNP $_{Design}$)

The first step in the process is to establish the value of structural number (SNP $_{Design}$) that is required for each homogeneous section of road for future traffic loading. This is achieved by using the AASHTO (1993) equation for flexible pavements, as shown in eq. 2.9 below:

$$\log_{10}(W_{8.16}) = Z_R \times S_0 + 9.36 \times \log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.2-1.5}\right]}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 \times \log_{10}(M_R) - 8.07$$

…………………………………………………………………………………………… (Eq. 2.9)

Where:

$W_{8.16}$ = predicted number of 8.16 tonne ESALs

Z_R = Standard normal deviate for required reliability

S_0 = Combined standard error of the traffic and performance predictions

ΔPSI = drop in serviceability over the performance period

M_R = subgrade resilient modulus in psi

SN = structural number to carry $W_{8.16}$ ESALs

The recommended reliability factors and decrease in pavement serviceability index (PSI) used in the equation are shown in Table 2.10 below. The standard deviation is set at 0.49 as recommended by AASHTO (1993). The calculated values of SNP $_{design}$ for various traffic levels, expressed as ESA, are presented in Table 2.11.

Table 2.10: AASHTO Design Criteria: Reliability factors and Serviceability Indices

Road Class	Reliability	Standard Deviation	Terminal PSI	Decrease in PSI
International	90	0.49	2.7	1.5
Primary	90	0.49	2.2	2.0
Secondary	85	0.49	2.0	2.2
Local	50	0.49	1.7	2.5

Source: Kenyan Design Manual, Draft- 2009

Table 2.11: Design SNP

	Future Traffic (Million ESA)						
Road Class	<0.5	0.5–1	1-2	2-5	5-10	10-20	20-50
A	-	-	-	5.68	6.25	6.84	7.67
B	-	-	-	5.22	5.76	6.28	-
C	3.54	3.93	4.32	4.90	5.40		-
Local	2.93	3.25	3.57	4.05	4.45		

Source: Kenyan design manual, draft- 2009

2.8.13 Structural Deficiency

It is necessary to plot the 'structural deficiency', that is the difference between the required design structural number of the road (SNP $_{Design}$) and the existing structural number at each FWD test (SNP $_{Existing}$), for each FWD test. This is expressed in equation 2.10:-

$$StructuralDeficiency = SNP_{design} - SNP_{existing} \quad \text{…………………. (Eq.2.10)}$$

After calculation the structural deficiency is plotted as a bar chart, which allows the identification of actions for the homogeneous sections based on the criteria given in Table 2.12.

Table 2.12: Structural Deficiency Criteria

Mean Structural Deficiency	Action	Notes
Zero or negative	Maintain	A thin overlay may be required to correct other defects
0 to 0.6	Thin overlay	Remedial works possible
0.6 to 1.5	Thick overlay (40/50mm)	Remedial works probable
> 1.5	Reconstruction probable	

Source: Draft Kenyan pavement design manual, 2009

2.9 Current pavement design techniques

The normal design procedures require determination of equivalent standard axle load (ESAL) of a road. This enables determination of the appropriate traffic class. From materials investigation the corresponding subgrade classification is determined. Then the alternative pavement structure options are shortlisted. Using the current price as a unit cost for each pavement alternatives, among all selected options being computed and the least cost (construction cost) will be chosen. The availability of the material in the area will be considered as a main factor to select the option. But if all the listed options are available in the area, relatively the least construction cost be given a priority and be picked among all alternatives. There are a number of alternative approaches to the design of pavement structures. Pavement design methods generally fall into the following four categories (TAC 2011):

1. Standard sections

2. Empirical pavement design methods

3. Mechanistic pavement design methods

4. Mechanistic-empirical pavement design methods

i. Standard section pavement design methods: - In this method an appropriate pavement design for given set of design conditions based on experience of past performance is selected. The primary limitation of these methods is that they are only applicable to the specific set of conditions under which they were developed (TAC 2011).

ii. Empirical pavement design methods are based solely on the results of experiments or experience. Observations of pavement responses to known traffic loading and subgrade conditions are used to establish correlations between pavement design inputs and pavement performance. The primary advantage of empirical methods is that they avoid the issue of defining theoretically the complex cause-effect relationship between pavement design and observed pavement distresses. The primary disadvantage of empirical pavement design methods is that the validity of the relationships is limited to the conditions under which they were observed. Extrapolating these relationships to other conditions requires assumptions that may undermine the accuracy of the method. The most commonly used empirical method for designing new and rehabilitated pavements in Canada and the United States is the AASHTO 1993 guide for design of pavement structures (AASHTO 1993, TAC 2011).

iii. Mechanistic pavement design methods use the theories of engineering mechanics to relate traffic loading and environmental conditions to pavement structural behavior and performance. In mechanistic methods stresses, strains, and deflections at critical points in the pavement structure based on specified traffic loading and environmental conditions are determined. The pavement structure is modeled as a multi-layered linear elastic system to capture the dynamic responses of the various pavement materials. One disadvantage of mechanistic pavement design models is that they are strictly theoretical and do not incorporate observed pavement performance in the field. In addition, the assumption of linear-elastic material behavior is generally incompatible with the prediction of nonlinear inelastic pavement distresses (Carvalho & Schwartz 2006). Since pavement performance is defined in terms of pavement distresses and not pavement structural responses, this is a significant limitation of purely mechanistic pavement design methods. For these reasons, attempts to develop fully mechanistic pavement design approach have generally been unsuccessful. (Carvalho et al. 2006)

iv. Mechanistic-empirical pavement design methods afford the advantages of mechanistic pavement design while addressing its primary limitations. The mechanic component of the model calculates pavement structural responses (i.e. stresses, strains, deflections) resulting from traffic loading, environmental conditions, and material properties. These pavement responses are then related to pavement performance through the use of empirical pavement distress prediction models. The empirical distress prediction models are developed and calibrated using observed pavement performance in the field. The most comprehensive mechanistic-empirical pavement design method is the mechanistic-empirical pavement design guide (MEPDG), which was developed under NCHRP Project 1-37A (ARA, 2004).

The mechanistic empirical pavement analysis software (mePADS) from South Africa contains a mechanistic-empirical design method, using layered elastic theory combined with a transfer function that is an adaptation of the mechanistic empirical pavement design guide (MEPDG) of AASHTO 2002 and 2007 versions. The pavement structure capacity analysis consists of the following input parameters:

Pavement structure

The 'Pavement structure' worksheet contains the following input boxes for defining the pavement system:

a. Number of layers: defines the unique layers in the pavement structure. A maximum of 5 layers can be defined.
b. Material: Refers to the type of pavement material, according to the South African material classification in TRH4. Select the material type from the drop-down list.
    i. AC: Continuously graded asphalt surfacing
    ii. AG: Gap graded asphalt surfacing
    iii. C1 – C4: Lightly cement treated materials
    iv. G1 – G6: Granular materials
    v. EG4 – EG6: Equivalent granular materials
    vi. Soils: In-situ or imported subgrade material
    vii. BC: Asphalt bases
c. Thickness: Layer thickness in mm. A rigid layer will be assumed to exist at the bottom of the last layer, unless a value of zero is specified, in which case the rigid

layer will be assumed to exist at 1000 mm below the defined pavement. No provisions have been made for semi-infinite pavements.

d. E-modulus: The modulus of elasticity of the selected material in MPa. Suggested value will be displayed as default, when the material type is selected.
e. Number of phases: defines the number of design phases to be considered in the analysis, as a result of the multi-phase nature of cemented materials. The number of phases in the analysis will be automatically selected depending on the number of cemented layers in the structure. This may be changed if a different number of phases in the analysis are required. Remember to also provide the material codes, E-moduli and Poisson's ratio for each of the phases.
f. Climatic region: Refers to rainfall region.
g. Road category: Defines the design reliability:
    i. A : 95 % reliability
    ii. B: 90 % reliability
    iii. C: 80 % reliability
    iv. D: 50 % reliability
h. Terminal rut: Failure rut-depth criteria for Subgrade rutting.
i. Design traffic class (in standard axles)
    i. ES0.003: 0 to 3 000
    ii. ES0.01: 3 000 to 10 000
    iii. ES0.03: 10 000 to 30 000
    iv. ES0.1: 30 000 to 100 000
    v. ES0.3: 100 000 to 300 000
    vi. ES1: 300 000 to 1 000 000
    vii. ES3: 1 000 000 to 3 000 000
    viii. ES10: 3 000 000 to 10 000 000
    ix. ES30: 10 000 000 to 30 000 000
    x. ES100:30 000 000 to 100 000 000

Loads and Evaluation Points:

Design location: The point at the pavement surface where the pavement design is to be carried out.

Load definition: The number, magnitude (kN & kPa) and position of wheel loads. At least 1 load must be defined.

Stresses and strains: The location in the pavement for evaluating stresses and strains. This analysis will be done independently from the bearing capacity analysis and the results are reported on the "Stresses and Strains" worksheet.

Load position plot: shows a plan view of the loads defined in the system. Press the update plot button to refresh the plotted loads.

Design parameters:

The stress and strain parameters at critical points in the pavement are displayed on this worksheet. These parameters are used in the bearing capacity calculations. The parameters and critical points vary for different material types as follows:

Asphalt layers: The horizontal tensile strain at the bottom of the layer controls the fatigue life of the layer.

Cemented layers: The horizontal tensile strain at the bottom of the layer controls the fatigue life of the layer, while the vertical compressive stress at the top of the layer defines the crushing life.

Granular layers: The principal stresses at the middle of the layer controls the shearing capacity of the layer.

Soil (subgrade) layers: The vertical compressive strain at the top of the layer controls the rutting life of the layer.

The output parameters:

The worksheet displays the main design outputs of the software. The worksheet will only become visible once a successful design has been completed after the Calculate button has been clicked

Layer bearing capacity: The bearing capacity (in terms of the defined load) of the layers at the selected design reliability. The design traffic class (in terms of standard axles) is also shown as lines on the bar chart. The bearing capacity is calculated using transfer functions, specially formulated for the material type. Certain materials, such as asphalt, have various transfer functions depending on the thickness and grading.

Approximate pavement life distribution: The distribution of pavement lives obtained by varying the design reliability input in the transfer functions.

Crushing cemented layers: The bearing capacity of the cemented layers with respect to failure by crushing.

Cemented life: The effective duration of the cement phasing out of the cemented layer.

Calculation Table:

Provides the transfer function outputs for a selected design reliability. This functionality is provided so that detailed information on the calculation procedure can be viewed. Select the desired reliability level and view the results in the table.

Contour plot:

It provides a contour plot of the selected stress or strain parameter for an area in the pavement, on a vertical or horizontal plane. The desired plot region, plane and parameter can be selected, and clicking on the plot button will generate the plot.

v. - AASHTO 1993 flexible pavement design

The AASHTO 1993 flexible pavement design method is based fundamentally on the relationship between traffic loading, subgrade strength, and the functional performance of the pavement. The AASHTO 1993 method uses equation 2.9 of section 2.8.12 above for the design of flexible pavements.

To complete a flexible pavement design using the AASHTO 1993 method, the pavement designer must first determine the representative resilient modulus of the underlying subgrade materials (MR). This can be determined either directly through laboratory testing of representative samples of subgrade material, or assumed based on soil classification and anticipated drainage conditions. The designer must also determine the cumulative traffic loading experienced over the performance period of the pavement (E80). The AASHTO 1993 method characterized traffic loading in terms of number of equivalent single axle loads (ESALs). An ESAL represents the damage experienced by a pavement structure as a result of loading from an 8,160 kg single axle. All traffic loading from a mixed stream of traffic of different axle loads and axle configurations predicted over the design life of the pavement is converted into an equivalent number of ESALs for design. The designer must also select a suitable value for design reliability. Reliability represents the probability that the pavement design will meet or exceed its design life, and is typically based on the highway functional classification and the risk associated with premature failure of the pavement. Finally, the designer must select the deterioration rate in terms of loss of serviceability (∆PSI). The AASHTO 1993 design method characterized pavement performance solely in terms of functional performance as measured using the pavement serviceability index (PSI). Equation 2.11 below is used to calculate PSI (TAC 2011):

$$PSI = 5.03 - 1.91\log(1 + SV) - 0.01(C + P)^{0.5} - 1.38RD^2 \ldots\ldots\ldots\ldots\ldots \mathrm{Eq.}\ 2.11$$

Where:

SV = longitudinal cracking in the wheel path
C = cracked area
P = patched area
RD = average rut depth for both wheel paths

As shown in equation 2.11 above, PSI is a composite performance measure that is influenced primarily by pavement roughness. The selection of suitable initial and terminal serviceability values is typically dependent on highway functional class and local agency policy. The output of the AASHTO 1993 flexible pavement design method is a structural number (SN) required for the pavement to function adequately over the design period at the specified level of reliability. The pavement SN is related to pavement layer thicknesses and drainage conditions using equation 2.12 (AASHTO 1993):

$$SN = a_1 D_1 + a_2 m_2 D_2 + a_3 m_3 D_3 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.\ldots\ldots Eq. 2.12$$

Where:

a_i= structure layer coefficient (e.g. 0.42 asphalt, 0.14 granular base, etc.)

m_i= drainage layer coefficient (e.g. 1.0 good drainage, 0.9 fair drainage)

D_i = layer thickness

The designer must select the individual pavement layer thicknesses to satisfy the required SN with consideration to producing a cost-effective design. According to AASHTO 1993, there are minimum requirements for the pavements that need to be looked into for the specific range of cumulative traffic load which is stated in table 2.13 below.

Table 2.13: Minimum Thickness (inches)

Traffic, ESAL's	Asphalt Concrete	Aggregate Base
Less than 50,000	1.00 (or Surface Treatment)	4
50,001 – 150,000	2.00	4
150,001 – 500,000	2.50	4
500,001 – 2,000,000	3.00	6
2,000,001 – 7,000,000	3.50	6
Greater than 7,000,000	4.00	6

Source: AASHTO, 1993

AASHTO, 1993 has outlined guidelines for the lengths of analysis period of different condition of roads (Highways) have been mentioned in Table 2.14 below.

Table 2.14: Guidelines for length of analysis period

Highway Conditions	Analysis Period (years)
High-volume urban	30 - 50
High-volume rural	20 - 50
Low-volume paved	15 - 25
Low-volume aggregate surface	10 - 20

Source: AASHTO, 1993

2.10 Work done outside Kenya similar to related subject

A) - Comparisons of AASHTO 1993 and MEPDG Pavement Designs

Since to the completion of NCHRP Project 1-37A in 2004, a number of studies have directly compared pavement structures obtained using the AASHTO 1993 and MEPDG pavement design methods. This section provides an overview of their key findings.

Carvalho et al. (2006) compared flexible pavement designs and performance between the NCHRP 1-37A method (M-E version 0.700) and AASHTO 1993. Flexible pavement designs were completed for five locations selected to be representative of the range of climates, subgrades, material properties, and local design preferences in the United States. All of the flexible pavement designs consisted of asphalt layers over granular base layers. Three traffic loading scenarios were examined: low (3.8 million ESALs), medium (15 million ESALs), and high (55 million ESALs). A design reliability of 95% was used for both the AASHTO 1993 pavement designs and MEPDG pavement performance predictions. Carvalho et al.(2006) correctly noted the difficulty of a direct comparison of the two methods due to the disparity in the number and detail of design inputs required between the two methods, the dependence of design thickness on specified design criteria, and the ability for multiple designs to satisfy the same performance criteria. Instead of design thickness, this study examined the M-E predicted performance of flexible pavements designed using the AASHTO 1993 method based on the same loss of serviceability. It was assumed that the flexible pavements designed for the same loss of serviceability with the AASHTO 1993 method should exhibit similar predicted performance in the ME models, and any discrepancies would be indicative of one design method being more conservative than the other. It was also assumed that the M-E design method was the more accurate of the two based on the extent of its national calibration. Based on the above, the following conclusions were reached:

- The AASHTO 1993 method underestimated rutting and bottom-up fatigue cracking (i.e. overestimated performance) for flexible pavements in warm climates.
- The AASHTO 1993 method underestimated rutting and bottom-up fatigue Cracking (i.e. overestimated performance) for flexible pavements with High traffic loading (i.e. 55 million ESALs).
- The AASHTO 1993 designs were less reliable at higher traffic levels as

demonstrated through increased variability in predicted pavement distresses.

•The AASHTO 1993 designs had low variability in predicted pavement distresses for pavements with low traffic loading and low to moderate temperatures.

B) Comparison of Ontario pavement designs using the AASHTO 1993 empirical method and the mechanistic-empirical pavement design guide method.

Jonathan N. Boone (2013) conducted a comparative analysis of Ontario (Canada) structural pavement designs using the AASHTO 1993 guide for design of pavement structures and the mechanistic-empirical pavement design guide. Historical flexible, rigid, and asphalt overlay pavement designs completed using the AASHTO 1993 pavement design method for the MTO were evaluated using a two-stage procedure. First, the nationally-calibrated MEPDG pavement distress models were used to predict the performance of the pavements designed using the AASHTO 1993 method. The purpose of this stage of the analysis was to determine whether the two methods predicted pavement performance in a consistent manner across a range of design conditions typical of Ontario. Finally, the AASHTO 1993 and MEPDG methods were compared based on the thickness of the asphalt concrete or Portland cement concrete layers required to satisfy their respective design criteria.

The results of the comparative analysis demonstrate that the AASHTO 1993 method generally over-predicted pavement performance relative to the MEPDG for new flexible pavements and asphalt overlays of flexible pavements. The MEPDG predicted that most of the new flexible pavements and asphalt overlays of flexible pavements designed using the AASHTO 1993 method would fail primarily due to permanent deformation and / or roughness. The asphalt layer thicknesses obtained using the MEPDG exceeded the asphalt layer thicknesses obtained using the AASHTO 1993 method, and a poor correlation was observed between the asphalt layer thicknesses obtained using the two methods. Many of the new flexible pavements and asphalt overlays of existing flexible pavements could not be re-designed to meet the MEPDG performance criteria by increasing the asphalt layer thickness.

2.11 Work done in Kenya with a related subject

A)-Structural behavior of flexible pavements

Long term studies on flexible pavements in Kenya by Gichaga (1979) suggest an increase in pavements strength (assessed on the basis of 9 deflections) with pavement age and a decrease in strength with cumulative traffic loading. Higher pavement deflections during periods of high rainfall and temperatures are also reported. The author recommends the regular monitoring of factors affecting performance in order to facilitate proper financial planning for pavement strengthening and routine maintenance (Gichaga, 1979).

B)-Performance study on flexible road pavements in Kenya

Murunga (1983) did a research on the performance of flexible road pavements in Kenya. The research set out to evaluate the performance of some six road test sections located on in-service bitumen standard roads in and around Nairobi, Kenya. The study found out that pavement age, traffic and climate were some of the major factors affecting pavement performance. Evidence was found to suggest that for cracked pavement sections, rebound deflections provided reliable indications of pavement weaknesses. This relationship was however found not to hold in the case of rutting. One of the recommendations of this study was the need for further research on the compaction characteristics of subgrade soils in Kenya.

C) - Deflection characteristics for flexible road and airport pavements in Kenya

Mwea and Gichaga (2004), reported in a paper presented in The 8th Conference on asphalt pavements for Southern Africa on the basis of research which they were able to relate the magnitudes of deflections induced by axle loads on a pavement structure with the pavement structural condition. As traffic traverses a flexible pavement, the axle loads induce a downward deflection of the pavement surface. This downward deflection was measured by tracing the profile of the surface behind loaded wheels of a vehicle moving at creep speed.

D) - Cost comparison of concrete versus flexible pavement designs for steep to rolling sections along A104 road (Nakuru – Eldoret)

Kipyator (2013) , seeks to carry out both a comparative design and costs for a concrete and flexible pavement in Northern corridor A104 highway at Timboroa, with a focus on steep to rolling sections after observations indicated that the steep to rolling sections deteriorate faster than the sections on flat terrains. The study covers the design of both a concrete and flexible pavement and computation of respective costs over a study period of forty years for the section, to inform whether it will be cheaper to introduce concrete pavement on the steep to rolling section or retain the flexible pavement and concluded that in the long-run concrete road is less expensive than flexible pavement.

E) - The study of pavement design of Nairobi-Thika highway

Temu (2012) involved in a study of both alignment soils and axle load data of the Nairobi-Thika (A2) road. The objectives of his study were to establish the variation of engineering properties of soils with depth particularly at the deep cut sections, to establish axle load data for the Nairobi-Thika Road and to compare year 2011 axle loading with the 1971 axle load and come out with a conclusion that within 40 year the axle load data has increased by almost 4 times.

Summary

The study road had done the alignment soil for determination of subgrade classes. From the FWD and IRI values homogeneous sections have been determined. Visual condition survey, FWD and roughness data gave the highlight about the condition of pavement.

Murunga (1983), studied on the performance of flexible pavements in Kenya and suggested there is a need for further research on the compaction characteristics of subgrade soils, Kipyator (2013), dealt with cost comparison of concrete versus flexible pavement design along A104 road (Nakuru - Eldoret) at Timboroa and established that on the long-run or 40 years later concrete road is less expensive than flexible pavement and Temu (2012), compared axle load data for the year 2011 & 1971 and came out with a conclusion that within 40 years the axle load data has increased by almost 4 times.

So many researches have been done on subgrade and flexible pavement but comparing the design methods was not tried and the researcher is going to fill the gap which design method is more economical for a flexible pavement and give us a sustainable structural pavement layer.

The study had considered into the Kenyan road design manual (1987), (Draft, 2009) - Part III & IV, and Tanzanian pavement manual (1999), empirical pavement design method (AASHTO 1993) and mechanistic-empirical pavement analysis methods (mePADS) were used on the data analysis.

CHAPTER THREE

METHODOLOGY AND DATA COLLECTION

3.1 Introduction

This section shows how the data was obtained, analyzed and presented. The data was obtained from the site and related institutions.

Some of the data sources were from:-

- Raw data from the field
- Ministry of transport and infrastructure
- Kenya national highways authority
- Consultants (Intercontinental consulting technocrats & runji and partners consulting engineers)

3.2 Overview of the Study

The traffic survey data was collected in in the field on 2016 in association with consultants ICT (Intercontinental Consulting Technocrats), Runji & Partners Consulting Engineers as well as the materials data was collected in association with ministry of transport and infrastructure due to the availability of the necessary laboratory machineries.

3.3 Research design

After having looked into all methods of design by mechanistic or empirical, then the research focused on design technique which can really give an alternative good options either economically viable road or structural sustainable. The design methods will address the challenges we are having with new and rehabilitation roads, stress, strain and deflection developed by the heavy vehicles in the design period. The data was fed to the software is in accordance to Kenyan provision for flexible pavement taking in to consideration all the embodied (default) data for the software from South Africa.

The flow chart of the mePADS software in the Figure 3.1 shows the steps in the analysis. The model details highlight how the software works. The interpretations have been explained under section 2.12(iv) above.

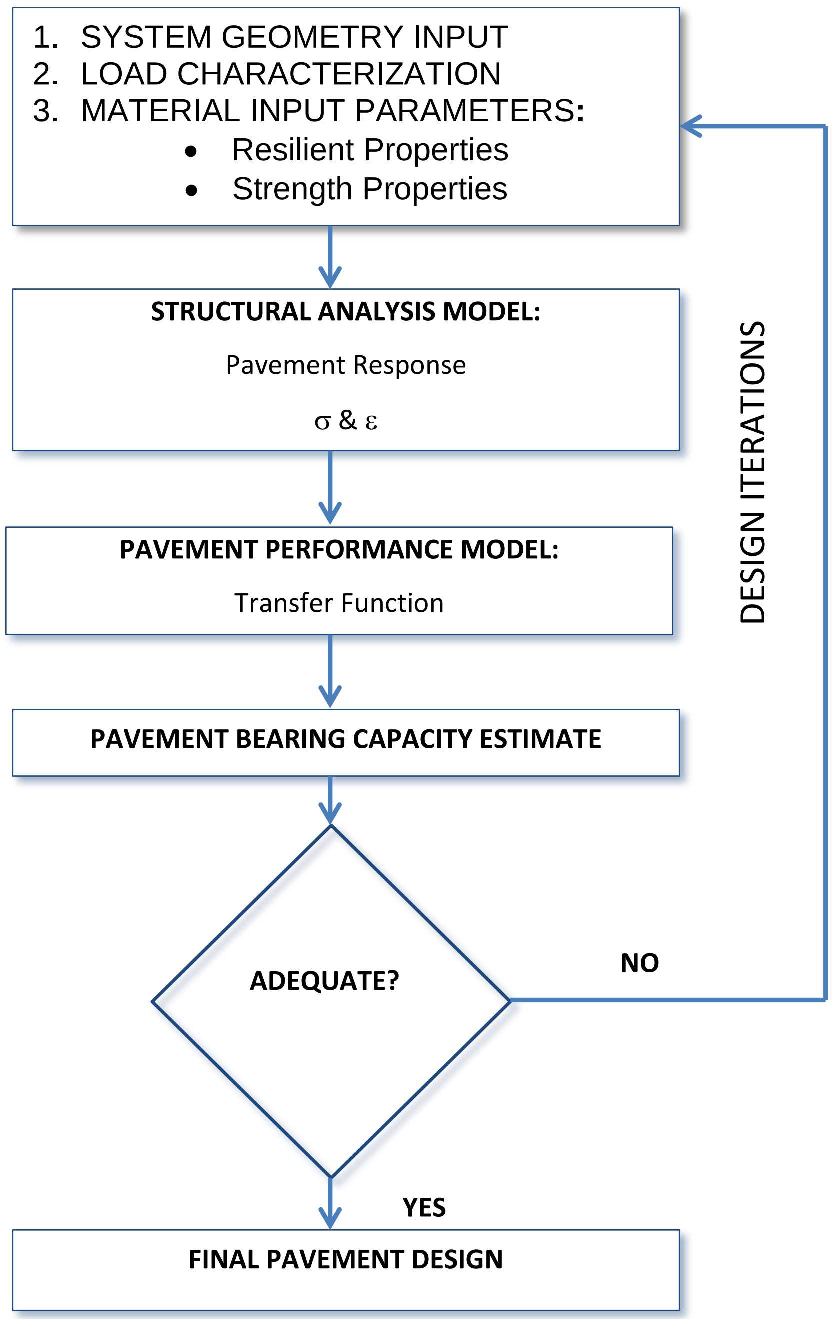


Figure 3.1: Flow diagram for a South African mePADS

Source: H L Theyse and M. Muthen, 1996

3.4 Data Collection

The traffic survey data was collected at the junctions of Nakuru, Bahati junction, Subukia and Nyahururu. The alignment soil was collected according to Kenyan design manual and standard specification at an interval of 500 meters. For the non-destructive tests FWD (falling weight deflectometer) was done with an interval of 100 meters in a staggered way from both directions. Roughness (Bump integrator) was also done with an interval of 100 meters in a staggered manner from both directions. The data collection was done in conjunction with the crew from ministry of roads and infrastructure materials department and all the data are very consistent and reliable.

Manual traffic count was carried out with trained enumerators. Historical data which has been done previously on the road in 2012 on B5 road a stretch from Nakuru up to Nyahururu was collected.

The traffic count was used for the following purposes:

- To know the traffic at the time of survey
- To predict traffic in the future
- To determine the cumulative equivalent axle load

According to Kenyan road design manual (RDM, 1987), the classification of vehicles is mentioned as follows in table 3.1:-

Table 3.1: Vehicles classification

Private cars	**: (cars) are all passenger motor vehicles seating not more than nine persons; including the driver.**
Light Vehicles	:are all goods vehicles of not more than 15kN un-laden Weight
Medium Goods Vehicles	: are all two-axle goods vehicles of more than 15kN un-laden Weight
Heavy Goods Vehicles	: are all goods vehicles having more than two axles.
Buses	:are all passenger motor vehicles seating more than 9 Persons, including the driver
Commercial Vehicles	: include buses and goods vehicles of more than 15 kN Un-laden weights.

Source: Kenyan road design manual, 1987

Historical data from KeNHA, 2012 on B5 had been analyzed, and from the analysis there was a tremendous increase in traffic growth along the study road as in table 3.2.

Table 3.2 Historical data for B5 road

HISTORICAL DATA FOR B5 (2012/2013)										
Segmented Traffic on B5 road		Average Daily Traffic Flow by Vehicle type (2012/2013)								
No	From - To	Car	(LGV)	Matatu	M. Goods	M. Goods Tanker	H. Goods	H. Goods Tanker	Bus	Total
1	MARUA-NYERI	1262	1677	397	178	7	38	5	35	3599
2	NYERI – NYAHURURU	309	623	288	83	4	13	1	32	1353
3	NYAHURURU - NYERI	430	523	502	168	11	20	8	25	1687

Source: KeNHA, 2012/2013

Due to erratic traffic growth in Kenya and unprecedented future traffic growth that may come due to the upgrading of Isiolo - Moyale and LAPSSET project corridor it was very difficult predict the real traffic growth rate in the area. For the purpose of calculation, the Kenyan economic growth rate GDP obtained from IMF and Kenyan national bureau of statistics (KNBS) has been presented in table 3.3.

Table 3.3: GDP growth of Kenya starting 2004 - 2015

Year	**2004**	**2005**	**2006**	**2007**	**2008**	**2009**	**2010**	**2011**	**2012**	**2013**	**2014**	**2015**
GDP	4.9	5.7	6.1	7.0	1.5	2.7	8.4	6.11	4.56	5.69	5.33	5.59

Source: IMF and KNBS, 2015

From the table above the average GDP economic growth rate is 5.1%. A growth rate of 5% was adopted.

3.5 Material and Pavement Investigation

3.5.1 Alignment Soil Investigation

The alignment soil collected was sent to the ministry of transport and infrastructure laboratory for engineering characteristics testing. The subgrade field property tests are summarized in the Table 3.4.

Table 3.4 Test results of Subgrade

Date Tested	Location	Right Way	Layer Thick (mm)	FDD Kg/m^3	Field Moisture (%)	Max.Dry Density Kg/m^3	OMC (%)	Subgrade DCP-CBR (%)
25.11.16	3+000	RHS	185	1361	11.8	1651	13	19
25.11.16	6+000	LHS	160	1404	19.6	1427	22.6	14
25.11.16	9+380	RHS	173	1109	34	1107	34	13
25.11.16	12+580	LHS	150	1335	25	1307	24.7	28
25.11.16	17+550	RHS	160	1453	21	1286	15	32
25.11.16	22+250	LHS	155	1205	19	1200	18	12
25.11.16	25+880	RHS	150	1159	33.3	1262	13.5	5
20.11.16	28+850	LHS	165	1450	20	1512	20	37
25.11.16	32+350	RHS	156	1460	18	1512	14.5	38
25.11.16	35+200	LHS	150	1372	23.3	1368	25.2	30
25.11.16	40+400	RHS	170	1459	24	1489	16	34
25.11.16	55+820	LHS	151	1389	25.5	1389	25.1	40
25.11.16	59+000	RHS	150	1089	29.8	1192	31.4	36
25.11.16	60+000	LHS	149	1352	23.4	1313	23.2	38

Source: Author, 2016

Soil classification according to AASHTO and RDM, 1987 as well as other engineering properties of soil were explained on table 3.5 & 3.6 respectively.

Table 3.5 AASHTO soil Classification

AASHTO soil classification	A-6	A-2-6	A-2-7	A-7-5	A-7-6
No. of samples	2	6	4	1	1
Range of LL (%)	37	35-40	37-38	61	51
Range of PI (%)	18	17-20	17-19	30	25
MDD (g/cm^3)	1.842	1.94-1.99	1.845-1.905	1.65	1.830
OMC (%)	16.50	10.9-12.0	12.9-14.7	19.80	16.90
Range of CBR (%)	13.0	48-56	14.0-19	5.20	12.20
Linear Shrinkage %	9.0	9.0-10.0	9.0-10	15	13.0

Source: Author, 2016

Table 3.6 Subgrade soil class according to Kenyan RDM, 1987

Kenyan design manual classification	S2	S3	S4	S5	S6
No of samples	1	2	1	2	8
Range of CBR	2.0-5	10.0-13	14-18	19-30	>30

Source: Author, 2016

The alignment soil on the project road with their respective CBR values in (%) and the corresponding chainage in (Km) were presented in the figure 3.2 below.

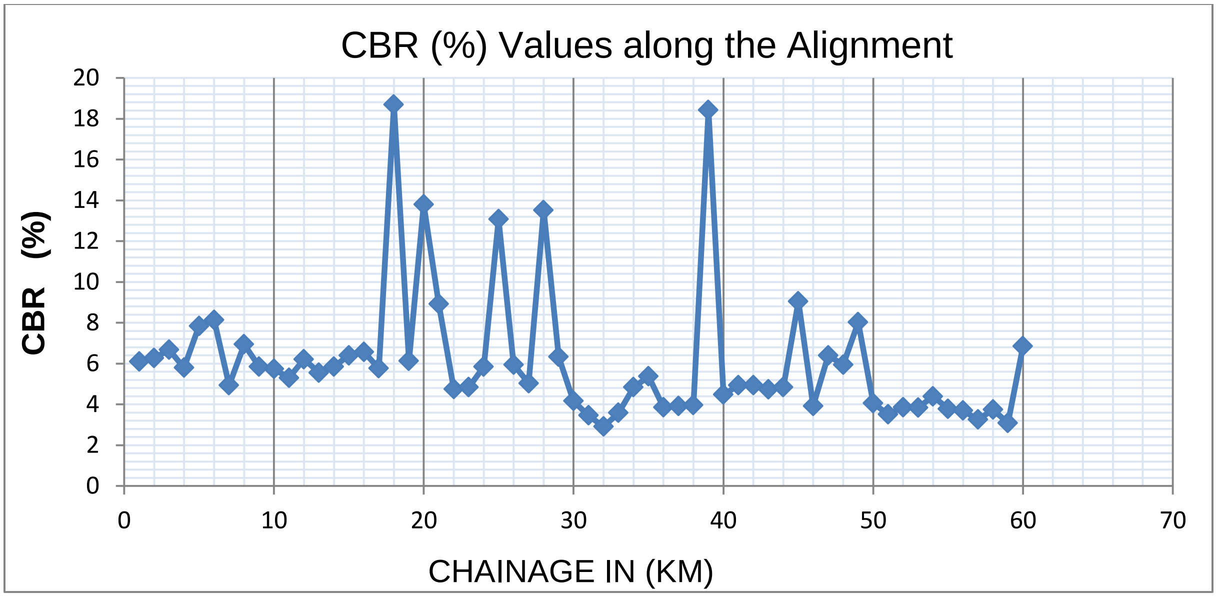


Figure 3.2: CBR value on the alignment soil from Nakuru – Nyahururu

Source: Author, 2016

Also the CBR values of some of the borrow pit subgrade soils on the road section are indicated in the figure 3.3 below.

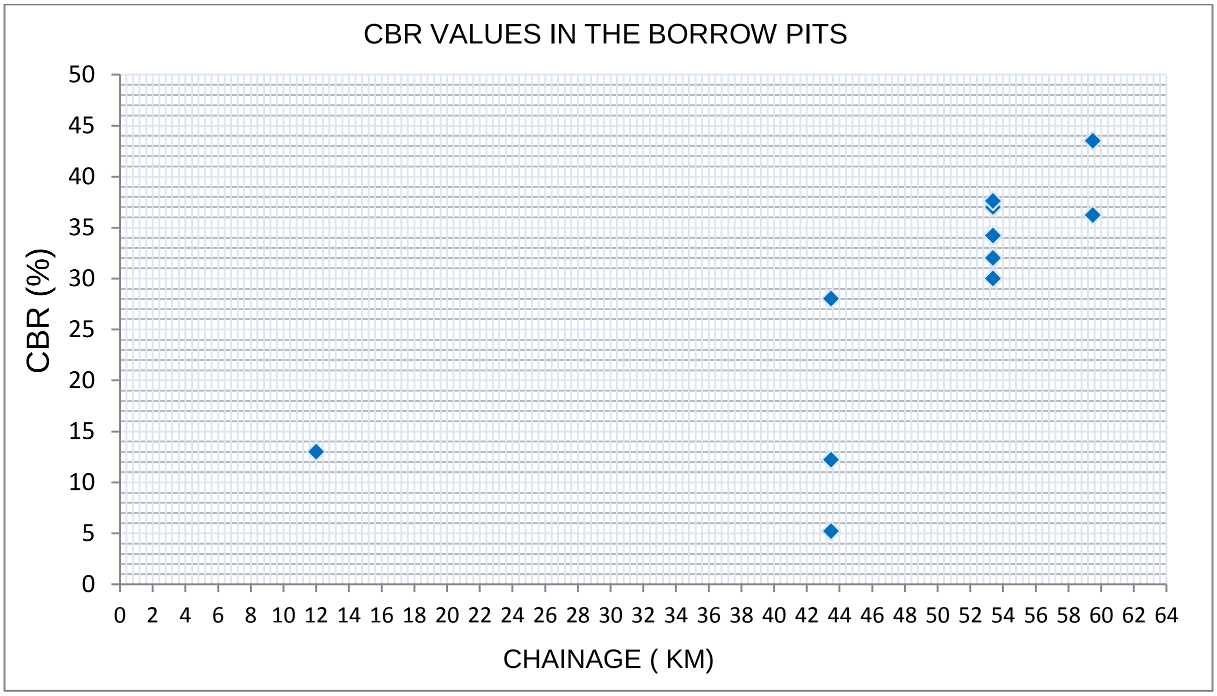


Figure 3.3: CBR value of borrow pits

Source: Author, 2016

3.5.2 Pavement investigation

According to visual inspection survey done on the project road; the study road had developed a pot holes, rutting, raveling and edge break will be seen in most part of the road. The pavement composition of the road was shown in Figure 3.4.

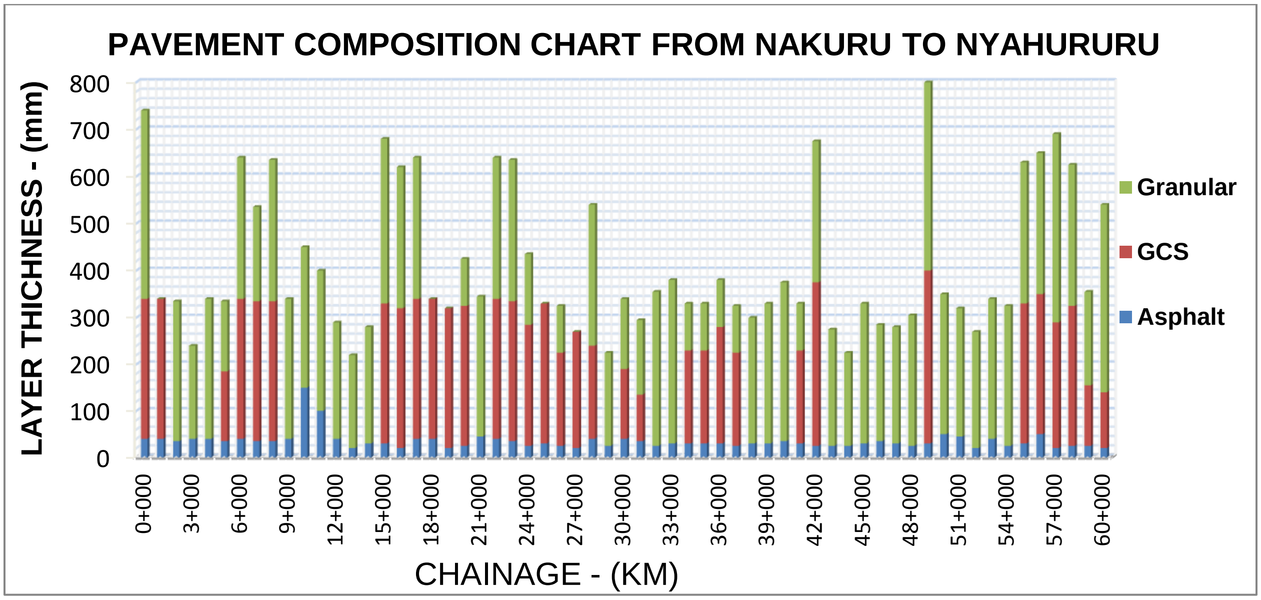


Figure 3.4: Pavement composition chart of the road from Nakuru – Nyahururu

Source: Author, 2016

3.5.3 Pavement surface defects

The rating of damage noticeable on the carriageway and shoulders was based on two criteria, namely severity of distress and extent of occurrence. In order to categorize the severity of defects found along the road, the rating system as set out in the CEBTP-LCPC, manual for rehabilitation of flexible pavement in tropical countries was used as the basis. This is a supplement to the TRL overseas road note 18 (ORN). In the rating of the level of distress on the pavement, severity of damage was expressed below.

3.5.4 Project Condition Survey

i- Detailed Visual Condition Survey

The project visual condition survey (VCS) is more detailed than the network survey, covering the whole of the road. It is carried out on foot, during the survey each sample length of (5, 10 or 20 metres) of the road being examined to identify defects in the wheel-paths.

A. Evaluation of Existing Pavement

(i) Minor Surface Distress And Defects:

a. Bleeding
b. Raveling or stripping (Aggregate loss)
c. Longitudinal and traverse cracks
d. Disintegration

(ii) Deformation:

a. Rutting:
b. Rutting without cracks
c. Shoving
d. Corrugations

(iii) Structural Distress:

a. Alligator (fatigue) cracking or crocodile cracking
b. Depressions or settlements
c. Rutting with cracks

(iv) Edge Breaks And Shoulder Wear

B. Pavement Surface Defects:

In the rating of the level of distress on the pavement, severity of damage has been expressed in three (3) levels as follows:

S1: Sound;

S2: Warning;

S3: Severe;

In adopting the above basis, it is to be noted that the levels referred to differ quantitatively for various defects.

Similarly, the extent of occurrence has been distinguished in three levels as follows:

①: <10% of distress

②: 10%-50% of distress

③: >50% of distress

Subsequently, the significance of the various levels of damage noticeable on the existing pavement has been derived from a combination of the extent and severity of the recorded defects thereon.

Criteria for Rut depth:

Rut Depth <10mm: Sound

10mm<Rut Depth<20mm: Warning

Rut Depth>20mm: Severe

The rating, of damage on the pavement has therefore been determined from a five point scale presented in the table 37.

Table 3.7: Damage Rating:

Cracks Deformation / **Rut Depth**	①	②	③
S1(Sound)	1	2	3
S2 (Warning)	3	4	5
S3 (Severe)	5	6	7

Source: SAPEM, 2014

A surface condition after combination of surface defects and deformation is defined as follows in table 3.8:

Table 3.8: Surface Conditions:

Visual Conditions	**Descriptions**	**Conditions Index**	**Surface condition rating**
Good	Few or without Cracks	1	Sound
Fair	Cracks without Rutting	2-3	Warning
Poor	Cracks and Rutting	4-5	Severe
Very poor	High Rutting and Cracks	6-7	

Source: - SAPEM, 2014

3.5.5 Roughness (Bump integrator)

The roughness study was carried out in order to determine the international roughness index (IRI). IRI is used to define a characteristic of the longitudinal profile of a travelled wheel track and constitutes a standardized roughness measurement. IRI is used to classify road section in homogeneous section and to know the extent to which the road is damaged. Vehicle mounted Roughometer (Version II) equipment was used to record the longitudinal profile of the travelled wheel track. The roughness of the road (LHS) under study from Nakuru to Nyahururu with respect to chainage is graphically indicated in Figure 3.5.

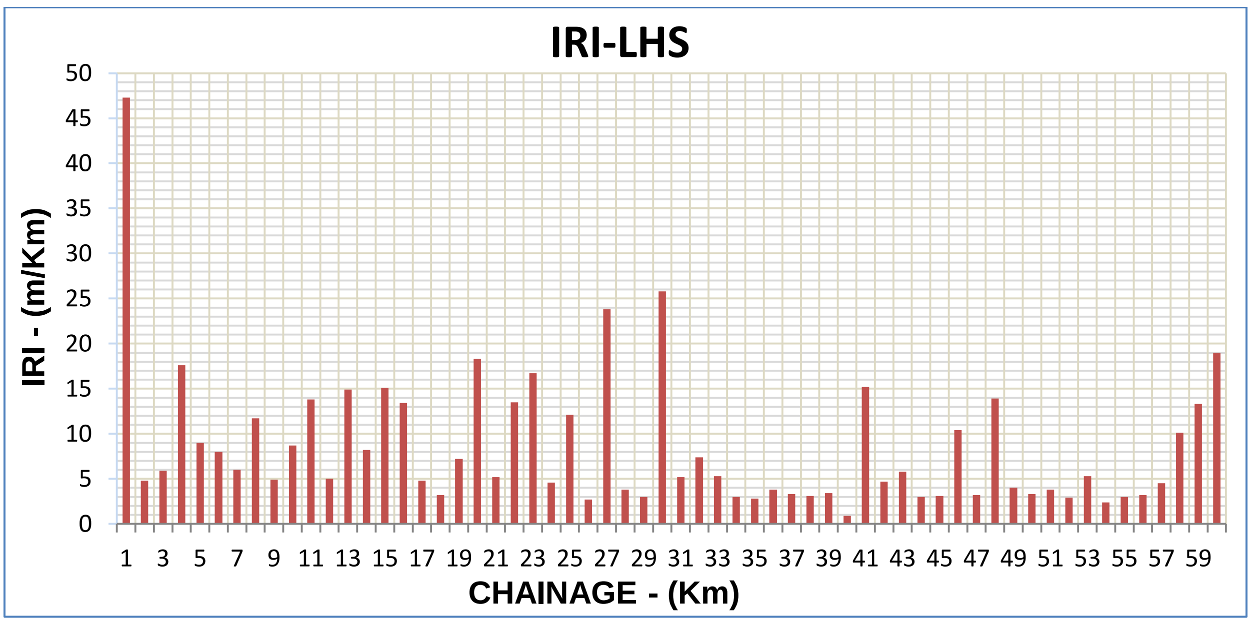


Figure: 3.5: Roughness profile along the Nakuru - Nyahururu road

Source: Author, 2016

3.5.6 Falling weight deflectometer

The deflections were carried out according to the ASTM D4694 - 09 standard test methods for deflections with a falling-weight-type impulse load device. This test method covers the determination of pavement surface deflections as a result of the application of an impact and impulse load to the pavement surface.

The resulting deflections are measured at the center of the applied load and at various distances away from the load and in this case at nine consecutive geophone points of 0, 200, 300, 600, 900, 1200, 1500, 1800, and 2100 mm. To obtain sufficient data for statistical analysis, measurements were taken at intervals of approximately 100 m. The result of FWD data measurement at (D_0) was presented in figure 3.6.

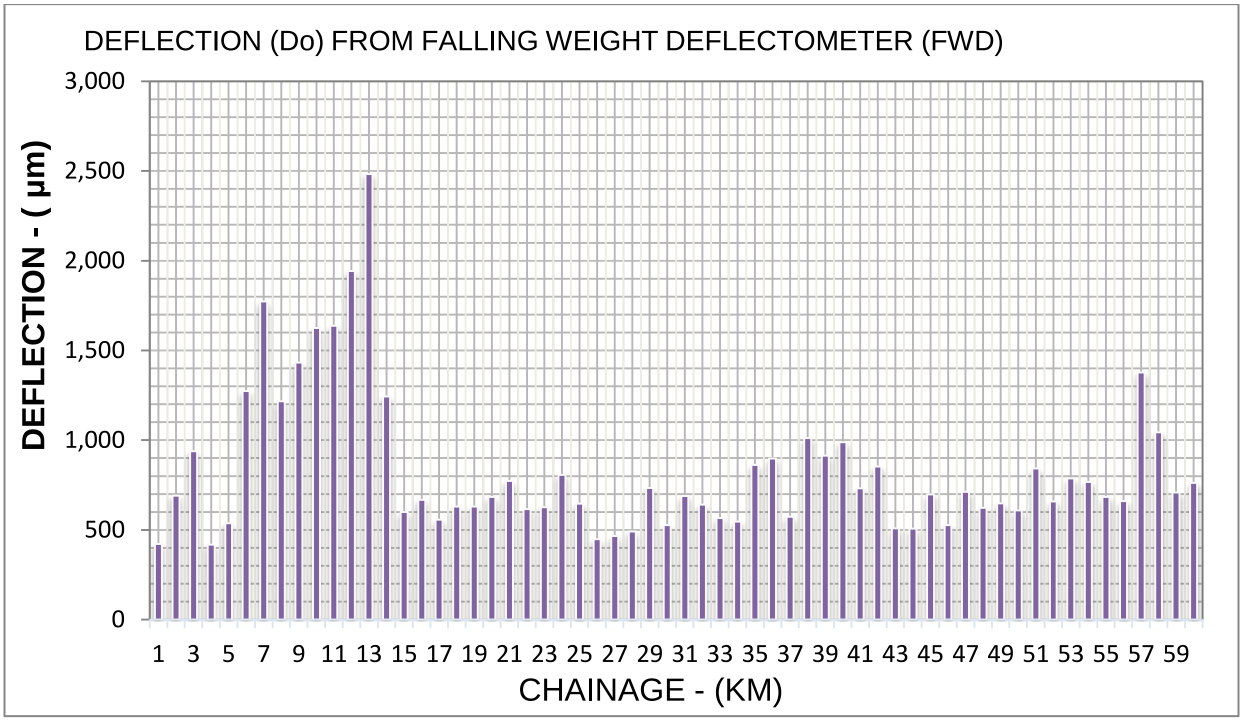


Figure 3.6: Deflection measurement at (D_0) on Nakuru – Nyahururu road

Source: Author, 2016

For the purpose of clarification, some of the pictures taken during data collection and pavement condition survey have been presented in Figure 3.7(a-i).

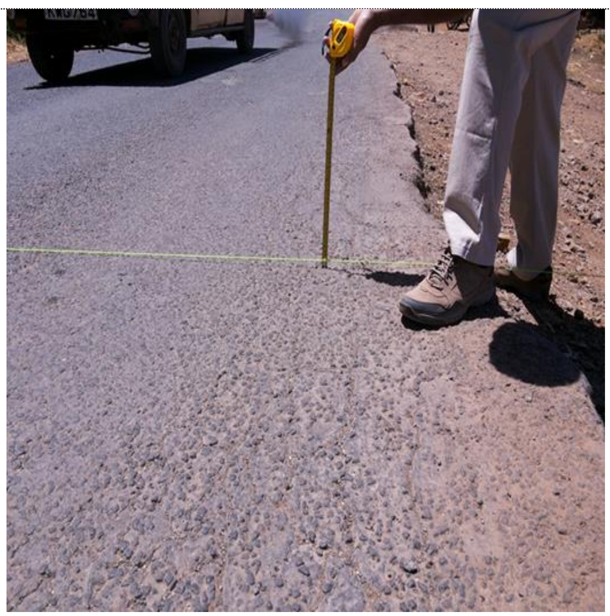
PICTURES TAKEN AT

PAVEMENT CONDITION SURVEY AND DATA MEASUREMENTS

AT THE TIME OF PAVEMENT CONDITION SURVEY

a- Edge break past Kabazi town

b- Rutting on LHS at chainage 50+000



c- Raveling, rutting and edge break past Bahati

d- Quarry site outskirt of Nakuru (4+000)

Non-destructive test (Roughness and falling weight deflectometer)

e- ROUGHOMETER II reader

f- Wheel mounted roughness measurement

g- Standard trailer mounted FWD being operated & monitored from the Range Rover

h- Sensors of FWD touching the ground & ready for testing

	
i- Test-pit of alignment soil on the Nakuru – Nyahururu (42+000)	

Figure 3.7 (a-i): Pictures taken at the time of pavement condition survey

Source: Author, 2016

CHAPTER FOUR

DATA ANALYSIS AND DISCUSSION

4.1 Introduction

In this section; after analysis of the data from trial pits and non-destructive tests, the most optimum remedial measure is investigated. Rather than sticking to only Kenyan design manual (RDM, 1987), it was necessary to compare mechanistic-empirical pavement analysis design software (mePADS). Then come up with the most economical one among Kenyan design manual, Tanzanian design manual, AASHTO 1993 and mechanistic-empirical pavement analysis design software (mePADS) of South Africa. This was done with the assumption that a routine maintenance of each year after new construction and periodic maintenance of 5 years intervals till the end of design period.

4.2 Design life determinations

The research dealt with how to have a design period of more than 15 years from other options as well. It is possible to design either for 15, 20, 25, 30, 40, 50 years using mechanistic-empirical pavement design analysis (mePADS) and AASHTO 1993. But for the analysis of the study and just to compare the economically justifiable pavement which method has less cost than the other, it was decided to take 15 years as a design life for all due to the restriction on most of the design manuals (catalogues).

4.3 Traffic data analysis

4.3.1 Traffic survey data

The traffic data in Nakuru, Bahati junction, Subukia and Nyahururu have been summarized in the Table 4.1.

Table 4.1 Summary of daily traffic

CATEGORY OF VEHICLES	NAKURU		BAHATI		SUBUKIA		NYAHURURU	
	UP	DOWN	UP	DOWN	UP	DOWN	UP	DOWN
CARS	8,474	8,773	5,372	5,468	2,218	4,110	15183	15262
LIGHT GOODS VEHICLE	1,098	1,042	558	817	248	390	784	966
MEDIUM GOODS VEHICLE	1,182	1,216	734	812	384	337	1226	1165
HEAVY GOODS VEHICLE	693	574	453	532	193	130	337	282
TRACTORS	21	13	32	28	21	11	65	56
BUSES	8,137	7,730	4,187	4,094	2,219	2,332	6617	6727

Source: Author, 2016

Legend :- (UP = Nyahururu – Nakuru; DOWN = Nakuru - Nyahururu)

The historic traffic data in table 3.2 and the current data on Table 4.1 show that, there was a decrease in light vehicle goods as well as medium vehicle goods. On the contrary, there was an increase of private cars, buses and high vehicle goods (3% increase of high vehicle goods, and 4% increase of buses).

4.3.2 Classification of project road in to homogeneous sections

Sections were classified into homogeneous by CUSUM (cumulative sum) method. In this method the deflection at the center D_0 of the FWD (falling weight deflectometer) had a mean value. The mean value was deducted from each deflection (D_0); the cumulative of all was plotted against the chainage of the road section as in equation 2.5. For uniformity of traffic the first two homogeneous section-1(hs-1) and 2(hs-2) are summarized as homogeneous section-I (HS-I), the middle section hs-3 as (HS-II) and the last two homogenous sections homogeneous section-4(hs-4) and 5 (hs-5) are summarized as homogeneous section-III (HS-III) as indicated in Figure 4.1.

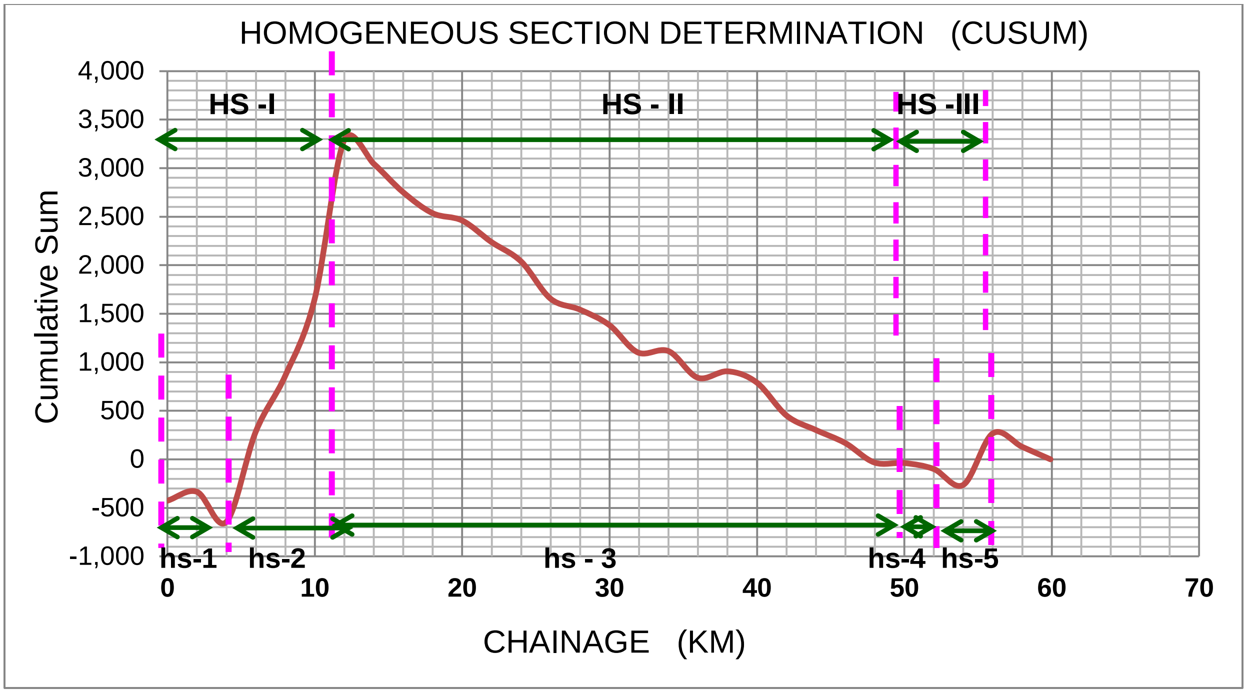


Figure 4.1 Homogeneous section determinations by FWD

Source: Author, 2016

4.3.3 Equivalence factor

The damaging effect of vehicles on the new pavement was factored for each category of vehicles. The equivalent factor was then turned into single standard axle load (ESAL). This was done by use of Liddle's formula - Equation 2.1. The summary an equivalent factor for each homogeneous section is indicated under Table 4.2:

Table 4.2 Average equivalent factor for all homogeneous sections

VEHICLE TYPE	EQUIVALENCE FACTOR		
	HS-I	HS-II	HS-III
Minibuses	0.1	0.10	0.1
Small Bus (24-45) Pass.	0.25	0.25	0.25
Large Buses (>45 seats)	1.34	0.73	0.79
Light Goods Vehicles (LGV)	0.26	0.04	0.25
Medium Trucks (MGV)	1.73	2.78	1.97
Heavy Trucks (HGV)	8.56	9.11	6.84
Articulated Truck	12.12	15.97	16.06
Agri. machinery/ Earth Moving Equipment	12.12	15.97	16.06

Source: Author, 2016

Equivalent standard axle per day (ESA/day) was computed from equivalent factor in table 4.2 above and the number of vehicles. Table 4.3 shows the equivalent standard axles (ESA) for the project roads.

Table 4.3 Equivalent standard axle per day at every homogeneous section

VEHICLE TYPE	HS-I			HS-II			HS-III		
	Total	VEF	ESA / Day	Total	VEF	ESA / Day	Total	VEF	ESA / Day
Minibuses	2,221	0.1	222	1,158	0.1	116	1,712	0.1	171
24-45 Seats	76	0.25	19	38	0.25	9	117	0.25	29
> 45seats	60	1.34	81	34	0.73	25	154	0.79	122
LGV	318	0.26	83	204	0.04	7	260	0.25	65
MGV	356	1.73	616	230	2.78	638	355	1.97	700
HGV	150	8.56	1,282	114	9.11	1,040	70	6.84	478
Art. Truck(AT)	38	12.12	467	32	15.97	513	22	16.06	356
Agri. Machine	5	12.12	61	9	15.97	141	18	16.06	289
ESA/day			2,829			2,490			2,210

Source: Author, 2016

4.3.4 Calculation of cumulative standard axle and traffic classes

From figure 4.1 above; the first homogeneous section (HS-I) started from Nakuru to Nyahurur junction up to Bahati junction, the second homogeneous section (HS-II) is from Bahati junction to Nyahururu outskirt (Km 53+400) and the third homogeneous section (HS-III) started from outskirt of Nyahururu (Km 53+400) to the end of other side of Nyahururu which is at Km 60+000. According to equation 2.2 and table 2.2, the traffic class for each homogeneous section was indicated in Table 4.4.

Table 4.4-Traffic classes for each homogeneous sections

Homogeneous Sections	**HS - I**	**HS - II**	**HS - III**
Cumulative number of standard axle	29.08 m	25.60 m	22.70 m
Traffic class	T_1	T_1	T_2

Source: - Author, 2016

Result: Homogeneous section I & II were categorized under traffic class T 1 and the third homogeneous section HS- III was classified in T 2.

4.4- Pavement Condition Survey Results

Pavement condition survey was evaluated by visual inspection and road roughness investigations in order to sub-divide road sections into the following:-

- Where only minimal routine or periodic maintenance is needed;
- Where major treatment, such as reconstruction is needed, and
- To those of intermediate condition

The visual condition survey taken on the existing Nakuru – Nyahururu road was indicated as in Table 4.5.

Table 4.5 Pavement condition survey on homogeneous section

Pavement Condition Survey							
Homo Sec.	Crack% Area	Rutting depth	Potholes % Area	Patching % Area	Edge Break	Raveling % Area	Drain(Lined / Unlined)
HS-I	0	30	0	0	0	1	Unlined
HS-II	0	50	3	3	90	30	Unlined
HS-III	0	30	9	9	0	51	Unlined

Source: Author, 2016

4.4.1 Pavement Surface Defects

The various distresses observed were rated and summarized as below. In the rating of the level of distress on the pavement, severity of damage was expressed in three levels as discussed in section 3.5.4 surface conditions after combination of surface defects and deformation were defined in section 3.5.4 and is presented in Table 4.6. Since the surface condition rating was “Severe”, the homogeneous section 1, 2 & 3 had a condition index of 4-5 or 6-7 as shown in section 3.5.4 and Table 3.8.

Table 4.6: An average rutting and rating on each homogeneous section

From: (km)	To: (km)	Average rutting(mm)	Rating
0+000	13+250	40	Severe
13+250	54+000	48	Severe
54+000	60+000	50	Severe

Source: Author, 2016

4.4.2- Deflection data analysis and curvature indices

i. - Characteristic deflection, D_{90}

From normal or Gaussian equation, the distribution of deflection in determining homogeneous sections is expressed by equation 4.2.

D_{90}= d +1.3 x σ ……………………………………………………...Eq. 4.2

Where:-

D_{90} - is the characteristic Deflection,

d- is the average deflection for the homogeneous section

σ- is standard deviation for the deflections D_0

Therefore; the results of deflection characteristics D_0 for each homogeneous section have been indicated in table 4.7.

Table 4.7: Characteristics deflection D_{90} of homogeneous section

Section	Homogeneous Section	From	To	Characteristic Deflection D_{90} (μ m)
1	HS - I	0+000	13+250	1,688.06
2	HS - II	13+250	54+000	1,024.87
3	HS - III	54+000	60+000	1,198.53

Source: Author, 2016

ii. - Criteria for deflections D_{90}

The allowable deflection limits and pavement condition rating are explained in Table 4.8 according to South African pavement design method. Table 4.9 was the pavement condition of each homogeneous section:

Table 4.8 Deflection limits for pavement condition rating

Deflection	Lower Limit: < 300μm	Middle Limit : 300-600μm	Upper Limit: > 600μm
Bearing capacity	High	Medium	Poor
Pavement condition rating	Sound	Warning	Severe

Source: SAPDM, 2014

Table 4.9 Pavement condition based on allowable deflection for homogeneous section

Section	Homogeneous Section	From	To	Pavement condition rating
1	HS - I	0+000	13+250	Severe
2	HS - II	13+250	54+000	Severe
3	HS - III	54+000	60+000	Severe

Source: Author, 2016

iii. - Characteristic radius of curvature (RoC)

The existing alignment soil is a granular base as per the South African design manual 2014. Table 4.10 shows the radius of curvature for the homogeneous sections obtained by application of Equation 2.4.

Table 4.10 Pavement condition rating based on RoC for homogeneous sections

NO	Section	From	To	Radius of Curvature (RoC)	Pavement Condition Rating
1	HS – I	0+000	13+250	51	Severe
2	HS – II	13+250	54+000	61	Severe
3	HS – III	54+000	60+000	51.8	Severe

Source: Author, 2016

iv. - Combination of visual conditions and structural conditions

From the structural deflection on Table 4.11 below, the D_{90} was greater than 600m for all homogeneous sections and hence the pavement condition rating is “Severe”. Also from visual condition assessment the rutting on the stretch is classified under “Q5”.

Table 4.11 Combination of visual condition with respect to structural condition

Structural deflection / Conditions: visual conditions	D_{90}< 300µm	300µm< D_{90}<600µm	D_{90}> 600µm
Rating	**Sound**	**Warning**	**Severe**
Sound: 1	Q1	Q2	Q3
Warning:2-3	Q2	Q3	Q4
Severe: 4-7	Q3	Q4	Q5

Source: SAPEM, 2014

The manual for rehabilitation of flexible pavement in tropical Africa suggests that for traffic class T1 & T2 and the corresponding pavement quality of Q5, as stated from section 2.9.2 and clearly mentioned in table 2.9 for the road under study from Nakuru to Nyahururu all parameters lead us to a recommendation of reconstruction.

4.5 Subgrade classifications

Alignment soils on the road stretch were tested and also the nearby quarry sites at near Menengai, Kabazi town, and outskirts of Nyahururu were tested. The test results are summarized in the table 3.4.For each homogeneous section the CBR values were as follows:

HS-1 = 13, 14, 19

HS-2 = 5, 12, 28, 30, 32, 34, 37, 38

HS-3 = 36, 38, 40

According to Tanzanian pavement and materials design manual (1999), from section 2.6.2 and figure 2.4, using the 90%-ile of CBR value for the design CBR of the homogeneous section, “d” value was calculated from eq. 4.3:-

d= 0.1x (n-1)…………………………………………………………….……….Eq. 4.3

Where:-**d**= the values in the horizontal axis starting from 1

n=number of tests used in the design

A - Homogeneous section -I (HS - I)

Among all the samples for homogeneous section- I, the section calculated CBR value came out to be 13 with a subgrade class of S_4 as seen in Figure 4.2.

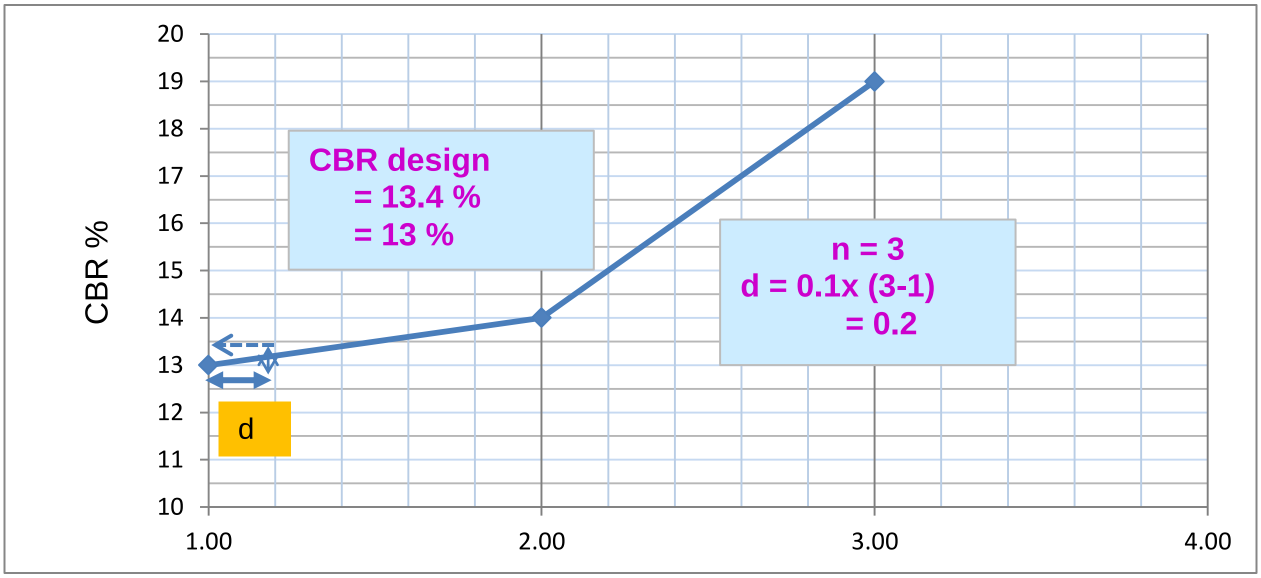


Figure 4.2: CBR design as the 90%-ile value for homogeneous section- I

Source: Author, 2016

B -Homogeneous section - II (HS – II)

The CBR values were drawn in an ascending order for determination of "d" as shown in Figure 4.3.

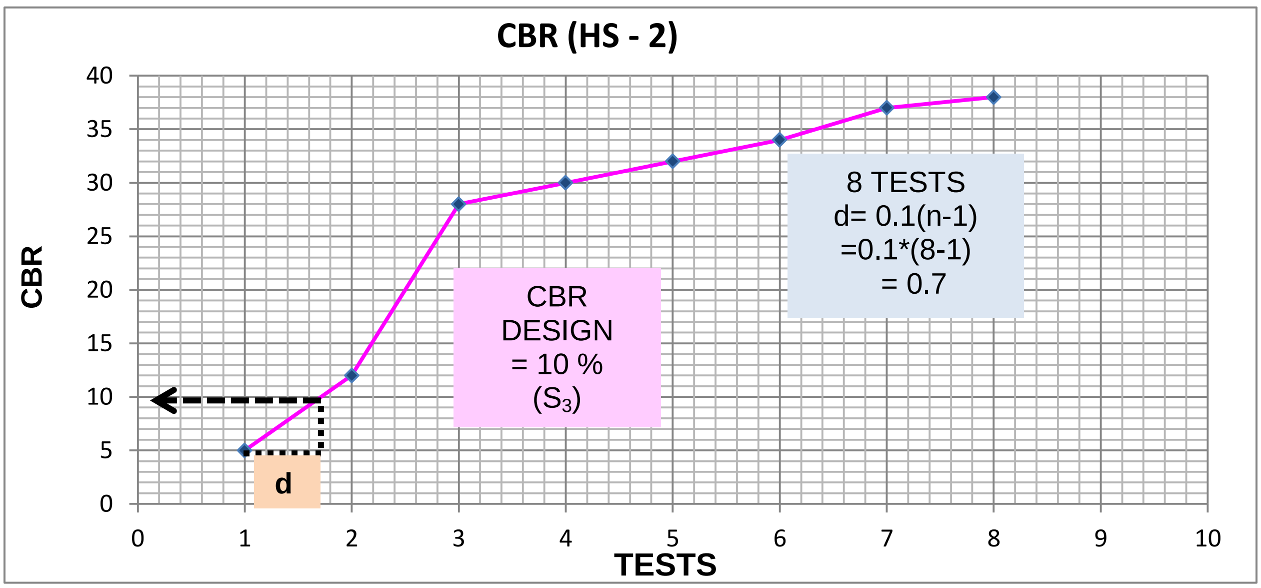


Figure 4.3: CBR design as the 90%-ile value for homogeneous section-II

Source: Author, 2016

C -Homogeneous section -III (HS- III)

The section calculated CBR value came out to be 36 which were categorized under a subgrade class of S_6 of all the samples for homogeneous section- III.

4.6 Current pavement design techniques

4.6.1 Designing using Kenyan road design manual

The road section between Nakuru and Nyahururu is of a mountainous, rolling terrain in which part of an extension of Aberdare Mountain with a lot of small to medium rivers flowing. The area had an availability of gravel and quarry stone site potentials and.

i. Homogeneous section - I

For subgrade class S_4 and traffic class $T_{1;}$

The available options were as follows: - type-4, type-5, type-8 and type-11. However GCS base was more considered uneconomical due to lack of aggregate in the project area. The corresponding pavement type selected according to the Subgrade classification and traffic load class on the Kenyan RDM, 1987 was sorted out and then, the unit price rate for each pavement section from an average of current contractor's price rate in Kenya was indicated on table 4.12 below.

Table 4.12: Average unit price of materials

SN	ITEM	UNIT	RATE (Ksh)
1	Asphalt concrete	m^3	24,735.00
2	DBM	m^3	21,640.00
3	GCS, Base & Sub-base	m^3	3,579.00
4	Improved material	m^3	1,526.00
5	Subgrade & earthen shoulder	m^3	749.00
6	Lean concrete	m^3	21,850.00

Source: KeNHA, 2012

According to the unit rate from table 4.12 the cost of each pavement options was calculated and mentioned in table 4.13 below.

From Table 4.13 it was found that, Type-5 was more economical.

Table 4.13 Cost comparison of homogeneous section - I

Options	**Type 4**	**Type 5**	**Type 8**	**Type 11**
Cost per kilometer	25.2millions	19millions	25millions	42millions

Source: Author, 2016

According to the minimum pavement rate (construction cost), “Type-5” has a pavement type and thickness described in table 4.14 below.

Table 4.14: Economical option for homogeneous section – I

Type - 5		
Surfacing	Asphalt concrete	100
Base	Cement stabilized gravel	150
Sub Base	Cement or lime improved material (Base quality)	200

Source: Kenyan road design manual, 1987

ii. Homogeneous section - II

For subgrade class S3 and traffic class T1, from Kenyan road design manual- III (RDM, 1987) the available suitable options are type-4, type-5, type- 8 and type-12 listed in table 4.15 among all type-5 were preferred due to availability of the material and minimum cost than other types.

Table 4.15 Cost comparison of available options in homogeneous - II

Options	**Type 4**	**Type 5**	**Type 11**	**Type 12**
Cost per Kilometer	36 millions	18 millions	26 millions	47 millions

Source: Author, 2016

“Type-5” has a pavement type and thickness as in table 4.16.

Table 4.16: Economical option for homogeneous section – II

TYPE - 5		
Surfacing	Asphalt concrete	100
Base	Cement stabilized gravel	150
Sub Base	Cement or lime improved material (Base quality)	225

Source: Kenyan Road Design Manual, 1987

iii. Homogeneous Section – III

According to section 2.6.2 the subgrade classification of soil is S3 and section 2.3.1 and table 4.4 the traffic class is T2. Subgrade class S_3 and traffic class T_2, from Kenyan part-III (RDM, 1987) type-4, type-11, type-12, type-13 and type-14 are considered options and type-4 is selected as the available resource in the area as well as with the minimum cost as in table 4.17.

Table 4.17 Cost Comparison of Options in Homogeneous Section - III

Options	**Type 4**	**Type 11**	**Type 12**	**Type 13**	**Type 14**
Cost Per Km	21million	40million	42million	46million	61million

Source: Author, 2016

Therefore; the preferred option “Type-4” has a pavement type and thickness as in table 4.18.

Table 4.18: Economical option for homogeneous section – III

TYPE - 4		
Surfacing	Asphalt Concrete	75
Base	Cement Stabilized Gravel	200
Sub Base	Graded Crushed Stone (Base Quality)	125

Source: Kenyan Road Design Manual, 1987

4.6.2 Designing using Tanzanian design manual (1999)

Tanzanian design manual is limited in options. The subgrade materials are all supposed to have a CBR greater than 15% (>15%). The options in the manual for pavement layers included:-

i. Homogeneous Section – I,II & III

Since the traffic class is 29.05m, 25.60m & 22.7m for homogeneous sections I, II & III respectively, it is in between 20-50 million E80, it is classified under TLC 50 according to Tanzanian manual 1999. Among the available ones, only two options are available from the manual which fulfills the requirements.

Among all options with a base course of bituminous mix was selected based on minimum cost as indicated in table 4.19.

Table 4.19: Economical option for homogeneous section – I, II & III

Base Course- Bituminous Mix		
	Layer Types	Thickness, mm
Surfacing	AC	50
Base course	Bituminous mix	200
Sub-base	Cemented material(CM + CM)	250

Source: Tanzanian design manual, 1999

4.7 Analysis of pavement options using (mePADS)

The preferred most economical options from each homogeneous section and their thicknesses were used in the software and were iterated to fulfill the required traffic capacity for the specified design period.

The software used to calculate for a design period of 15 years. The E-moduli for the materials used for the modeling is from RDM-III 1987 and RDM –V 1988, is detailed in table 4.20.

Table 4.20: E-Moduli for Base, Sub base and Subgrade materials

Surfacing Materials	E-Modulus (MPa)
Asphalt concrete type 1 (AC)	4,000
Asphalt concrete type 2	2,500
Base Materials	E-Modulus (MPa)
Natural gravel (G4)	300
Cement or lime improved materials (C4)	1,000
Cement stabilized gravel (C3)	4,000
Graded crushed stones (C1)	400
Dense bitumen macadam (BC2)	5,000
Lean concrete	10,000
Sub-Base Materials	E-Modulus (MPa)
Natural gravel (G5)	200
Cement or lime improved materials (C4)	300
Graded crushed stones (C2)	300
Subgrade Materials	E-Modulus (MPa)
Subgrade classes S1, S2, S3, S4, S5, S6	15, 50, 65, 90, 125, 250

Source: Kenyan RDM-III 1987 and RDM- IV 1988

4.7.1 Analysis of Kenyan road design manual options using mePADS

All pavement options were analyzed to determine whether they would be structurally viable or not.

i. - Homogeneous section – I & II

a- Selection from the catalogue

Based on the unit price selection above in the tables 4.15 & 4.17, pavement “type–5” was selected from material availability and minimum construction cost point of analysis and the layers, type and thickness are as in table 4.21.

Table4.21: Selected option in homogeneous section I & II

LAYERS	DESCRIPTION	THICKNESS
Surfacing	Asphalt concrete	100mm
Base	Cement stabilized gravel	150mm
Sub-Base	Cement(lime) improved material	200/225mm

Source: Author, 2016

b- Analyzing Using (mePADS)

The pavement structures were analyzed using the South Africa (mePADS). Five layers were identified as required by the software. Because of the altitude, the climate was considered wet, rutting was limited to 20mm according to Kenyan design manual. The road category “B” was adopted. This translated to an assumption of 90% of the road being serviceable and with an input of Table 4.22 above.

The output using software from South Africa (mePADS) is shown in Figure 4.4 below.

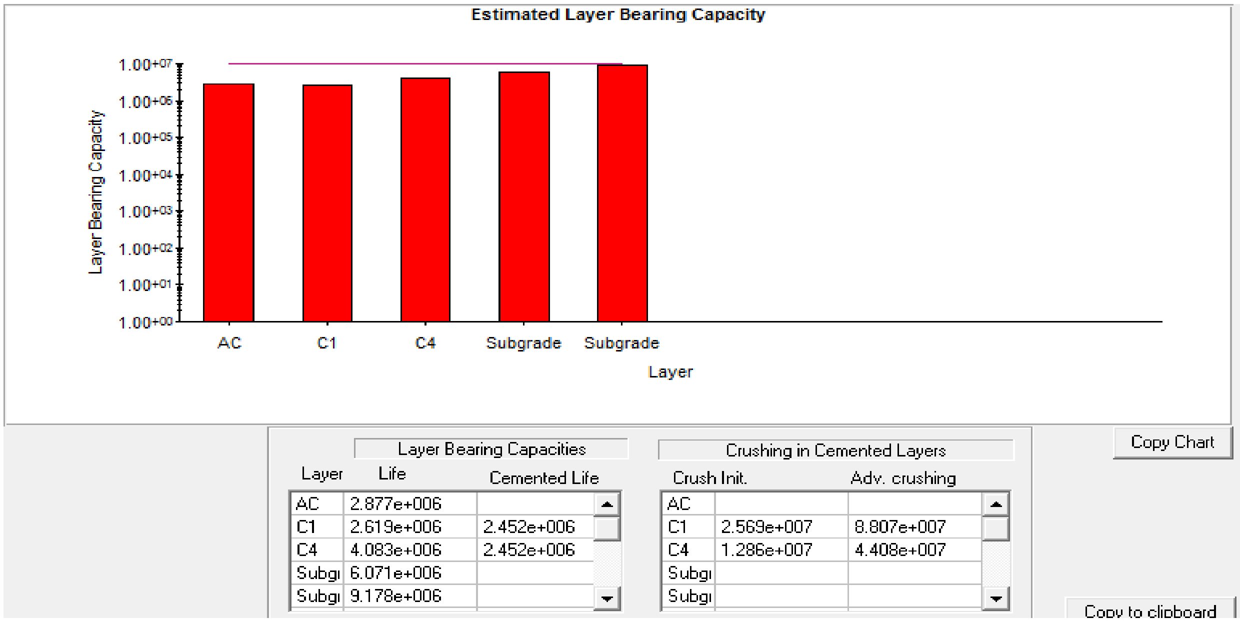


Figure 4.4: mePADS analysis of Kenyan RDM pavement type-5 (HS-I & II)

Source: Author, 2016

Result: For type "B" road category, 90% of the road will reach its serviceability lifetime within 3-4 years after construction is complete.

ii. - Homogeneous Section – III

a. Choosing from the catalogue

Homogeneous section –III has subgrade class of S6 and traffic class of T2, among all options type-4 is selected as minimum construction cost according to the unit price on the table 4.16 above and found to be economical cost wise. Keeping all things the same as to that of homogeneous section I & II, the pavement structure in type-4:-

Surfacing	: AC – 75mm
Base	: Cement stabilized gravel – 200mm
Sub-base	: GCS (Base quality) – 125mm

b. Analyzing using (mePADS)

Three pavement layers both the cement layers in base and sub-base; the pavement climatic condition is wet, axle load ranges from 10-30 Million (E30) and rutting of 20mm. The pavement options for category "B" which their reliability of 90% of the road is serviceable. The analysis is given in Figure 4.5 below.

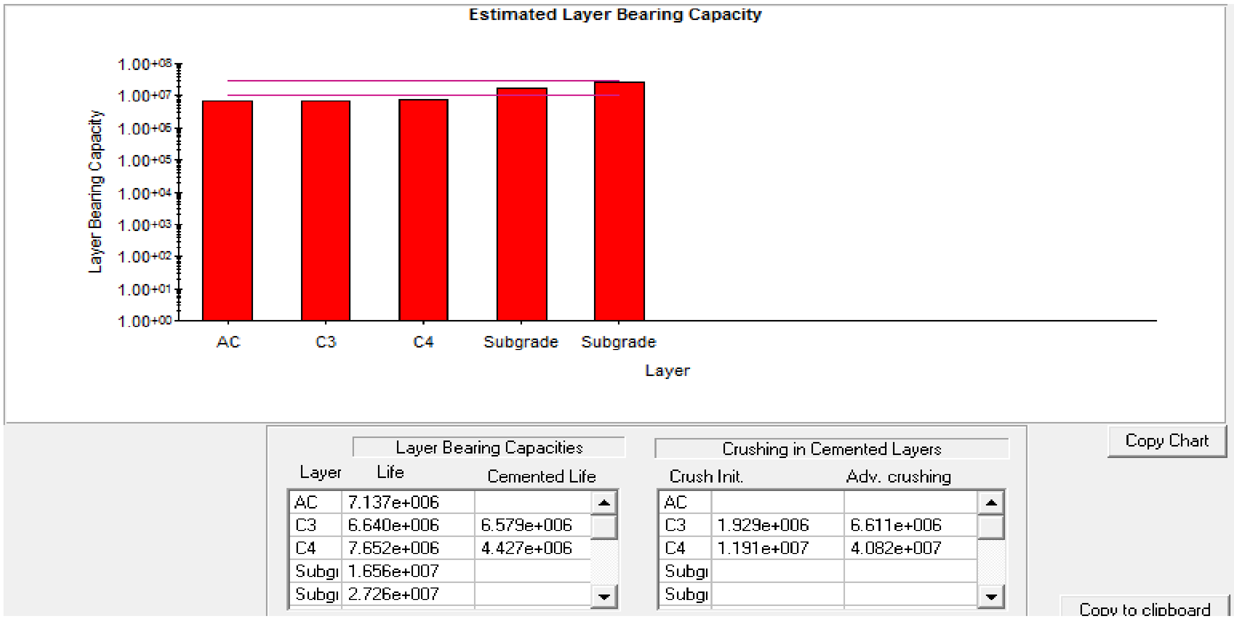


Figure 4.5: mePADS analysis of Kenyan RDM type-5 (HS – III)

Source: Author, 2016

Results: From the analysis, the subgrade is in good condition because of the high CBR which is > 30 % but the surfacing, base and sub-base did not reach the minimum axle load range of the software which is 10 million. So the serviceability of the road with category B will last up to 7 years after completion of construction. Therefore; our selection from the minimum construction cost for homogeneous section I, II & III did not last till end of design period. It was a must to have overlaid for the remaining years for the design period to be equal for us to compare all options.

In order to do that, the structural number was calculated for the existing and the required one. The difference was considered as a structural deficiency for the overlay required as shown in section 4.8 and table 4.24.

4.7.2 Tanzanian manual (1999)

A- Homogeneous section I, II & III

i. Choosing from the catalogue

After comparing and contrasting all the available options from the Tanzanian design manual the following is selected as economical options as shown on the table 4.22.

Table 4.22: Selected pavement for homogeneous section I, II & III

LAYERS	DESCRIPTION	THICKNESS
Surfacing	AC	50
Base	Bituminous mix	200
Sub-base	Cemented material(cm + cm)	250

Source: Tanzanian manual, 1999

ii. Analyzing using mePADS

The input layers and their corresponding values in the table 4.21 above and the output of the analysis are given in Figure 4.6 below.

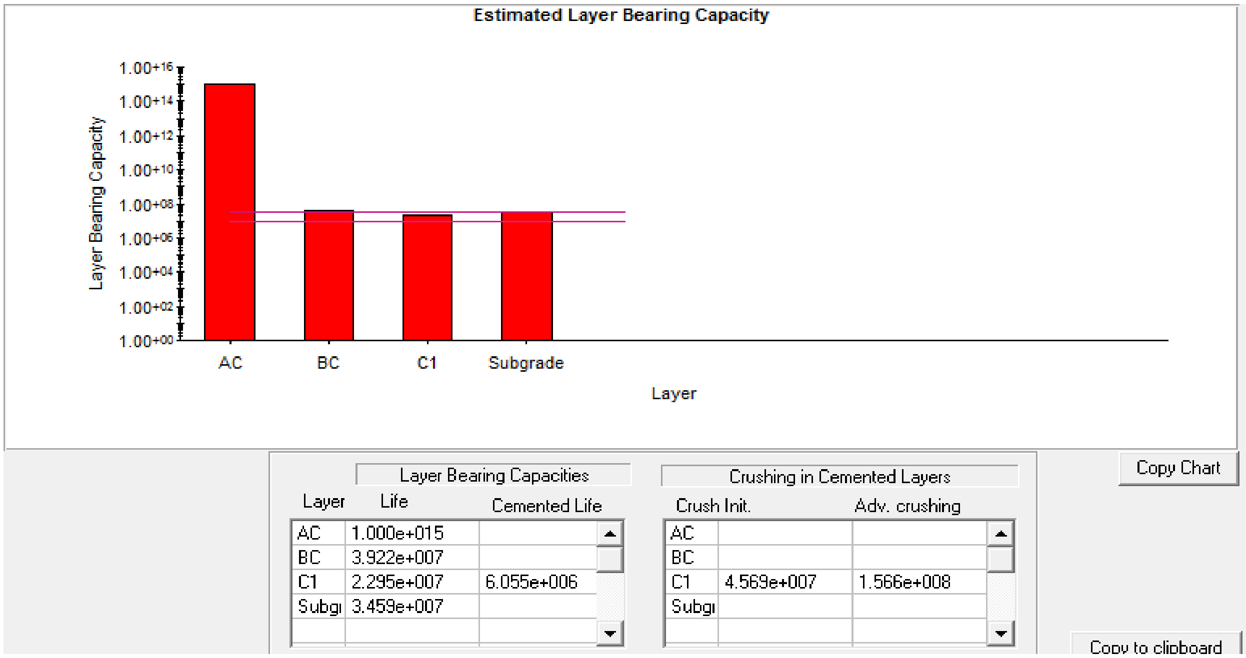


Figure 4.6: mePADS analysis of Tanzanian manual (Bituminous base course)

Source: Author, 2016

Result: From figure 4.7, was noted that the AC would be intact within the range. The base layer would perform well after the design life is over. The sub base and subgrade could stay safely within the design life with a reliability of road category B 90%.

4.8 Overlaying for the remaining design periods (Kenyan design manual)

According to the RDM, 1987, the pavement options would serve for the 4 years after construction, therefore a design for the overlay of the remaining 11 years to accommodate the traffic would be required. The existing structural number of the pavement and the structural number required for carrying the cumulative axle load on the road within the design period was calculated by the Equation 4.4:-

$$\mathrm{SN} = a_1 D_1 m_1 + a_2 D_2 m_2 + a_3 D_3 m_3 \quad \text{Eq.4.4}$$

Where;

SN= Structural number

m_i= Drainage coefficient

D_i= Thickness of pavement layer

A. Kenyan road design manual, 1987

i. Homogeneous section – I

The existing SN was calculated adding the structural number of each pavement section, which is multiplying the layer coefficient of each pavement with the thickness of the corresponding one. The layer coefficient for existing is stated in section 2.10.5 and table 2.10, was indicated in Figure 4.7 and the results are calculated as in Figure 4.8.

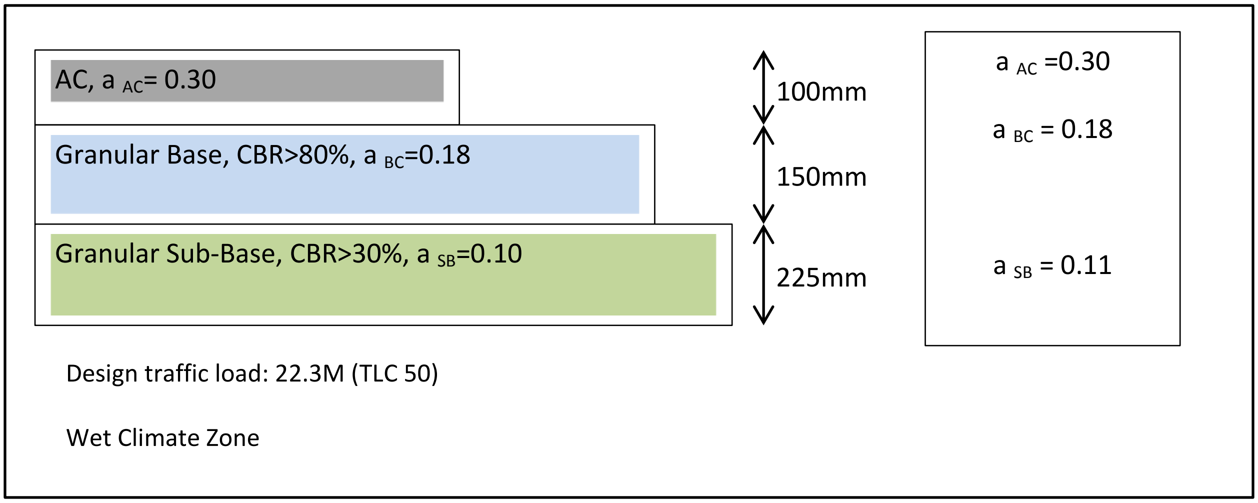


Figure 4.7: Determination of overlaying pavement

Source: Author, 2016

The deficiency of structural number of each layer is calculated in Figure 4.8 below.

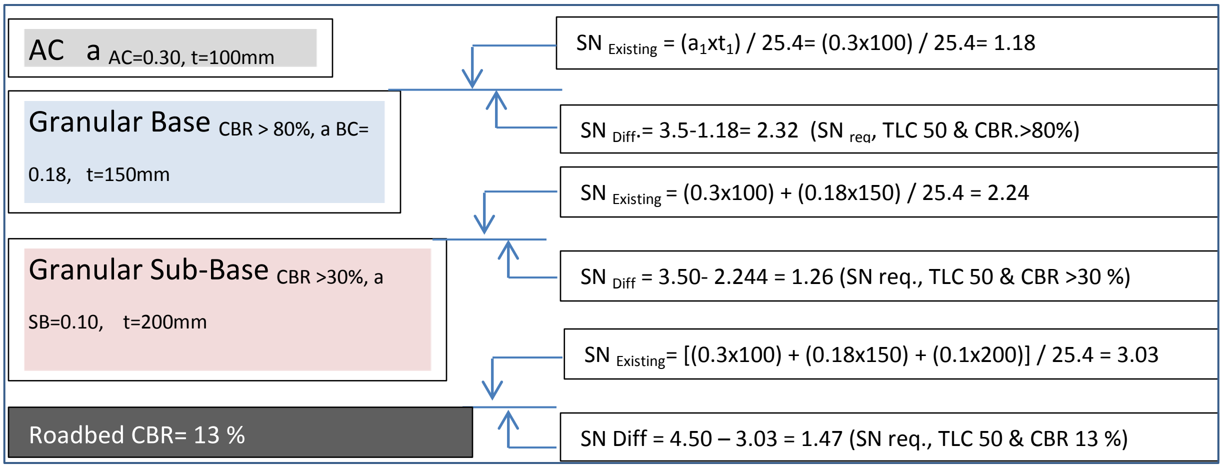


Figure 4.8: Overlaying calculation for RDM Kenya

Source: Author, 2016

Among the values of $SN_{Diff.}$ above of each layer; 2.32, 1.26 & 1.47 the highest value will be used to govern the evaluation of the rehabilitation the pavement. Therefore; using the $SN_{Diff.}$ = 2.32, the nearest value from the table on the overlay alternative is $SN_{Diff.}$ = 2.25 which helped choose from various options and the selected alternatives are indicated in the table 4.23 below.

Table 4.23: Overlay alternatives

SN Diff.	**Overlay Alternatives**			
	Asphalt concrete overlay	Bituminous mix for base course	Penetration macadam	Granular base course
2.25		AC 50 mm Bitum. 130 mm	AC 50 mm PM 80 125 mm Alternatively: ST PM 60 100 mm PM 60 100 mm	

Source: Tanzanian manual, 1999

So it is preferred it to be an AC of 50 mm and a bituminous thickness of 130mm so as to the pavement reach the remaining years of design period.

ii. – Homogeneous sections II & III

With subgrade CBR of 36, 10 and a traffic load of both TLC 50, the structural number required (SN Req.) for homogeneous two and three are 3.5 and 4.50 respectively. After calculating the SN $_{Diff.,}$ the selected options are:-

Homogeneous -II: - AC-50 mm, Bituminous mix (DBM) -130 mm

Homogeneous-III: - AC-50 mm, Bituminous mix (DBM) -130 mm

B. Tanzanian manual, 1999(catalogue)

From the Tanzanian manual in all homogeneous sections the selected options were analyzed with South African software mePADS are within the range of provision of the design limits and therefore we don't require an overlay until the time of design period. The selected pavement options were taken for evaluation and cost analysis with other design methods.

4.9- Design using mePADS (mechanistic-empirical pavement analysis design software) of South Africa

The allowable stresses and strain as well as the deflections were kept the same to the South African standard. The surfacing of an asphalt concrete (AC), a Base of dense bitumen macadam and sub-base of a cement improved gravel for all homogeneous i, ii & iii section with their corresponding subgrade values 13, 10, & 36 was adopted. The cumulative traffic load of 22.3 million, 19.6 million & 17.4 million respectively belong to E30 (TLC 10-30) in accordance with South African Software. The standard design load data were as shown in fig.4.9.

i. Homogeneous Section – I

With the parameters w_{18}= 29.05 million, pavement parameters as in table 4.24.

Table 4.24 Selected pavement parameters of homogeneous section-I

Surface Type	Modulus of Elasticity	Poisson's Ratio
AC	4,000 MPA	0.35
Base Course	5,000 MPA	0.35
Sub Base Course	1,000 MPA	0.40
Subgrade	90 MPA	0.45

Source: Kenyan RDM, 1987

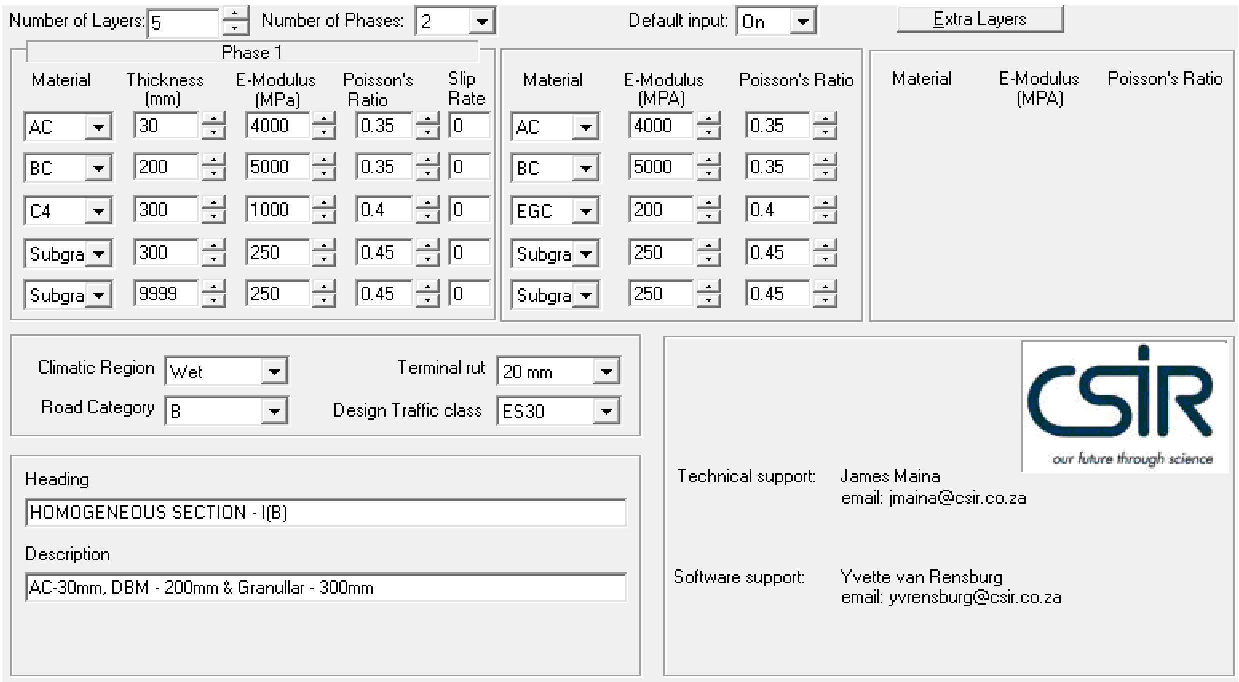


Figure 4.9: Pavement determination using mePADS homogeneous section-I

Source: Author, 2016

From the analysis the pavement required is:-

Surfacing: - AC - 30mm,

Base Course: - DBM - 200mm and

Sub-base: - Cement improved Gravel - 300 mm

From the analysis, two layers of subgrade; the lower subgrade was considered. This was to be milled from the existing asphalt with all base and sub-base layers and have a CBR of more than 30% and the upper subgrade was to be improved as a subgrade class of S_6 with a CBR value greater than 30 %. And the result is seen diagrammatically as shown in Figure 4.10.

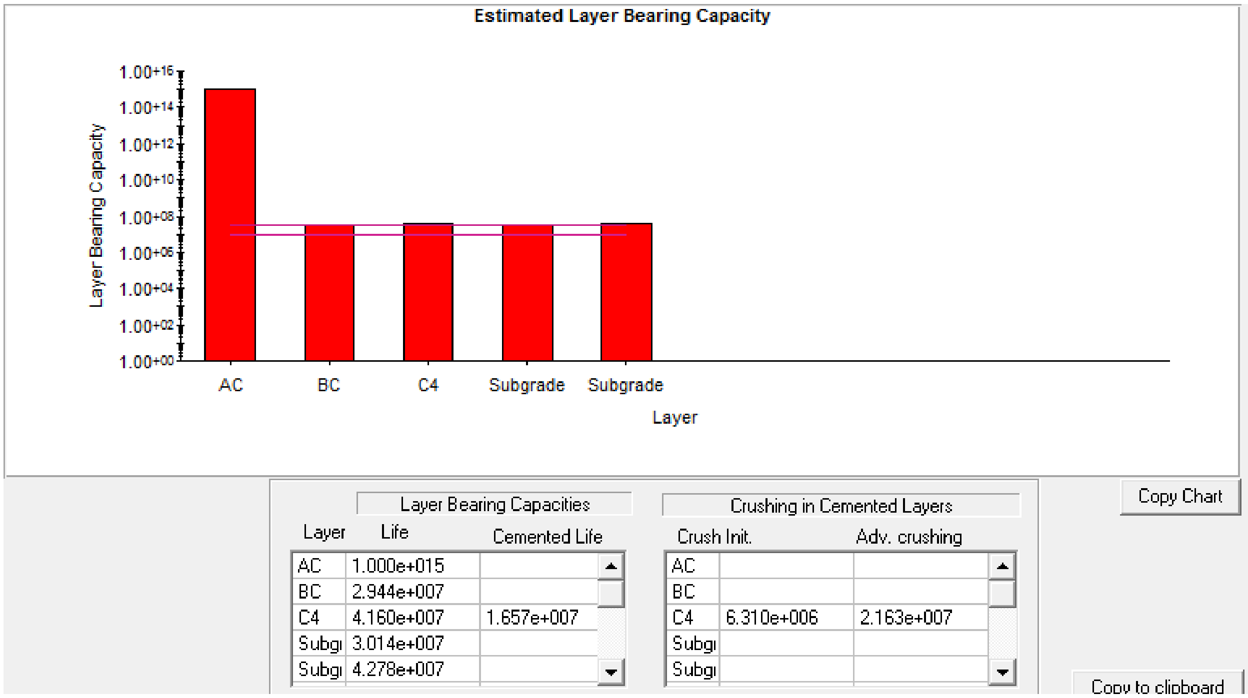


Figure 4.10: Pavement analysis using mePADS homogeneous section- I

Source: Author, 2016

Figure 4.10 above, all parameters were on the range but the subgrade required improvement from class S_4 with modulus of Elasticity of 90 to S_6 a modulus of elasticity of 250 with a CBR of more than 30%.

ii. - Homogeneous Section –II

With the parameters w_{18}= 25.6 million, pavement options as shown in table 4.25.

Table 4.25: Selected pavement options for homogeneous section-II

Surface Type	Modulus of Elasticity	Poisson's Ratio
Asphalt concrete	4,000 MPA	0.35
Base course	5,000 MPA	0.35
Sub base course	1,000 MPA	0.40
Subgrade	90 MPA	0.45

Source: Author, 2016

The standard load and other design parameters are as shown in Figure 4.11.

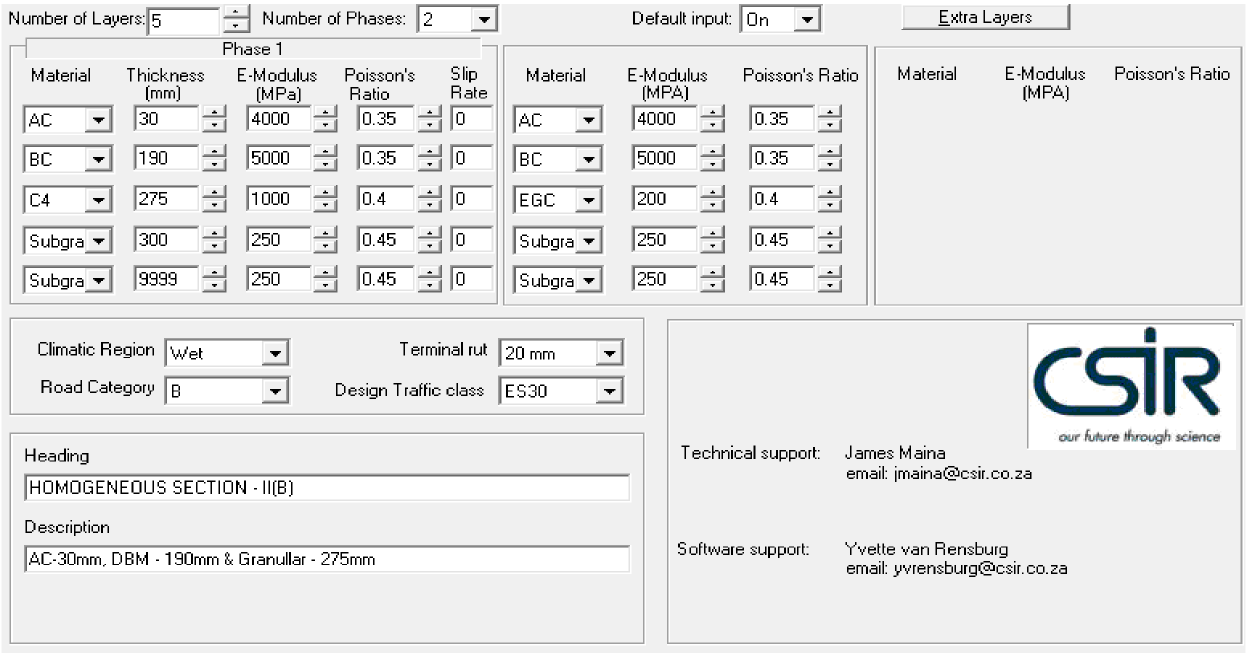


Figure 4.11: Pavement determination using mePADS homogeneous section-II
Source: Author, 2016

Figure 4.12 below shows that all data are up to the design life of the homogeneous section which is 25.60 million cumulative standard axles and the subgrade class needs to be raised to a modulus of elasticity of 250 MPa with a thickness of 300mm with a CBR value of more than 30%.

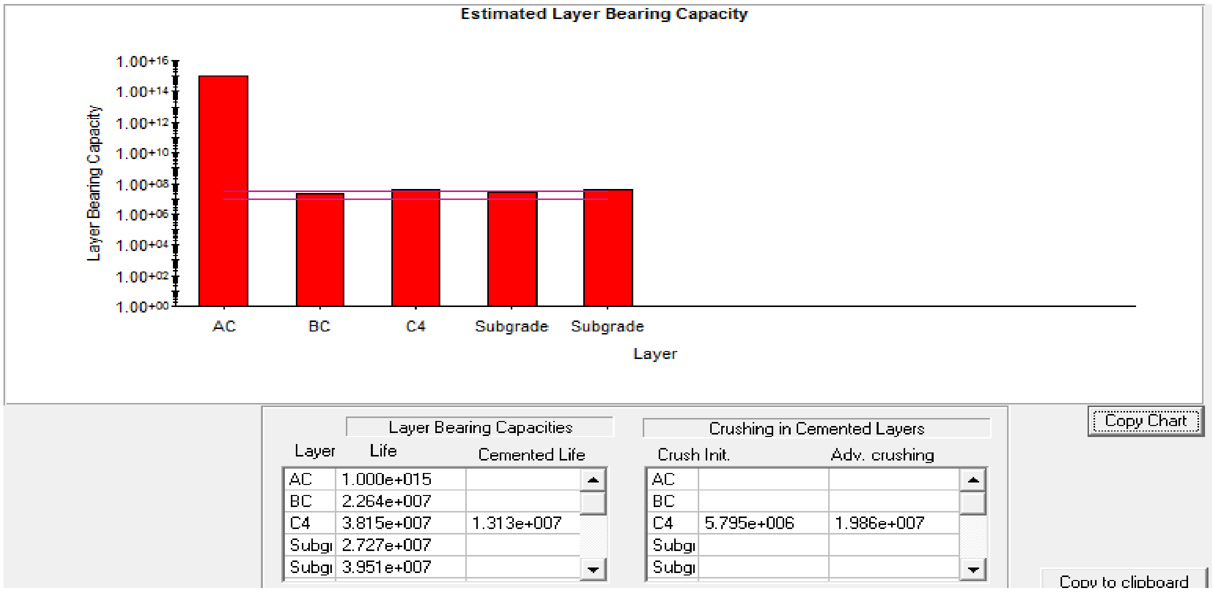


Figure 4.12: Pavement analysis using mePADS homogeneous section-II

Source: Author, 2016

From the analysis the pavement required is:-

- Surfacing: AC- 30mm
- Base course: DBM – 190mm
- Sub-base course: CM – 275mm (CM- cement modified gravel)

iii. - Homogeneous section –III

With the parameters w_{18} (E80) = 22.7 million, pavement options as in table 4.26.

Table 4.26: Pavement options for homogeneous section-III

Surface Type	Modulus of Elasticity	Poisson's Ratio
AC	4,000 MPA	0.35
Base course	5,000 MPA	0.35
Sub-base course	1,000 MPA	0.40
Subgrade	250 MPA	0.45

Source: Author, 2016

The standard design and other parameters were as shown in fig.4.13

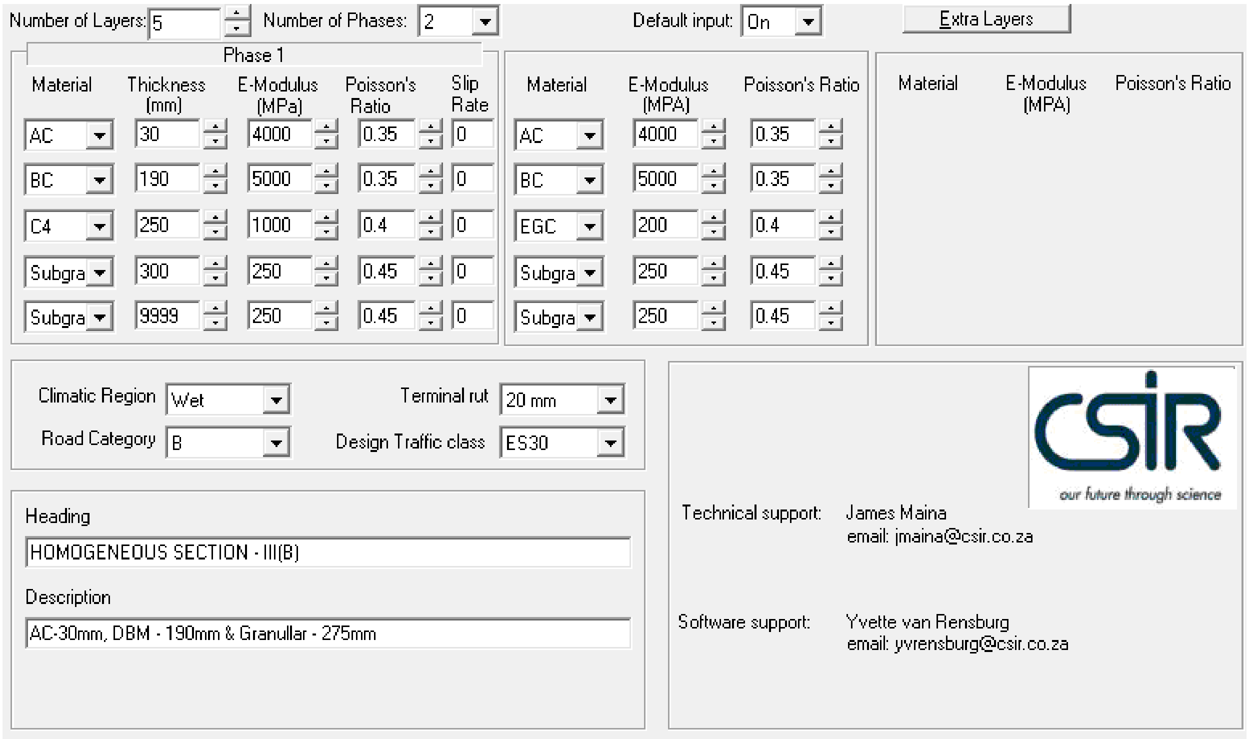


Figure 4.13: Pavement determination using mePADS homogeneous section-III
Source: Author, 2016

From the analysis of the software for homogeneous- III as shown in figure 4.14, a surfacing AC of 30 mm and a base of DBM 190 mm and cement improved of 250 mm has stable pavement structure within the design period. The reduction in the sub-base thickness is due to the high CBR value of existing subgrade which has high moduli of elasticity 250 MPA.

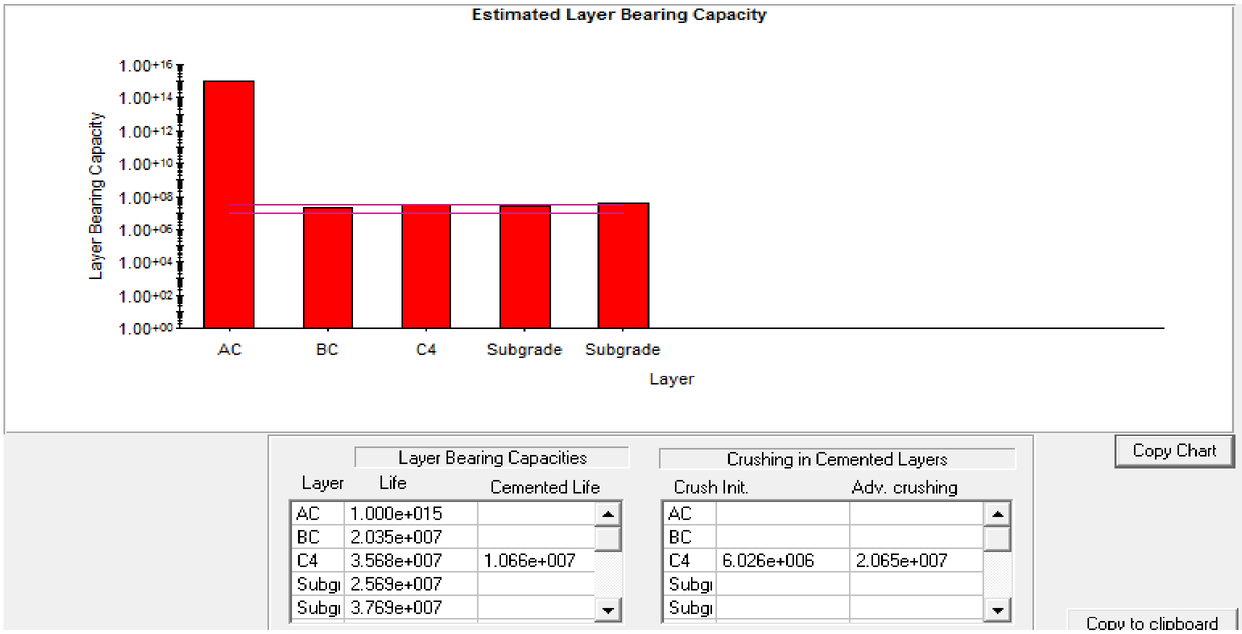


Figure 4.14: Pavement analysis using mePADS homogeneous section-III

Source: Author, 2016

For all homogeneous sections; the two layers of subgrade; the lower subgrade is considered to be the milled existing asphalt with all base and sub-base layers which has a CBR of more than 30% and the upper subgrade is to be designed with a subgrade class of S_5 or S_6 with a CBR value greater than 15% & 30% respectively according to RDM, 1987.

4.10- Design using AASHTO 1993 empirical method

Even though AASHTO 1993 method was developed in America; with American soil characteristics and climatic condition was used as a design method for comparison with appropriate adjustments. According to AASHTO 1993, there are minimum requirements for the pavements that need to be looked into for the specific range of cumulative traffic load. This is stated in section 2.12(v).

i. - Homogeneous Section- I

From AASHTO 1993 manual, using the following parameters into consideration, the structural number of the pavement under study was determined:-w_{18} = cumulative equivalent standard axle load, S_o= overall standard deviation, Mr = modulus of resilience, ∆PSI = serviceability loss, reliability (R %), drainage coefficient (m_i) as shown in fig.4.15 and fig.4.16.

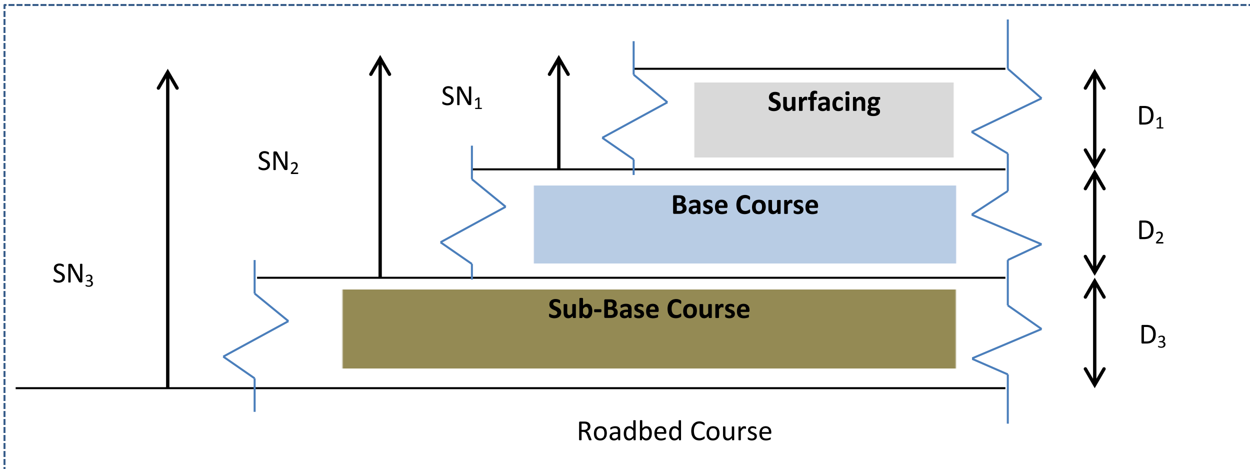


Figure 4.15: Pavement analysis using AASHTO 1993 for homogeneous section-I
Source: Author, 2016

W_{18}= 29.05 M, So = 0.45, ∆PSI = 4.5-2.5= 2, R=80%, m_1=1.0 (Drainage Coefficient)

SURFACING (AC)

$SN_1 = a_1 \times D_1$; $3.0 = 0.4 \times D_1$; $D_1 = 7.5$ (8 inches)

$SN_1 = 0.4 \times 8 = 3.2$

BASE (DBM)

$D_2 = (SN_2 - SN_1)/a_2 = (4.1-3.2)/0.3 = 3.0$ (3 inches)

$SN_2 = 0.3 \times 3 = 0.90$

SUB BASE (Modified Material)

$D_3 = (SN_3-(SN_1+SN_2))/a_3 = (4.5-(0.9+3.2))/0.11 = 3.6$(4 inches)

$SN_3 = 0.11 \times 4 = 0.44$

Therefore; AC=200 mm, DBM= 75 mm, Cement Modified (Improved) sub base=100 mm

ii. - Homogeneous section - II

Using the following requirements: - W_{18} =25.6 M, So=0.45, ΔPSI = 2, R =90%, m_2=1.0

Surfacing (AC)

$SN_1 = a_1 \times D_1$; $2.9 = 0.4 \times D_1$; $D_1 = 7.25$ (7 inches)

$SN_1 = 0.4 \times 7 = 2.8$

Base (DBM)

$D_2 = (SN_2 - SN_1)/a_2 = (4.0-2.8)/0.3 = 4.0$ (4 inches)

$SN_2 = 0.3 \times 4 = 1.2$

Sub Base (modified material)

$D_3 = (SN_3-(SN_1+SN_2))/a_3=(4.5-(1.2+2.8))/0.11=4.5$ (5 inches)

$SN_3 = 0.11 \times 5.0 = 0.55$

Therefore; AC =175mm, DBM = 100mm, cement modified sub-base= 130 mm

iii. - Homogeneous section - III

With the following requirements: - w_{18} = 22.70 M, So= 0.45, ΔPSI = 2, R = 90%, m_3=1.0

Surfacing (AC)

$SN_1 = a_1 \times D_1$; $2.8 = 0.4 \times D_1$; $D_1 = 7$ (7 inches)

$SN_1 = 0.4 \times 7 = 2.8$

Base (DBM)

$D_2 = (SN_2 - SN_1)/a_2 = (3.8-2.8)/0.3 = 3.3$ (3.5 inches)

$SN_2 = 0.3 \times 3.5 = 1.05$

Sub Base (Modified Material)

$D_3=(SN_3-(SN_1+SN_2))/a_3=(4.5-(1.05+2.8))/0.11=5.91(6 \text{ inches})$

$SN_3= 0.11 \times 6=0.66$

Therefore; AC =175mm, DBM = 90mm, Cement Modified Sub-base= 150mm

Summary

Kenyan and Tanzanian design manuals were used to come up with pavement options. Then those options were analyzed using South African software. Kenyan option would not reach its design period. Whereas, the Tanzanian design manual option would exceed the design period. Also AASHTO 1993 was used to come up with an alternative option. The pavement type and thicknesses of each and every option is summarized in the table 4.27 below.

Table 4.27 Summary of Pavement type and thickness

DESIGN OPTIONS	PAVEMENT TYPE AND THICKNESS			
	HS-I	HS-II	HS-III	OVERLAY
KENYAN	AC-100 mm CSG-150 mm CLIM-200 mm	AC-100 mm CSG-150 mm CLIM-225 mm	AC-75 mm CSG-200 mm GCS-125 mm	HS-I: AC-50, DBM-130 HS-II: AC-50, DBM-130 HS-III: AC-50,DBM-130
TANZANIAN	AC- 50 mm DBM-200 mm CLIM-250 mm	AC- 50 mm DBM-200 mm CLIM-250 mm	AC- 50 mm DBM-200 mm CLIM-250 mm	
AASHTO	AC-200mm DBM-75mm CLIM-100mm	AC-175 mm DBM-100 mm CLIM-130 mm	AC-175 mm DBM-90 mm CLIM-150 mm	
mePADS	AC-30 mm DBM-200 mm CLIM-300 mm	AC-30 mm DBM-190 mm CLIM- 275 mm	AC- 30 mm DBM-190 mm CLIM-250 mm	

Source: Author, 2016

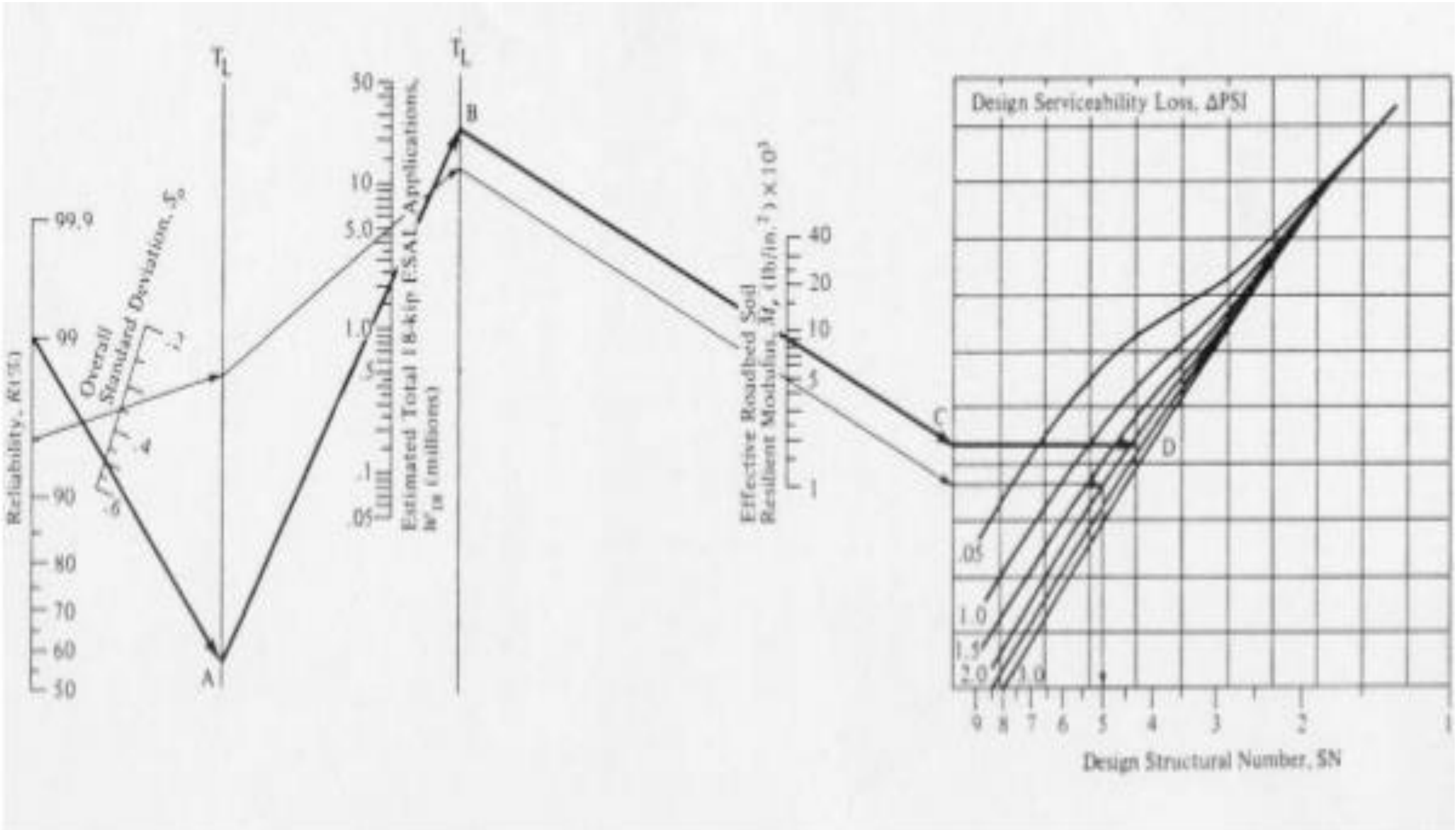


Figure 4.16: Structural Number determination using AASHTO, 1993

Source: AASHTO, 1993

4.11 Cost analysis of each pavement design methods

After analyzing the pavement options from the catalogue of Kenyan, Tanzanian manual, AASHTO 1993 and the South African Software was tried to evaluate their cost effective way of selecting the pavement design which would be most efficient and structurally sustainable to design a pavement. It was compared using the same design period of 15 years and 3 years of construction. When the pavement would not reach the expected design period, the pavement was overlaid. The routine and periodic maintenance rate is taken from the average contractor's unit price submitted to KeNHA (Kenya national highway authority) for budget allocation to similar type of roads of B-Class.Tables 4.28 & 4.29 provide comparison of the summary for construction cost of each category of design methods. The periodic maintenance would be held after 5 years. Routine maintenance would be done 1 year after new construction with an increase of 10% each year (10% was assumed for the annual inflation and variation in prices).

Table 4.28: Total cost of Kenyan RDM design option

Kenyan Design Manual			
Year	Activity	Cost	Cumulative Cost
2016	Year of Construction		
2017	Year of Construction		
2018	Year of Construction		
2019	Initial	5,375,144,006	5,375,144,006
2020	Routine Maintenance	15,000,000	5,390,144,006
2021	Routine Maintenance	16,500,000	5,406,644,006
2022	Routine Maintenance	18,150,000	5,424,794,006
2023	Traffic & Overlay (HS-I)	631,969,278	6,056,763,284
2024	Routine Maintenance	15,000,000	6,071,763,284
2025	Routine Maintenance	16,500,000	6,088,263,284
2026	Traffic & Overlay (HS-II & III)	2,591,527,740	8,679,791,024
2027	Routine Maintenance	15,000,000	8,694,791,024
2028	Routine Maintenance	16,500,000	8,711,291,024
2029	Routine Maintenance	18,150,000	8,729,441,024
2030	Routine Maintenance	19,965,000	8,749,406,024
2031	Periodic Maintenance (PM)	60,000,000	8,809,406,024
2032	Routine Maintenance	15,000,000	8,824,406,024
2033	Routine Maintenance	16,500,000	8,840,906,024
2034	Routine Maintenance	18,150,000	8,859,056,024

Source: Author, 2016

Table 4.29: Total cost comparison of Tanzanian manual, AASHTO 1993 and mePADS

Year	Tanzanian Manual		AASHTO 1993		mePADS	
	Cost	Cum. Cost	Cost	Cum. Cost	Cost	Cum. Cost
2016	Year of Construction		Year of Construction		Year of Construction	
2017						
2018						
2019	8,532,767,891	8,532,767,891	7,798,760,013	7,798,760,013	7,821,522,446	7,821,522,446
2020	15,000,000	8,547,767,891	15,000,000	7,813,760,013	15,000,000	7,836,522,446
2021	16,500,000	8,564,267,891	16,500,000	7,830,260,013	16,500,000	7,853,022,446
2022	18,150,000	8,582,417,891	18,150,000	7,848,410,013	18,150,000	7,871,172,446
2023	19,965,000	8,602,382,891	19,965,000	7,868,375,013	19,965,000	7,891,137,446
2024	60,000,000	8,662,382,891	60,000,000	7,928,375,013	60,000,000	7,951,137,446
2025	15,000,000	8,677,382,891	15,000,000	7,943,375,013	15,000,000	7,966,137,446
2026	16,500,000	8,693,882,891	16,500,000	7,959,875,013	16,500,000	7,982,637,446
2027	18,150,000	8,712,032,891	18,150,000	7,978,025,013	18,150,000	8,000,787,446
2028	19,965,000	8,731,997,891	19,965,000	7,997,990,013	19,965,000	8,020,752,446
2029	21,961,500	8,753,959,391	21,961,500	8,019,951,513	21,961,500	8,042,713,946
2030	60,000,000	8,813,959,391	60,000,000	8,079,951,513	60,000,000	8,102,713,946
2031	15,000,000	8,828,959,391	15,000,000	8,094,951,513	15,000,000	8,117,713,946
2032	16,500,000	8,845,459,391	16,500,000	8,111,451,513	16,500,000	8,134,213,946
2033	18,150,000	8,863,609,391	18,150,000	8,129,601,513	18,150,000	8,152,363,946
2034	19,965,000	8,883,574,391	19,965,000	8,149,566,513	19,965,000	8,172,328,946

Source: Author, 2016

4.12 Discussion of the findings

From figure 4.17and 4.18 below, it was noted that the findings of the analysis as:-

i- For the Kenyan road design manual 1987, all the available pavement options start with a less cost initially but a relatively high cost at the end of design period. When the Kenyan design manual pavement options were analyzed through mePADS, the pavements were structurally unsustainable throughout the design period unless overlaid.

ii- The Tanzanian design manual 1999 pavement options have a high initial construction cost as well as final maintenance cost. For the Tanzanian design manual 1999 in case of high traffic road, the selected option was structurally sustainable throughout the design period.

iii- AASHTO 1993 and mePADS both had a relatively lower cost at the end of design period. Their maintenance costs are uniform throughout the design period. But

AASHTO 1993 will reach its serviceability time one or two years before the design period.

iv- From the analysis it was recommended that the road be designed with mePADS and/or AASHTO-1993 without any limitations.

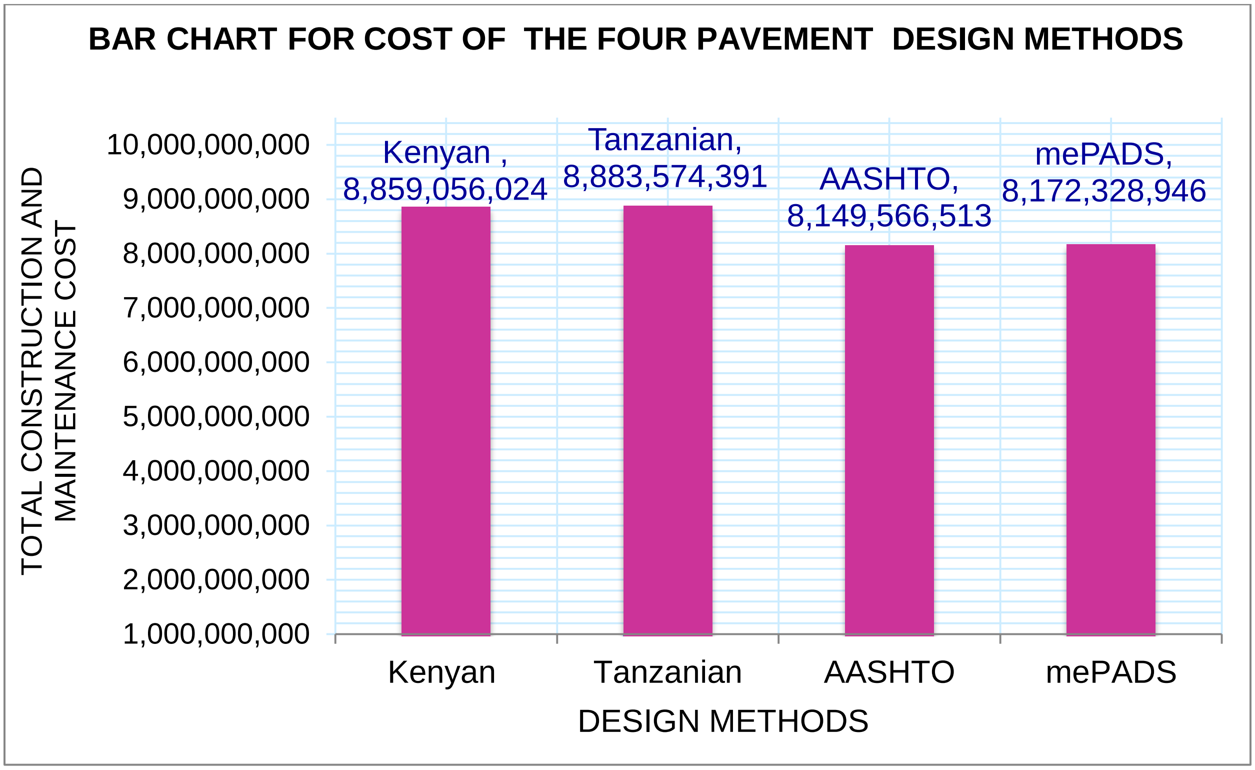


Figure 4.17 Bar chart cost for Kenyan, Tanzanian, AASHTO and mePADS

Source: Author, 2016

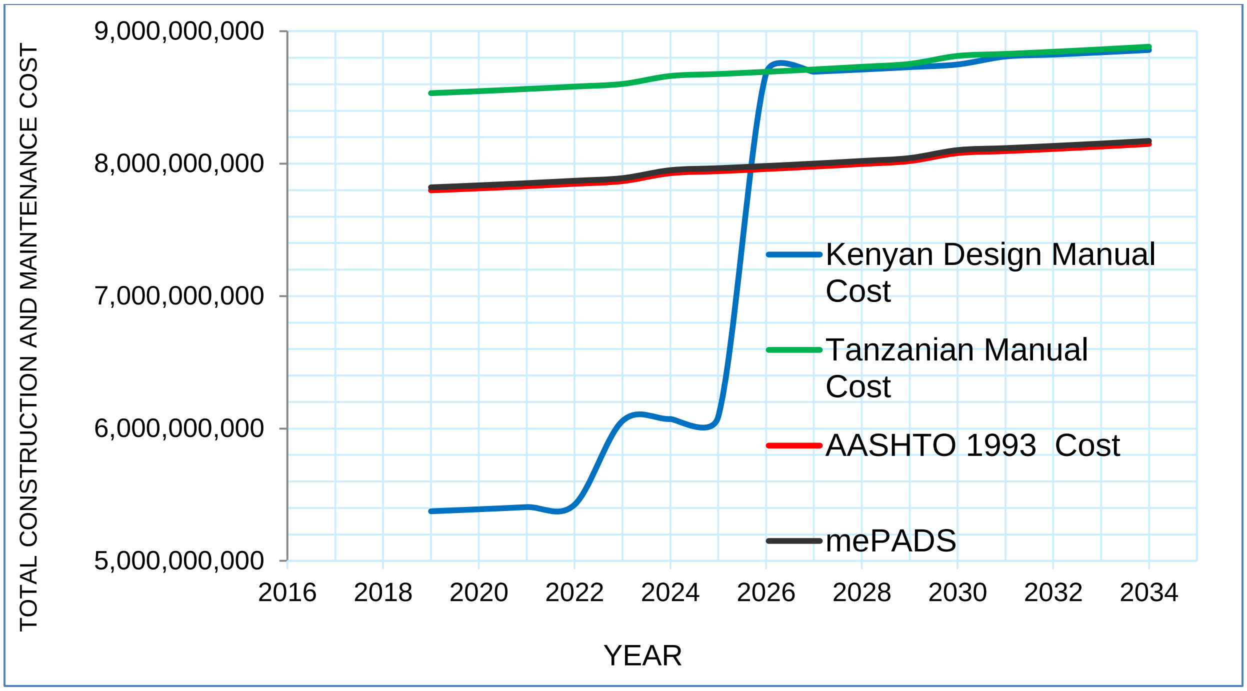


Figure 4.18: Graphical chart for cost of all pavement design methods

Source: Author, 2016

Carvalho et al. (2006) tried to compare flexible pavement designs and performance between the NCHRP 1-37A method (M-E version 0.700) and AASHTO 1993.This was done with a range of climates, subgrades and material properties. It was with the suggestion that the mechanistic-empirical was more precise among the two based on the degree of its calibration and was mentioned in section 2.12 (A).

In section 2.12 (B), Jonathan N. Boone (2013) conducted a study on evaluation of AASHTO 1993 and mechanistic –empirical pavement design guide in Ontario (Canada) a flexible pavement design done on new asphalt. From the outcomes of the comparative analysis the thickness of asphalt found using mechanistic-empirical was greater than the AASHTO 1993 and also AASHTO 1993 will fail early due to permanent deformation and/or roughness.

There was great similarity with the finding of this study and that of Carvalho et al. (2006) and Jonathan (2013). There is a difference in getting the thickness of each layer. AASHTO 1993 pavement fails one or two years before the design period is reached.

This was noted on the base layer when analyzed by the mechanistic-empirical pavement analysis software.

The Tanzanian design approach has a good sustainability as it exceeded the design period without failure. However; it was found uneconomical. The Kenyan design manual pavement fails midway the design period.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATIONS

5.1 Conclusions

After carrying out a pavement design by use of road design manuals from Kenya and Tanzania, AASHTO-1993 and South African software mePADS (mechanistic empirical pavement analysis design software) with a design period of 15 years the construction and maintenance costs were analyzed. The comparisons from the findings of various methods were done and came out with those recommendations:

1- It is possible to design a road with a design period of more than 15 years using the mePADS South African software and AASHTO 1993 also can get the corresponding layer thicknesses for a specified cumulative axle load and subgrade.
2- Designing using the Kenyan manual is not economical in the long run due to strengthening needed before the end of the design period.
3- Using mePADS and AASHTO 1993 was found more economical than the Kenyan as well as Tanzanian design manual when the long term cost of maintenance over the design period is taken into account.
4- Kenyan RDM, 1987 pavement was found to be structurally insufficient when analyzed by mePADS.

5.2 Recommendations

By going through analyzing rehabilitation of flexible pavement using the Kenya design manual (RDM, 1987) and Tanzania designs manual 1999, AASHTO 1993 and mePADS of South Africa, the following recommendations were drawn:-

1- The Kenyan design manual RDM, 1987 needs to be revised for the design and construction of economical and sustainable road pavement structures.
   The overlapping of the subgrade values in category of classes should be avoided.

2- The revised design manual of Kenya should be analyzed using the software from South Africa (mePADS) and the type and thicknesses in the catalogue should correspond to the result obtained from the software.

3- There is a need to establish a research center for pavement performance monitoring and research.

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